APPENDIX E
Estimation of Rock Mass Strength Properties and Evaluation of Rock Mass Structure

APPENDIX E.1
WSDOT Memorandum Dated April 8, 2009

Washington State Department of Transportation

Memorandum

April 8, 2009

TO: S. Golbek
South Central Region, Union Gap

FROM: T. M. Allen and T.C. Badger
E&EP Geotechnical Division, 47365

SUBJECT: I-90 Snoqualmie Pass East – Hyak to Keechelus Dam
Snowshed shaft foundations

The Geotechnical and Bridge & Structure Divisions have considered the design recommendations provided by URS in their *Final Technical Memorandum No. 3* (dated July 2008) for the extreme loading condition of seismogenic landslide forces on the rock-socketed shafts at the Snowshed. Based on the expected loads estimated by Bridge using the DFSAP model, the rock mass compressive strength of 350 psi estimated by URS will not support the structure. Because of the impacts associated with these findings, we have

- reviewed URS' estimation of rock mass strength discussed in Appendix D;
- further considered the AASHTO design methodology for rock-socketed shafts; and
- reevaluated the rock mass conditions and strengths for the estimated loads.

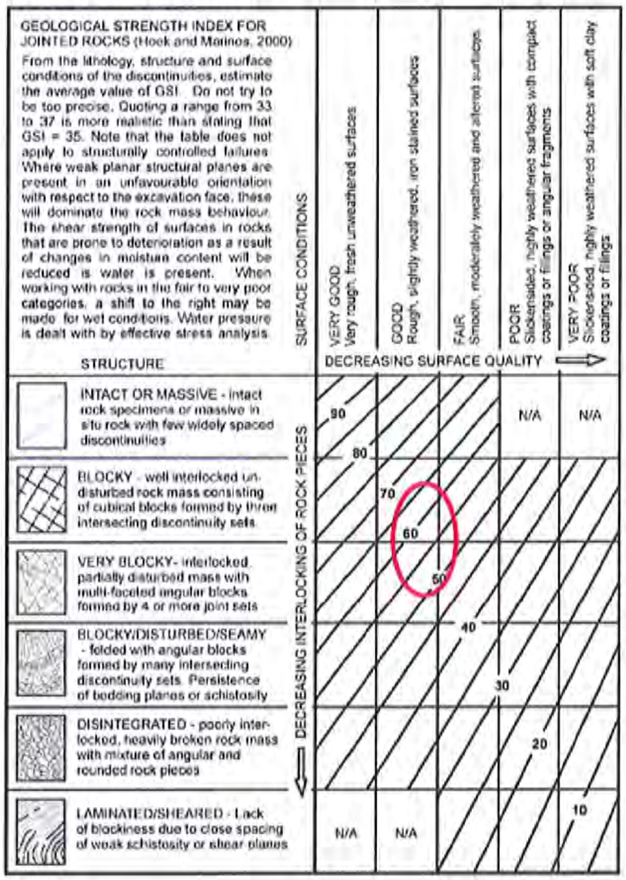
As pointed out by URS, AASHTO uses dated references for estimating rock mass compressive strengths – Rock Mass Rating (RMR) of Bieniawski (1984) and an empirical strength criterion of Hoek and Brown (1988) – and provides no explicit guidance for interpolating between Hoek-Brown classifications. The result, in our opinion, has resulted in an overly conservative estimation of rock mass compressive strength. Both AASHTO and WSDOT's Geotechnical Design Manual allow for the use of more current methods to estimate strength, which we have employed to consider a strength range that may more closely approximate interpreted rock mass conditions.

URS performed 30 point load tests to estimate the compressive strength of the intact rock. Based on a statistical evaluation of the test results, URS selected an intact compressive strength of 10,500 psi for design. To validate the point load test correlations, we performed six uniaxial compressive tests on representative cores proximal to URS point load tests, which yielded the following results:

Boring #	Sample Depth *(ft)*	Comments	Compressive Strength *(psi)*	Point Load UCS Estimate *(psi)*
SSD-002-07	9.9	slightly weathered healed fracture mid-sample	5,990	NA
SSD-002-07	27.5	fresh, no apparent discontinuities	27,670	43,400
SSD-002-07	40.9	fresh, no apparent discontinuities	29,430	35,400
SSD-003-07	24.4	fresh, no apparent discontinuities	7,600	10,000
SSD-003-07	34.0	slightly weathered, no apparent discontinuities	15,500	22,100
SSD-003-07	45.9	fresh, no apparent discontinuities	18,490	18,500

For the specific conditions, the point load correlations generally overestimate the uniaxial compressive strengths. The lowest UCS results were from samples taken within 5 ft of the bedrock surface, representing what we would interpret to be more weathered and disturbed, lower strength conditions. Examination of the cores and test results suggest that compressive strengths rapidly increase with depth. For design values, we support URS' selection of 10,500 psi for intact strength, but feel this represents a low bound and that a range up to 15,000 psi should be considered. Copies of the core photos and the UCS test results are attached to this memorandum.

Current rock engineering practice generally uses the Geologic Strength Index (GSI), rather than the Rock Mass Rating (RMR), to estimate rock mass strengths. We have used a current reference (http://www.rocscience.com/hoek/pdf/11_Rock_mass_properties.pdf) for estimating the GSI of the rock mass. Based on our observations and the RQDs of the cores, we would characterize the rock mass as "blocky" to "very blocky" with generally "good" surface (discontinuity) conditions, yielding a GSI range of 50 to 65, as shown in the figure below.



We followed Hoek et al. (2002) for estimating rock mass strength, which was also cited by URS. To allow for parametric iterations, we used the RocData v. 4.0 software by Rocscience. We considered a range of intact compressive strength values between 10,000 and 15,000 psi. We used a suggested range of m_i values for intact, fine-grained igneous or sedimentary rock of 10 to 15; and a disturbance factor, D=0, similar to URS.

Hoek et al. (2002) present two forms of in situ compressive strength for a fractured rock mass, one referred to as the *uniaxial compressive strength*, σ_c, and the other as the *global rock mass strength*, σ'_{cm}. The latter form is presented (with limited supporting documentation) as a better representation of the overall rock mass behavior, and perhaps more suitable for the complex loading conditions of 3D confinement around a shaft. We computed both forms of rock mass compressive strength for the above-mentioned range of input values (see attached plots), representing a potential lower and upper bound strength:

uniaxial compressive strength (σ_c) = 600 to 2100 psi

global rock mass strength (σ'_{cm}) = 1700 to 4400 psi

To ascertain exactly how the requisite DFSAP input parameter was used and to verify our approach for estimating rock mass compressive strength, we contacted Mohamed Ashour, the developer of DFSAP. He first verified that the requisite input parameter was the compressive strength of the weathered rock mass, and that the program used one half of this value to derive the shear strength of the rock mass as, by definition, the rock mass shear strength is equal to one-half the unconfined compressive strength. He followed AASHTO's approach of using RQD to estimate rock mass modulus, assuming that the ratio of intact to rock mass modulii would also apply to shear strength. Using this assumption and AASHTO (Table 10.4.6.5-1) for RQDs in the range of 50 to 70% for open fractures, a modulus ratio of 0.1 would yield a rock mass compressive strength of 1050 to 1500 psi for the above range of intact compressive strengths. While developed for shaft foundations, the assumption that the ratios in that table can be applied to shear strength is, in our opinion, questionable at best, and it appears to provide an overly conservative estimation of rock mass compressive strength.

While Hoek-Brown failure criterion is empirically derived from and is primarily used for excavations, it is the best available estimation for compressive strength of a fractured rock mass. We expect that because the rock is confined by soil overburden and not exposed, the application of Hoek-Brown to this foundation problem is likely to be conservative for the proposed 15-foot embedment into rock. For these reasons, we feel that rock mass compressive strengths between 2000 to 3000 psi would be more appropriate for the design of the shafts.

We recognize that this range represents a 6- to 8-fold increase for the value of 350 psi provided by URS. In light of our findings, we request that URS reevaluate their recommendations considering these new data and using the more current methodology to estimate rock mass compressive strength.

If you have questions or require further information, please contact Tom Badger at (360) 709-5461 or Tony Allen at (360) 709-5450.

REFERENCES

Bieniawski, Z.T., 1984. *Rock Mechanics Design in Mining and Tunneling.* A.A. Balkema, 272p.

Hoek, E. and E.T. Brown, 1988. "The Hoek-Brown Failure Criterion – A 1988 Update", *Proceedings, 15th Canadian Rock Mechanics Symposium*, Toronto, Canada, pp. 31-38.

Hoek, E., C. Carrazna-Torres, and B. Corkum, 2002. Hoek-Brown Failure Criterion – 2002 Edition; *5th North American Rock Mechanics Symposium and 17th Tunneling Association of Canada Conference*: NARMS-TAC, pp. 267-271.

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PROFESSIONAL ENGINEER

TMA/TCB
Attachments

cc: R. Giles, South Central Region
T. Moore, Bridge & Structures Office,

(14)

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION - MATERIALS LABORATORY
PO BOX 167 OLYMPIA, WA. 98507-0167/1655 SO. 2ND AVE TUMWATER, WA. 98512

Physical Testing Section
PCC Compressive Strength Test Report
Test Method AASHTO T 22

Work Order No. XL2779
Lab ID No. J-0000324507

Transmittal No. 516633
Bid Item No.
Org. Code 306310
F.A. No.

Date Received: 02/05/2009
S.R. No.: 90
County: KITTITAS
Section: SNOQUALMIE PASS EAST-PHASE 1A HYAK TO CRYSTAL SPR
Contractor:

Sent To Region
FEB 10 2009
From State Materials Lab

Cement: Type: Mill Test No.:

Mixing Plant Location: Cert. No.:

Admixtures		Mix Data
Air Entrainment	Per 100 Wt.:	Cement lb/cy:
Water Reducer	Per 100 Wt.:	Gravel Source:
High-Range Water Reducer	Per 100 Wt.:	Sand Source:
Water	Per 100 Wt.:	Approx. Slump:
Water/Cement Ratio	W/C :	Percent of Air:

Class:
Date Made: 02/05/2009 Date Tested: 02/06/2009 Test Age: 1 Days

Concrete Placement Location: SSD-02-07 / 9-9.5', 27.5-28', 40.9-41.4'

Cylinder No.	1	2	3
Diameter (in.)	2.40	2.40	2.41
Cross-Section Area (sq. in.)	4.52	4.52	4.56
Original Length (in.)	5.54	5.62	5.73
Original Weight (lbs.)	2.35	2.40	2.45
lbs/cf	162.2	163.3	162.0
L/D Ratio	2.31	2.34	2.38
Correction Factor	1.00	1.00	1.00
Maximum Load (lbf)	27,070	125,070	134,190
Compressive Strength (psi)	5,990	27,670	29,430
Fracture Type	Columnar	Cone & Shear	Columnar

Avg. Comp Strength (psi)

Distribution:

Materials File		X
Region Construction	45	X
Project Engineer:		
TOM BADGER		X(2)

Results: INFORMATIONAL
Remarks:

THOMAS E. BAKER, P.E.
MATERIALS ENGINEER
Michael Polodna, P.E. By:____
Date: 02/06/2009
Phone: (360)709-5446

T46E- T46H- T46Y-
T46N- 3.0 T46X-

cylinder.dfr 3/02

15

WASHINGTON STATE DEPARTMENT OF TRANSPORTATION - MATERIALS LABORATORY
PO BOX 167 OLYMPIA, WA. 98507-0167/1655 SO. 2ND AVE TUMWATER, WA. 98512

Physical Testing Section
PCC Compressive Strength Test Report
Test Method AASHTO T 22

Work Order No. XL2779
Lab ID No. J-0000324508

Transmittal No. 516634
Bid Item No.
Org. Code 306310
F.A. No.

Date Received: 02/05/2009
S.R. No.: 90
County: KITTITAS
Section: SNOQUALMIE PASS EAST-PHASE 1A HYAK TO CRYSTAL SPR
Contractor:

SENT TO REGION
FEB 10 2009
FROM STATE MATERIALS LAB

Cement: Type: Mill Test No.:

Mixing Plant Location: Cert. No.:

Admixtures		Mix Data
Air Entrainment	Per 100 Wt.:	Cement lb/cy:
Water Reducer	Per 100 Wt.:	Gravel Source:
High-Range Water Reducer	Per 100 Wt.:	Sand Source:
Water	Per 100 Wt.:	Approx. Slump:
Water/Cement Ratio	W/C :	Percent of Air:

Class:
Date Made: 02/05/2009 Date Tested: 02/06/2009 Test Age: 1 Days

Concrete Placement Location: SSD-03-07/24.4-24.9',34-34.5',45.9-46.4'

Cylinder No.	4	5	6
Diameter (in.)	2.41	2.41	2.42
Cross-Section Area (sq. in.)	4.56	4.56	4.60
Original Length (in.)	5.79	5.64	5.54
Original Weight (lbs.)	2.50	2.40	2.40
lbs/cf	163.6	161.3	162.7
L/D Ratio	2.40	2.34	2.29
Correction Factor	1.00	1.00	1.00
Maximum Load (lbf)	34,671	70,691	85,074
Compressive Strength (psi)	7,600	15,500	18,490
Fracture Type	Cone & Shear	Columnar	Columnar

Avg. Comp Strength (psi)

Distribution:

Materials File		X
Region Construction	45	X
Project Engineer:		
TOM BADGER		X(2)

Results: INFORMATIONAL
Remarks:

THOMAS E. BAKER, P.E.
MATERIALS ENGINEER
Michael Polodna, P.E. By: MP
Date: 02/06/2009
Phone: (360)709-5446

T46E- T46H- T46Y-
T46N- 3.0 T46X-

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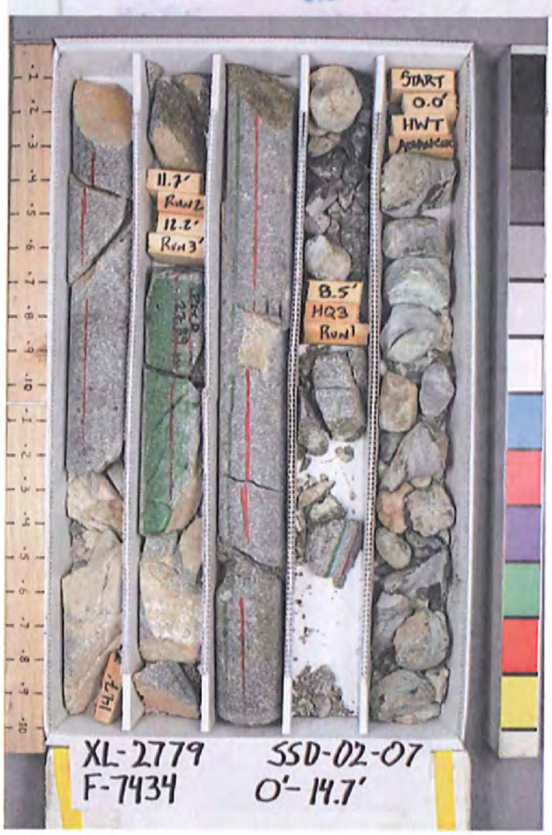
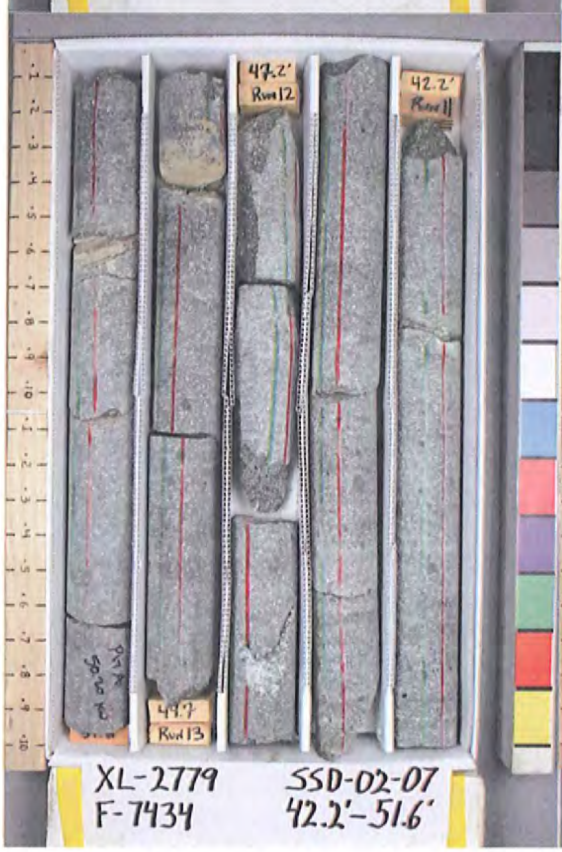
XL-2779
F-6875
H-23-06
6.5'-17.5'

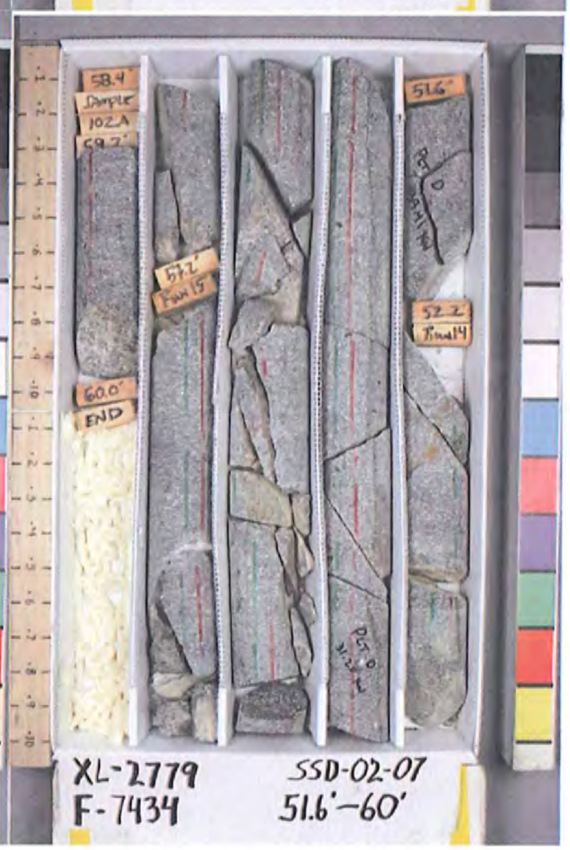

START
0.0'
HWT
11.7'
12.2'
8.5'
HQ3
XL-2779
F-7434
SSD-02-07
0'-14.7'

Box
missing


XL-2779
F-7434
SSD-02-07
23'–33'


XL-2779
F-7434
SSD-02-07
33'–42.2'


XL-2779
F-7434
SSD-02-07
42.2'–51.6'

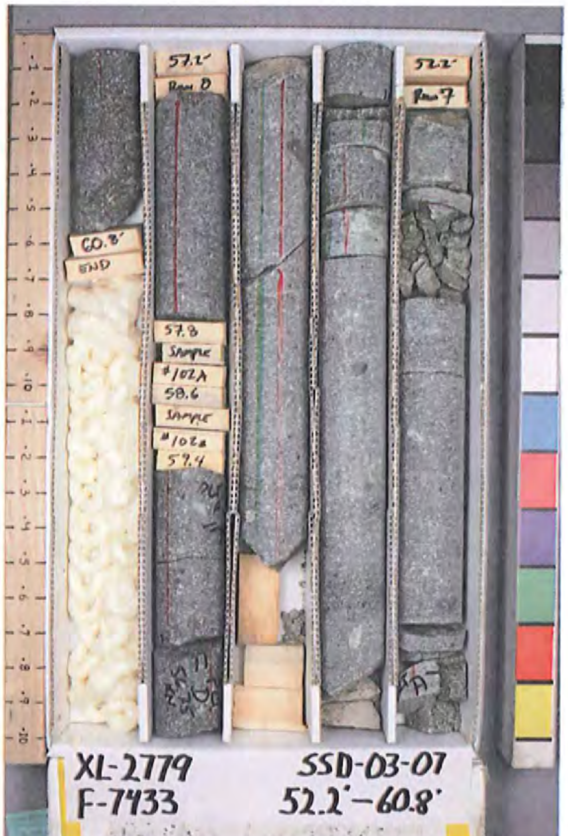

XL-2779
F-7434
SSD-02-07
51.6'–60'

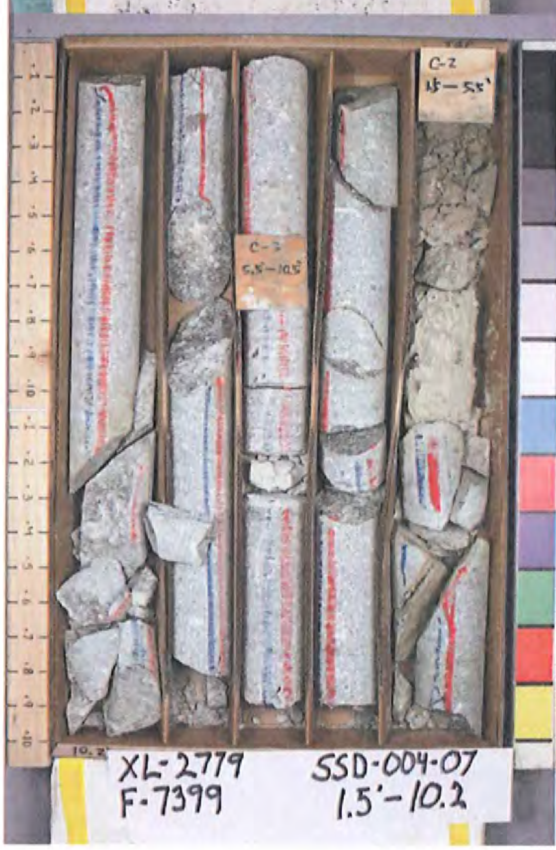

XL-2779
F-7433
SSD-03-07
0'–24.2'


XL-2779
F-7433
SSD-03-07
24.2'–35'


XL-2779
F-7433
SSD-03-07
35'–43.4'


XL-2779
F-7433
SSD-03-07
43.4'–52.2'


60.8'
END
XL-2779
F-7433
SSD-03-07
52.2'–60.8'

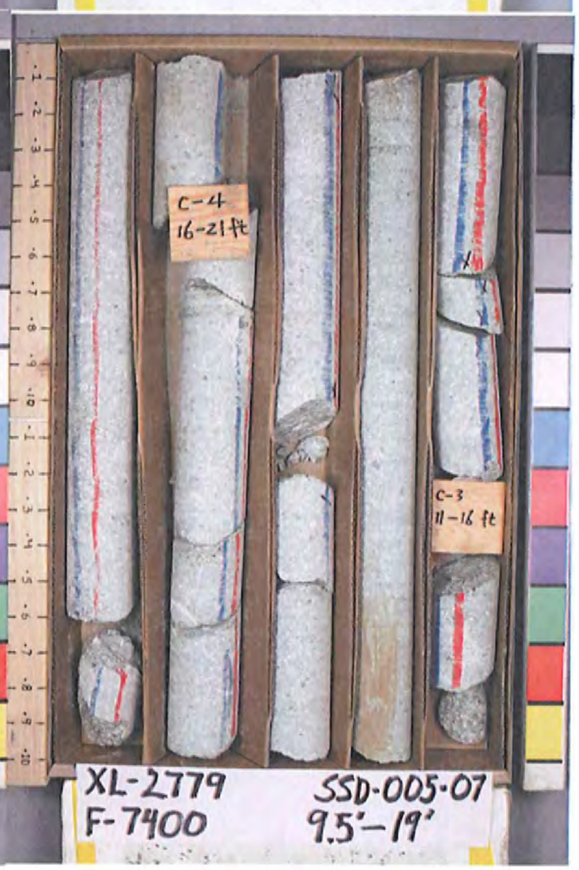

C-2
1.5–5.5'
C-3
5.5–10.5'
XL-2779
F-7399
SSD-004-07
1.5'–10.2

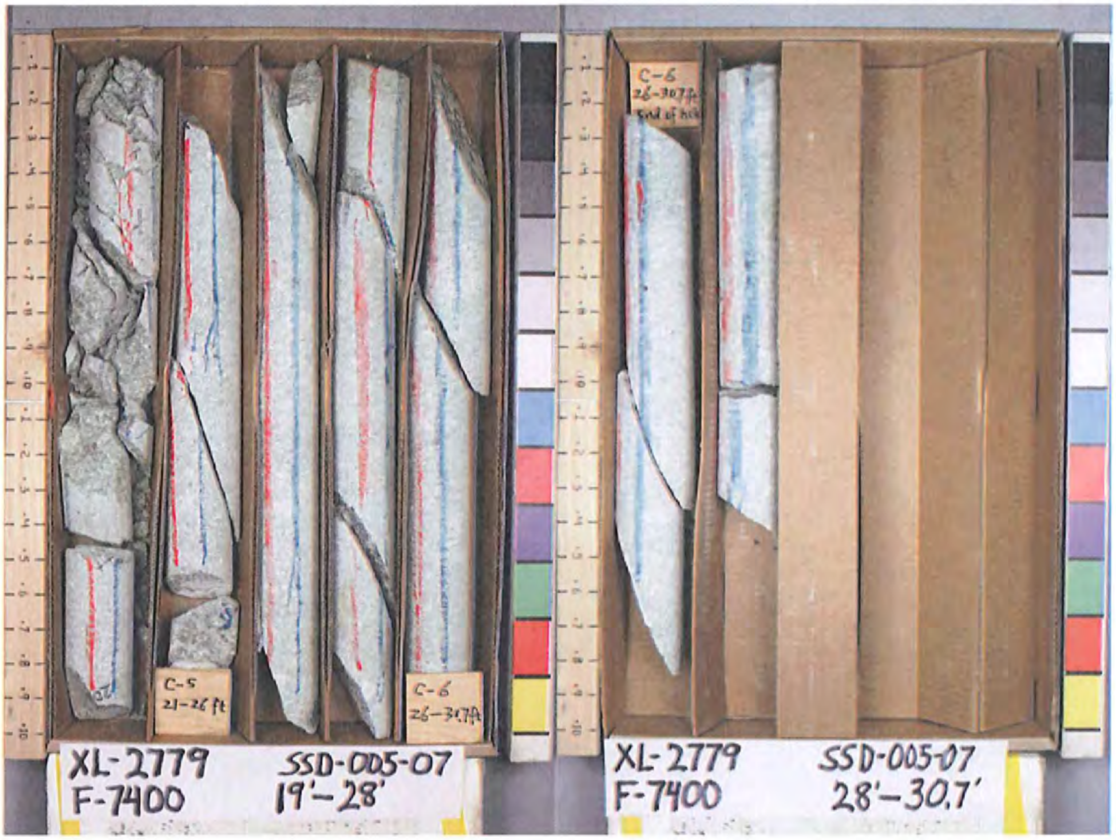

C-4
10.5–15.5'
C-5
15.5–20.5'
XL-2779
F-7399
SSD-004-07
10.2'–19.2'


C-6
20.5–25.5
C-7
25.5–30.5
XL-2779
F-7339
SSD-004-07
19.4'–30'


bottom
30.5'
End
C-7
25.5–30.5
End
XL-2779
F-7399
SSD-004-07
30'–30.5'

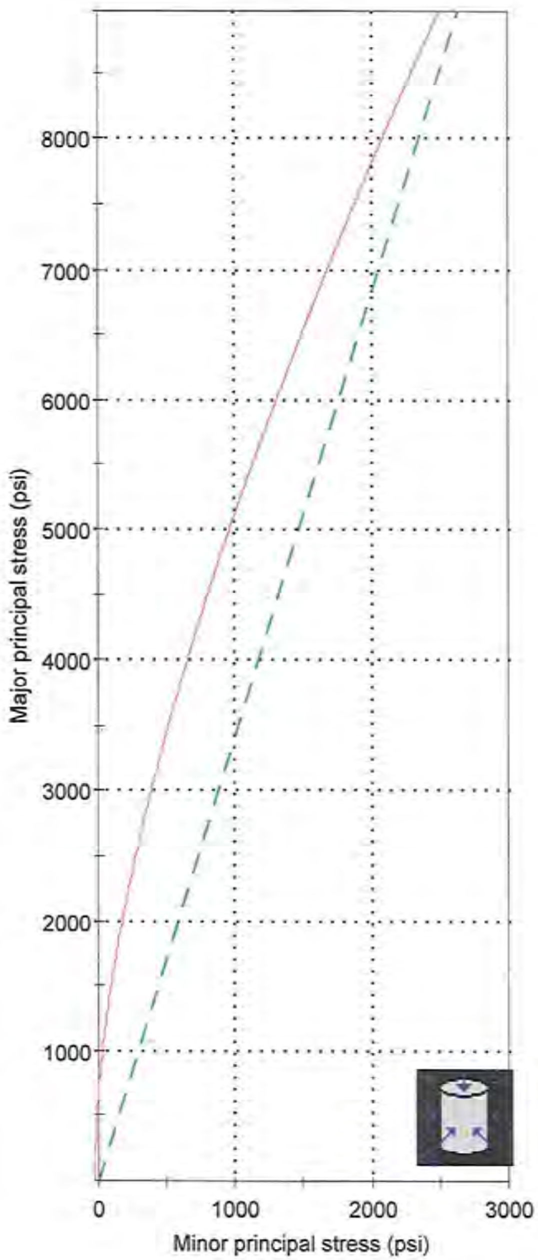

C-1
0–6 ft
C-2
6–11 ft
XL-2779
F-7400
SSD-005-07
0'–9.5'


C-4
16–21 ft
C-3
11–16 ft
XL-2779
F-7400
SSD-005-07
9.5'–19'


C-5
21-26 ft
C-6
26-30.7 ft
XL-2779
F-7400
SSD-005-07
19'–28'
C-6
26-30.7 ft
End of hole
XL-2779
F-7400
SSD-005-07
28'–30.7'

XL-2779 Hyak to Keechelus Dam
Snowshed shafts – rock cores

I-90 Snowshed shafts - low end strength

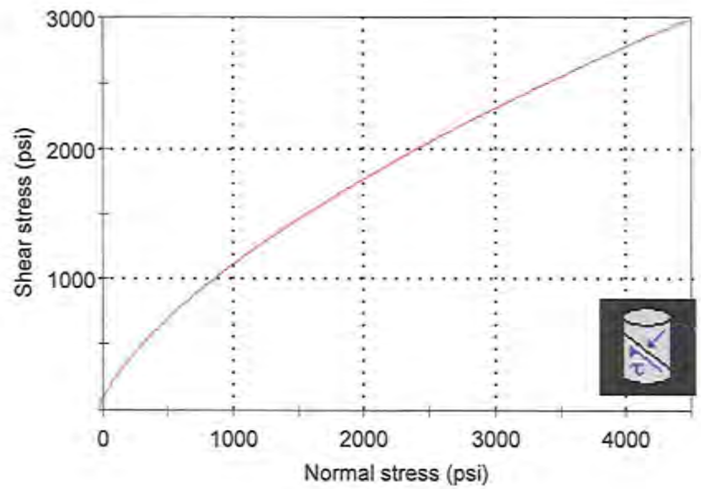


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 10000 psi
GSI = 50   mi = 10   Disturbance factor = 0
intact modulus (Ei) = 3e6 psi
modulus ratio (MR) = 300

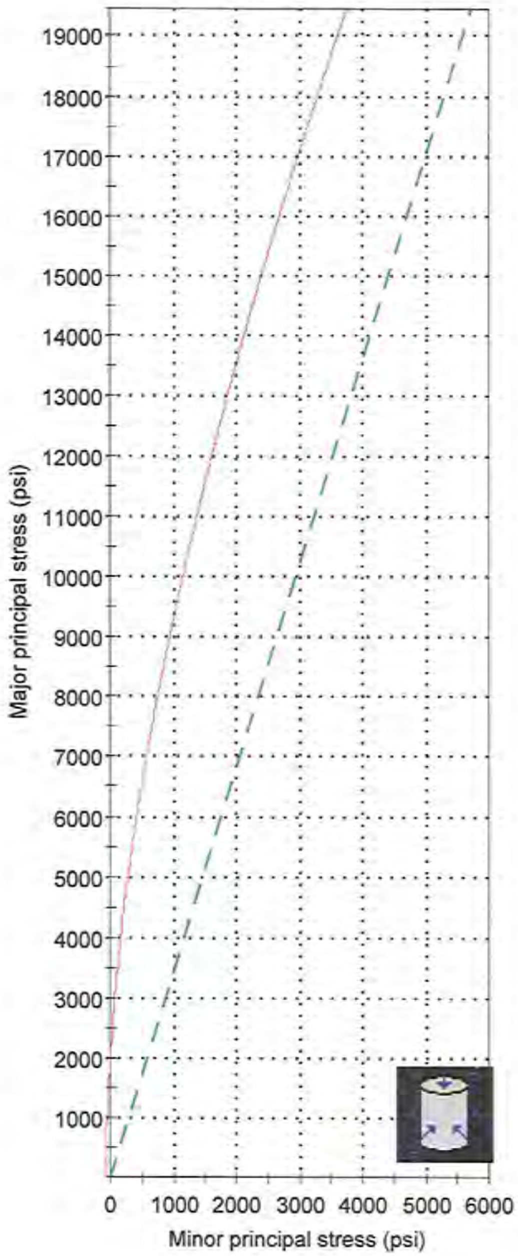
Hoek-Brown Criterion
mb = 1.677   s = 0.0039   a = 0.506

Mohr-Coulomb Fit
cohesion = 498.046 psi   friction angle = 30.52 deg

Rock Mass Parameters
tensile strength = -23.056 psi
uniaxial compressive strength = 602.272 psi
global strength = 1743.335 psi
modulus of deformation = 921557.70 psi



I-90 Snowshed shafts - high end strength

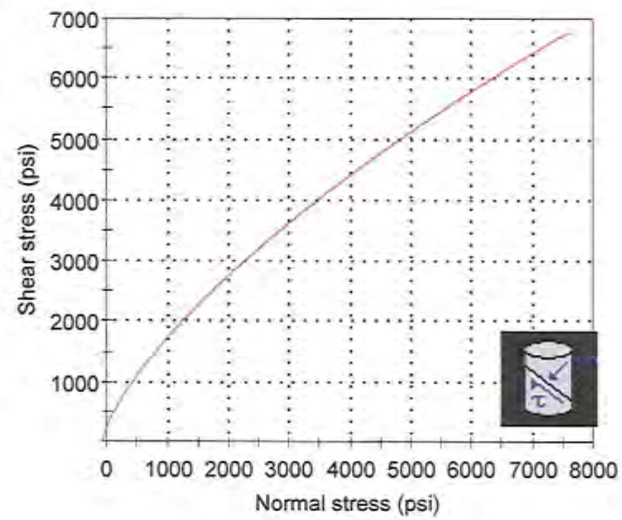


Hoek-Brown Classification
intact uniaxial comp. strength (sigci) = 15000 psi
GSI = 65 mi = 15 Disturbance factor = 0
intact modulus (Ei) = 4.5e6 psi
modulus ratio (MR) = 300

Hoek-Brown Criterion
mb = 4.298 s = 0.0205 a = 0.502

Mohr-Coulomb Fit
cohesion = 1060.929 psi friction angle = 38.47 deg

Rock Mass Parameters
tensile strength = -71.441 psi
uniaxial compressive strength = 2129.579 psi
global strength = 4396.075 psi
modulus of deformation = 2842737.35 psi



APPENDIX E.2
URS Memorandum Dated April 29, 2009

Snowshed Shaft Foundations: Rock Strength Estimates for DFSAP Analysis – I-90 Snoqualmie Pass East

TO: Tony Allen and Tom Badger; WSDOT

FROM: Chuck Vita and Cecil Urlich; URS

DATE: April 29, 2009

Copies: Randy Giles, Scott Golbek, Jerry Wood, Patrick Cooper; WSDOT

This memo responds to your (WSDOT) memo of 08 April 2009 to Scott Golbek on the foundation rock compressive strengths for the DFSAP analysis on the Pier 2 shafts of the proposed replacement Snowshed for the I-90 Snoqualmie Pass East project.

WSDOT requested that URS reevaluate its 350-psi (50,000-psf) Rock Mass Unconfined Compressive Strength (UCS) [1] recommendation for DFSAP analysis because that estimate, according to WSDOT, "will not support the structure." WSDOT, based on their evaluation, concluded that "rock mass compressive strengths between 2000 to 3000 psi would be more appropriate for the design of the shafts."

URS presented its 350-psi Rock Mass UCS in Table 8 "Pier 2 Design Parameters for Lateral Load Analysis Using DFSAP" of the Final Technical Memorandum 3 – Snowshed Replacement, July 2008 (TM3). The assumptions and methodology used to estimate a 350-psi Rock Mass UCS is documented in TM3 Appendix D. Appendix D contains URS' 05 May 2008 memo to WSDOT, "Discussion of Rock Mass Unconfined Compressive Strength: Estimates for Snowshed DFSAP Analysis." It is important to note that the Rock Mass UCS was a DFSAP recommendation, and not the shaft rock socket vertical and lateral capacities recommended in TM3.

Table 1 Results: A Preliminary Look

Table 1 presents the rock mass compressive strength calculations requested by WSDOT. Both the Rock Mass UCS and Global Rock Mass Strength (RMS) have been calculated, as well as the Rock Mass Modulus, for the range of Intact Rock UCS and Geologic Strength Index (GSI) values suggested by WSDOT, along with supplementary estimates that URS thinks are relevant.

Based on interpretation of the Table 1 results and the supporting analyses discussed in this memo, URS opines that a rock mass compressive strength for DFSAP analysis can be increased from 350 psi to 2,000 psi *if* the Global RMS is used and if WSDOT can accept an estimated 50% probability that the rock at one of the 44 shafts will have *less* than 2,000-psi Global RMS, and a less but still significant probability that more than one shaft will have less than 2,000-psi Global RMS.

Significantly reducing the probabilities of such potentially "under-performing shafts" would require essentially confirmatory, supporting results from additional rock boring and testing, in our

[1] UCS, or unconfined compressive strength, is also referred to as "uniaxial compressive strength."

opinion. As will be discussed, there is currently very limited rock strength data along the proposed Snowshed Pier 2 alignment.

Increasing the rock mass compressive strength estimate increases the potential for one or more shafts having compressive strengths less than the estimate. URS agrees with WSDOT that, given the available data, 2,000 psi to 3,000 psi can be justified for *general* or average conditions along the Pier 2 alignment. However, general conditions currently appear unlikely to apply at all 44 shaft locations. Using the available data, the TM3 350-psi estimate had aimed at having a greater than 50% probability (reliability) that *none* of the 44 shafts would have an actual rock compressive strength that was effectively less than that 350-psi estimate.

If the Global RMS is used in place of the Rock Mass UCS, the 350-psi would increase to 1,300 to 2,000 psi, a factor of nearly 4 to 6. This increase is for Tuff, the rock type along the Pier 2 alignment having the deepest depths to rock. The other Snowshed rock types, Andesite and Agglomerate, would have higher relative increases. Global RMS significantly exceeds Rock Mass UCS, all other rock parameters and qualities being equal.

The Table 1 results will be discussed further following the next discussions, which add context for understanding and evaluating those results.

Differences Between TM3 and WSDOT Rock Parameters

Both URS and WSDOT based their rock mass strength estimates on the 2002 Hoek-Brown failure criterion.[2] The differences between the estimates described in URS TM3 and the WSDOT memo has to do with an appropriate value, or value-range, for the following parameters:

1) Hoek-Brown input GSI (Geological Strength Index)
2) DFSAP input "Compressive Strength of Rock Mass" (not explicitly a Hoek-Brown parameter)
3) Intact Rock UCS (qu_i or σ_{ci})

WSDOT suggested that a GSI range of 50 to 65 and an Intact Rock UCS range of 10,000 psi to 15,000 psi be used to estimate the rock mass strength. WSDOT also suggested that the Global RMS could be more appropriate for DFSAP analysis than the Rock Mass UCS used in TM3.

Geological Strength Index (GSI) Range

WSDOT interpreted a GSI range of 50 to 65 using the latest Hoek-Brown guidance. TM3 used a GSI of 40 based on an RMR (Rock Mass Rating) of 45 that was interpreted from available data following 2008 AASHTO Section 10.4.6.4 "Rock Mass Strength" to the extent practical.

The AASHTO guidelines are not consistent with Hoek-Brown 2002, as discussed in TM3 Appendix D. Although 2008 is AASHTO's latest guidance, it may be considered less-appropriate and obsolete compared to the latest Hoek-Brown guidance. Also, because AASHTO does not use

[2] Hoek-Brown 2002: Evert Hoek, Carlos Carranza-Torres, and Brent Corkum, "Hoek-Brown Failure Criterion--2002 Edition."

GSI, TM3 used the relation GSI = RMR-5, for RMR>23, to estimate GSI for use with the 2002 Hoek-Brown Criteria.[3]

URS can agree with WSDOT's range of 50 to 65 for general conditions along Pier 2. However, based on the same Table 5 cited by WSDOT[4] and conditions documented for boring SSD-002-07, it appears that a GSI as low as 40 to 45 could be appropriate for some limited portions of Pier 2. According to Table 5, a GSI as low as 40 (or greater than 35) to 50 (or less than 55) applies to a "very blocky" structure with "fair" surface (discontinuity) quality.

At SSD-002-07, discontinuities in approximately the upper 20 ft of rock are logged as "closely to moderately spaced and in poor to fair condition" with RQDs of about 40% between rock depths of 4 and 15 feet. SSD-002-07 is one of only a limited number of borings that are even close to the Pier 2 alignment, and is thus unlikely to represent the worst conditions that will be encountered at the 44 Pier 2 shaft locations. A minimum GSI of 40 is used in Table 1 (a GSI of 40 was used in TM3) to represent potential localized minimums affecting shafts.

A comparison can be made with the 1957 failure at Slide Curve. Norm Norrish reports that for the Domain 3 rock at Slide Curve which directly contributed to the 1957 failure and which is the poorest quality rock encountered along the alignment, an RMR range of 30 to 41 was assessed. This would relate to a GSI range of 25 to 36, and would constitute a lower bound estimate.

DFSAP Input Parameter "Compressive Strength of Rock Mass"

Depending on assumptions and details of the DFSAP modeling theory and computer code, the more appropriate 2002 Hoek-Brown parameter for DFSAP analysis could be the Global RMS, as suggested by WSDOT, rather than Rock Mass UCS, as used in TM3.[5]

Exactly how the subject input parameter is used in DFSAP and how it relates to model calibration or accuracy remains unclear. A parametric sensitivity analysis of DFSAP results would be helpful in evaluating appropriate input values, particularly if the sensitivity is limited to the longer shafts or is effectively location dependent.

The Rock Mass UCS is a significantly more conservative interpretation of rock mass compressive strength than is the Global RMS. Rock Mass UCS is like a localized "yield strength" whereas Global RMS is more like a generalized "peak failure strength." It seems that the Rock Mass UCS could be unrealistically conservative, depending on how it is used in DFSAP.

It is clear that the DFSAP input parameter "rock mass compressive strength" is inherently uncertain, subject to significant model uncertainty and parameter uncertainty, both of which affect the DFSAP results and should inform their interpretation.

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[3] NCHRP 2006: *Rock-Socketed Shafts for Highway Structure Foundations*, TRB. GSI=RMR-5 is Eq 7, p.24.
[4] Rocscience web site: "Rock Mass Properties," http://www.rocscience.com/hoek/pdf/11_Rock_mass_properties.pdf.
[5] Rock Mass UCS is commonly symbolized "qu" or, as in Hoek-Brown, "σ_c"; the Hoek-Brown Global RMS, is symbolized "σ_{cm}."

That said, and while there are apparently reasonable arguments for using either parameter (or both), the DFSAP final report (April 2006) simply calls for entering the “Compressive Strength of Rock Mass” as *the value of the unconfined compressive strength (qu) of the rock mass*, which is the Rock Mass UCS, not the Global RMS.

Consistent with the DFSAP documentation, URS has generally used Rock Mass UCS as the recommended input for DFSAP analysis, including the Gold Creek Bridges and Slide Curve Bridge. A decision to use the Global RMS instead of the Rock Mass UCS would, presumably, also affect the DFSAP analyses for these other structures.

Intact Rock UCS

Another question is the appropriate value for the Intact Rock UCS. TM3 used 10,500 psi based on a statistical analysis and evaluation of available Point Load Test (PLT) results. WSDOT’s range of 10,000 psi to 15,000 psi was based on their interpretation of the DM3 PLT data and their UCS lab testing of six rock core specimens, three each from borings SSD-002-007 and SSD-003-007.

Intact Rock UCS will be discussed in some detail because it points to questions regarding the basis of *generally* using increased rock compressive strengths, given the available boring data, for all 44 shafts along Snowshed Pier 2. Appropriate Intact Rock UCS, and GSI, values are thus imbedded in a bigger issue of limited rock data at shaft locations along Snowshed Pier 2, as elaborated next.

Limited Rock Data for Snowshed Pier 2 Shafts

The proposed Snowshed Pier 2 includes 44 shafts of 8-ft diameter over a length of approximately 1,100 ft. URS has not identified any borings with rock data adequate for design at any of the 44 shaft locations. There are also no borings actually along the Pier 2 alignment. However, the URS rock characterization did make use of nine borings that are considered near enough to the Pier 2 alignment to provide limited extrapolated data for conceptual-level shaft sizing.

The closest borings to the Pier 2 alignment with reasonably complete rock data such as rock descriptions, RQDs, and PLT data, are SSD-002-07 and SSD-003-07. These borings were drilled approximately 160 and 140 ft, respectively, upslope of the Pier 2 alignment. These two borings are located upslope from the deepest depths to bedrock along Pier 2, the portion of Pier 2 that has the highest lateral loading on the shafts and will require the longest shafts. The rock at both borings is Lapilli Tuff.

Two other borings have very limited PLT data. These are H-23-06 (1 PLT specimen), located 200 ft west of the west end of Pier 2 and offset 100 feet south; and H-24-06 (2 PLT specimens), located 350 ft east of the east end of Pier 2, and no offset. The rock at both of these borings is Andesite.

Five other, relatively nearby borings that have rock descriptions, but no PLT data, are 90 ft to over 600 ft from Pier 2. The rock at these borings is Lapilli Tuff, Andesite, or Agglomerate. There are numerous, much older borings near Pier 2 that help define depths to rock, but include only brief and ambiguous rock descriptions that provided no additional information useful to quantitatively estimating rock compressive strengths.

Statistical Reliability Analysis of Boring-Averaged Intact Rock UCS (qu_i)

This section updates the TM3 Intact Rock UCS estimate (10.5 ksi) that was based on a statistical analysis and interpretation of available rock PLT data. That data consisted of 33 PLT results on 22 specimens from four borings. Figure 1 graphs the 33 PLT results and WSDOT UCS results as Intact Rock UCS (horizontal axis) against test-specimen depth below the bedrock surface (vertical axis). Figure 2 graphs both the 33 PLT results and the RQD measured at each of the nine borings near the Pier 2 alignment.

Figure 3 includes only the two borings in Tuff with PLT data and WSDOT UCS measurements. The depth variability and testing gaps apparent in Figure 3 make it impractical to quantify a correlation between PLT and UCS because it is not clear which PLT results, if any, should be paired with the UCS results, which greatly increases the uncertainty of any quantified correlation or trend. However, the average of the six UCS results (17.4 ksi, standard error = 4.0 ksi) and 22 PLT results (21.2 ksi; standard error = 2.3 ksi) were not statistically different at the 95% confidence level (critical alpha > 0.10), supporting the decision to combine available PLT and UCS data in the statistical reliability analysis.

The statistical reliability analysis assumed that the available PLT and UCS laboratory testing results gave representative and unbiased estimates of the (uncertain) actual Intact Rock UCS (qu_i) along the Pier 2 alignment—recognizing that the available data were in fact only spatially proximate to Pier 2 and that some of the PLT results could be questionable. Ten specimens having two or three test results at effectively constant depth were averaged to yield one specimen value. Three borings had more than one specimen result.

The specimens were averaged at each of the three borings having more than one specimen to yield boring averages. Using boring averages significantly reduced variability from specimen results. The boring averages were interpreted as a rough estimator of effective “shaft rock-socket averages.”

The three boring averages were 25.5 ksi (SSD-007-02), 20.3 ksi (SSD-007-03), and 18.1 ksi (H-24-06); these yielded an average boring-average of 21.3 ksi with a standard deviation of 3.81 ksi and a coefficient of variation of 18%. Including the WSDOT UCS lab results changed the two affected boring averages to 24.2 ksi (SSD-002-07) and 18.7 ksi (SSD-003-07), yielding an updated average boring-average of 20.3 ksi with a standard deviation of 3.35 ksi and a coefficient of variation of 16%. (Considering only the six UCS results the boring averages were 21.0 ksi at SSD-002-07 and 13.9 ksi at SSD-003-07, yielding a 2-boring, N=2, average boring-average of 17.4 ksi, with a standard deviation of 5.07 ksi and a coefficient of variation of 29%; the standard deviation for the six samples was 9.81 ksi).

The statistical analysis assumed that the shaft (boring) averages were normally distributed with the average equal to the average of the three boring averages and standard deviation equal to the standard deviation of the three boring averages. The reliability estimates were based on an effective sample size of N=3 boring averages. The analysis reflects both inherent spatial variability ("aleatory" uncertainty), through the normal distribution and standard deviation of the averages, and effects of limited data ("epistemic" uncertainty), measured by the sample size, N=3.

The Statistical Reliability Analysis Results table below summarizes results both with and without the WSDOT UCS data. There is no significant difference in the results within the accuracy of the analysis, so only the results that include the UCS lab data will be discussed.

To a rough but useful first approximation, and given the available data (the N=3 boring averages and standard deviation), the analysis calculated the boring (shaft) <u>average</u> Intact Rock UCS, or qu_i, that has a probability (reliability) R of being exceed by a Proportion "P" of the 44 shaft locations along Pier 2. The lowest P for less than one shaft being weaker than the qu_i estimate is P=0.98. From the table, a qu_i estimate of:

- 13.1 (13) ksi has an estimated R=50% reliability (probability) of being exceeded at all 44 shafts
- 10 ksi has (by interpolation) an estimated R=60% reliability of being exceeded at all 44 shafts.

Statistical Reliability Analysis Results

Reliability R that qu_i-Actual Will Exceed qu_i-Estimate	Proportion "P" of Borings (Shafts) Having Average Intact Rock UCS qu_i <u>Above</u> Selected Estimate					
	0.75	0.90	0.95	0.98	0.99	0.999
	Equivalent Number of Shafts Out of 44 Having Strengths qu <u>Below</u> Selected Estimate					
	11	4.4	2.2	0.9	0.4	0.0
	Original TM3 PLT Data (Avg Boring Avg = 21.3 ksi; Std Dev = 3.81 ksi)					
Reliability R	**qu_i Estimate of Shaft Intact Rock UCS, ksi**					
50%	18.7	16.4	15.0	13.1	12.4	9.5
75%	15.7	11.8	9.3	5.8	4.7	ID
90%	11.4	5.1	ID	ID	ID	ID
95%	6.8	ID	ID	ID	ID	ID
	With WSDOT UCS Data (Avg Boring Avg = 20.3 ksi; Std Dev = 3.35 ksi)					
Reliability R	**qu_i Estimate of Shaft Intact Rock UCS, ksi**					
50%	18.1	16.0	14.8	13.1	12.5	10.0
75%	15.4	12.0	9.8	6.7	5.7	ID
90%	11.6	6.1	2.5	ID	ID	ID
95%	7.6	ID	ID	ID	ID	ID

ID = Inadequate Data for Estimate

For an estimated reliability of R=75% the UCS would decrease to 6.7 ksi. There is inadequate data (ID) to achieve an estimated reliability of 90% or greater for P=0.98. However, if P was reduced to 0.95, such that 2 of the 44 shafts could be weaker than the estimate, then the qu_i estimate could be increased to 15 ksi for R=50%. For P=0.95, an estimate of 10 ksi yields R=75%. For P=0.75, a qu_i estimate of 15 ksi yields R=75%; 10 ksi yields R>90%; and so on. At P=0.50, half of the 44 shafts could be weaker than the estimate; for P=0.50 and R=50% the qu_i estimate is equal to the average borehole average, which is 20 ksi.

The results also show that while using a qu_i estimate of 13 ksi would provide R=50% for P=0.98, there would still be an estimated 10% to 20% probability that more than 10 shafts would have an actual qu_i of less than 13 ksi (interpolated from P=0.75 between R=75% and R=90%). With a qu_i estimate of 10 ksi there would still be an estimated probability between 5% and 10% that more than 10 shafts would have an actual qu_i of less than 10 ksi (interpolated from P=0.75 between R=90% and R=95%).

Interpreting the P and R aspects of these qu_i results in terms of the calculated Rock Mass UCS and Global RMS requires consideration of the variability and uncertainty in the GSI and parameter m_i (discussed in the next section). For *constant* GSI and m_i, the P and R values associated with the qu_i results apply directly to both Rock Mass UCS and Global RMS. A more refined analysis could directly consider (in the statistical model) the additional affects of variability and uncertainty in GSI and m_i on computed Rock Mass UCS and Global RMS (these affects include potential correlations between the parameters).

Global Rock Mass Strength (Global RMS)

According to Hoek et al., the Global RMS represents the “overall behavior of a rock mass” and, unlike the Rock Mass UCS, includes effects of confinement pressure. Calculation of the Global RMS differs from the Rock Mass UCS in that its calculation requires the additional intact-rock parameter, termed “m_i.” For a given rock mass, the Global RMS exceeds the Rock Mass UCS, with the relative degree of exceedance depending on:

a) GSI, decreasing with increasing GSI, and
b) m_i, increasing with increasing m_i.

Hoek-Brown Intact Rock Parameter m_i for Snowshed Rock Types

Based on the Rocscience publication referenced by WSDOT[6], parameter m_i is (like the inherently uncertain *actual* GSI and qu_i) a probabilistic variable that depends on rock type. For the three rock types identified along the Pier 2 alignment (Lapilli Tuff, Andesite, and Agglomerate), m_i values are as follows: [7]

- Tuff: 13 ± 5
- Andesite: 25 ± 5
- Agglomerate: 19 ± 3

[6] The reference cited in footnote 4.
[7] Ibid Table 3

For a constant GSI, the Tuff will have the least increase in Global RMS over Rock Mass UCS because its m_i values are less than the m_i for Andesite or Agglomerate. From Table 1 the increase for Tuff ranges from a factor of nearly 4 to 6.

Besides having the least relative strength increase from Rock Mass UCS to Global RMS, Tuff is also the rock type located along the Pier 2 alignment having the deepest depths to bedrock. This means that the longest shafts, which have the largest lateral loads, will be in Tuff.

Tuff is, therefore, the critical rock type in terms of rock strengths along the Pier 2 alignment. For the same intact rock UCS and GSI, both Agglomerate and Andesite will have a higher Global RMS than Tuff, while all three will have the same Rock Mass UCS, according to Hoek-Brown 2002 methodology.

Conclusions and Recommendations

Table 1 shows the calculated Rock Mass UCS and Global RMS following WSDOT's recommendations for the range of Intact Rock UCS or qu_i of 10,000 psi to 15,000 psi and GSI of 50 to 65. While WSDOT used m_i values of 10 to 15, Table 1 uses m_i values of 13 ± 5 (i.e., 8, 13, and 18), or 8 to 18, consistent with the m_i values for Tuff in "Rock Mass Properties, Table 3" on the Rocscience web site, as cited above.

The quantities for the WSDOT parameter sets are included as Cases 1-12 in Table 1. There are six additional cases, for a total of 18 cases in Table 1, as explained below.

The Table 1 values only include Tuff. Tuff is the critical rock type for Pier 2 because the longest shafts with the highest lateral loads will be in Tuff. Tuff also has a lower m_i than either Andesite or Agglomerate, which means it has a lower Global RMS than either for the same GSI and qu_i.

Calculated Rock Modulus values are also included in Table 1. Two calculations are included: one using the equation of Hoek-Brown 2002, and one based on 2008 AASHTO (10.4.6.5-1). It is noted that the Rock Modulus exceeds the Rock Mass UCS by a factor of about 2,000 for cases in Table 1. Table 1 also shows sets of values for:

1) qu_i = 10,500 psi and GSI = 40, which was used in TM3 (Cases 13-15)
2) qu_i = 13,000 psi and GSI = 40 (Cases 16-18).

The first of the two sets quantifies the increase in rock mass compressive strength from the 350-psi recommended in TM3 by simply changing the operative definition from Rock Mass UCS to Global RMS. That change to Global RMS results in a rock mass compressive strength of 1,300 to 2,000 psi, a factor of nearly 4 to 6 greater than the Rock Mass UCS.

The second set represents the maximum estimates that, in URS' opinion, the limited available data can, arguably, defensibly support in a statistical sense as being "more probable than not" that the *actual* rock mass compressive strength (either Rock Mass UCS or Global RMS) will be equal or greater than the *calculated* values at all 44 shaft locations. For this case, the calculated Rock Mass

UCS is 430 psi and the calculated Global RMS is 1,600 psi to 2,400 psi with a median "best estimate" of 2,000 psi. Therefore, *on this qualified basis:*

- URS can recommend a maximum rock mass compressive strength of 2,000 psi for DFSAP analysis.

Higher values or higher reliability would require additional borings and testing along the Pier 2 alignment at actual proposed shaft locations. Also, it is considered that additional borings and testing may be cost effective for the design of the longest shafts having the highest lateral loads.

Recommended SSD 2009 Borings

It is tentatively suggested that six borings be drilled as part of the 2009 geotechnical field program at proposed shaft locations as shown on the accompanying plan view and profile (separate files from this memo). It is recommended that the borings, called out as SSD-006-09 to SSD-011-09, be placed on 100-ft centers, as close to shaft locations as practical, and span the 500-ft section of Pier 2 having greatest depths to bedrock. This area is identified on the accompanying profile from approximately WB station 1355 to 1360.

We also believe that rock testing should include at least three UCS tests (ASTM D4719) per boring on representative rock core specimens. To the extent practical, the UCS testing should be paired with co-located PLT so that a relevant correlation with PLT can be established.

Besides the boring's apparent design value, their likely benefits from a construction and bidding standpoint of increasing the quality and quantity of rock information at or near the Pier 2 shafts may greatly increase their ultimate cost-effectiveness.

Other Possible Was to Increase Shaft Performance

While outside the scope of this memo, it is briefly noted that there are other possible ways to increase shaft performance. These could include the use of deeper or larger-diameter rock sockets; selective use of upslope "guard-type shafts" to shadow critical shafts; or shaft-location-specific ground treatment to increase downslope soil or rock passive restraint on critical shafts.

Closure

URS recommends that a conference call or meeting be held to confirm a mutual understanding and "meeting of the minds" on the WSDOT needs and the discussions and recommendations in this URS memo. In particular, our better understanding of the DFSAP analysis and results could significantly help in resolving questions of a suitable rock mass compressive strength.

If you have technical questions about this memo, please feel free to email or call Chuck Vita (206-438-2348) at any time.

Table 1.—Rock Mass Unconfined Compressive Strengths (UCS), Global Rock Mass Strength (RMS), and Rock Mass Modulus, as a Function of Input Parameters: Intact Rock UCS (qu_i), GSI, and Hoek-Brown Parameter m_i.

Parameter Case	Intact Rock Unconfined Compressive Strength, UCS or qu_i	GSI	Rock Mass Unconfined Compressive Strength, Rock Mass UCS or qu Hoek-Brown 2002		Hoek-Brown Parameter m_i for Tuff, m_i = 13 +/- 5	Global Rock Mass Strength, Global RMS Hoek-Brown 2002		Rock Mass Modulus Hoek 2002	Rock Mass Modulus AASHTO 2008	Ratios: Global RMS to Rock Mass UCS	Ratios: Modulus to Rock Mass UCS Hoek 2002
	psi		ksf	psi		ksf	psi	ksi	ksi		
1	10,000	50	87	602	8	227	1,579	1,204	1,934	2.6	2,000
2	10,000	50	87	602	13	286	1,984	1,204	1,934	3.3	2,000
3	10,000	50	87	602	18	334	2,321	1,204	1,934	3.9	2,000
4	10,000	65	204	1,420	8	331	2,298	2,856	4,585	1.6	2,012
5	10,000	65	204	1,420	13	402	2,791	2,856	4,585	2.0	2,012
6	10,000	65	204	1,420	18	462	3,212	2,856	4,585	2.3	2,012
7	15,000	50	130	903	8	341	2,369	1,475	1,934	2.6	1,633
8	15,000	50	130	903	13	429	2,976	1,475	1,934	3.3	1,633
9	15,000	50	130	903	18	501	3,481	1,475	1,934	3.9	1,633
10	15,000	65	307	2,130	8	496	3,447	3,498	4,585	1.6	1,642
11	15,000	65	307	2,130	13	603	4,186	3,498	4,585	2.0	1,642
12	15,000	65	307	2,130	18	694	4,817	3,498	4,585	2.3	1,642
13	10,500	40	50	347	8	190	1,317	694	1,087	3.8	1,999
14	10,500	40	50	347	13	241	1,674	694	1,087	4.8	1,999
15	10,500	40	50	347	18	284	1,970	694	1,087	5.7	1,999
16	13,000	40	62	430	8	235	1,631	772	1,087	3.8	1,796
17	13,000	40	62	430	13	298	2,073	772	1,087	4.8	1,796
18	13,000	40	62	430	18	351	2,439	772	1,087	5.7	1,796

Notes:

1) Hoek-Brown 2002: Evert Hoek, Carlos Carranza-Tores, and Brent Corkum, "Hoek-Brown Failure Criterion--2002 Edition."

2) Parameter mi based on Table 3 "Rock Mass Properties," http://www.rocscience.com/hoek/pdf/11_Rock_mass_properties.pdf

3) AASHTO 2008: Section 10.4.6.5 Rock Mass Deformation, Equation 10.4.6.5-1.

Figure 1.— Available Point Load Test (PLT) results for the Snowshed. WSDOT (2009) UCS test results are also graphed. Note that borings H-02-07 is SSD-002-07, and H-03-07 is SSD-003-07.

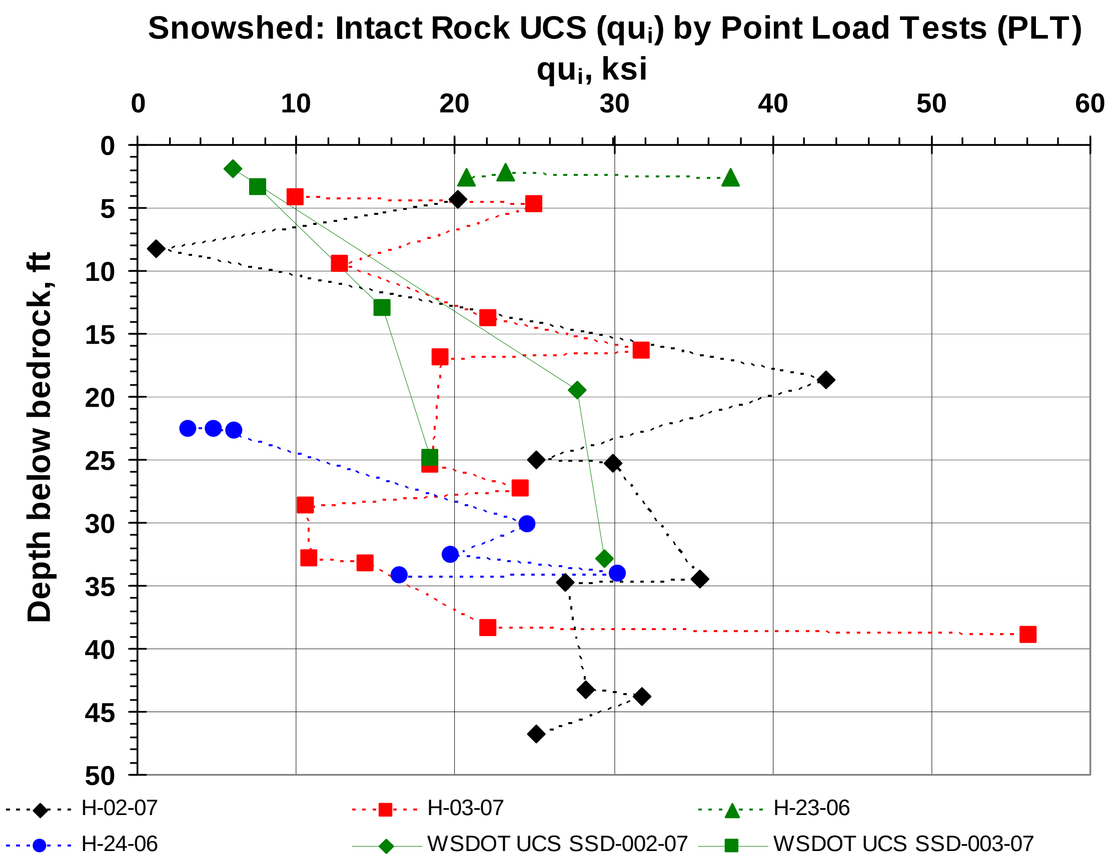


Figure 2.— Available PLT results and boring RQDs for the Snowshed. Note that borings H-02-07 is SSD-002-07, and H-03-07 is SSD-003-07.

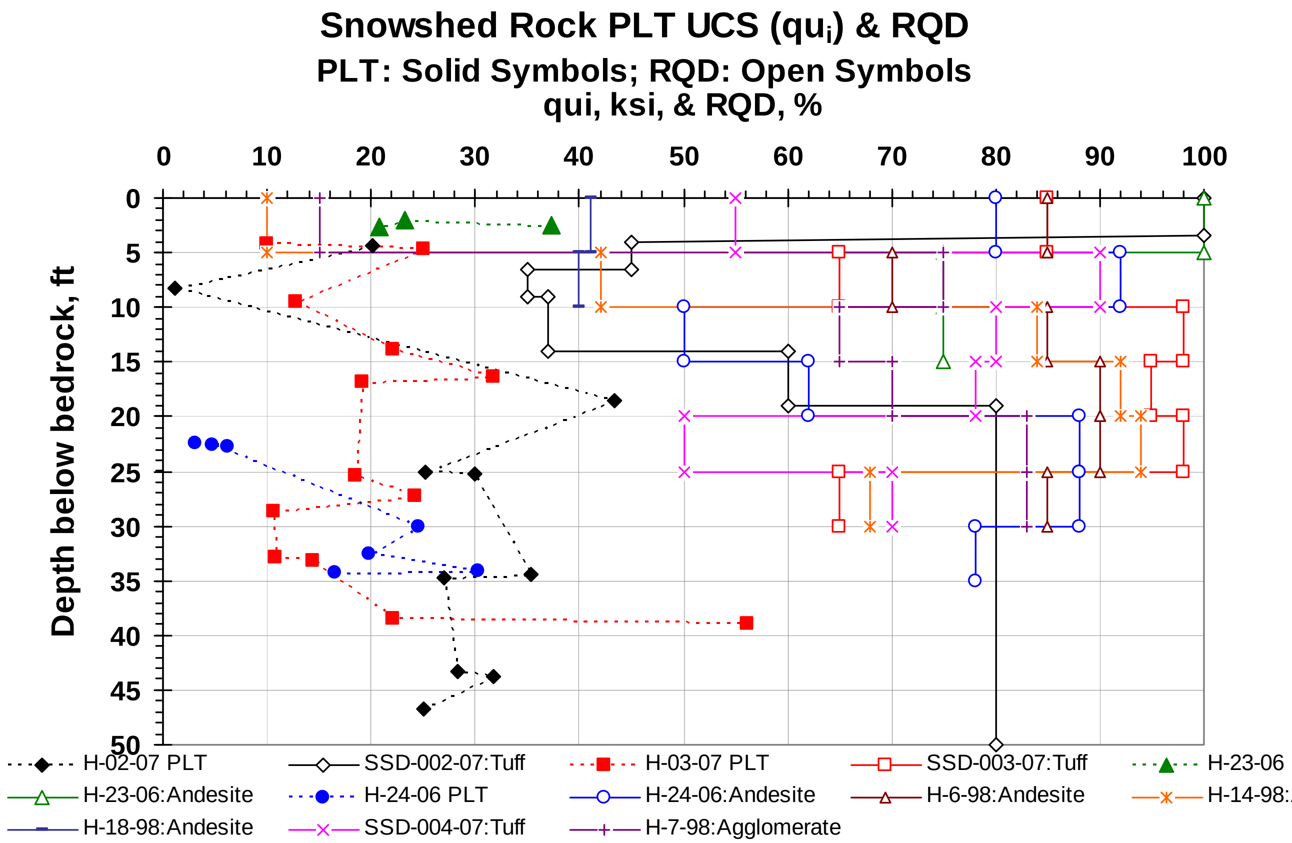
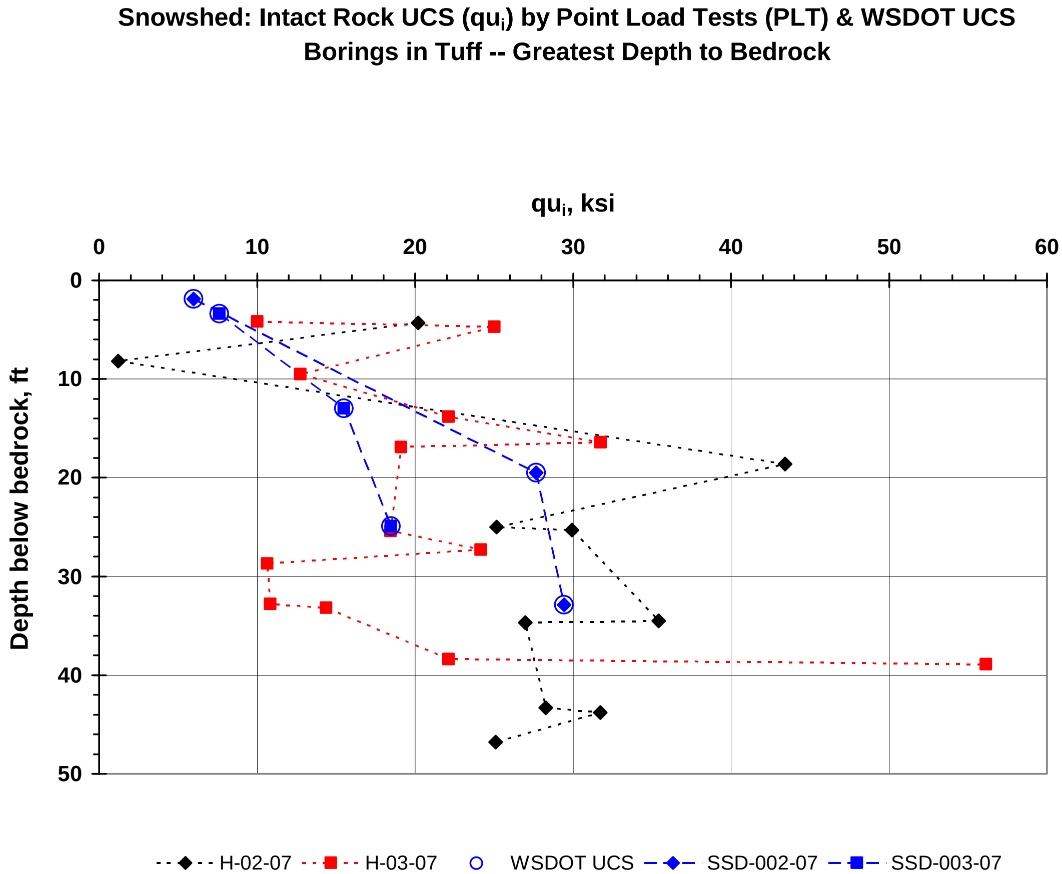


Figure 3.— Available PLT and WSDOT (2009) UCS test results for the two borings in Tuff. Note that borings H-02-07 is SSD-002-07, and H-03-07 is SSD-003-07.



APPENDIX E.3
W&N Memorandum dated June 18, 2010

Memorandum

To: Chuck Vita, P.E. (URS Corporation)

From: Norman I. Norrish, P.E. (Wyllie & Norrish Rock Engineers Inc.)

Date: June 18, 2010

Re: Bedrock Global Stability Evaluation
I-90 Snoqualmie Pass East Snowshed

Background

As required under Work Order number 33758662.03000 Task DO, Wyllie & Norrish Rock Engineers Inc. (W&N) is pleased to present this technical memorandum with the results of our stability evaluation for the proposed snowshed (SSD) located within Phase 1C of the I-90 Snoqualmie Pass East Project. The specific scope for W&N as quoted from the Task Order was to "Evaluate the potential for adverse bedrock structure that could affect the global stability of the snowshed and provide findings in writing".

The main reference documents for this study are:

- Memorandum from T.M. Allen and T.C. Badger to S. Golbek dated April 8, 2009 and titled "I-90 Snoqualmie Pass East – Hyak to Keechelus Dam Snowshed Shaft Foundations". This memorandum will be referred to as **WSDOT**, **2009**.

- Memorandum from Chuck Vita and Cecil Urlich (URS) to Tony Allen and Tom Badger (WSDOT) dated April 29, 2009 and titled "Snowshed Shaft Foundations: Rock Strength Estimates for DFSAP Analysis – I-90 Snoqualmie Pass East". This memorandum will be referred to as **URS, 2009**.

- Memorandum by M. Molinari and M. McCabe titled "I-90 Snoqualmie Pass East Project, Slide Curve Bridge and Walls, Geologic and Geotechnical Interpretation of Significant Discontinuities" dated February 5, 2010. This will be referred to herein as **URS (2010a)**.

- Various data packages containing borehole logs, core photographs, plans, profiles and sections, and COBL televiewer logs transmitted via emails from URS 7/15/09, 04/06/10, and 05/28/10. This information will be collectively referenced as **URS, 2010b**

- The Wyllie & Norrish final design report for the new I-90 alignment dated April, and titled "Phase 1C – Rock Slope Engineering Report, 2008 Geotechnical Program". The report will be referred to as **W&N, 2009.**

Methodology

Structural Analysis:

Rock slope stability evaluations are a two-part process. The initial step, and the subject of this study, is to analyze the structural geology to determine if geologic discontinuities (joints, faults, flow boundaries etc) are adversely oriented such that structure-bound blocks can displace with respect to the orientation of a slope under consideration. This is often referred to as a kinematic analysis. The discontinuities of most concern are those with the greatest persistence (i.e. continuity) and the lowest shear strength (i.e. clay-filled faults or flow boundaries).

A structural analysis is performed by means of a graphical presentation of discontinuity measurements referred to as a stereonet. In essence, stereographic projection is a technique to visualize and analyze three-dimensional data in a two-dimensional representation. This is useful because kinematic analyses are concerned with the spatial relationships between planes representing discontinuities and planes representing slope surfaces. On a stereonet planes can be represented either as great circles or as poles. Poles represent the normals to planes and plot as points on a stereonet. The degree to which these poles plot in tight groups or clusters, the more ordered or systematic is the structural fabric. These groups are referred to as "sets". Highly ordered structural fabric is conducive to kinematic analysis because it is predictable. In contrast, the stereonet for a random structural fabric devoid of preferred set orientations has a "shotgun" appearance and is difficult to adapt to kinematic analysis.

Available Structural Data:

Previous drilling at the SSD has been performed over multiple exploration campaigns by WSDOT and by URS in 2007, 2008 & 2009. Core logging generally recorded rock parameters such as core recovery, Rock Quality Designation (RQD), qualitative rock strength, fracture frequency along with geologic descriptions. Based upon the logging, URS (2009, 2010b) reported general rock mass characteristics with GSI values from 50 to 65, but locally as low as 40. Also noteworthy at the SSD is the apparent lack of clay-filled discontinuities as contrasted to Slide Curve Bridge (URS, 2010a).

The drilling of greatest relevance to the current study was a series of seven 2009 boreholes, designated SSD-006-09 to SSD-012-09 inclusive, in which in situ structural information was obtained using downhole optical and acoustical imaging. Approximately 83% of the 212 feet of bedrock drilling was successfully logged using televiewer techniques. After editing to remove duplicate orientations from the dual logging methods, a total data set of some 237 discontinuity measurements was developed. Note that the genetic types of discontinuities, such as joints or bedding planes, are not readily discriminated in the downhole logging techniques.

The stereonet in Figure 1 summarizes the data set for the seven boreholes. Poles have been plotted and contoured to accentuate preferred orientations using Rocscience software "DIPS", Version 5.108. Since the seven boreholes were all drilled vertically, the raw data set has a tendency to under-represent steeply dipping discontinuities. To offset this bias, the stereonet was modified using a standard Terzaghi correction as supplied with the software. Figure 1 indicates three discontinuity sets similar in general orientation to results developed for the rock slope stability studies for the roadway alignment (W&N, 2009). For the SSD data these sets are tabulated as follows:

Set	Dip	Dip / Direction
1	40°	226°
1a	40°	268°
2	72°	053°
3	56°	310°

Sets 1 and 1a are inferred to be variants within a common set. The pole and the great circle representations of each mean set orientation are plotted in red on Figure 1. Sets 1, 1a and 3 have westerly dip directions while Set 2 dips to the east. Furthermore, Sets 1 and 1a have strike directions that are parallel to Set 2 but with opposite dip directions.

For comparison, the stereonet for the highway alignment (Design Sector VIII) is shown in Figure 2. The notable difference is the absence of Set 4 in the downhole logging for the snowshed. The other three sets are present but with slight orientation differences.

In order to analyze for potential instability, the orientation of the bedrock surface must be determined. This was accomplished by measuring the inclination of the inferred bedrock surface at seven cross sections developed by URS (2010b). The corresponding dip direction for each inclination was determined from the contour at 2450 feet MSL. It is recognized that this contour may not mirror the underlying bedrock surface, but for the purposes of this analysis the similarity should be adequate. The resultant bedrock orientation data is summarized in the following table:

Station (EB)	Inclination (deg)	Dip Direction
1351+45	13	225
1353+03	35	249
1354+92	41	238
1357+46	35 (overall) 51 (local)	225
1358+78	41	275
1361+22	39	252
1362+01	34	260

For design purposes, a maximum inclination of 40° and dip direction range of 225° to 275° were adopted for overall slopes. The great circle representations of these two limiting orientations are plotted in green on Figure 1.

<u>Salient Observations with Respect to Snowshed Bedrock Stability</u>

The important observation from the kinematic analysis of Figure 1 is that <u>none of the defined discontinuity sets are oriented so as to give potential to **planar or wedge** type failures for the orientations of the overall bedrock surface.</u> Furthermore, Sets 1 and 1a are coincident with the range limits for the bedrock surface as demonstrated by the similarity between the respective red and green great circles. This means that there is a high probability that at least portions of these bedrock surfaces are dip slopes; that is slopes formed by discontinuity surfaces.

The exception to the above assertion concerning **planar** failure relates to the localized 51° bedrock inclination interpreted at EB Station 1357+46. At this location, Set 1 with a dip of 40°could define a thin slab that daylights on the 51° bedrock surface (Figure 3). A stability assessment of this slab includes consideration of the following:

- The 51° bedrock inclination is an inferred orientation lacking corroboration with site specific exploration. The steep local slope may not be present.
- Stable slopes steeper than 51° are inferred upslope of the SSD and forming the cut slope behind the wall (see Pier 1 area Figure 3).

Given the uncertainty, a possible course of action is to probe the top-of-bedrock during construction of the temporary soil nail wall or the tiebacks for the shafts. If steep bedrock dips are confirmed, an evaluation of the need for passive localized rock reinforcement through the Pier 1 foundation could be performed at that time.

Set 2 dips steeply to the east with an orientation that could generate block **toppling**. Given the diminutive character of Set 2 it is inferred that the discontinuities are neither closely spaced nor highly persistent thereby minimizing the potential for this failure mode on the natural bedrock slopes proximal to the foundation. If steep temporary rock cuts are required during construction, localized reinforcement to preclude rockfall from the Set 2 features may be required.

A potential hybrid failure mechanism that is consistent with the structural geology is referred to as the "**bi-planar**" model and is illustrated in the conceptual sketch shown on Figure 4. This mechanism is also referred to as the active-passive block model. Bi-planar failures require highly specific structural orientations and are relatively uncommon. The most favorable geologic regime is that of dip slopes formed by bedding in sedimentary rock types. The essential structural features for this mechanism are three-fold and include:

A. Discontinuities parallel to a dip slope.
B. A cross-over discontinuity at or near the toe that forms the potential surface of movement for the passive block.
C. Steeply dipping discontinuities at the transition from the active to passive blocks to facilitate differential movement between the blocks.

The correlation between these three prerequisites and the current structural model is shown in the lower part of Figure 4. Requisite "A" is fulfilled by Sets 1 and 1a. Requisite "C" is fulfilled by Set 2. However, there are very few televiewer-logged features that fulfill the requisite "B" cross-over feature.

For the volcanic bedrock slopes at the SSD, the following observations are made respect to the potential for bi-planar failures:

- Sets 1 and 1a are inferred to be joints of limited persistence based on studies for the roadway alignment. This means that shear strength for a large scale requisite "A" surface would comprise both joint strength and rock mass strength. The resultant friction and cohesion for the surface would cause the slab resting on the 40° surface to be stable.

- The cross-over feature does not correspond to a concentration of measured discontinuities (yellow shaded area on stereonet). Therefore failure would require shearing through intact rock at the toe. The low shear stresses for slopes of this magnitude would be insufficient to overcome the shear strength of the rock mass.

It is concluded that the bi-planar is not viable for the SSD foundation slopes and that additional stability analyses to determine safety margins are not warranted.

A final failure mechanism for consideration is the classical **slip circle** employed for soil slopes. In the case of rock slopes, the favored structural conditions for this mechanism are extremely closely fractured, weak rock masses such as found in tectonic regimes or highly weathered rock masses. This is not the situation for the subject slopes as reported by WSDOT(2009) and URS (2009) wherein unconfined compressive strengths range from 10 to 30 ksi and GSI values from 50 to 65 (but locally as low as 40). Application of the Hoek-Brown Failure Criterion to such values would predict rock mass friction angles greater than 50° and cohesion values greater than 10,000 psf, sufficiently high to preclude the circular failure mechanism for the moderately inclined natural bedrock slopes.

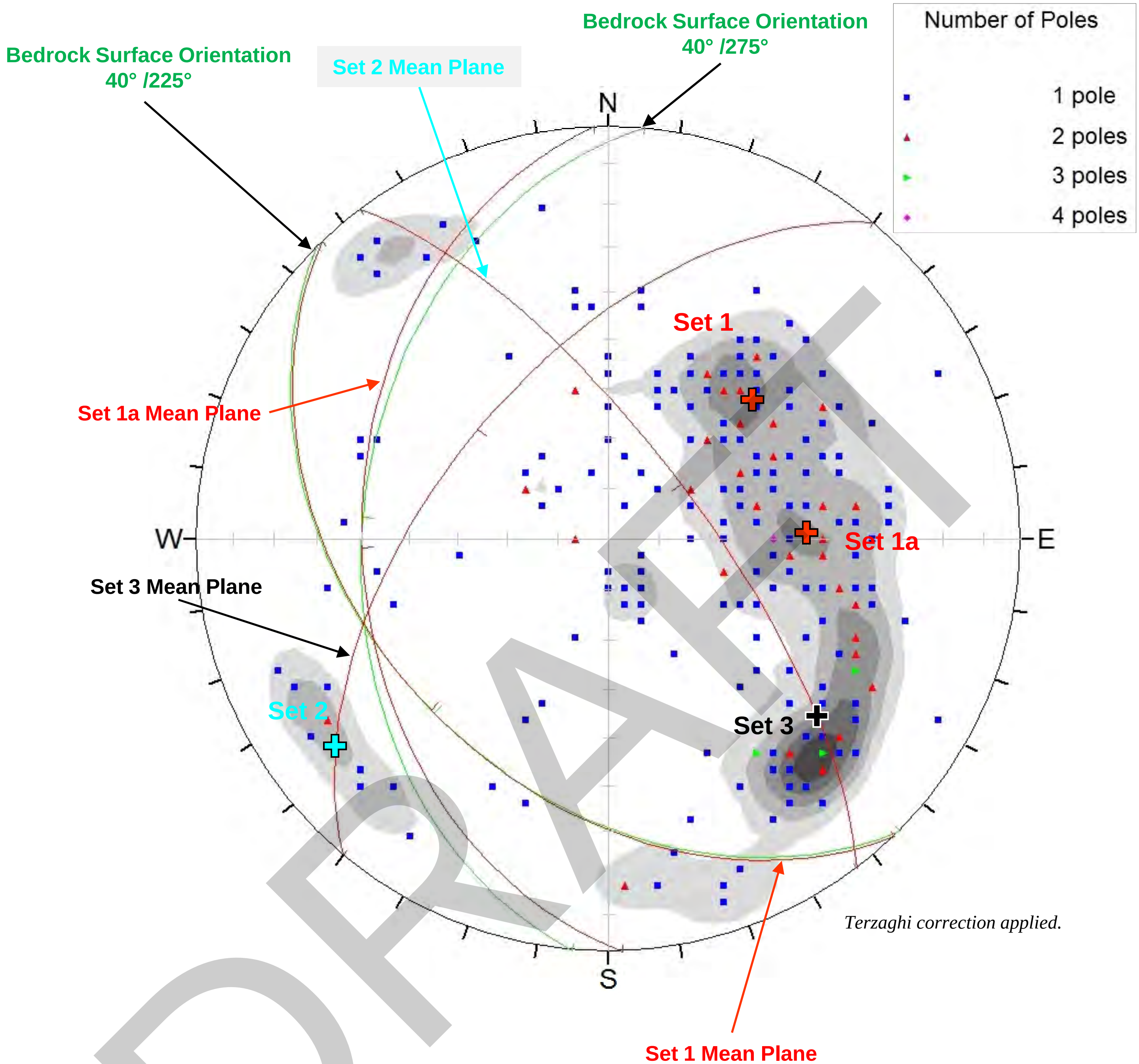
Summary

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

Potential for localized planar failure was identified at EB 1357+46 due to an inferred steep bedrock surface down slope of Pier 1. Construction observation should be utilized at this location and as a general practice for the entire SSD. Specific recommendations are:

1. Found the entire width of the Pier 1 foundation on bedrock.
2. Correlate the top-of-rock between the Pier 1 excavation and the drill holes for the tiebacks at the shaft and/or for the soil nail wall. If bedrock surfaces steeper than 40° are confirmed, passive reinforcement through the base of Pier 1 should be implemented.
3. If the bedrock surface is steeper than 40°, locate the entire bond zone for the tiebacks at Pier 2 beyond an assumed +40° plane extending upslope from the shaft centerline.

The conclusions herein are based on the assumption that the structural fabric reported by the televiewer logging is representative of "average" bedrock conditions. There is always the possibility, especially in volcanic regimes, that a rogue surface, as yet unidentified, could control bedrock stability. Construction observation and monitoring should be established for such contingency.



Mean Set Orientations			
Set	Pole Symbol	Dip	Dip Dir
1	✚	40º	226º
1a	✚	40º	268º
2	✚	72º	053º
3	✚	56º	310º

Equal Area
Lower Hemisphere
237 Poles
237 Entries

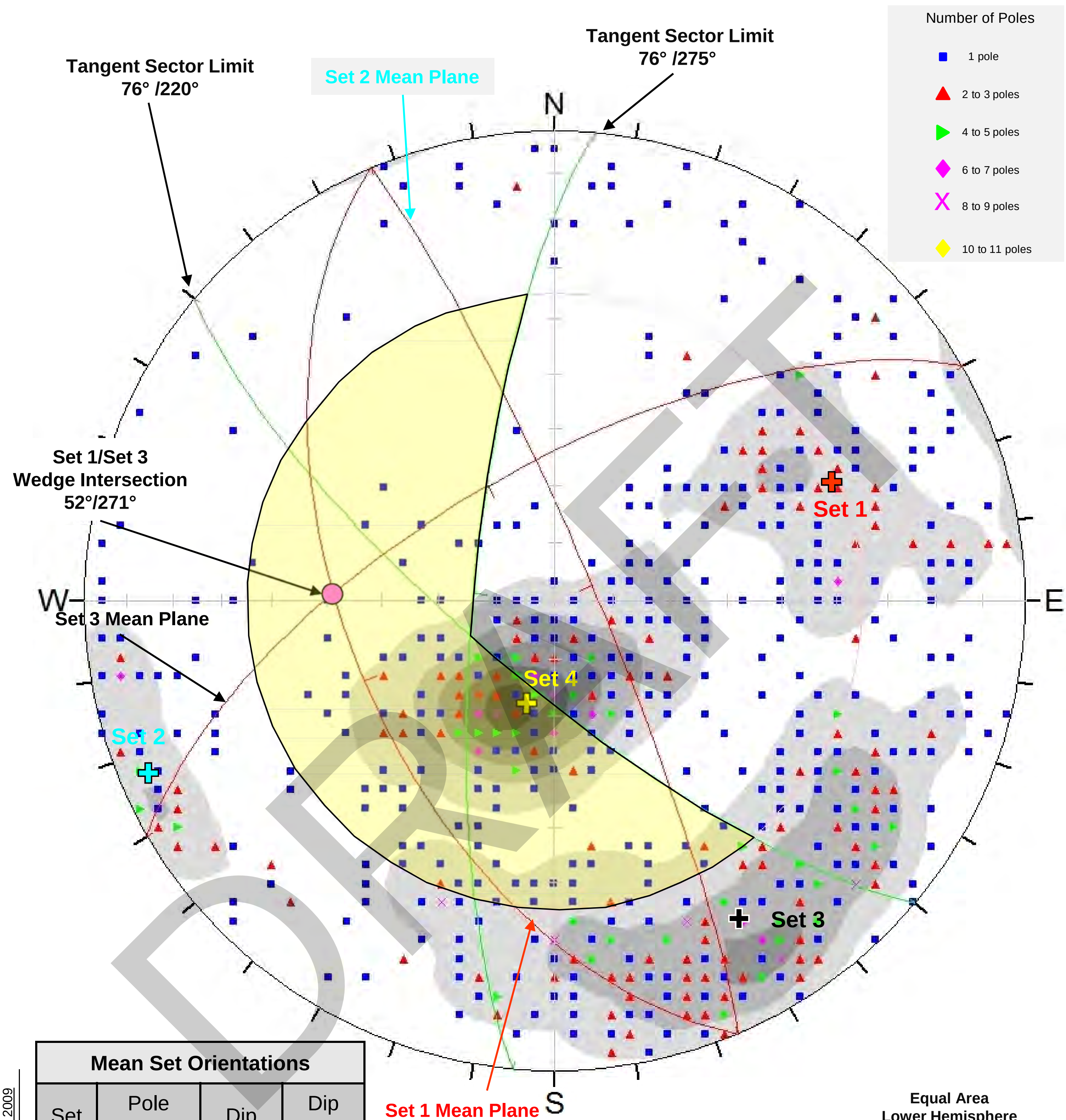
Project No. 062-2002 Date: May , 2010

Figure 1

Snowshed Rock Slope Kinematic Analysis

wylie & norrish ROCK ENGINEERS

I-90 Snoqualmie Pass East
Snowshed



Equal Area
Lower Hemisphere
777 Poles

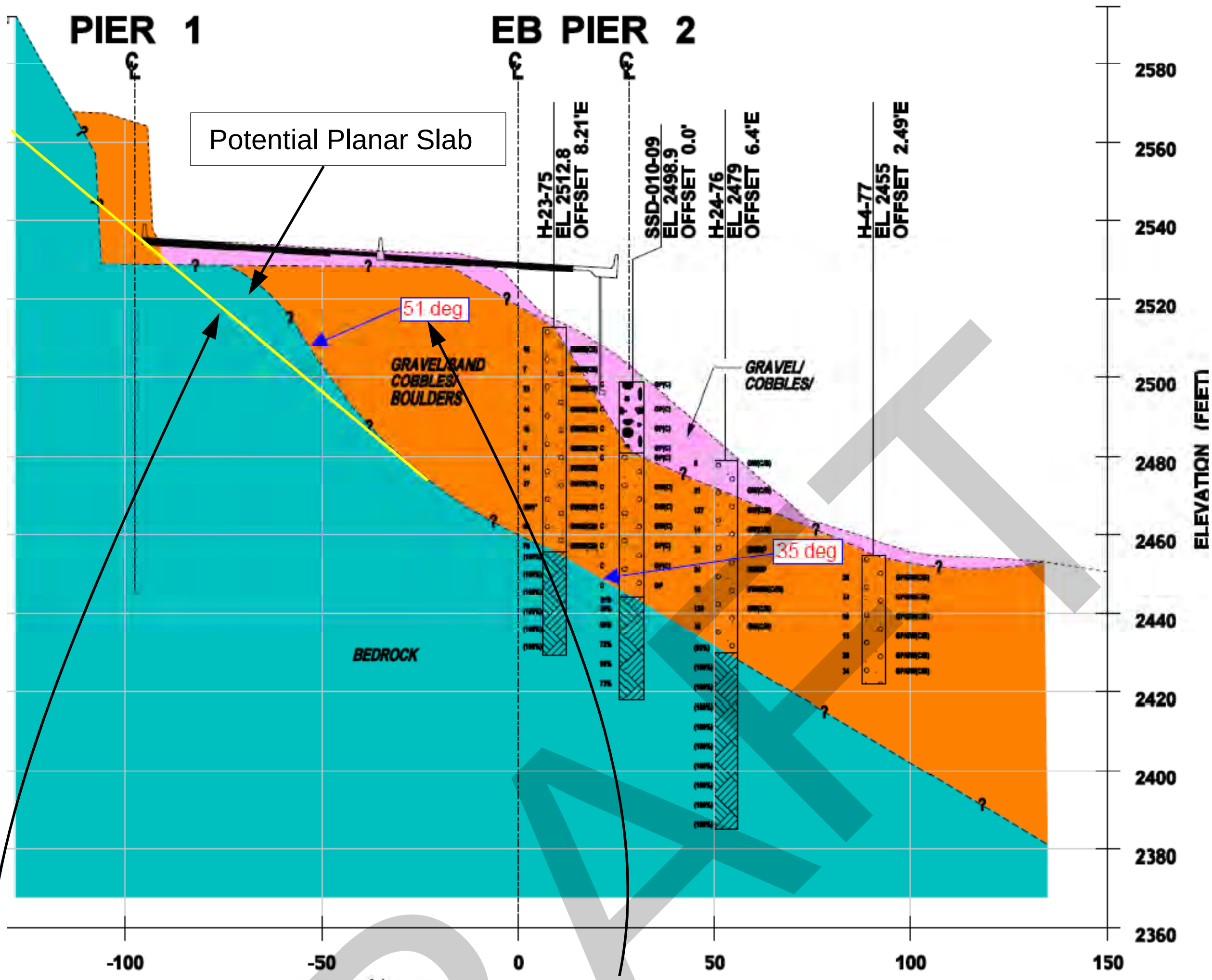
Mean Set Orientations			
Set	Pole Symbol	Dip	Dip Dir
1	✚	54°	247°
2	✚	83°	067°
3	✚	67°	330°
4	✚	18°	014°

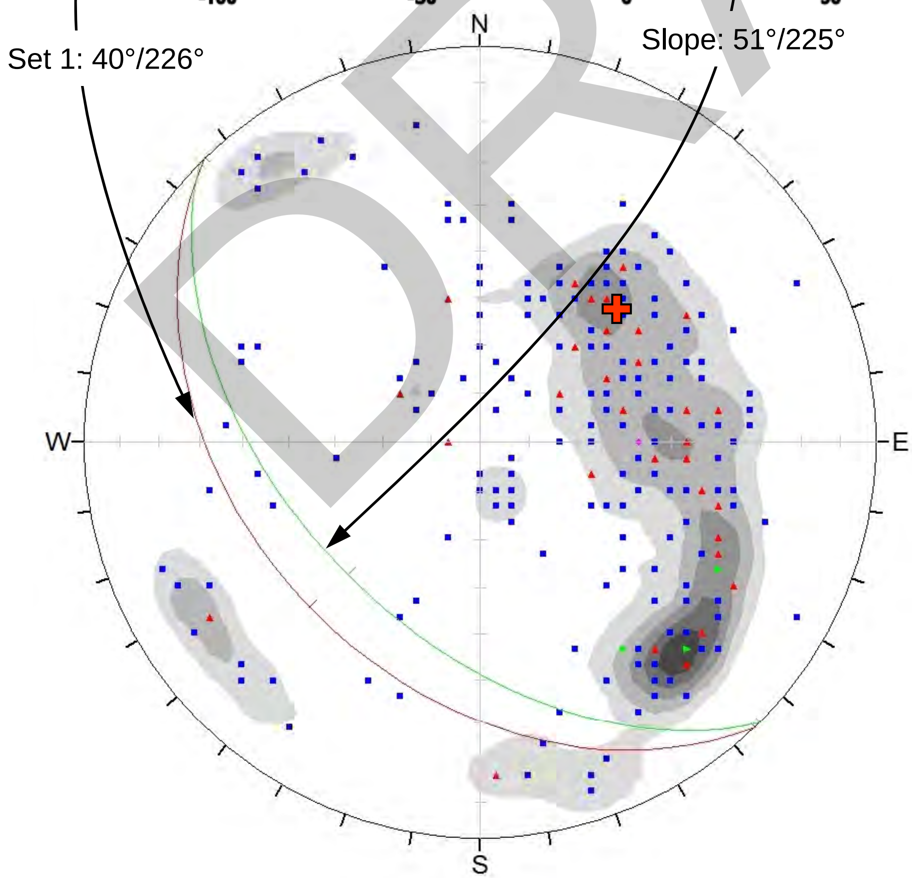
Project No. 062-2002 Date: February, 2009

Figure 2

Design Sector VIII (Snowshed) – Detailed Kinematic Analysis

I-90 Snoqualmie Pass East
Hyak to Keechelus Dam

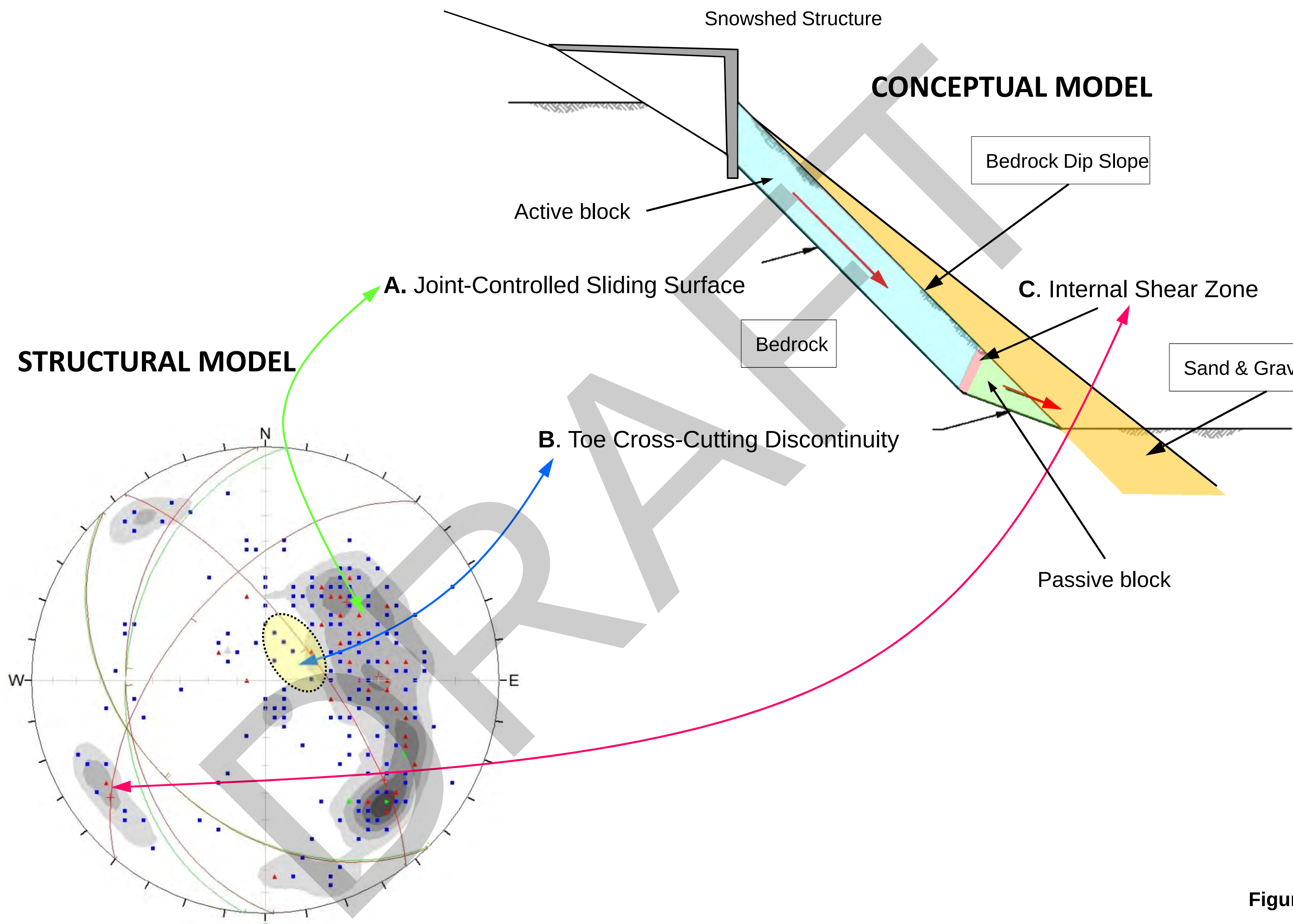




Stereonet shows only Set 1 and local bedrock orientation.

Figure 3

EB Station 1357+46 Detail



Stereonet from COBL Data (see Figure 1)

Project No. 062-2002 Date: May 2010

Figure 4

Bi-Planar Dip Slope Failure Mechanism

wyllie & norrish ROCK ENGINEERS

I-90 Snoqualmie Pass East Snowshed

APPENDIX E.4
URS Memorandum Dated February 5, 2010

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Technical Memorandum

To: File **Date:** February 5, 2010

From: Martin McCabe, Mark Molinari

cc: John Zeman

Subject: I-90 Snoqualmie Pass East Project
Slide Curve Bridge and Walls
Geologic and Geotechnical Interpretation of Significant Discontinuities

Introduction

The purpose of this technical memorandum is to provide a geologic and geotechnical engineering basis of design with respect to bedrock discontinuities for the structures within the Slide Curve Bridge and Walls project. The geologic and geotechnical engineering interpretation consisted of investigating and analyzing the nature and continuity of potentially significant soil-filled discontinuities encountered in the bedrock and the impact of that these discontinuities would have on design and construction of the main bridge, half-bridge,, tied shaft wall, and possibly, the soil nail wall.

Background

The presence of a three-dimensional combination of unfavorably oriented, potentially significant discontinuities, may result in a portion of the sloping rock foundation sliding into the lake under static and/or seismic conditions and thus compromising the bridge foundation. A discontinuity is defined as a planar surface between two rock masses. Discontinuities may be of limited extent or persistent, smooth or rough, open or closed, and may be clean or have mineral coatings or infilling of soil or rock fragments. Potentially significant discontinuities are those identified in rock core recovered during drilling and the COBL logs, or in rock outcrops, that may represent persistent, low strength and unfavorably-oriented weakness planes that may impact the stability of rock supporting the foundation for the planned bridge.

A plan view of the Slide Curve Bridge area is shown on Figure 1 and includes all the known boring locations within this area. A three-dimensional illustration of the overburden and bedrock at Slide Curve Bridge is shown on Figure 2. The potential adverse effect of the discontinuities in question was first considered when a discontinuity with 0.8 feet of soil infilling was encountered in Boring SCB-022-09 (west end of the bridge). Additional discontinuities were also encountered at the east end of the bridge in Boring SCB-021-08 and SCB-006-8, where the soil infilling was 1 and 0.2 to 0.4 feet, respectively. The presence of these potentially significant discontinuities warranted further drilling at the west end of the bridge (SCB-026-09 and SCB-027-09) and the east end of the bridge (SCB-028-09, SCB-029-09, and SCB-030-09) in October 2009. Borings SCB-028-09, SCB-029-09 and SCB-030-09 at the east end of the bridge all encountered “wide” (equal to or greater than 0.5 inches thick) potentially significant discontinuities.

It was concluded that due to the limited discontinuities at the west end of the bridge, additional evaluation was not warranted. However, due to the potentially significant discontinuities at the east end of the bridge, a more detailed investigation and analysis was performed and is presented in the following sections.

East Slide Curve Bridge

The east end of Slide Curve Bridge is located below “Design Sector XII” for the rock slopes above the highway, where the nearest borings are RKS-039-08, RKS-040-08, RKS-013-07 (refer to Phase 1C -

Rock Slope Engineering Report dated April 2009). Estimates of the subsurface profile for locations at the east end of the bridge have indicated that the top of rock surface slopes downward toward the lake at approximately 25 to 30 degrees, based on most logs of borings and results of geophysical testing performed perpendicular and parallel to the slope face. A slightly more complex rock surface geometry is suggested at the profile through borings SCB-021-08, SCB-020-09 and SCB-028-09 (see Figure 3), where rock on the upper part of the slopes appears to be inclined at roughly 50 degrees, while rock on the lower part of the slope below boring SCB-028-09 is inclined at the more typical 30 degrees. Overall, the rock surface slopes to the west towards the lake with an inclination azimuth of approximately 260 to 280 degrees, and is overlain by roughly 15 to 40 feet of colluvium and rockfill.

Some uncertainty exists regarding the steep upper rock inclination between Boring SCB-021-08 and SCB-020-09 due to the significant "no-recovery" zone in Boring SCB-020-09 between the depths of 18 to 43 feet below the ground surface. The most likely explanation for the "no-recovery" zone was the presence of loose or fine overburden soil washed out by the coring process, or highly weathered and incompetent rock. Boring SCB-028-09 was drilled near and downslope of the location of Boring SCB-020-09. Bedrock was encountered at a shallower elevation at boring SCB-028-09 than at SCB-20-09; thus the rock surface slopes easterly between these two borings (Figure 3).

Evaluation of Discontinuities

The evaluation of the discontinuities at the east end of Slide Curve Bridge were further evaluated as follows:

1. <u>Identification of Potentially Significant (Wide) Discontinuities</u>
   A listing of the potentially significant discontinuities for all borings at the east end of the bridge are shown in Table 1. The nine most recent borings included two on the upslope side of the bridge (SCB-011-08 and SCB-021-08) and seven on the downslope side (SCB-006-08, SCB-010-08, SCB-020-09, SCB-024-09, SCB-028-09, SCB-029-09 and SCB-030-09), as shown on Figure 1.

   Boring logs of two older upslope borings (SW1-005-07 and H-26-06) were also reviewed, but did not provide as much useful information. Four of the nine borings (SCB-010-08, SCB-011-08, SCB-020-09, SCB-024-09) showed no potentially significant discontinuities, while the remaining five borings (SCB-006-08, SCB-021-08, SCB-028-09, SCB-029-09 and SCB-030-09) showed from 1 to 10 potentially significant discontinuities in each, for a total of 30 potentially significant discontinuities. For reference, in borings where Crux Oriented Borehole Logs (COBL) data was obtained (SCB-024-09, SCB-028-09, SCB-029-09 and SCB-030-09), typically 40 to 70 or more planar features were identified in the optical trace of 30 to 50 feet of rock core for each boring (2 to 3 fractures per foot). Most of these are either rock fabric features (e.g., foliation) or are not apparent in the recovered core. The potentially significant discontinuities therefore represent from less than 5 percent to up to 20 percent of the total number of potential discontinuities identified in the COBL interpretation of any one boring, and less than roughly 10 percent of the total number of joints in the group of borings at the east end of the bridge.

   The widest discontinuity encountered was the 1-foot wide feature in upslope Boring SCB-021-08, occurring at a depth of about 43 feet below the top of rock. In general, the wider filled joints could occur at any depth, and were not concentrated at any particular depth.

2. <u>Discontinuity Orientation, Persistence and Properties</u>
   The most important discontinuity factors are orientation (dip and dip direction) and persistence (length), which will define whether the structure foundation is controlled by an unfavorably oriented single plane or a series of planes of weakness.

   Visual Examination of Photos and Logs
   The boring logs, COBL logs, and photographs of the rock core from the borings in the area of the west and east end of the proposed bridge were reviewed to assess whether there was visual or other qualitative evidence that the potentially significant soil-filled discontinuities observed in the prior

borings SCB-021-08 and SCB-022-09 were apparent in the more recent borings SCB-026-09 through SCB-030-09.

COBL logs are available for SCB-024-09 as well as SCB-026-09 through SCB-030-09. The discontinuity values recorded on the COBL logs were reviewed and an interpretation was made regarding the correlation between these values and the potentially significant discontinuities observed on the photographs of the core. Photographs of the core in the split tube core barrel (SCB-026-09 through SCB-030-09) were used instead of the photographs of the core in the core boxes because the core was more intact.

The depths recorded for potentially significant discontinuities by the COBL and on the boring logs can be typically correlated to within about 0.5 foot, but in some cases appeared to deviate up to about 1 foot. Therefore, a definitive direct correlation between the COBL discontinuity values and potentially significant discontinuities observed in the core photographs could not always be made.

It should be noted that because COBL measurements were not made in Boring SCB-021-08, there was no formal quantitative identification of the dip direction and dip angle for the large 1-foot wide discontinuity. However, examination of the recovered rock core on either side of the soil filling suggests that this discontinuity has a low angle dip of approximately 10 degrees. For conservatism, the dip azimuth of this discontinuity was assumed to be downslope to the west (i.e. 275 degrees). A check on the depth of potentially significant discontinuities in the borings directly downslope from SCB-021-08 (i.e., Borings SCB-020-09 and SCB-028-09), including a review of photographs of the core box and split tube core, did not identify a potentially significant discontinuity with infill material similar to that in SC-021-08. In addition, the following conclusions were developed:

- Potentially significant discontinuities were not present (e.g. SCB-020-09)
- Potentially significant discontinuities dipped into the slope
- Potentially significant discontinuities were sufficiently deep that if the discontinuities were connected, the weakness plane would dip so steeply that it could not daylight on the lower rock slope face.

A qualitative evaluation of the potential strike continuity and persistence of potentially significant discontinuities between the borings was performed in the east foundation area. The evaluation was based on the stereographic projection discussed below and cross-sections developed to assess dip continuity downslope where two or more borings are roughly aligned down the slope (e.g. SCB-020-09, SCB-021-08, and SCB-028-09),

The average orientation of the rock slope below the proposed bridge location is approximately N12^{o}E-S12^{o}W (i.e. 12 to 192 degrees), and the typical slope inclination is 25 to 30 degrees to the west. Therefore, only discontinuities with dip azimuths to the west and dips of 45 degrees or less were considered to be a potential concern.

A cross-section was constructed along the slope (oriented 172 – 352 degrees) in the east foundation area that includes borings SCB-024-09, SCB-028-09, SCB-029-09, and SCB-030-09. The potentially unfavorable significant discontinuities on the COBL logs were plotted with their apparent dip in the line of section to assess whether one or more could potentially be correlated along strike. While one or more discontinuities may be correlative between any two adjacent borings, the available data do not provide evidence of a persistent discontinuity throughout the east foundation area.

Visual assessment of the core photos and physical inspection of the core indicates that the uppermost approximately 5 feet of rock is typically more weathered and of poorer quality than the underlying rock. In addition, there are several 1-foot thick zones of rock with more frequent discontinuities in the borings. Within these zones, there are typically a variety of planar orientation based on the COBL values or the predominant discontinuity planes dip easterly into the slope; therefore, these zones of discontinuities do have not a potential for planar or wedge failure.

Stereographic Projections
Equal angle stereographic projections (stereonets) were created using the software package DIPS V-5.0 from Rocscience. Stereonets were created for only the potentially significant discontinuities (23 entries with sufficient information), and also for all discontinuities identified in the COBL data for borings at the east end of the bridge (286 entries). The Terzaghi (1965) correction for sampling bias in vertical boreholes was not incorporated in the stereonets.

Many of the “discontinuities” mapped on the COBL images are either not an actual planar feature or rock fabric features and are not very apparent in the core box or sample tube core photographs. Others appear to be only minor features that are not potentially significant discontinuities. Therefore a further examination of the COBL “discontinuities” was made for Borings SCB-029-09 and SCB-030-09 where photos had been taken of the core while still in the inner tube of the core barrel. Many of the COBL discontinuities apparent in the photographs primarily consisted of color changes, textural changes, healed joints, or other features and were interpreted to be rock fabric features, not actual physical planar separations in the rock mass; therefore, these were not included in a modified discontinuity data set. Based on this procedure, 50 to 60 percent of the COBL discontinuities were eliminated. A separate stereonet was produced from the remaining discontinuities.

The resulting stereonet from the 23 filtered data set was not markedly different from the unfiltered 286 data set. The “great circles,” representing the faces of the slope (general case of 30 degree inclination and a special case of a 50 degree upper slope inclination), were also plotted in the stereonets to verify whether any of the joint “sets” or wedges created by joint planes actually day-lighted on the slope face. The Terzaghi correction would add 3 to 4 additional discontinuities to the 23 discontinuity data set for the typical 30 degree rock slope inclination in the east end area.

The stereonet for the 23 potentially significant discontinuities with a filling thickness of at least 1-inch was used to select six possible “joint sets” (Figure 4), based on a weak concentration of poles, with no joint set having more than 2 to 3 poles each. This stereonet has only a few minor similarities with the stereonet developed for the nearby Design Sector XII slopes, as shown in Phase 1C Rock Slope Engineering Report of April 2009. Only one joint plane (Joint Set 2m) day-lighted on the 30-degree slope face, but at an angle of only about 11 degrees. A possible wedge formed by two planes appears to dip at about the same angle as the slope inclination. However, the “planes” forming the wedge are not strongly indicated in the data for the potentially significant discontinuities or in data for all discontinuities in the east end borings. For the special case of a portion of the upper slope inclining at 50 degrees, Joint Set 1m inclining at 44 degrees in the direction 282 degrees could daylight. Even if this steeper joint set did not daylight, its position sub-parallel to the rock surface could adversely influence the stability of the foundation.

The stereonet for all discontinuities had only very broad concentrations of poles, the most significant of which suggested a low angle plane dipping into the slope (Figure 5). This stereonet had only slight resemblance to stereonet created for the nearby rock slope above the roadway in Design Sector XII. The most significant similarity was that Joint Set 4 (dip 15 degrees, dip direction 134 degrees) for the rock slopes above the highway and Joint Set 1m (dip 16 degrees, dip direction 140 degrees) for the east end borings matched reasonably well, although this joint set does not represent an adverse condition for the bridge foundation slope.

Joint set 1a (dip 28 degrees, dip direction 270 degrees) for the rock slope above the highway and Joint Set 4m for the east end borings both dip downslope, but the difference in dip angle is substantial (28 degrees versus 44 degrees). The lack of stereonet similarity is not wholly unexpected since the two rock masses (i.e., above the highway versus below the highway), appear to be different flows. A flow boundary was noted in the slope face near the highway level, and the upper flow appears to have pumice fragments and iron staining that are not prevalent in the Slide Curve Bridge borings.

Features common to both Figures 4 and 5 are the low angle joint set (dip angle 10 to 11 degrees, dip direction 288 to 298 degrees) and the steeper joint set (dip angle 44 degrees, dip direction 271 to 288 degrees) that both dip roughly parallel to the slope direction. These should be considered during the analysis of the Slide Curve Bridge project structure foundations.

At present there is no firm information about the persistence of potentially significant discontinuities with apertures greater than 1.0 inch. The Phase 1C – Rock Slope Engineering Report stated that on the slope above the roadway, many joints are persistent and without significant filling material, while others are filled with up to 3-inches of clay.

3. Nature, Origin and Thickness of Soil Filling of Discontinuities
The soil fill in the potentially significant discontinuities has been identified as ranging from soft clay to silty sand based on visual inspection. The most common description for the soil fill was "silty sand," used for 16 of the total 30 discontinuities. "Clay" was used to describe fill in six of the discontinuities, with the remaining were described as silt or sandy silt. Laboratory testing of two samples of the silty sand infilling in Boring SCB-028-09 indicated these soils actually had 42 to 50 percent fine gravel content. Clay infilling from Boring SCB-022-09 at the west end of the bridge was found to have low plasticity (PI = 7 to 9 percent), with peak and residual friction angles of 28 degrees and 24 degrees, respectively, based on laboratory testing.

The thick 12-inch filling in Boring SCB-021-08 is interpreted to most likely be a flow boundary similar to one identified in a roadway cut slope and in Boring RKS-39-08, which is further upslope. It appears unlikely that this thick infilling is fault gouge because: there is no shearing; larger fragments of rock were observed within the infilled soil; and, the discontinuity boundaries do not appear to be altered, substantially weathered or discolored. The material could represent a later ash flow settling into irregularities at the top of a flow boundary. If a more definitive determination of the origin of this infill material is desired, then a mineralogical analysis of the material is recommended.

4. Shear Strength of Discontinuities
The presence of potentially significant discontinuities should be incorporated into the stability analysis of the bridge and other major structures. Even if the potentially significant discontinuities cannot be shown to be persistent, their contribution to the overall rock mass strength could be assessed by estimating the shear strength parameters of the discontinuities, and estimating an anisotropic mass rock strength using proportions of discontinuity and rock mass strength values that are appropriate to the anticipated orientation of the failure plane.

The shear strength of the potentially significant discontinuities can be estimated using the method of Barton and Bandis (1990), which utilizes the base friction angle for the rock fabric along with the joint wall compressive strength (JCS) parameter and joint roughness coefficient (JRC) as defined by Barton (1976). Base friction angle values for the lapilli tuff was measured to be in the range from about 28 to 33 degrees (Phase 1C - Rock Slope Engineering Report). Unconfined compressive strength values for the joint walls was not measured by the traditional Schmidt Hammer method, but was estimated using the point load test and unconfined compression strength test results presented in previous reports and obtained from measurements in the field and lab in 2008 and 2009 (see Table 1). Values of JRC coefficient must be made at the rock mass scale, and therefore, cannot be accurately measured from core samples. Instead, values were taken from field measurements made on the rock slopes above the highway in the vicinity of Sta. 1382+75 to 1390+00, as presented in the Phase 1C - Rock Slope Engineering Report (Appendix B). The values ranged from 4 to 20, with an average minimum of 10 and an average maximum of 12.

Using values that are slightly below average, including a JRC of 8, a JCS of 9 ksi and a base friction angle of 33 degrees, the Barton-Bandis method predicts large scale joint shear strength parameters of approximately 45 to 50 degrees friction and 2 to 3.5 psi (300 to 500 psf) cohesion. These friction values are higher than previously estimated for this general area based on lab testing, back-analysis of rock slope performance, and other theoretical considerations. We recommend that a friction angle of 40 degrees, with a cohesion of 3.5 psi (500 psf), be used for stability analysis of bridge, tied shaft wall and other major structure foundations.

5. General Characterization of Rock Mass
A very wide variation in rock mass character can be observed in the borings at the east end of the Slide Curve Bridge. A recent re-examination of rock core from these borings has suggested that rock in the vicinity of bridge pier No. 9 (Sta. 1380+80), as reflected in Borings SCB-006-08 and SCB-011-08 ,should be considered separately from the remainder of the east end of the bridge. This discussion is presented in Section 5.2, below.

East of Bridge Pier #9 (Sta. 1380+80)
Most of the borings at the east end of the bridge have average Rock Quality Designation (RQD) values from 25 percent (Poor Rock) to 70 percent (Fair Rock). The weighted average RQD value for all 9 borings is approximately 53 percent in the upper 25 feet.

An unrealistically low average unconfined compressive strength of the intact rock equal to 3.7 ksi was recorded in SCB-024-09, based on 13 tests. The average RQD for the six other borings nearby ranged from 7 to 16 ksi, with an overall average of about 11 ksi. The standard deviation of the strength measurements was typically 40 to 70 percent of the average value.

Rock Mass Rating (RMR) values were estimated at various locations using the Geomechanics Rock Mass Classification System per AASHTO 2007 Bridge Design Specifications and ASTM D5878-08. Of the five parameters comprising the evaluation, the “Condition of Discontinuity” is the most difficult to estimate. Typical joint walls were slightly weathered to fresh. A rating of 20 was used despite a description of “highly weathered walls” of discontinuities, which does not apply to this rock, because the other ratings did not adequately account for the presence of wide discontinuity wall separations that were occasionally encountered. Based on the following additional parameter, an RMR value of 52 to 57 was obtained for the rock mass:

- Strength = 7
- RQD = 8 or 13 (dependent on whether the RQD was >50 percent or <50 percent)
- Spacing = 8
- Groundwater = 9.

For application to slopes, a RMR value of 27 to 32 was obtained, which is listed in the “Poor Rock” category. An adjustment of -25 was assumed for “fair” joint orientations with respect to slopes. For applications to foundations, an adjustment of only -7 was assumed for the same “fair” orientation with respect to joints, resulting in an RMR value of 45 to 50 (i.e., in the “Fair Rock” category).

The upper 5 to 6 feet of rock in many of the borings at the east end of the bridge indicate a slight tendency towards a higher degree of weathering and discoloration, although no systematic reduction in RQD, increase in fracture frequency, or decrease in compressive strength of intact pieces can be shown numerically.

An estimate of rock mass strength by the Hoek-Brown method, as reflected in the Rocscience software “RocLab,” is recommended for use in evaluating foundation design and slope stability issues. The recommended input parameters are as follows:

- Intact Rock Uniaxial Strength = 9000 psi. This was determined by taking the average rock uniaxial strength and subtracting one-half the standard deviation to account for the assumption that the results of point load and unconfined compression tests likely overestimate the overall rock strength (better quality core pieces were used for lab testing).

- Geological Strength Index (GSI) = 50 to 65. The GSI was selected for most of the east end of the bridge, signifying a "very blocky" structure and "fair" to "good" joint surface conditions. The GSI value can be reduced to 43 to 48 to reflect a worse joint surface condition falling in the "fair to poor" transition zone, where a large number of joints having infilling thicknesses of 1-inch or more were encountered (e.g., Boring SCB-028-09 and SCB-030-09). The potentially significant discontinuities are not considered "poor," which is defined as "slickensided highly weathered surfaces, with compact coatings or fillings containing angular rock fragments".
- Mi = 13, as suggested by the authors of the software (Hoek, Carranza-Torres and Corkum, 2002) for tuff rock.
- Modulus ratio = 300, as suggested by the authors of the software for tuff rock.
- Disturbance Factor D = 0 for minimal disturbance during construction.
- Values of parameters "mb", "a" and "s" are calculated by default by the software when the other parameters above are entered.

Use of these parameters results in Mohr Coulomb strength parameters of approximately 55 degrees for the friction angle and cohesion in the range of about 100 to 180 psi for most rock at the east end of the bridge. The accompanying values of rock mass "uniaxial compressive strength" range from about 500 to 1300 psi, while the "global rock mass strength" ranges from 1800 to 2500 psi.

For zones in the rock mass containing frequent potentially significant discontinuities (GSI from 43 to 48), the friction angle reduces to approximately 53 degrees with cohesion in the range from about 80 to 90 psi. The accompanying values of rock mass "uniaxial compressive strength" range from about 400 to 500 psi, while the "global rock mass strength" ranges from 1500 to 1700 psi.

At Bridge Pier #9 (Sta. 1380+80)
It should be noted that the very low RQD values at Boring SCB-006-08 and SCB-011-08 are apparently the lowest encountered within the I-90 project corridor. Similarly, RQD values in some of the borings on the slopes above the highway at this location were typically lower than elsewhere on the project.

A recent re-examination of core samples indicated that a substantial number of healed fractures could be observed, and the core could be broken up by hand. Given some drying-related deterioration of this core, there seems to be a general reduction in the strength of the rock fabric that may have resulted from tectonic pressures or other disturbance at this location. Rock does not outcrop above the roadway upslope from this pier.

Based on the depth to rock measurements from borings and geophysical surveys, there is apparently a buried sidewall drainage channel (trough) in the rock surface that extends from the lake level up to the slope above the highway. Reduced wave velocities for rock can be seen in geophysical test data here. The unconfined compressive strength measurements listed in Table 1 are for the small percentage of reasonably competent pieces of core produced during the drilling process. Those compressive strength values are not representative of most rock at this location. A more appropriate value of perhaps 25 to 30 percent of the measured values should be assumed for analysis of the bridge, tied shaft wall, and other major structure foundations. Despite the lower RQD and more intense fracturing, the rock is still considered "very blocky" instead of "blocky/disturbed/seamy;" therefore warranting a GSI of 40 when the joint surface condition of "fair" is applied.

6. Sources of Uncertainty
Decisions about the character and performance of geologic materials must always take into consideration the natural variability of the materials in question and the limitation of tools for observations and measurements. The variability of characteristics and properties of the meta-welded lapilli tuff rock at the Slide Curve Bridge location can be regarded as moderately high to high. For example, the standard deviation for unconfined compressive strength was usually about 40 to 70 percent of the mean, while the standard deviation for RQD measurements was typically 30 to 50 percent of the mean. Variability in the dip direction and dip angle of discontinuities is reflected in the absence of strong clustering of the joint sets on the stereonets.

Limitations in the tools for observation and measurement, for example, the inability to capture all core during the coring process, results in gaps in knowledge about the rock and its discontinuities. While the average percent recovery of core in borings at the east end of the bridge was approximately 91 percent (ranging from 65 percent to 100 percent), this still means that more than 30 feet of core and discontinuities was missing from the 9 borings east of Sta. 1381+00. A conclusion about the traceability of joints from one boring to another is affected by the possible absence of core at key locations. These sources of uncertainty should be considered during the analysis of abutment stability by assessing the sensitivity of the result to reasonable variations in parameters and joint system geometries as presented below in the Conclusions and Recommendations.

Conclusions and Recommendations

Based on the available data, it is unlikely that any of the potentially significant discontinuities encountered in borings at the east end of the Slide Curve Bridge are persistent from one side of the bridge foundation to the other. With no missing core and no apparent potentially significant discontinuities at key Boring SCB-020-09 located immediately downslope of the largest 12-inch thick potentially significant discontinuity in SCB-021-08, a persistent zone of potentially significant discontinuities cannot be hypothesized across the full width of the bridge foundation. Even with potentially significant discontinuities present in SCB-028-09, located less than 15 feet from SCB-020-09, there is no conclusive evidence that a persistent weak planes of exists.

Two joint sets of potential significance to the performance of the bridge foundations can be hypothesized from the stereonet plots. The first, identified on both Figure 4 and Figure 5, dips at a very shallow angle (10 to 11 degrees) with an azimuth of 288 degrees to 298 degrees. This set could theoretically daylight from either a localized 50 degree upper rock slope or the general 30 degree rock slope. The second, identified only in Figure 4, dips at 44 degrees to an azimuth of 271 degrees (i.e. directly downslope). This set could theoretically daylight from a localized 50 degree upper rock slope, or serve as a possible weak plane that sub-parallels either the 50 degree upper rock slope or the more general 30-degree rock slope. It should be noted that any failure plane exiting from the rock on this slope must still propagate through the overlying fill and colluvium to complete the failure process.

The presence of potentially significant discontinuities should be incorporated into the design process either by:

- Adjusting the rock mass strength and deformability to account for the lower quality rock that would result from the presence of these features. In particular the "Geological Strength Index" (GSI) value should be selected in the range of 43 to 48 in areas of multiple soil-filled joints for use in the Hoek-Brown rock mass strength estimation method; or
- Creating an anisotropic rock mass strength with the general strength parameters obtained using the Hoek-Brown estimation method with GSI in the range of 50 to 65, but with a reduction in those strength parameters for portions of the failure plane that are oriented parallel to one or both of the joint sets identified above. The amount of reduction should be estimated assuming that a failure plane propagating through the rock mass proceeds along the discontinuity, with shear strength parameters appropriate to the discontinuity, about 75 percent of the path length. Joint

shear strength parameters of 40 degrees friction and 3.5 psi cohesion are recommended. The remaining 25 percent of the failure path length at that orientation proceeds through portions of the rock mass that interrupt the persistence of the joint.

- For sensitivity analyses, a discontinuity/rock mass as percent of path length ranging from 67/33 to 100/0, ± 2 degrees in friction angle, and ± 25 percent for rock strength is recommended.

Bridge Pier #9 (Sta. 1380+85) appears to be located within a rock mass domain that is somewhat different in character from other areas underlying the bridge. The difference may be the result of tectonic disturbance that has created a weakened rock mass fabric for much of the rock mass volume. There is no evidence of major through-going discontinuities at the pier based on available surface and subsurface data. Rock mass strength estimated by the Hoek-Brown method should use a recommended GSI of 40 and an unconfined rock strength of 2 ksi (i.e., only 30 percent of the values recorded for the best pieces of core in the two borings at that location).

References

1. Barton, N.R. and Bandis, S.C. 1990. Review of predictive capabilities pf JRC-JCS model in engineering practice. In Rock Joints, Proc int symp on rock joints, Loen, Norway (eds N. Barton and O. Stephenson), p 603 – 610, Rotterdam, Balkema.

2. Barton , N.R. 1976. The shear strength of rock and rock joints. Int. J. Rock Mech Min Science& Geomech Abstr 13(10), p 1 – 24.

3. Terzaghi, R., 1965. Sources of error in joint surveys, Geotechnique, v. 15, p287 *as presented in* Goodman, R.E., 1976, Methods of Geological Engineering, West Publishing Company.

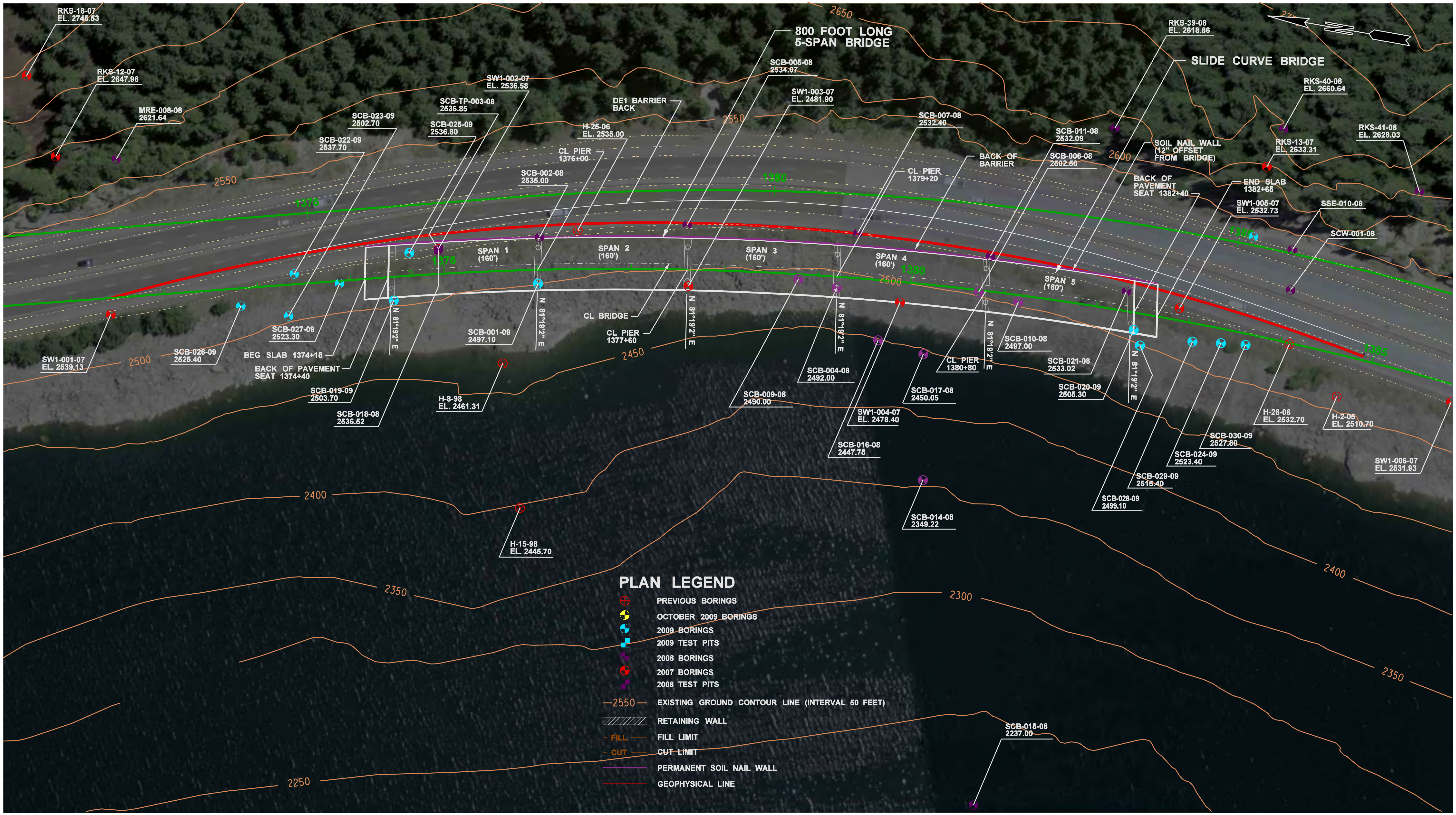
Table 1: Summary of Filled Joints and Rock Character for Borings on East Side of Slide Curve Bridge

Boring No.	Ave % Recov Upper 25-ft	Ave % Recov Lower 25-ft	Ave. RQD Upper 25-ft	Ave. RQD Lower 25-ft	Ave Fractures per Foot Upper 25-ft	Ave Fractures per Lower 25-ft	No. Discont > 1.0-in	Thickest Discont (in)	Depth Below GS (ft)	Elev (ft)	Dip (degrees)	Dip Direction (degrees)	Soil Description	Rock Ave. Unconf Compr Strength - PLT (ksi)	No. of PLT Tests	Rock Ave. Unconf Compr Strength - Uc (ksi)
SCB-06-08 *	80	86	15	18	2	3.1	1	5	71.5	2431	NA	NA	silt/sand	4.7	5	
SCB-10-08 *	98	93	53				0							7	6	
SCB-11-08	80	33	12	3	7.2	10	0							8.8	5	
SCB-20-09 *	80	73	25	27	2.5		0							15.6	7	
SCB-21-08	65	92	53	53	3.3		1	12	68	2465	(10)	(270)	silty clay	10.6	6	
SCB-24-09 *	91	97	45	62	4.6		0							3.7	12	
SCB-28-09 *	99	98	38	28	2.8		10	2	33.5	2467	NA	NA	brn silty sand	16.3	23	16.1
								1	36.5	2464	(40)	NA	gry sandy clay			
								2	50.1	2450	52	142	gry sandy silt			
								3	50.5	2450	42	345	gry silty sand			
								2	56.3	2444	42	60	gry silty sand			
SCB-29-09 *	95	100	70	71	1.5	2.3	8	1.5	32.5	2479	12	265	brn sandy silt	12.6	16	15.1
								2	43.5	2468	63	271	gry silty sand			
								1	57.3	2454	75	59	gry silty sand			
SCB-30-09 *	99.5	99.7	67	58	1.9		10	1.5	31.3	2494	(40)	NA	brn sandy silt	11.5	18	12.7
								1	34.6	2490	41	350	gry silty sand			
								1.5	61.2	2464	32	208	gry silty sand			
								2	64.2	2461	22	170	gry silty sand			
								2	69.2	2456	56	222	gry silty sand			
								1	83.4	2442	67	120	gry sandy silt			
SW1-005-07	97	NA	39				0									
H-26-06	98.6	100	92				0									

* indicates downslope side of bridge
RQD = Rock Quality Designation
PLT = Point Load Test

Notes

1. Only discontinuities with at least 1-inch of soil infill are shown here.
2. NA indicates not available.
3. Dip angles in parenthesis (35) were estimated from measurements in core box; all others are from COBOL measurements.
4. All dip direction values are from COBL measurements, except assumed value in parenthesis for SCB-21-09.



0 100 200
SCALE 1" = 100'

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TIME	11:21:51 AM
DATE	2/1/2010
DESIGNED BY	Kranti Maturi
ENTERED BY	larry_jiles
CHECKED BY	Dave Walker
ASST. PROJ. MNGR.	John Zeman
PROJ. MNGR.	Cecil Urlich

REVISION | DATE | BY

REGION NO.	STATE	FED.AID PROJ.NO.
10	WASH	

JOB NUMBER 33758623

CONTRACT NO. Y-9764

LOCATION NO.

P.E. STAMP BOX DATE

URS
CENTURY SQUARE
1501 4TH AVENUE, SUITE 1400
SEATTLE, WA 98101
PHONE: (206) 438-2700
FAX: (206) 438-2699

Washington State
Department of Transportation

I-90 Snoqualmie Pass East
Hyak to Keechelus Dam, Washington

SITE AND EXPLORATION PLAN

FIGURE
1

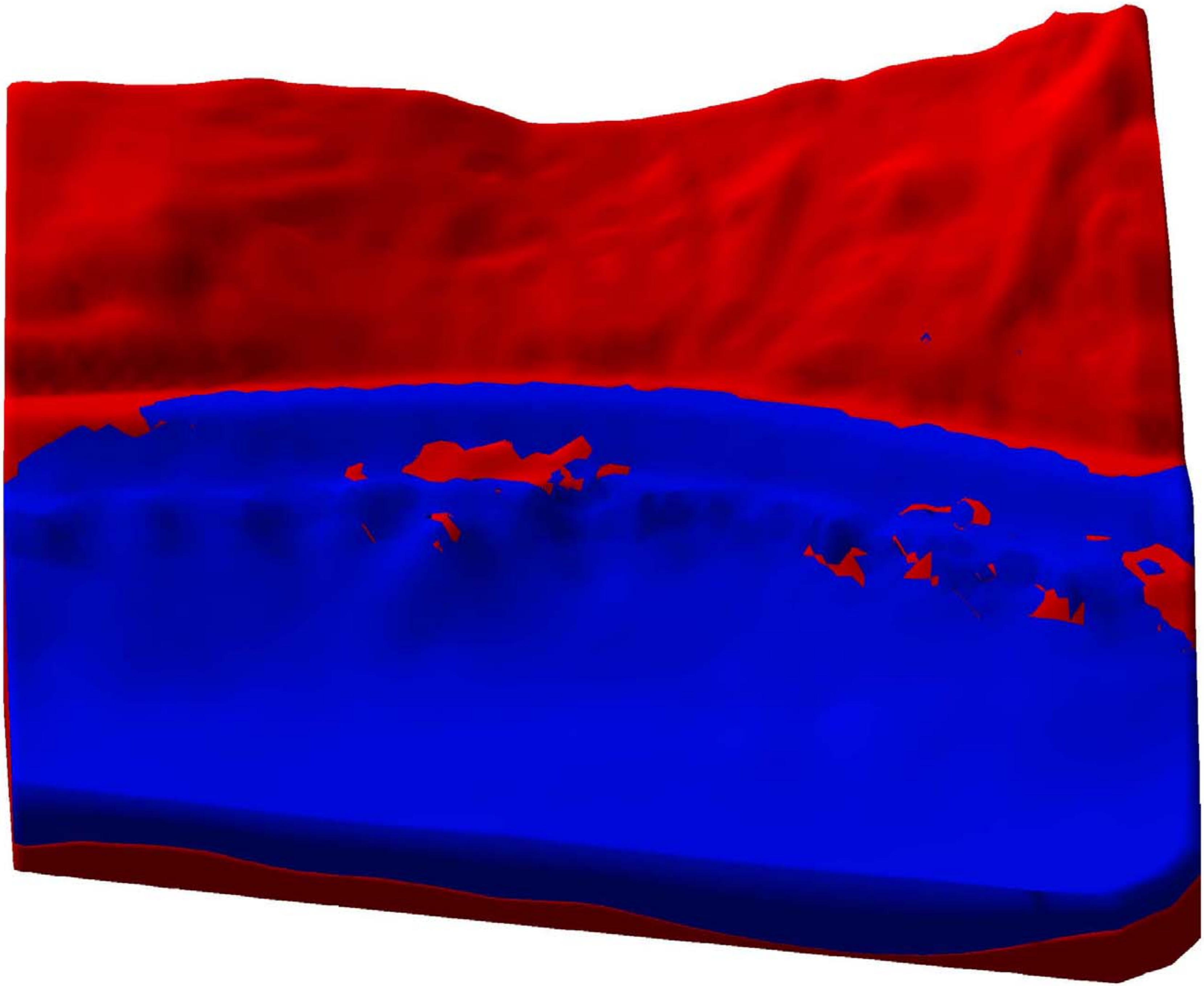
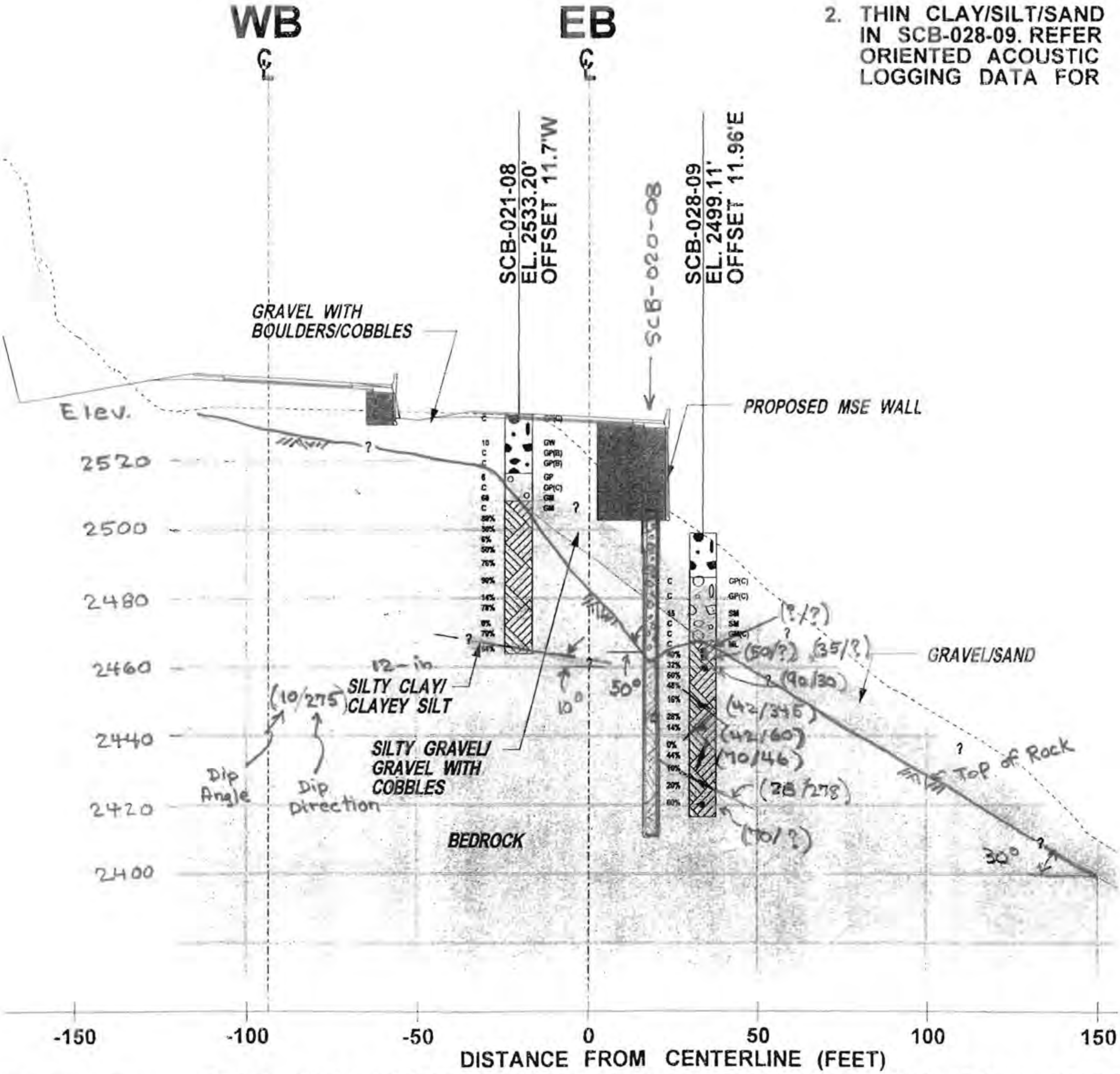


Figure 2

NOTES:

1. THIS CROSS-SECTION PROPOSED ALIGNMEN
2. THIN CLAY/SILT/SAND IN SCB-028-09. REFER ORIENTED ACOUSTIC LOGGING DATA FOR



)NTAL SCALE 1"=40'
:AL EXAGGERATION = 1x

EB STATION 1382+40 SLIDE CURVE BRI

CADD\07900 CADD 2009\Sheet Files\Slide Curve Bridge (SCB)\P1234_RS_Slide Curve Bridge_Section_7.dgn

REVISION	DATE	BY	REGION NO. 10	STATE WASH	FED.AID PROJ.NO.
			JOB NUMBER 33758623		
			CONTRACT NO. Y-9764		LOCATION NO.

P.E. STAMP BOX

DATE

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CENTURY SQUARE
1501 4TH AVENUE, SUITE 1400
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FAX: (206) 438-2699

Wa
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Figure 3

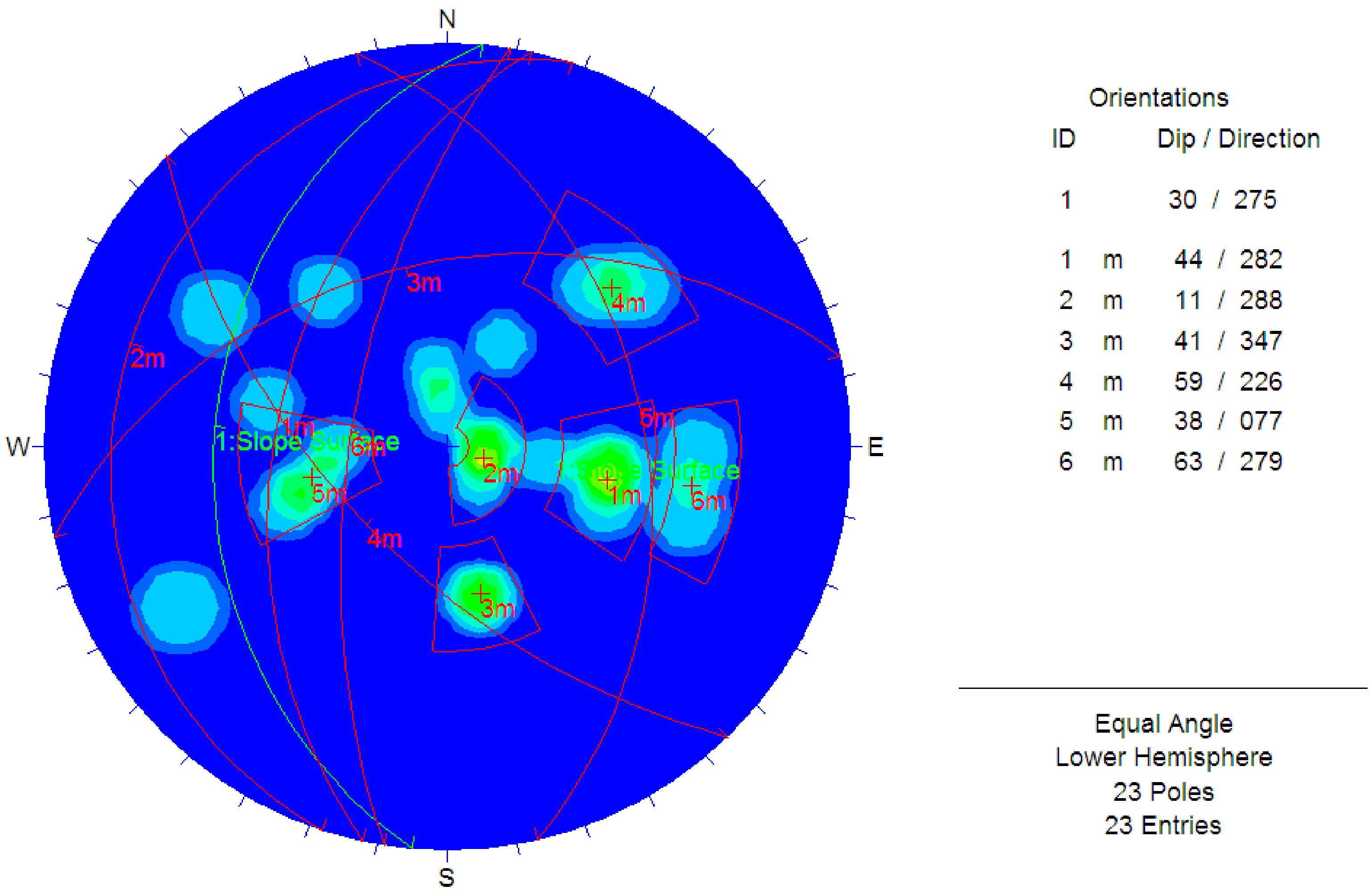
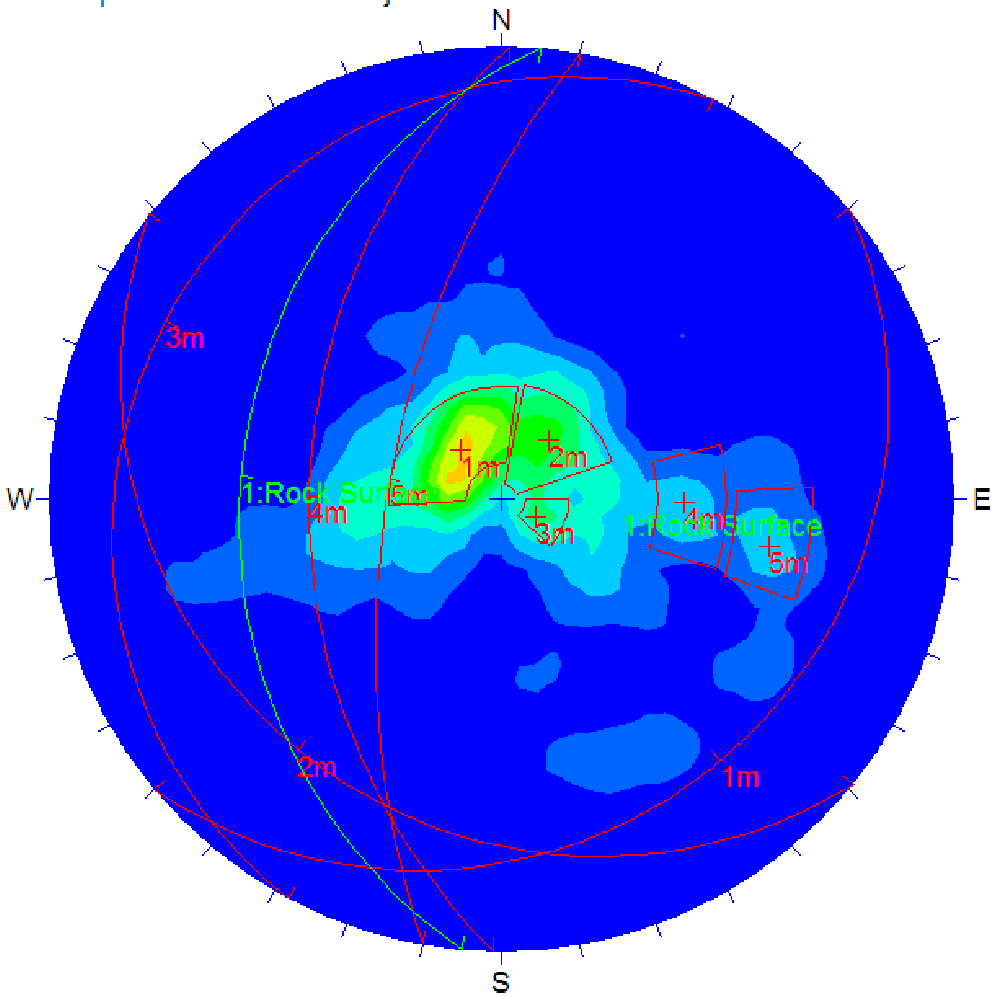

N
S
E
W
1:Slope Surface
1m
2m
3m
4m
5m
6m
Orientations
ID Dip / Direction
1 30 / 275
1 m 44 / 282
2 m 11 / 288
3 m 41 / 347
4 m 59 / 226
5 m 38 / 077
6 m 63 / 279
Equal Angle
Lower Hemisphere
23 Poles
23 Entries

Figure 4

I-90 Snoqualmie Pass East Project



East End of Slide Curve Bridge by K Yang, 11/30/09

Orientations

ID		Dip / Direction
1		30 / 275
1	m	16 / 140
2	m	19 / 219
3	m	10 / 298
4	m	44 / 271
5	m	62 / 280

Equal Angle
Lower Hemisphere
286 Poles
286 Entries

Figure 5