APPENDIX K
Phase 1C PS&E Support

APPENDIX K.1
Snowshed

Appendix K.1.1
Portal Tower Loads

Chuck Vita/Seattle/URSCorp

03/06/2009 10:23 AM

Files Attached: **0**
Total Email Size: **14 kb**

To
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cc
Dave Walker/Seattle/URSCorp@URSCORP, "Jenkins, David V (MLO)" <JenkinD@WSDOT.WA.GOV>, "Golbek, Scott" <GolbekS@WSDOT.WA.GOV>, "Moore, Tim" <mooret@WSDOT.WA.GOV>, "Stoddard, Richard" <stoddari@WSDOT.WA.GOV>
bcc

Subject
re I-90 Snowshed - Tower Loading

Lou -- I will answer your (second) question regarding lateral loads on the tower shafts, as shown in Fig 20 of TM3 (July 2008). Because they are at the ends of Pier 2 and close to the next interior shaft, the tower shaft loading P1 can be somewhat reduced from Fig 20. I would recommend, without further analysis, a maximum reduction of 1/3, not 1/2 -- so the recommended pressures are 2/3 of P1 in Fig 20 (but only for the tower shaft). If there is any P2 ("downslope passive resistance"), it may be reduced from Fig 20 as much as you want.

Regards,
Chuck

Chuck Vita, PhD, PE, GE
Principal Engineer
URS Corp, Century Square, 1501 4th Ave #1400, Seattle, WA 98101-1616
Office Phone: 206-438-2348. Cell: 206-390-9958.

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----- Forwarded by Dave Walker/Seattle/URSCorp on 03/06/2009 08:41 AM -----

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03/04/2009 04:07 PM

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Subject
I-90 Snowshed - Tower Loading

Hi Dave,

We have questions regarding to the avalanche loading on the towers and the slide earthquake pressures on the tower shaft foundations.

The Avalanche Mitigation Report does not cover what would be the avalanche load on the tower from the hillside. Currently we assume the avalanche pressure 1400 psf acts on the tower to height of 30 feet and taper to 0 at top, similar to figure 2-21. Is this adequate assumption? Would you consult with Arthur Mears.

The foundation of the towers on the lake side will be single shaft foundation, 8 feet in diameter. These two foundations will be about 11 feet in distance to pier 2 shaft at each end. We assume the earth pressures P1 and P2 are half of the P1 and P2 values of the figure 20, Technical Memorandum No. 3 - Snowshed, Are these adequate? Or you have different values for us to use.

We have three drawings of the tower details for you to review, see attachment.

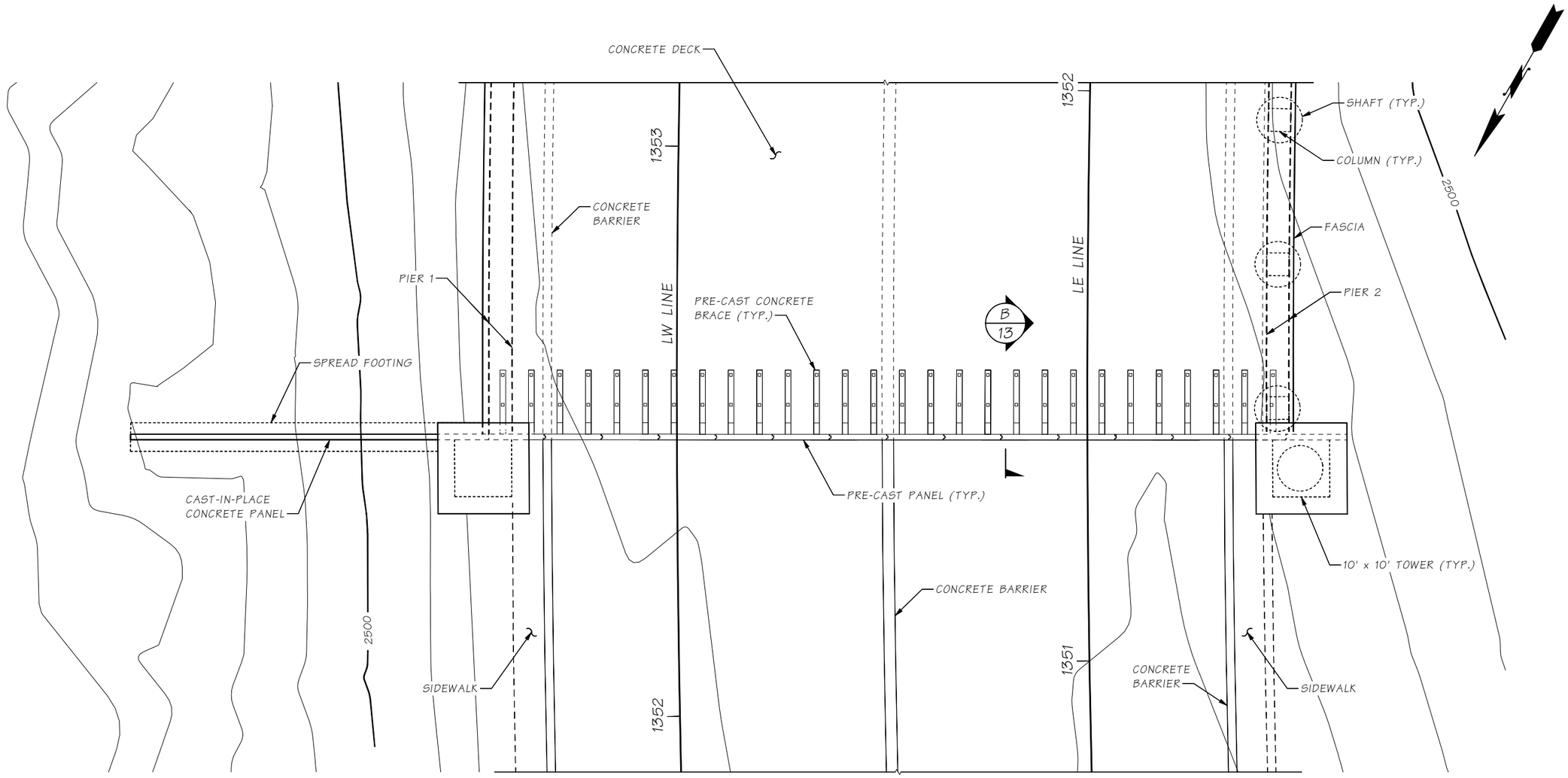
Please let us know that you need more information.

Thanks,

Lou

Luong(Lou) H. Tran, P.E.
Bridge and Structures Office
PO Box 47340
Olympia,WA 98501
tranluo@wsdot.wa.gov

[attachment "WEST PORTAL.PDF" deleted by Chuck Vita/Seattle/URSCorp] [attachment "PLAN.PDF" deleted by Chuck Vita/Seattle/URSCorp] [attachment "OPT. 1.PDF" deleted by Chuck Vita/Seattle/URSCorp]



PLAN

WEST PORTAL

I-90 FILE NO. ____ SHEET 10

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Designed By	Tran, L.	02/09				10	WASH.			
Checked By	Moore, T.	02/09								
Detailed By	Lemcke, D.	02/09				JOB NUMBER				
Bridge Projects Engr.										
Prelim. Plan By										
Architect/Specialist			DATE	REVISION	BY	APP'D				

Wed Mar 04 15:11:52 2009

BRIDGE AND STRUCTURES OFFICE

JUGESH KAPUR — STATE OF WASHINGTON — 25400 — STRUCTURAL ENGINEER — PROFESSIONAL ENGINEER

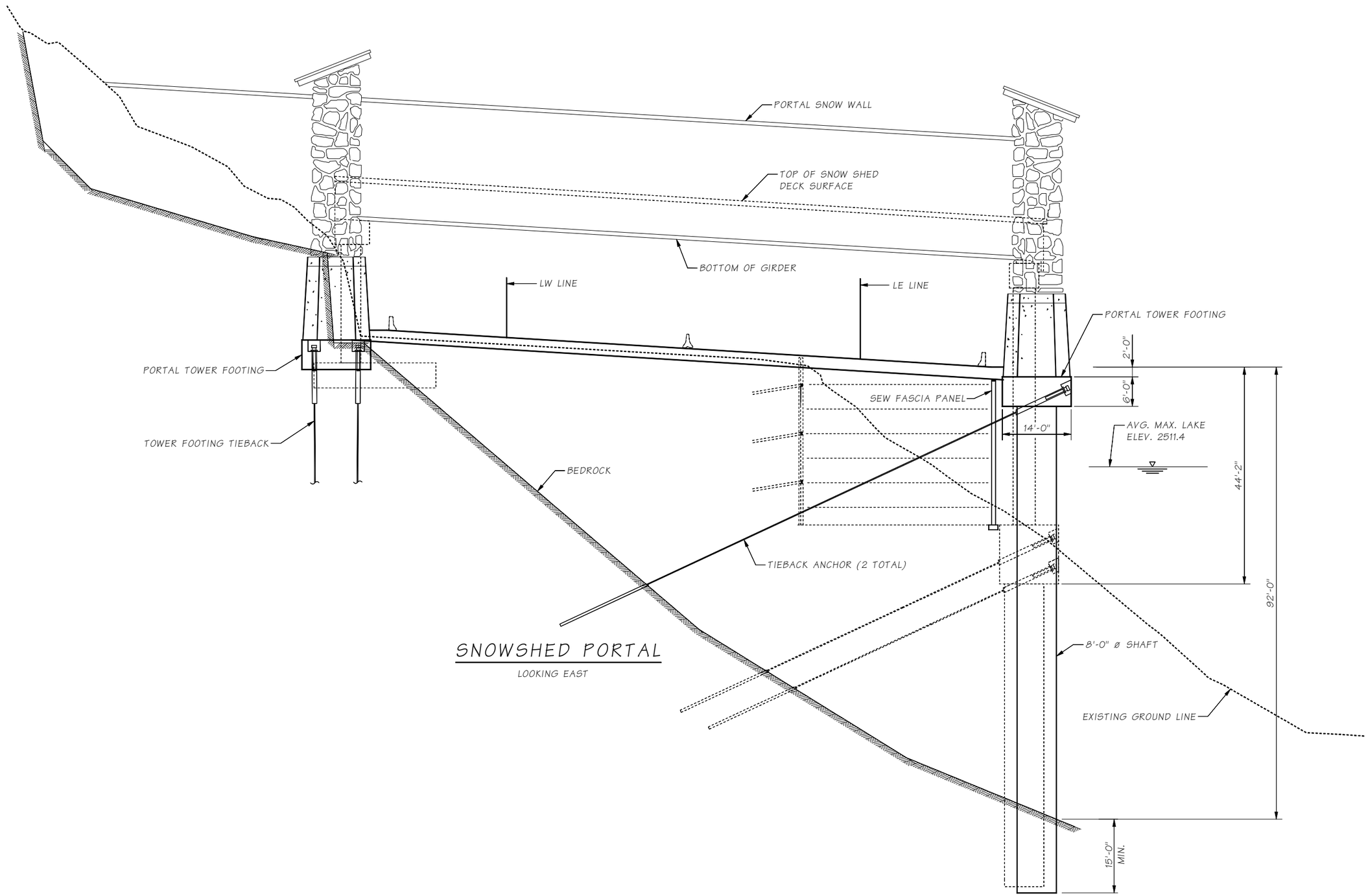
Washington State Department of Transportation

I - 90
SNOQUALIMIE PASS EAST
HYAK TO KEECHELUS DAM
SNOW SHED

WEST PORTAL - PLAN

BRIDGE SHEET NO. 10

SHEET OF SHEETS



SNOWSHED PORTAL

LOOKING EAST

I-90 FILE NO. ____ SHEET 9

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Supervisor					REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
Designed By	Tran, LH	02/09			10	WASH.			
Checked By	Moore, T.	02/09			JOB NUMBER				
Detailed By	Lemcke, DR	02/09							
Bridge Projects Engr.									
Prelim. Plan By									
Architect/Specialist		DATE	REVISION	BY	APP'D				

Tue Feb 24 12:49:45 2009

BRIDGE AND STRUCTURES OFFICE

BLAN KHALEGHI — STATE OF WASHINGTON — 27095 — STRUCTURAL ENGINEER — PROFESSIONAL ENGINEER

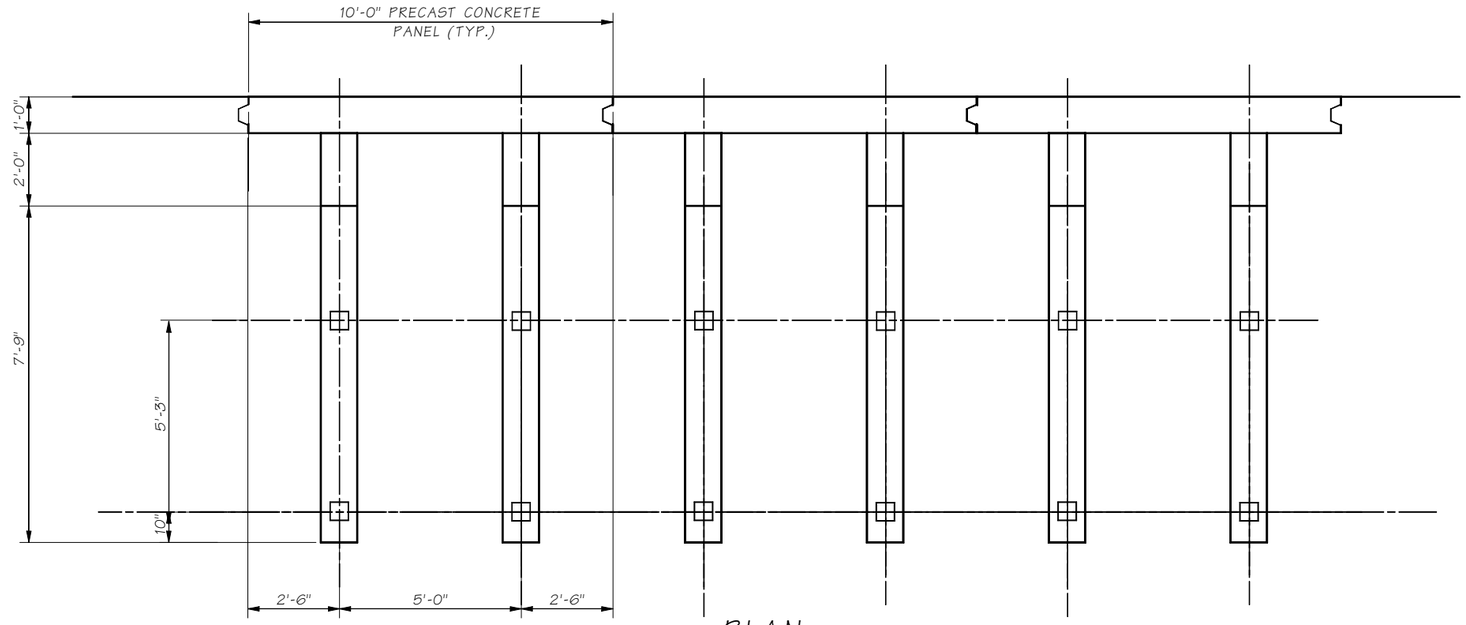
Washington State Department of Transportation

I - 90
SNOQUALIMIE PASS EAST
HYAK TO KEECHELUS DAM
SNOW SHED

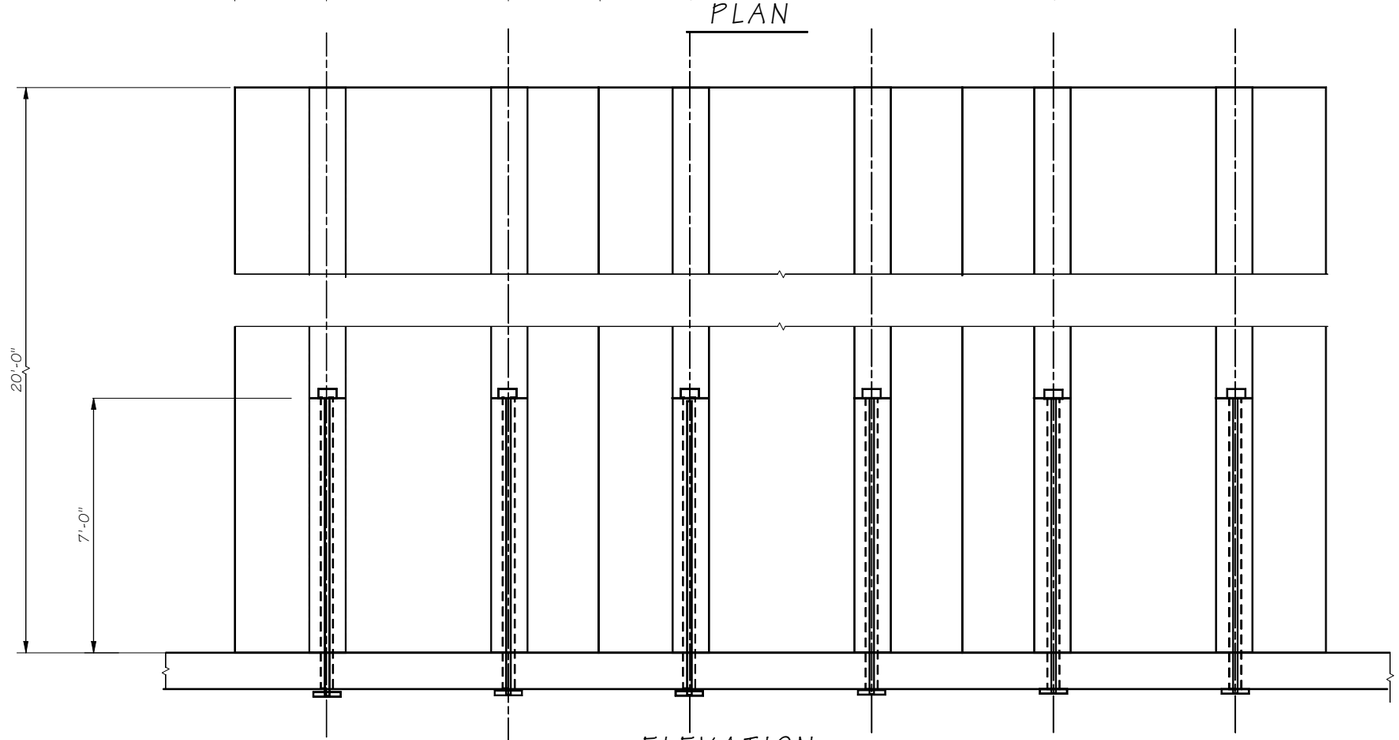
WEST PORTAL ELEVATION

BRIDGE SHEET NO. 9

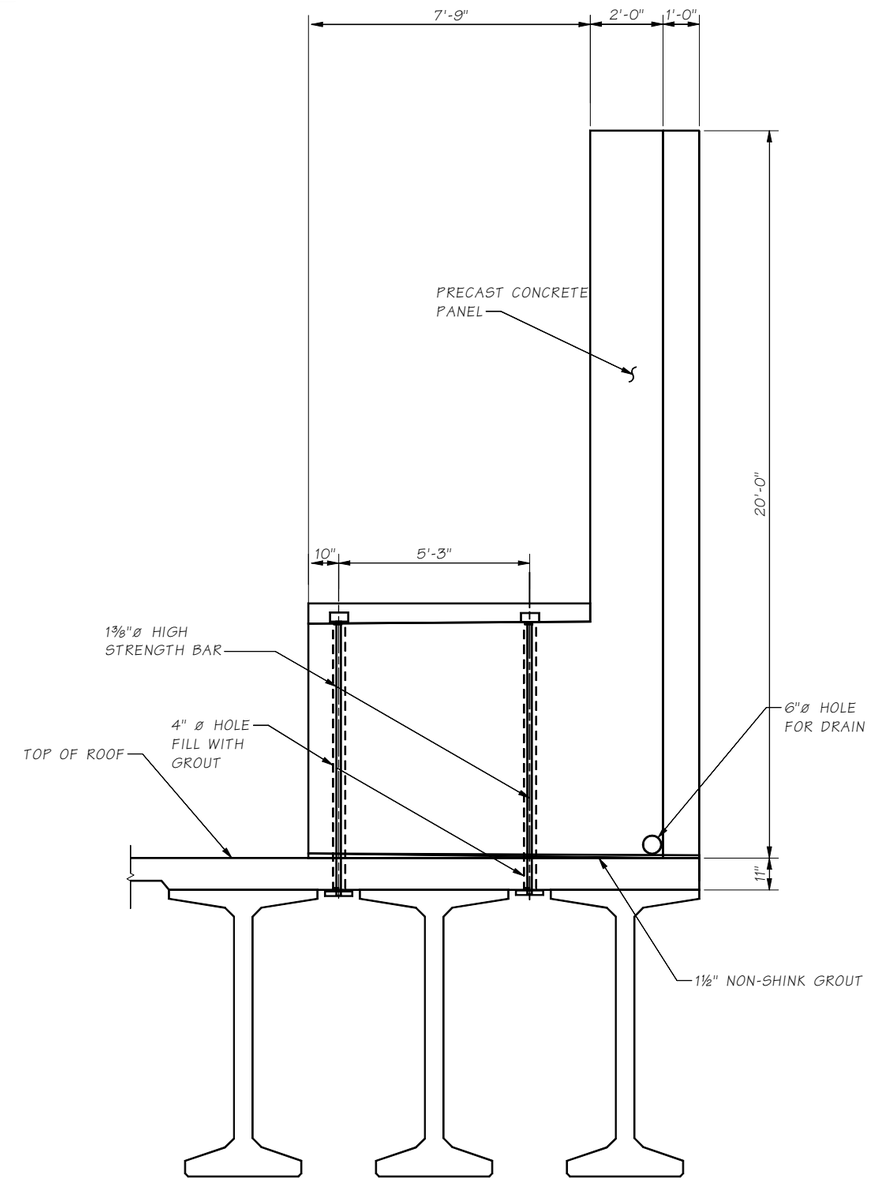
SHEET OF SHEETS



PLAN



ELEVATION



SECTION B
10, 12

Bridge Design Engr.			M:\X-Team\I-90 Snoqualmie Pass\PRELIMINARY SNOW SHED\window files\OPT. 1.WND						
Supervisor						REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
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Checked By	Moore, T.	02/09				JOB NUMBER				
Detailed By	Lemcke, D.	02/09								
Bridge Projects Engr.										
Prelim. Plan By										
Architect/Specialist			DATE	REVISION	BY	APP'D				

BRIDGE AND STRUCTURES OFFICE

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PROFESSIONAL ENGINEER

Washington State Department of Transportation

I - 90
SNOQUALIMIE PASS EAST
HYAK TO KEECHELUS DAM
SNOW SHED

PORTAL - PRECAST FASCIA
OPTION 1

BRIDGE SHEET NO. 13
SHEET OF SHEETS

I-90 FILE NO. ____ SHEET 13

Tue Feb 24 12:49:57 2009

Appendix K.1.2
Utility room

Dave Walker/Seattle/URSCorp

06/12/2009 08:52 AM

Files Attached: **1**
Total Email Size: **57 kb**

To
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bcc

Subject
RE: I-90 Snowshed Utility Rooms under the Roadway

Lou-

Yes, 3 tons per sq ft unfactored bearing resistance is OK.

The loading diagrams in the attached figure are OK, except that the mechanical room would also be subject to vertical snow loads. The avalanche debris load ("snow load static " in the figure) varies with the length of the shed. If the avalanche debris loads are problematic and the mechanical room and/or ducts don't run the full length of the shed (in particular, if they are not present beneath the main avalanche chute), it may be possible to reduce these loads. Snow loads do not need to be included for the extreme event limit state.

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Fax: 1-866-495-5288

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"Tran, Luong" <TranLuo@wsdot.wa.gov>
06/11/2009 08:54 AM

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Subject
RE: I-90 Snowshed Utility Rooms under the Roadway

Hi Dave,

Thank you for the information. One more question for the duct vaults sitting on ground behind pier 1, what would be the soil bearing pressure, 3 tons unfactored?

I drew up the loading diagrams for the duct vault and mechanical room on the attached sheet. Please let me know that I got the loading correctly.

Thanks,

Lou

Dave Walker/Seattle/URSCorp

06/08/2009 03:09 PM

Files Attached: **1**
Total Email Size: **52 kb**

To
"Tran, Luong" <TranLuo@wsdot.wa.gov>
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Chuck_Vita@URSCorp.com, "Cooper, Patrick" <CooperP@WSDOT.WA.GOV>, "Giles, Randy" <GilesR@WSDOT.WA.GOV>, "Golbek, Scott" <GolbekS@WSDOT.WA.GOV>, jeff_gibson@urscorp.com, "Lewis, Ron" <LewisR@WSDOT.WA.GOV>, Robert_Hamilton@URSCorp.com, "Stoddard, Richard" <stoddari@WSDOT.WA.GOV>, "Wood, Jerry" <WoodJe@WSDOT.WA.GOV>
bcc

Subject
RE: I-90 Snowshed Utility Rooms under the Roadway

Lou-

Attached are the lateral earth pressures that are recommended for design of the utility room near Pier 2. These pressures assume that the soil is not reinforced and that the utility room walls act rigidly. Soil cover between the base of the pavement and the top of the utility room is not required except as may be appropriate at transitions across the perimeter of the utility room. The pavement and utility room need to be designed to bridge the span carrying the vehicle loads.

If the utility room is placed behind Pier 1, the lateral earth pressures can be calculated using Figures 11 and 12 from Technical Memorandum No. 3 - Snowshed Replacement (same as recommended for Pier 1 wall design). The vertical soil pressure in psf can be calculated as 135 times the height of soil above the top of the box.

Pier 2 utility room earth pressures.pdf

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06/03/2009 02:09 PM

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Subject
RE: I-90 Snowshed Utility Rooms under the Roadway

Hi Dave,

These are reinforced concrete boxes, probably precast system with three or four sides, similar to the commercial utilidor system. We can design the reinforcement system to take any load, either increasing the wall thickness or more rebars. We also can design for the tie-back system on these boxes to resist the lateral loads if needed.

Please note we need the loading to design these boxes for either placing under roadway near pier 2 or behind pier 1. I do not think we have the vertical surcharge load for these boxes if placed behind pier 1. Can we use same lateral loads as pier 1 pier wall design?

Thanks,

Lou

"Tran, Luong" <TranLuo@wsdot.wa.gov>

05/28/2009 04:01 PM

To

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"Moore, Tim" <mooret@WSDOT.WA.GOV>, "Wood, Jerry" <WoodJe@WSDOT.WA.GOV>, "Lewis, Ron" <LewisR@WSDOT.WA.GOV>, "Stoddard, Richard" <stoddari@WSDOT.WA.GOV>

Subject

I-90 Snowshed Utility Rooms under the Roadway

Hi Dave and Dave,

We are looking options for utility room locations. One option is behind pier 1, another is place under the roadway near pier 2, see attachment. These rooms will be watertight with access either from top or side.
We have few questions for you:
What is the minimum soil cover that we need between the bottom of roadway pavement to top of the utility room?
What are the loadings for this utility room?
Do you have any concerns?
Please let us know your comments.

Thanks,

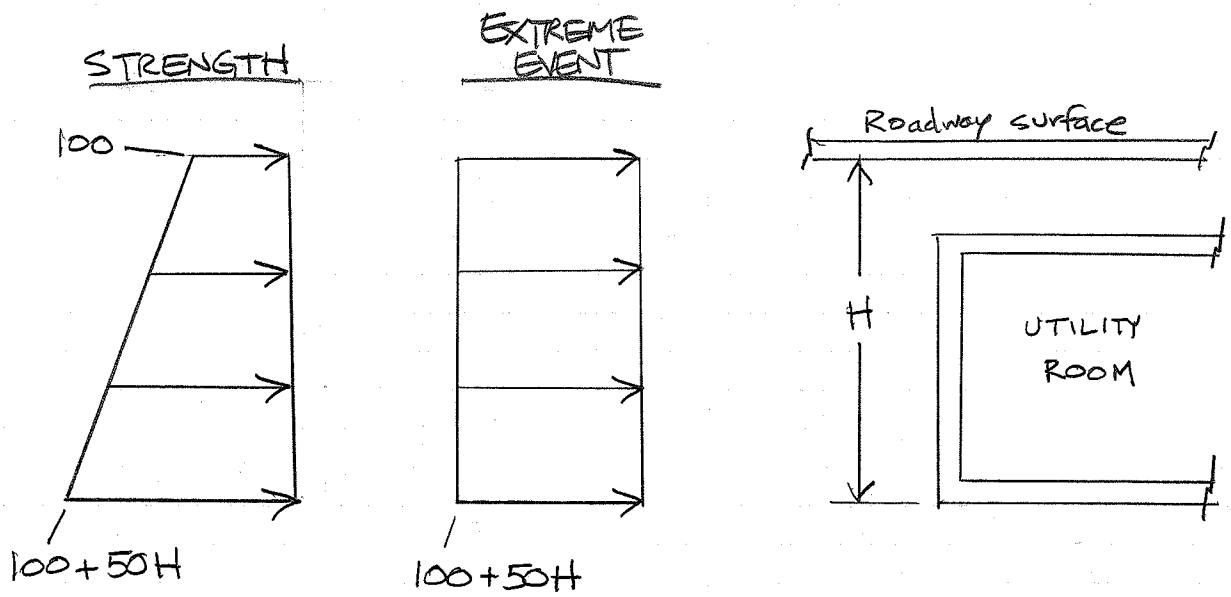
Lou

(See attached file: XSECT 1355 53 - A.PDF)

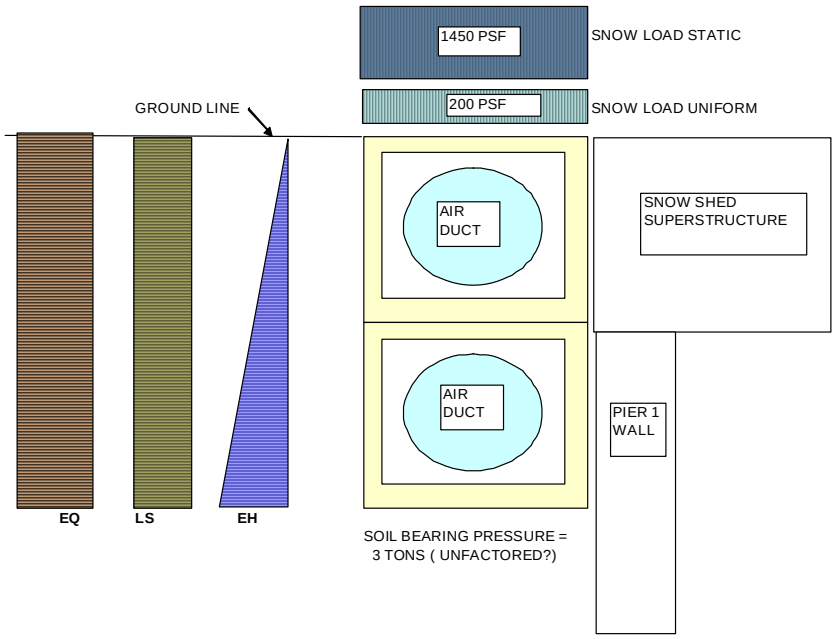
6/8/2009

SNOWSHED UTILITY ROOM

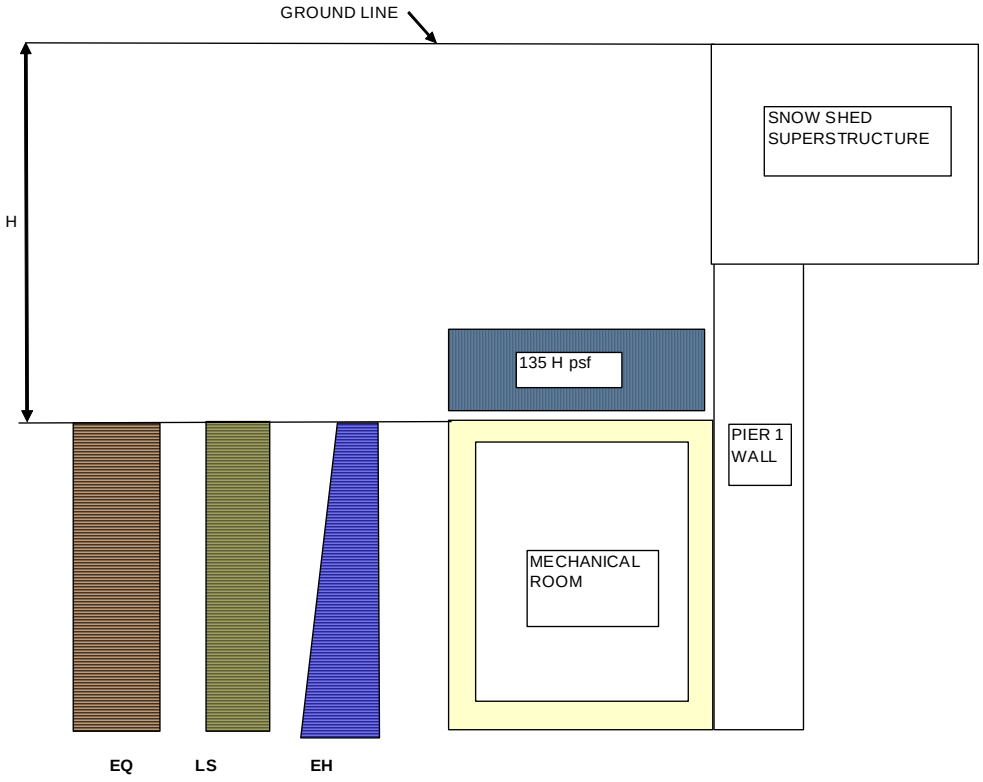
Lateral Earth Pressures in psf



TOPS. 35500



LOADING FOR THE AIR DUCT VAULT



LOADING FOR THE MECHANICAL ROOM

Appendix K.1.3
Sediment Transport

URS

Memorandum

Date: October 6, 2009

To: Tanmay Sampat, Jerry Wood
Washington State Department of Transportation

From: Phillip Mineart, Jeanne Gambino, and Dave Walker, P.E.

Subject: ***Preliminary Estimate of Sediment Transport in Keechelus Lake***

1.0 BACKGROUND

As part of the construction of a work bench to create access for building bridge shafts and a soil nail wall, excavated material will be side cast into Keechelus Lake. The side casting will be accomplished using dozers and excavators. Approximately 52,019 cubic yards (yds) will be side cast at Slide Curve Bridge and Walls (SCB) and approximately 56,577 yds will be side cast at the snowshed. It will take approximately 3 to 4 months to complete the work. To minimize impacts to the lake from the placement of sediment in the lake, a silt curtain will be installed offshore of the excavation to help prevent fine sediments from being transported away from the construction site and farther into the lake. The curtain will be up to 1500 feet long.

2.0 METHODS

It is estimated that the excavation rate will be limited by the rate at which the lake level drops. The maximum rate of lake level drop is 1.5 feet per day. For this analysis it was assumed that a 1.5-foot-thick lift of material would be removed over the 45-foot depth and 1,100-foot length of the bench in a day. It was assumed the material would be placed in the lake at a constant rate over the 1,100 foot length during each 8-hour work day. The sediment placement was assumed to occur within 10 feet from shore.

To provide a preliminary estimate of the sediment transport that could occur during the construction period, the US Environmental Protection Agency (EPA) Water Quality Analysis Simulation Program (WASP) version 7.3 model was used. This model predicts water quality responses to natural phenomena and discharges of pollutants. The model geometry is based on compartments (boxes) for the water column and underlying sediment. The “simple toxicant” model type was used to model the transport and deposition of silt. Silt was assumed to make up 8 percent of the total excavated material, based on laboratory testing performed at Slide Curve Bridge, as shown in Table 1. Settling of sand and gravel would exceed 0.1 foot per second. Therefore, the transport of sand and gravel was not modeled because it will settle out quickly and would not be transported a significant distance into the lake. The model provides an estimate of the concentration of sediment in the water column and the amount of deposition of the sediment on the bottom.

The geometry for the water column was represented by boxes having a width of 9.8 feet (3 meters), a length of 10 feet (3.05 meters), and a depth of 8 feet (2.44 meters). A single box was used to represent the water column within the first 10 feet from shore. The next 10 feet of the water column were comprised of 2 stacked boxes, and so on. This resulted in a model extending 70 feet (7 boxes out) from the shoreline with a bottom slope of 1.2 to 1 (horizontal to vertical). Two boxes were added underneath each column to represent the surface and subsurface sediment. The results of the model apply to the

portion of the lake along the 1,100-foot bench with the highest concentrations. The concentrations along the edges of the sediment plume would be lower due to along-shore dispersion.

The sediment is transported into the lake by advection and dispersion. Dispersion is only allowed to occur in the horizontal direction perpendicular to the shoreline. This is because it was assumed that sediment was discharged into the lake along the entire length of the excavation. The model requires the user to specify the flow path and fraction of flow between adjacent boxes, along with a total inflow rate. The inflow was specified as equivalent to the volume of excavated material, or 9,281 cubic feet per hour along 1,100 feet of shoreline (2.17×10^{-4} cubic meters per second per meter of shoreline). The inflow was added at the same time as the sediment. Horizontal dispersion of 0.01 square meters per second was included in the model to represent the diffusion within the lake.

Model runs were performed with and without a silt curtain to evaluate the effects of the silt curtain. For model runs that included a silt curtain, all flow was routed through the lowest box representing lake water at a location 50 feet from the shoreline to simulate flow under the curtain.

Table 2 lists the inputs to the model. The mass of silt in the average hourly volume of excavated material (8.4 cubic feet per hour per foot of shoreline [0.78 cubic meters per hour per meter of shoreline]) was determined by multiplying the total volume of excavated sediment by the assumed bulk dry density of 1,590 kilograms per cubic meter. It was assumed that 8% of the total mass was silt so that the load of silt into the lake was determined to be 99 kilograms per hour per meter of shoreline. Within a 3-meter-wide model box, the total hourly load is 299 kilograms.

The silt contents shown in Table 1 were measured using samples collected using a standard penetration test (SPT) split-spoon sampler. Consequently, only soil particles that were smaller than the inside diameter of the SPT sampler (1-3/8 inches) were collected and tested. Since the embankment material generally has a significant portion of coarse gravel to boulder size particles, the average silt content of the total material is less than 8%.

Grain-size testing has not been conducted on samples from the embankment at the snowshed. Based on the descriptions of soil in the boring logs, the excavated material at the snowshed is anticipated to have similar or coarser grain-size distribution compared to SCB.

3.0 RESULTS

Figure 1 shows the maximum concentrations of silt and the amount of silt deposited as a function of the distance from the shore in the lake for a single day. This figure reflects the potential movement of a sediment plume in the lake, including the distance a plume might migrate, the reduction of concentrations with distance from the shore, and the depths of maximum concentrations. Because the soil to be excavated is mainly coarse-grained, and the excavation does not occur 24 hours a day, sufficient time elapses for sediment to settle out of the water column, and the cumulative effects of multiple consecutive days of excavation would not exceed the effects from a single day of excavation.

The model indicates that more than half of the silt would be deposited within the first 10 feet from the shore, with over 90 percent deposited within 70 feet from the shore. Since it was assumed that 8 percent of the excavated material was silt, this means that less than one percent of the total volume of excavated material would be transported more than 70 feet into the lake.

The silt curtain limits the movement of sediment suspended higher in the water column, which allows a greater percentage of the material to settle out. The model indicates that the maximum concentrations of silt in the water column at distances over 70 feet from the shore would be less than 50 milligrams/liter with the silt curtain in place.

The model assumes that material would be placed directly into the lake. When material is excavated at elevations higher than the lake level, it will tumble some distance down the slope. Its momentum as it strikes the lake water will create some turbulence, which could increase transport by dispersion. It is expected that this effect would be limited to a few tens of feet from the shore. The silt curtain would limit effects beyond its assumed location of 50 feet from shore.

4.0 OTHER ISSUES

The potential increase in suspended sediment in the lake due to larger rocks tumbling down the slope toward the bottom of the lake was not included in the model. Neither were the affects of a thermocline. These issues are discussed below.

4.1 TUMBLING BOULDERS

It is expected that rock larger than about 12 inches in diameter will tend to tumble down the slope, with some of these rocks reaching the toe of the slope. A consideration is the possibility of these rocks impacting and suspending existing slope or lake bed material.

Soils encountered in borings drilled on the existing slope (H-1-73, H-5-74, H-6-98, H-8-98, H-14-98, and SCB-014-08) consist of coarse-grained soils (sand to boulder size) with small percentages of fine material. It is expected that any material that became suspended as a result of rocks tumbling down the slope would rapidly fall out of the water column.

Soils encountered in borings drilled in the lake bed in the vicinity of the slope toe have generally been coarse–grained sands and gravels with occasional boulders (SW2-008-08, SW2-009-08, and SW2-010-08) or plastic silts and clays (SCB-015-08). Where the lakebed soils are coarse-grained, it is expected that any material that became suspended as a result of rocks impacting the lake bed would rapidly fall out of the water column. Where the lakebed soils are plastic silts and clays, it is expected that these materials would not become suspended.

4.2 THERMOCLINE

It is anticipated that the work benches will first be excavated to the elevation of the high lake level. The work bench will then be brought down as the falling lake level allows. The lake level typically begins dropping around the beginning of July. However, in extreme high water years, the lake level has begun dropping as late as mid-August. Figure 2 shows the lake level fluctuation for an average year, a high water year (1996), and a low water year (2005).

During an average year, the lake level drops to the lowest bench elevation (El. 2480) by the middle of August. During an extraordinary high water year (1996), the lake level dropped to the lowest bench elevation by the middle of September. Assuming the bench is lowered as rapidly as the dropping lake level allows, side casting will be complete in mid-August to mid-September before the beginning of the lake "turnover," which typically occurs in November.

Should excavation occur at times when thermocline effects are present in the lake, it is unknown how the formation of the thermocline would be affected in the region of the lake between the shoreline and the silt curtain due to the construction activity in combination with the rapidly falling lake. If the thermocline were to form and remain during construction, it would only affect the transport of very fine silt and clay particles.

Table 1
Laboratory Testing Results for Slide Curve Bridge (SCB)

Boring No. & Sample Information				Water Content (%)	Grain Size Analysis		
Boring No.	Sample No.	Sample Depth (ft)	USCS/WSDOT Classification		Gravel (%)	Sand (%)	Silt + Clay (%)
SCB Area							
SCB-002-08	D-1	5.0-6.5	SW	9.7	45.9	46.1	8
	D-2	11.5-13.0	SW	9.7	45.9	46.1	8
	D-11	31.5-33.0	GW	7	53.2	38.2	8.6
	D-13	36.5-37.3	GW	7	53.2	38.2	8.6
SCB-004-08	D-2	2.5-4.0	SW	27.6	0.4	94.1	5.5
	D-8	17.5-19.0	GW	9.3	67.4	29.1	3.5
	D-10	22.5-22.9	GW	9.3	67.4	29.1	3.5
	D-12	27.5-29.0	SM	16.4	35.1	52.5	12.4
	D-14	32.5-34.0	SM-GM	13.6	44.3	44.4	11.3
SCB-005-08	D-1	5.0-6.5	GP	8.5	58.2	33.4	8.4
	D-2	10.0-10.2	GP	8.5	58.2	33.4	8.4
	D-4	16.5-18.0	GW	8.6	65	30.3	4.7
	D-9	31.5-33.0	GW	6.6	52.5	37.7	9.8
	D-11	36.0-37.5	SM	9.5	20.6	63.1	16.4
SCB-006-08	D-2	1.5-3.0	SW	16.4	26.9	67.7	5.4
	D-4	6.0-7.5	GP	10.9	74.3	22.5	3.3
	D-6	11.5-11.7	GP	10.9	74.3	22.5	3.3
	D-14	41.5-43.0	SW	14.8	43.9	46.5	9.5
	D-18	51.5-51.9	SM	13.1	30.7	56.8	12.5
SCB-007-08	D-2	5.0-6.5	GW	7	71.9	25.9	2.1
	D-6 a & b	26.5-28.0	GW	13	51.1	41.6	7.3
	D-13	56.5-57.2	SM	9.6	41.6	46.7	11.6
	D-15	61.5-63.0	SM	10.5	37.7	51	11.3
SCB-011-08	D-2	7.0-8.5	GW	6.7	55.2	37.8	7
	D-12	32.0-33.5	GP	11.9	66	27.6	6.4
	D-14	37.0-38.5	GP	11.9	66	27.6	6.4
SCB-021-08	D-2	7.0-8.5	GW	9.5	57.3	39.5	3.2
	D-5	17.0-18.5	GP	6.5	74.1	21.8	4.1
	D-7	22.0-23.4	GM	16.7	67.1	18.4	14.5

SUMMARY

Structure	Fines Content, %			
	Average	SD	Max	Min
SCB	7.8	3.7	16.4	2.1

Table 2
Inputs to Model

Parameter	Units	Value	Notes
Specific gravity of soil solids	--	2.65	
Bulk dry density of soil solids	kg/m^3	1,590	
Soil porosity	--	0.4	
Fraction of excavated soil assumed to be silt	--	8%	Average fines content from laboratory testing of soil samples at Slide Curve Bridge (see Table 1).
Loading rate of silt applied for 8 hours	kg/d (kg/hr)	7,179 (299)	Based on volume of 1.5-ft-thick lift of material removed over 45-foot depth and 1,100-foot length of the bench, assumed bulk dry density, and assumed fraction of silt applied to cell width of 3 meters. Within range of dispersion rates recommended for lakes.
Horizontal dispersion	m^2/s	0.01	
Inflow applied for 8 hours	m^3/s	0.00065	Based on volume of excavated material.
Silt settling velocity	m/s	0.00037	Based on Stokes settling velocity for particle diameter of 0.02 mm.
Silt resuspension rate	m/d	0	Assumed no resuspension.
Side slope of lake bottom (H:V)	--	1.2	

kg = kilogram(s)
m = meter(s)
d = day(s)
s = second(s)

Figure 1. Maximum Concentration of Silt in Suspension and Fraction of Silt Deposited in Keechelus Lake

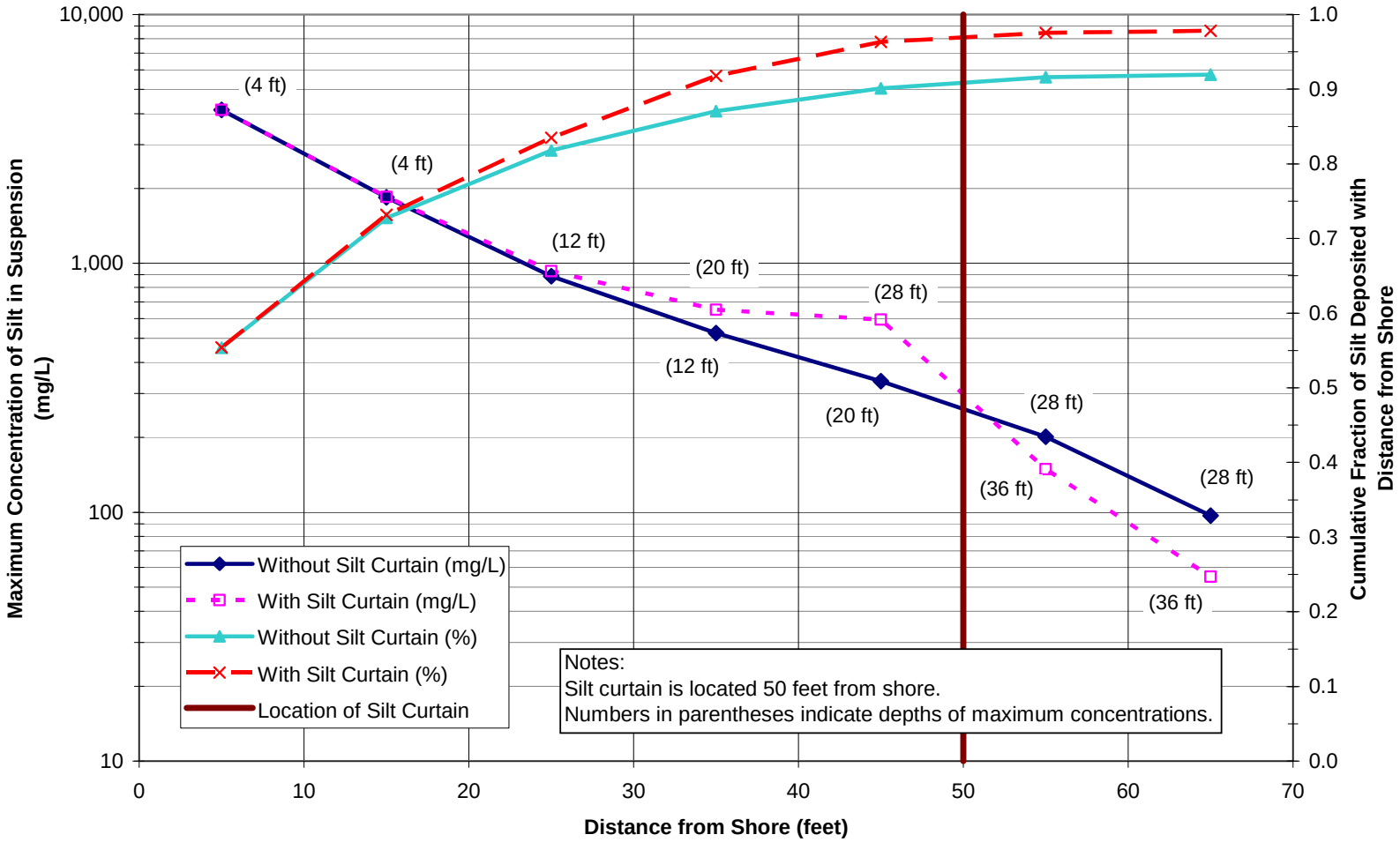
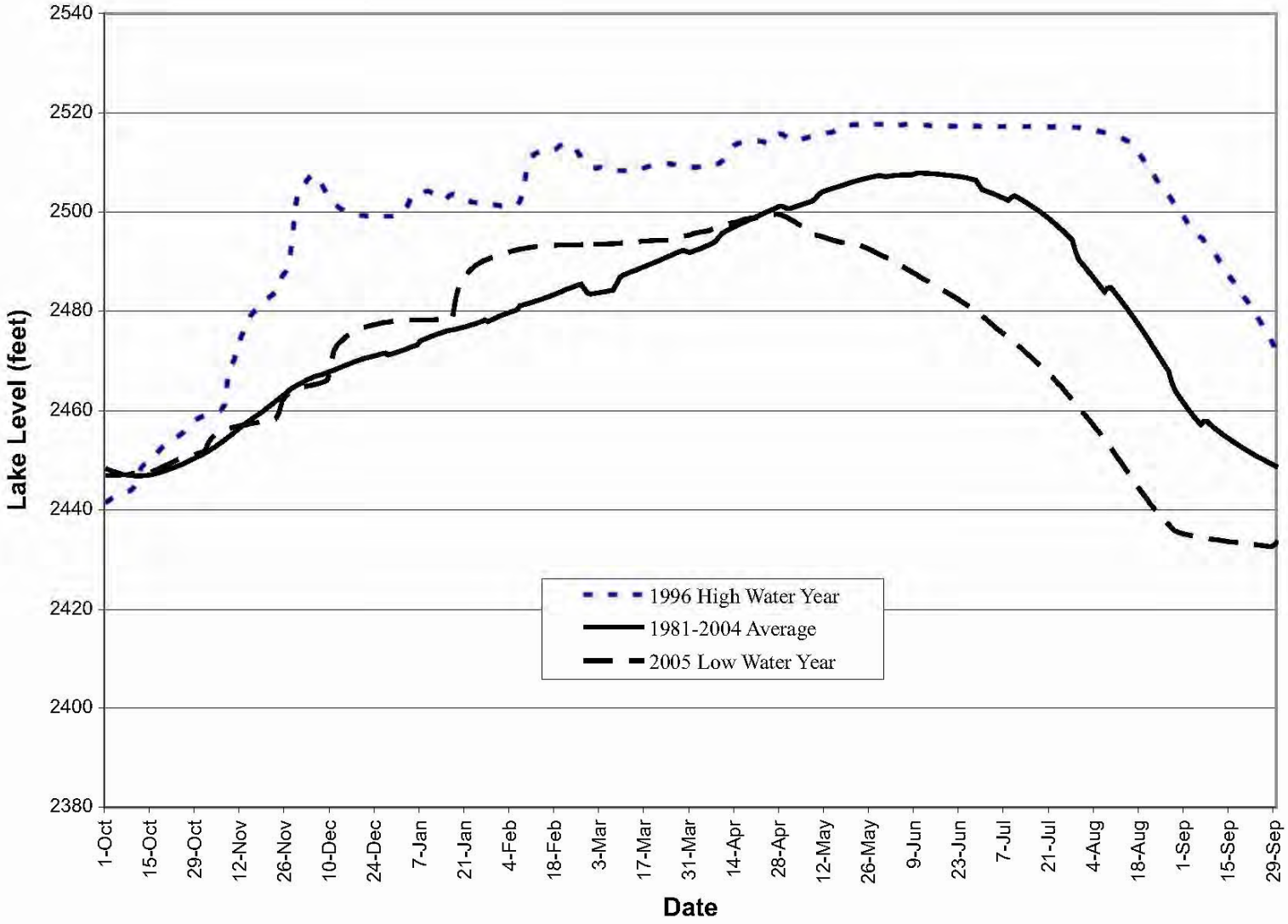


Figure 2. Keechelus Lake Reservoir Levels



Appendix K.1.4
Catchment Ditch Cut (Wall 1)

Dave Walker/Seattle/URSCorp
08/23/2010 01:31 PM

To "Golbek, Scott" <GolbekS@wsdot.wa.gov>
cc Chuck_Vita@URSCorp.com, "Cooper, Patrick" <CooperP@wsdot.wa.gov>, "Darwood, Doug" <DarwooD@wsdot.wa.gov>, "Hendrickson, Chase"
bcc
Subject RE: Wall 1 X-sec

This email discusses bedrock elevations along the north edge of a catchment that is proposed to be excavated west of the snowshed between approximately LW stations 1346+50 and 1352+50. The catchment will be at the base of the existing cut slope immediately north of the proposed WB shoulder. Sections of the proposed catchment are attached.

The south face of the catchment will be retained by proposed Wall 1. The purpose of this review was to help WSDOT assess whether overburden will need to be retained for excavation of the north face of the catchment.

The information reviewed includes:

- Cross section SSD 4-4' at LW 1352+66;
- Borings SSD-016-10 at LW 1350+95 and SSD-017-10 at LW 1351+91;
- The overburden soil nail wall plans; and
- Photographs of the existing cut face.

Bedrock is exposed in the existing cut slope west of approximately LW 1351+75 (Phase 1B Rock Slope Engineering Report, p. 58). In this area, bedrock exposed in the cut slope dips toward the east. It is assumed that bedrock continues to dip toward the east between LW 1351+75 and the east end of the catchment at LW 1352+50.. An interpretation of the bedrock surface perpendicular to the roadway has been made at LW 1352+66 (section SSD 4-4').

West of LW 1351+06, the overburden is retained by a soil nail wall. Between LW 1351+06 and 1351+75, it appears that the existing overburden thickness at the alignment of the north catchment face is less than five feet. This thickness was interpreted from the top of bedrock shown in the Phase 1B Rock Slope Engineering Report (p. 58) and should be field-verified.

East of LW 1351+75, lacking any more specific information, an idealized bedrock surface is interpreted by assuming that (1) the bedrock slope parallel to the road is uniform between LW 1351+75 and 1352+66 (2) the bedrock slope perpendicular to the road at all sections between LW 1351+75 and 1352+66 is the same as at LW 1352+66. The top of bedrock at the cut slope at LW 1351+75 is taken to be at the bottom of the existing ditch. Using these assumptions, the idealized bedrock surface can be plotted at any station between LW 1351+75 and 1352+66 using linear interpolation and the following plotting points:

Offset from LW, ft	Rock El. @ LW 1351+75	Rock El. @ LW 1352+66
72	2513	2508

-34	2538	2533
-46	2540	2535
-144	2600	2595

Wall 1 catchment sections.pdf

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Golbek, Scott" <GolbekS@wsdot.wa.gov>

"Golbek, Scott" <GolbekS@wsdot.wa.gov>
08/20/2010 03:11 PM

To <Dave_Walker@URSCorp.com>
cc "Cooper, Patrick" <CooperP@wsdot.wa.gov>, "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>, <Chuck_Vita@URSCorp.com>, "Hendrickson, Chase" <HendriC@wsdot.wa.gov>, "Darwood, Doug" <DarwooD@wsdot.wa.gov>
Subject RE: Wall 1 X-sec

Dave please proceed with providing the email opinions. Your request for 3 hours is approved for task order DO, 2.5.

From: Dave_Walker@URSCorp.com [mailto:Dave_Walker@URSCorp.com]
Sent: Wednesday, August 18, 2010 3:44 PM
To: Golbek, Scott; Hendrickson, Chase
Cc: Cooper, Patrick; Trudeau, Julee (Consultant); Chuck_Vita@URSCorp.com
Subject: Re: Wall 1 X-sec

Scott-

Chase and Julee are asking for bedrock information at Wall 1. It would be a significant effort to come up with cross sections of the bedrock surface, but I think an evaluation of rock elevations along the Wall 1 face should be adequate to evaluate whether the wall can be founded on rock and whether a soil nail wall may be needed.

My expectation is that we will find that bedrock is at shallow depth along the Wall 1 face. Based on this assumption, I estimate we could review the available information and write an email presenting our opinions and any data limitations for an effort of 3 hrs. The work could be done under TO DO, subtask 2.5.

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Hendrickson, Chase" <HendriC@wsdot.wa.gov>

"Hendrickson, Chase" <HendriC@wsdot.wa.gov>

To<Dave_Walker@URSCorp.com>

cc"Cooper, Patrick" <CooperP@wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>, "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>

08/18/2010 01:53 PM

SubjeWall 1 X-sect

Dave,

Attached are some x-sections that we would like to have the bedrock plotted on. It is for the area along Wall 1, west of the snowshed. We are concerned about the proposed tie-in to existing ground and if we will need a soil nail wall here or if we are close to bedrock. Please let Julee or myself know if you have any questions.

Chase

Chase Hendrickson, EIT

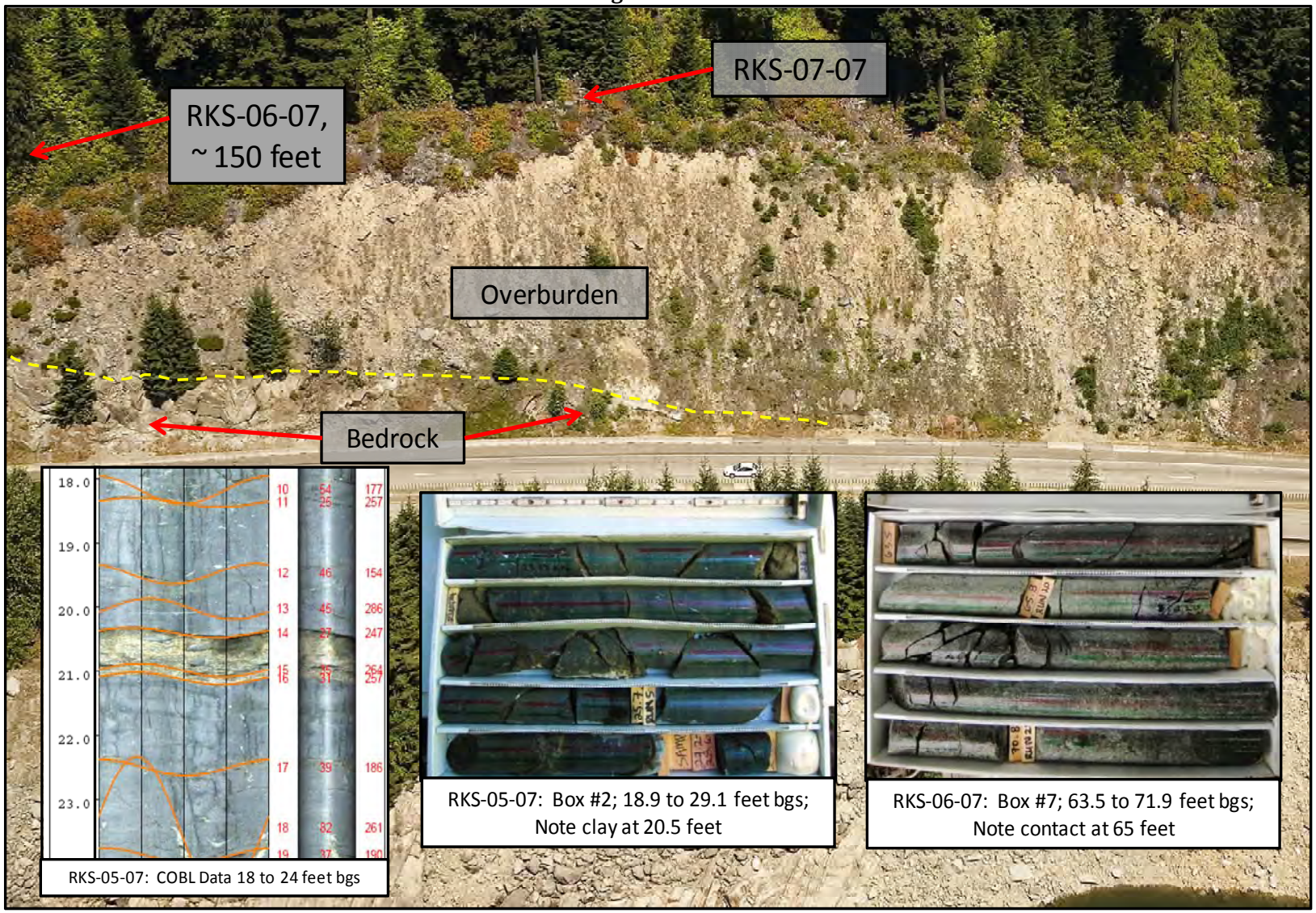
Transportation Engineer 1
I-90 Snoqualmie Pass East Project
HendriC@wsdot.wa.gov

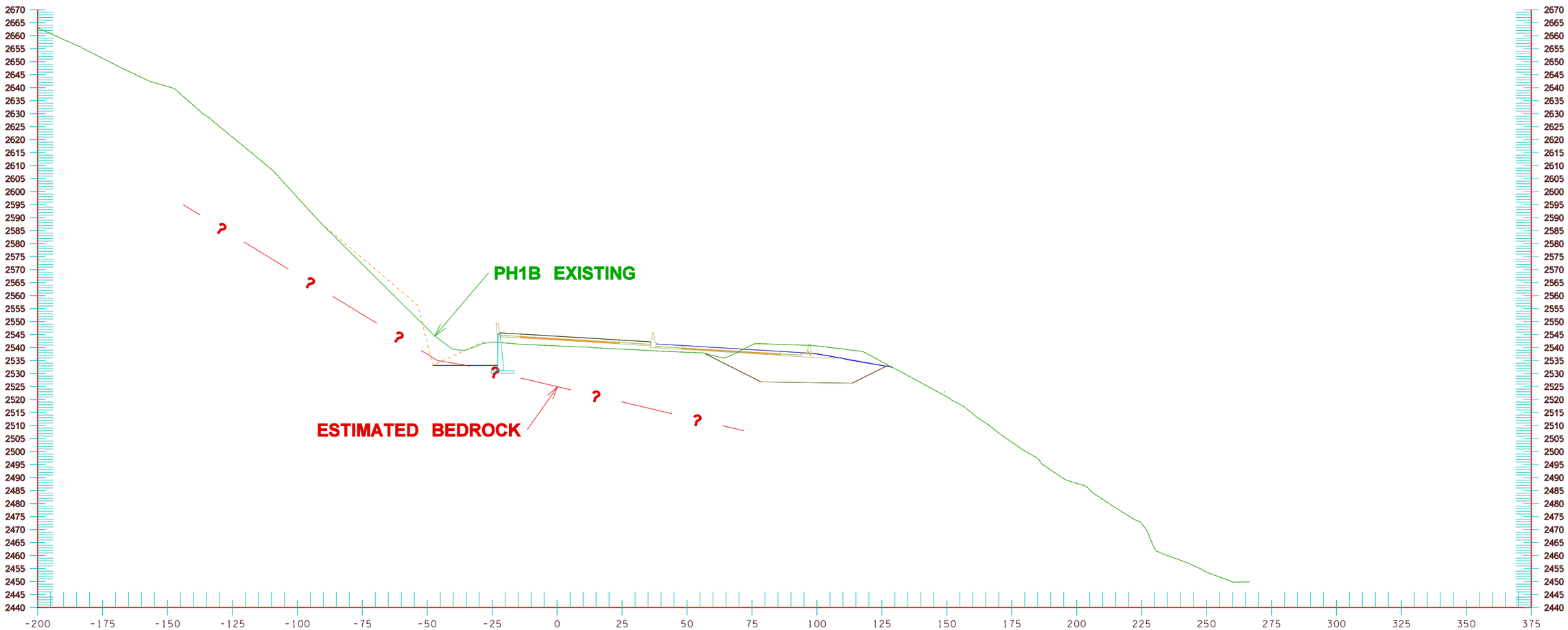
2809 Rudkin Road
Union Gap, WA 98903-1648

(509)-573-8331 (direct)
(See attached file: L2779C_PS_Wall1_R00B.PDF)

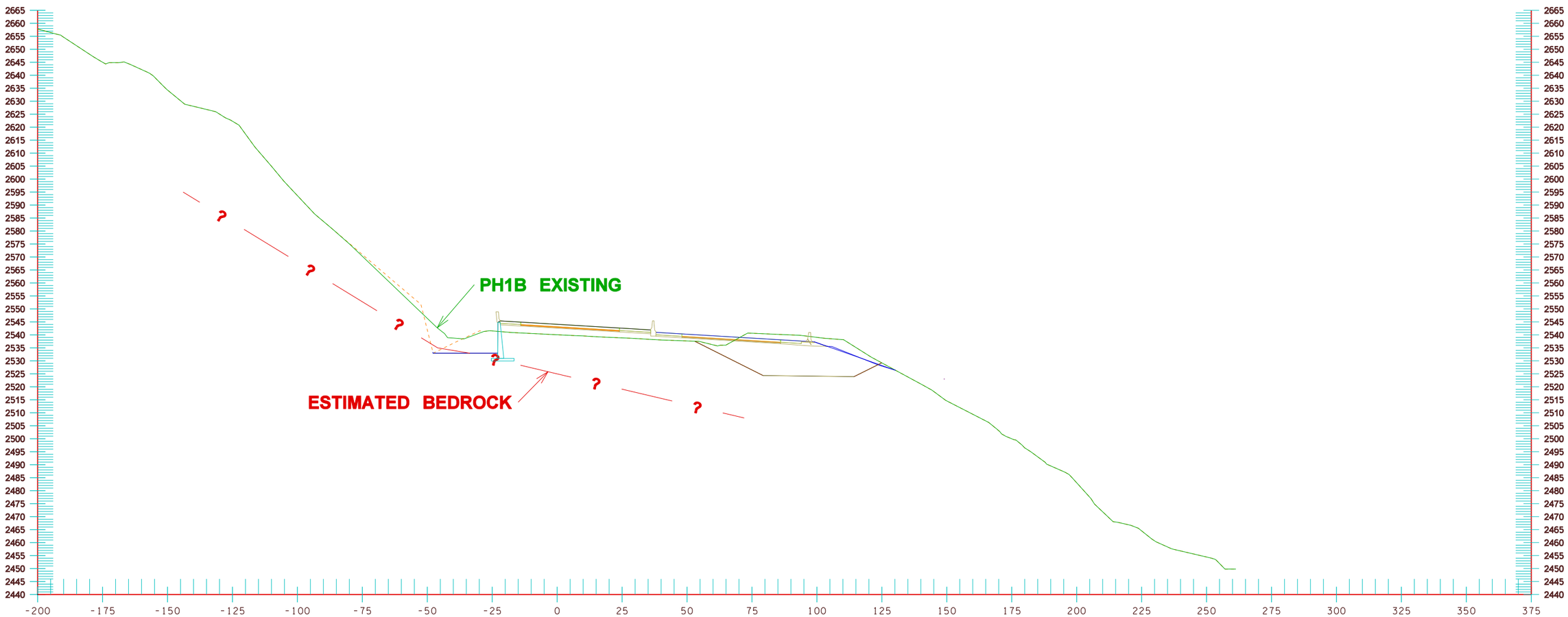
*** eSafe1 scanned this email for malicious content ***
*** IMPORTANT: Do not open attachments from unrecognized senders ***

Design Sector VIII





LW 1351+50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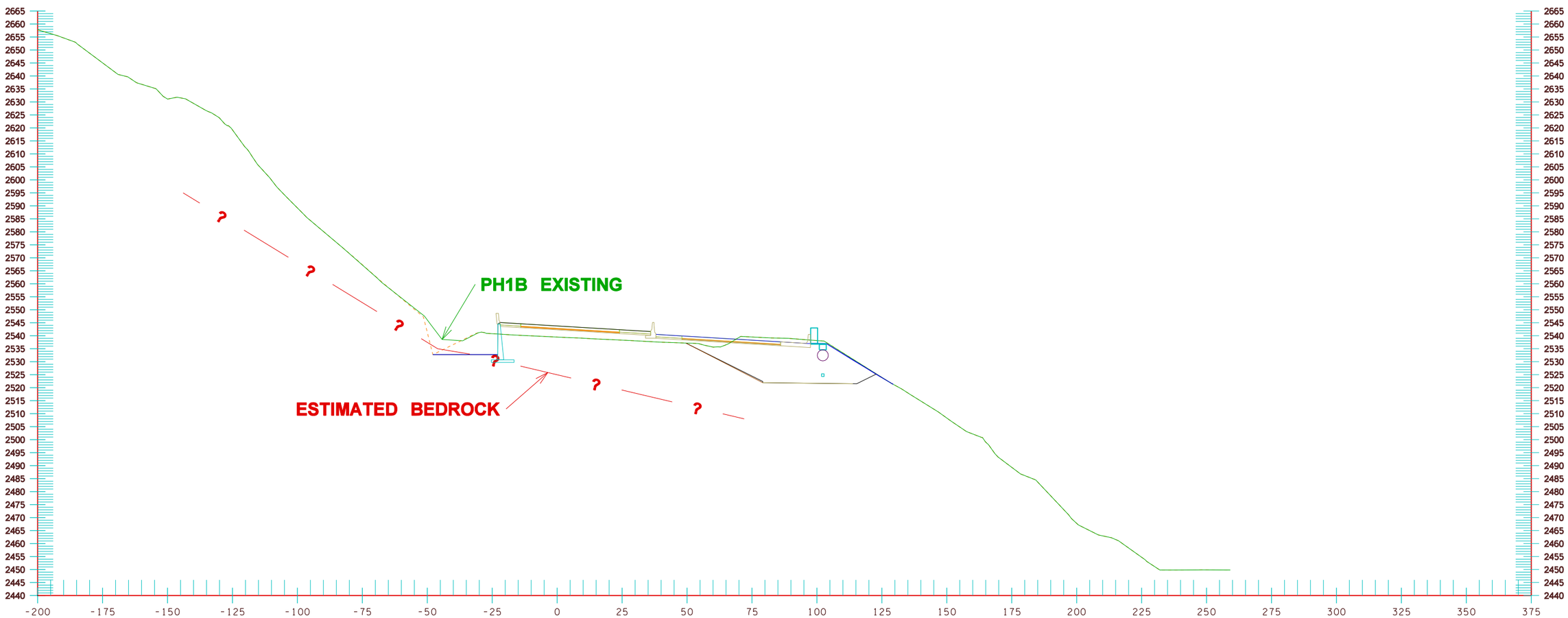
LW 1351+75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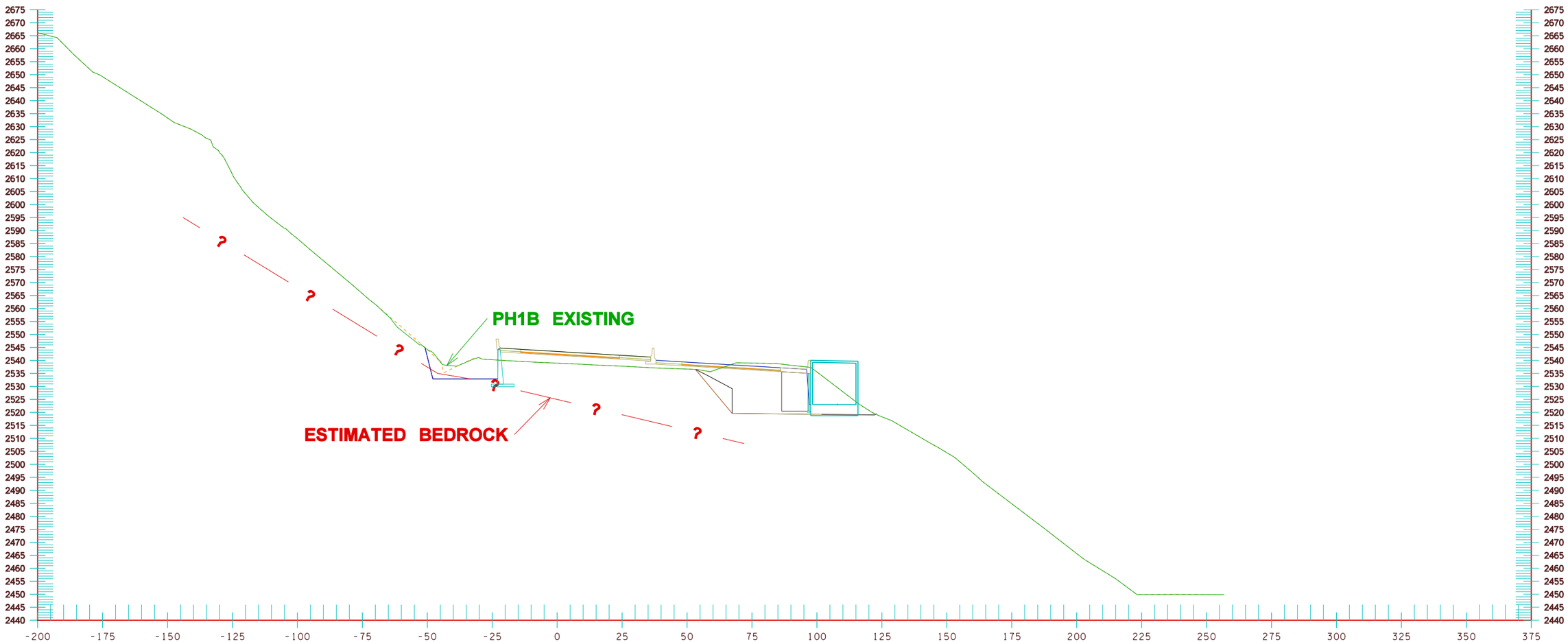
LW 1352+00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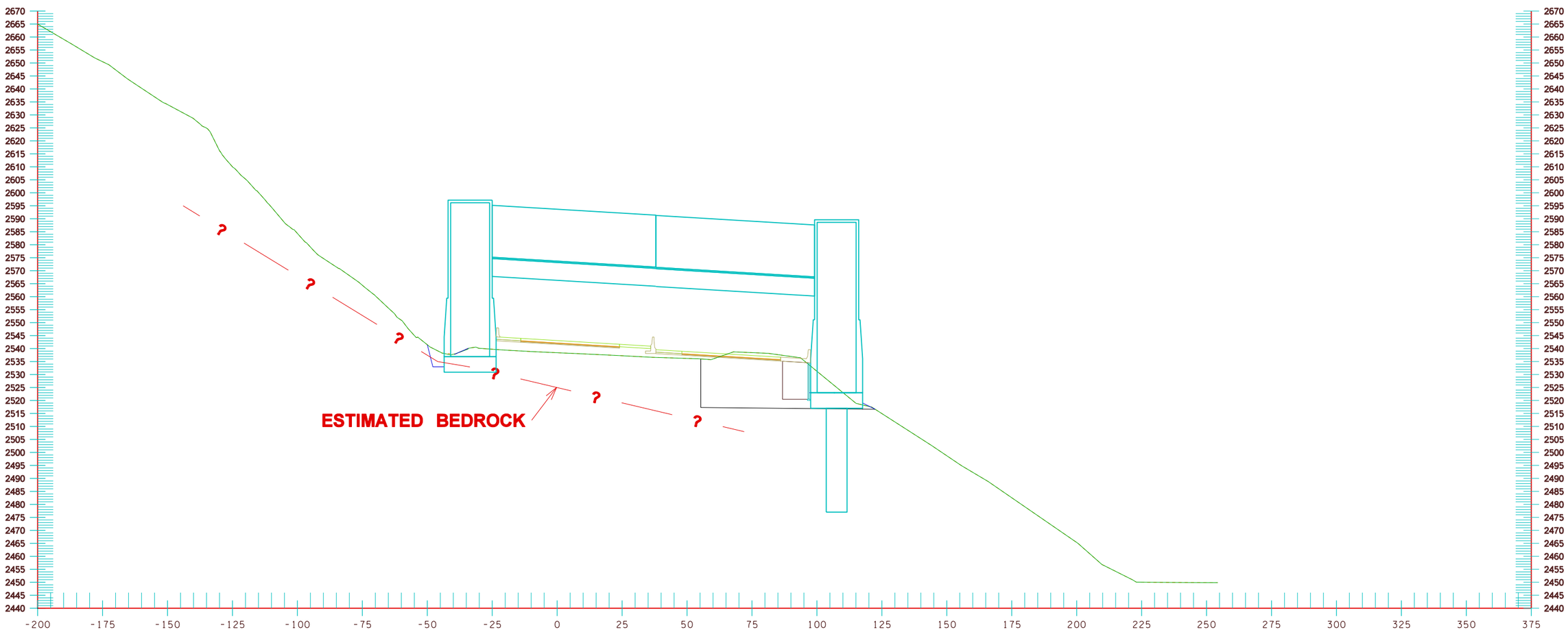
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LW 1352 + 50

Appendix K.1.5
Grade Separation Barrier

Dave Walker/Seattle/URSCorp
10/01/2010 02:25 PM

To "Tran, Luong" <TranLuo@wsdot.wa.gov>
cc Chuck_Vita@URSCorp.com, "Golbek, Scott" <GolbekS@wsdot.wa.gov>, I-90_SPE_Project@urscorp.com, "Jenkins, David V (MLO)"
bcc
Subject RE: I-90 Contract 1C Geotech Information for Grade Differential Barrier

Lou-

Attached are the July 6 email containing the grade separation barrier recommendations and the bearing resistance chart.

July 6, 2009 email.pdf Unfactored bearing resistance vs effective footing width.pdf

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Tran, Luong" <TranLuo@wsdot.wa.gov>

"Tran, Luong" <TranLuo@wsdot.wa.gov>
09/30/2010 08:43 AM

To <Dave_Walker@URSCorp.com>
cc "Golbek, Scott" <GolbekS@wsdot.wa.gov>, "Jenkins, David V (MLO)" <JenkinD@wsdot.wa.gov>, "Kendrick, Tad" <KendriT@wsdot.wa.gov>, <Chuck_Vita@URSCorp.com>, <I-90_SPE_Project@urscorp.com>
Subject RE: I-90 Contract 1C Geotech Information for Grade Differential Barrier

Hi Dave,

We do not have the July 6, 2009 information. Would you provide us a pdf or email of this information.

Thanks,

Lou

Luong(Lou) H. Tran, P.E.
Bridge Design Engineer
Bridge and Structures Office
(360)705-7195
PO Box 47340
Olympia,WA 98501
tranluo@wsdot.wa.gov

From: Dave_Walker@URSCorp.com [mailto:Dave_Walker@URSCorp.com]
Sent: Monday, September 27, 2010 8:56 AM
To: Tran, Luong
Cc: Golbek, Scott; Jenkins, David V (MLO); Kendrick, Tad; Chuck_Vita@URSCorp.com; I-90_SPE_Project@urscorp.com
Subject: Re: I-90 Contract 1C Geotech Information for Grade Differential Barrier

Lou-

URS provided recommendations for design of a grade-separation barrier up to 3 feet high and supported on compacted, coarse-grained embankment fill on July 6, 2009. Does your question relate to the snowshed? At the snowshed, the closest exploration we have to the proposed median is SSD-TP-001-08. Coarse-grained soil was encountered there. This material is probably representative of conditions at the proposed snowshed grade-separation barrier. In this case, the recommendations provided on July 6 would be applicable.

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Tran, Luong" <TranLuo@wsdot.wa.gov>

"Tra
n,

Luong" <TranLuo@wsdot.wa.gov>

09/23/2010 11:23 AM

To<Dave_Walker@URSCorp.com>

cc"Jenkins, David V (MLO)" <JenkinD@wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>, "Kendrick, Tad" <KendriT@wsdot.wa.gov>

SubjeI-90 Contract 1C Geotech Information for Grade Differential Barrier ct

---

Hi Dave,

I am sure that URS has produced any recommendation for the Grade Differential Barrier foundation yet, we had the information for contract 1B. Can we use the same information from 1B?

Regards,

Lou

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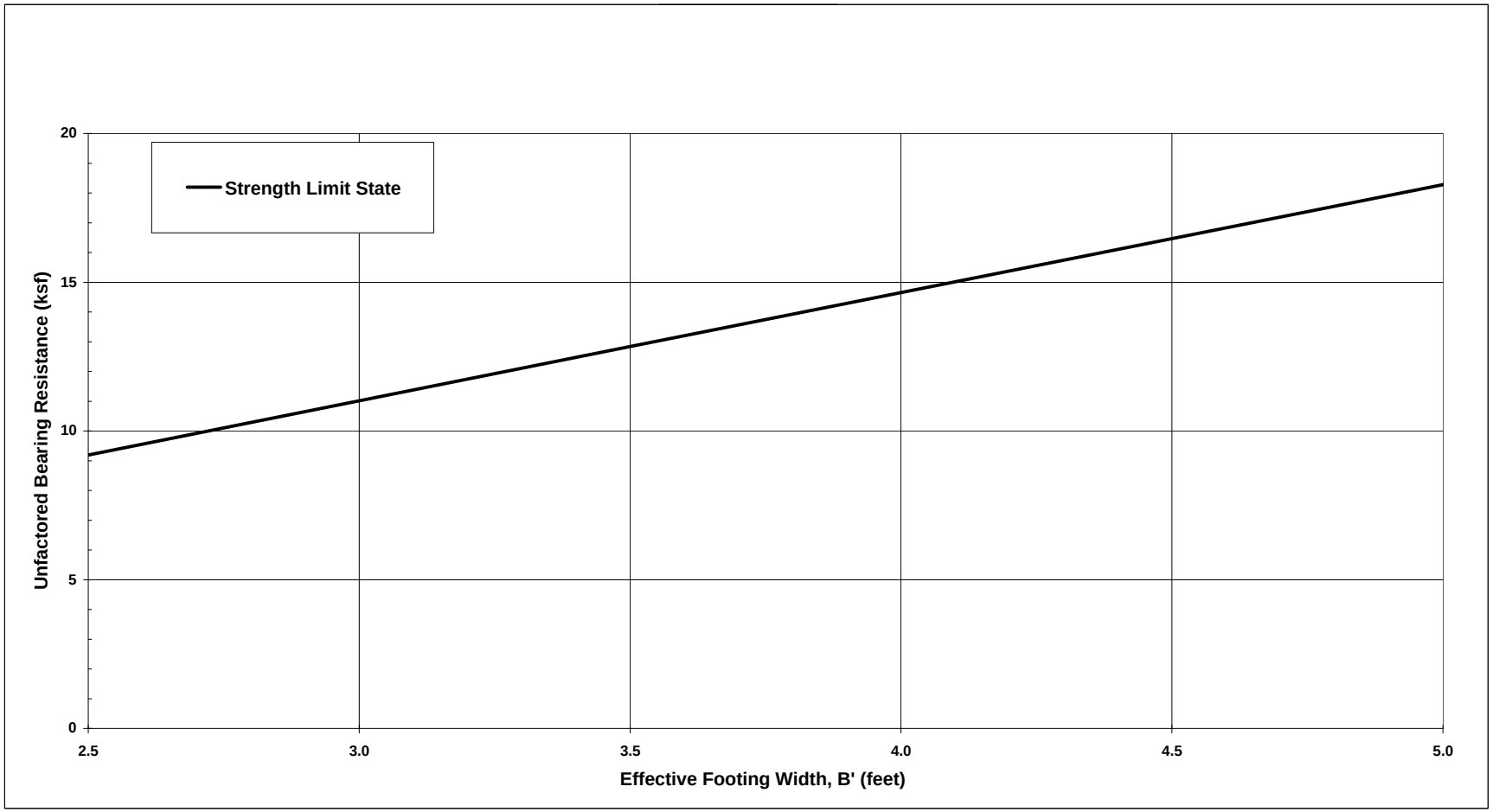

Notes:

1. Effective wall length (L') =200 feet.
2. Groundwater assumed to be at El. 2517

Unfactored Bearing Resistance vs Effective Footing Width

Job No.: 33758638

Grade Separation Median Barrier, WB Stas. 1200+00 to 1208+00 and 1318+00 to 1336+00

URS

WSDOT I-90 Snoqualmie Pass East
2008 Geotechnical Analyses & Report

SB - 7/6/2009

P:\26120 EMB\26124 Engineering Analyses\26124.08 Phase 1B Grade Separation\LRFD Spread Footing No overburden rev1 - LRFD Spread Footing No overburden rev1

Dave Walker/Seattle/URSCorp
10/01/2010 02:25 PM

To "Tran, Luong" <TranLuo@wsdot.wa.gov>
cc Chuck_Vita@URSCorp.com, "Golbek, Scott" <GolbekS@wsdot.wa.gov>, I-90_SPE_Project@urscorp.com, "Jenkins, David V (MLO)"
bcc
Subject RE: I-90 Contract 1C Geotech Information for Grade Differential Barrier

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July 6, 2009 email.pdf Unfactored bearing resistance vs effective footing width.pdf

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1501 4th Avenue, Suite 1400
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"Tran, Luong" <TranLuo@wsdot.wa.gov>

"Tran, Luong" <TranLuo@wsdot.wa.gov>
09/30/2010 08:43 AM

To <Dave_Walker@URSCorp.com>
cc "Golbek, Scott" <GolbekS@wsdot.wa.gov>, "Jenkins, David V (MLO)" <JenkinD@wsdot.wa.gov>, "Kendrick, Tad" <KendriT@wsdot.wa.gov>, <Chuck_Vita@URSCorp.com>, <I-90_SPE_Project@urscorp.com>
Subject RE: I-90 Contract 1C Geotech Information for Grade Differential Barrier

Hi Dave,

We do not have the July 6, 2009 information. Would you provide us a pdf or email of this information.

Thanks,

Lou

Luong(Lou) H. Tran, P.E.
Bridge Design Engineer
Bridge and Structures Office
(360)705-7195
PO Box 47340
Olympia,WA 98501
tranluo@wsdot.wa.gov

From: Dave_Walker@URSCorp.com [mailto:Dave_Walker@URSCorp.com]
Sent: Monday, September 27, 2010 8:56 AM
To: Tran, Luong
Cc: Golbek, Scott; Jenkins, David V (MLO); Kendrick, Tad; Chuck_Vita@URSCorp.com; I-90_SPE_Project@urscorp.com
Subject: Re: I-90 Contract 1C Geotech Information for Grade Differential Barrier

Lou-

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1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Tran, Luong" <TranLuo@wsdot.wa.gov>

"Tra
n,

Luong" <TranLuo@wsdot.wa.gov>

09/23/2010 11:23 AM

To<Dave_Walker@URSCorp.com>

cc"Jenkins, David V (MLO)" <JenkinD@wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>, "Kendrick, Tad" <KendriT@wsdot.wa.gov>

SubjeI-90 Contract 1C Geotech Information for Grade Differential Barrier ct

---

Hi Dave,

I am sure that URS has produced any recommendation for the Grade Differential Barrier foundation yet, we had the information for contract 1B. Can we use the same information from 1B?

Regards,

Lou

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Appendix K.1.6
Lateral Earth Pressure for Water Tank Vault

Dave Walker/Seattle/URSCorp
03/04/2011 01:48 PM

To Gustavo Franco/Seattle/URSCorp@URSCorp
cc golbeks@wsdot.wa.gov, binglej@wsdot.wa.gov
bcc
Subject Lateral earth pressure for water tank vault

Gustavo-

Please used the attached figure for calculating the lateral earth pressures on the water tank vault. This figure supersedes that provided on 11/5/2010. It differs from the previous version by having an updated value for the avalanche debris load, q. This value was updated based on the upslope vault wall being further left of LW than previously assumed. The value of q was interpreted from the Avalanche Mitigation Report, Figure 2-7 for the East Shed 2 avalanche path.

Water tank vault LEP diagram 030411.pdf

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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Dave Walker/Seattle/URSCorp
11/05/2010 09:00 AM

To "Tran, Luong" <TranLuo@wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>
cc "Cooper, Patrick" <CooperP@wsdot.wa.gov>, trudeaj@consultant.wsdot.wa.gov, Chuck Vita/Seattle/URSCorp@URSCORP, jenkind@wsdot.wa.gov
bcc
Subject Re: TO DO - Snowshed water tank lateral earth pressures

Lou-

The attached figure shows recommended unfactored lateral earth pressures and load factors for design of a buried water tank vault at the snowshed. The value of H2, which represents the location of the water table, can be evaluated separately for the strength and extreme event limit states, as appropriate. It may also be taken as 0 if the vault backfill is completely drained. In our opinion, H2=0 is appropriate for the extreme event limit state.

The avalanche surcharge load was interpreted from Figure 2-7, East Shed 2 100-Year Avalanche Debris Static Loading, of the Avalanche Mitigation Report.

Please contact us if you have questions.

Water tank vault LEP diagram 110510.pdf

Dave Walker, P.E.
URS Group, Inc.
1501 4th Avenue, Suite 1400
Seattle, WA 98101-1616
Direct: 206-438-2350
Fax: 1-866-495-5288

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"Golbek, Scott" <GolbekS@wsdot.wa.gov>

"Golbek, Scott" <GolbekS@wsdot.wa.gov>
11/04/2010 11:52 AM

To <Dave_Walker@URSCorp.com>, "Cooper, Patrick" <CooperP@wsdot.wa.gov>, "Tran, Luong" <TranLuo@wsdot.wa.gov>
cc "Darwood, Doug" <DarwooD@wsdot.wa.gov>
Subject Re: TO DO - Snowshed water tank lateral earth pressures

Approved.

From: Dave_Walker@URSCorp.com <Dave_Walker@URSCorp.com>

To: Golbek, Scott; Trudeau, Julee (Consultant); Cooper, Patrick; Tran, Luong
Sent: Thu Nov 04 11:14:23 2010
Subject: TO DO - Snowshed water tank lateral earth pressures

Scott-

As discussed at our meeting on 11/3, URS can provide lateral earth pressures for design of a buried water tank vault at the snowshed. Our estimated LOE to complete this work is 2 hours. The work can be done under TO DO, subtask 2.5.

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Unfactored pressures

$P_1 = K_o \gamma H_1$

$P_2 = K_o \gamma (H - H_2)$

$P_3 = P_2 + K_o(\gamma - \gamma_w) H_2$

$P_4 = K_o q$

$P_5 = \gamma_w H_2$

$P_6 = 0.5 K_{ae(r)} \gamma H$

$P_7 = P_6 - K_o \gamma_w H_2$

$\gamma = 135$ pcf

$\gamma_w = 62.4$ pcf

$K_o = 0.36$

$K_{ae(r)} = 0.51$

$q = 1{,}650$ psf (estimated avalanche debris loading for East Shed 2)

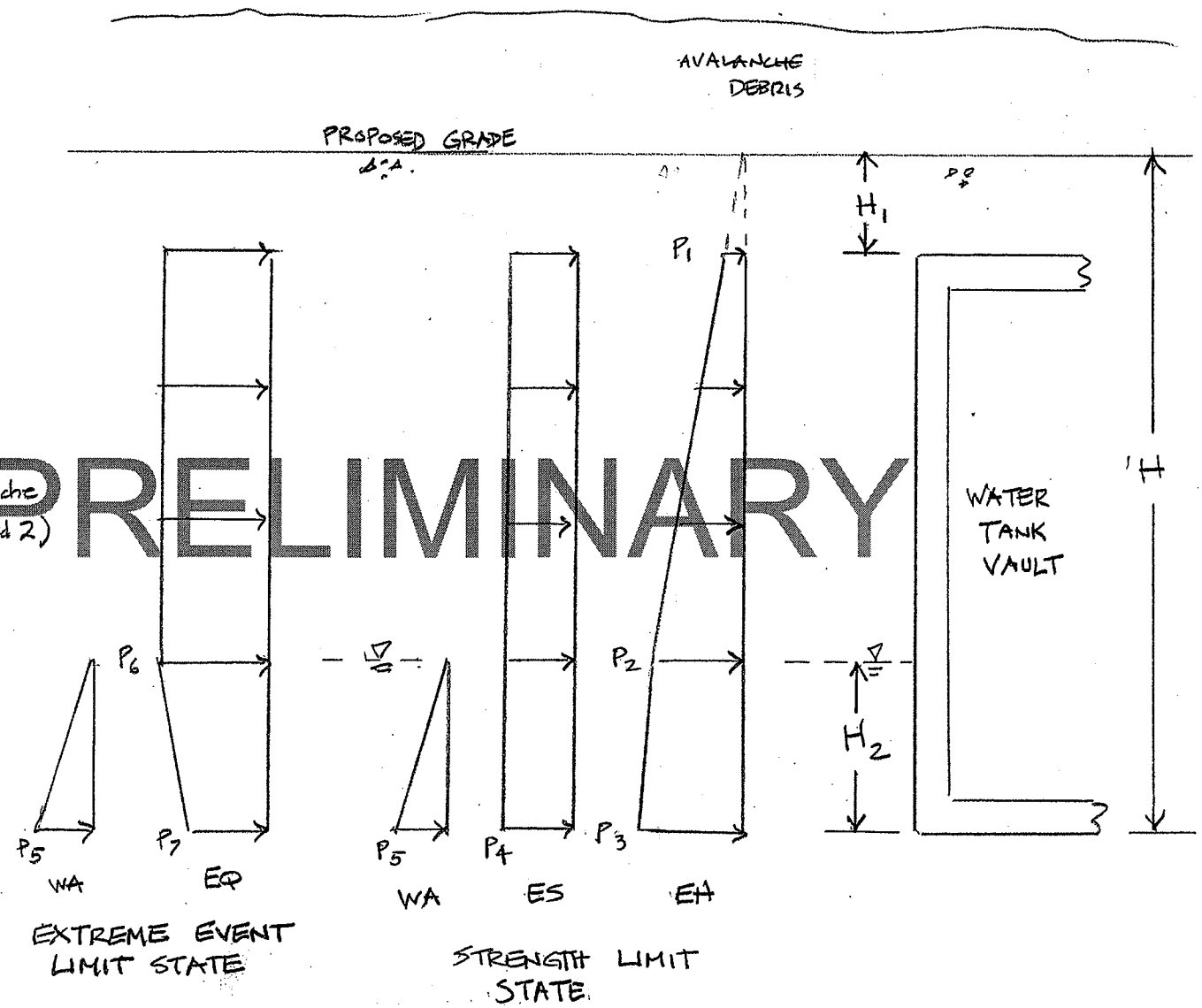
Load factors

$\gamma_{EH} = 1.35$

$\gamma_{ES} = 1.5$

$\gamma_{WA} = 1.0$

$\gamma_{EQ} = 1.0$



PRELIMINARY

LATERAL EARTH PRESSURES FOR WATER TANK VAULT

Appendix K.1.7
Wall 9D Rock Elevation and Bearing Capacity

Dave Walker/Seattle/URSCorp
11/05/2010 02:47 PM

To "Hendrickson, Chase" <HendriC@wsdot.wa.gov>
cc "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>, I-90 SPE Project/App/URSCorp@URSCorp
bcc
Subject RE: FW: Wall 9C and D

Chase-

Thanks for letting me review this. I'd like to adjust the information I gave you slightly. I made a simplifying assumption that makes more difference than I thought it would. I've changed the offset of the first point. This change makes the bedrock profile consistent with the interpretation in section SSD 4-4'.

W9D Station	Wall 9D Offset from LE, ft	Rock Elevation, ft
10+86.47	-114	2532
9+67.55	-216	2595
9+33.48	-249	2609

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"Hendrickson, Chase" <HendriC@wsdot.wa.gov>

"Hendrickson, Chase" <HendriC@wsdot.wa.gov>
11/05/2010 10:46 AM

To <Dave_Walker@URSCorp.com>
cc "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>
Subject RE: FW: Wall 9C and D

Dave,

Attached is an updated profile with your information incorporated. Just wanted to give you a chance to look at it before I sent it off to bridge to make sure this matched your recommendation.

W9D Station	Wall 9D Offset from LE, ft	Rock Elevation, ft
10+86.47	-105	2532
9+67.55	-216	2595
9+33.48	-249	2609

Thanks,

Chase

From: Dave_Walker@URSCorp.com [mailto:Dave_Walker@URSCorp.com]
Sent: Friday, November 05, 2010 9:15 AM
To: Trudeau, Julee (Consultant); Hendrickson, Chase; Golbek, Scott
Cc: Chuck_Vita@URSCorp.com; I-90_SPE_Project@urscorp.com
Subject: RE: FW: Wall 9C and D

Julee, Chase-

The table below can be used to estimate bedrock elevations below Wall 9D. The table is based on where bedrock was encountered in borings SSD-013-09, SS-018-10, and SSD-019-10. Linear interpolation can be used for locations between these points.

Wall 9D Offset from LE, ft	Rock Elevation, ft
-105	2532
-216	2595
-249	2609

For allowable bearing pressure, we recommend using the same values as provided for Walls 9A, B, and C, which are:

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

The Wall 9D bearing pressures and the loads from the avalanche debris that will accumulate behind Wall 9D must be accounted for in the design of the MSE walls. If the wall foundation spans rock and soil (including wall backfill), the potential exists for differential settlement. This could result in some cracking of the wall.

Please contact us if you have questions.

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"Golbek, Scott" <GolbekS@wsdot.wa.gov>

11/03/2010 01:31 PM

To <Dave_Walker@URSCorp.com>
cc "Darwood, Doug" <DarwooD@wsdot.wa.gov>
Subject RE: FW: Wall 9C and D

Your LOE request is approved.

From: Dave_Walker@URSCorp.com [mailto:Dave_Walker@URSCorp.com]
Sent: Wednesday, November 03, 2010 9:53 AM
To: Golbek, Scott; Trudeau, Julee (Consultant); Hendrickson, Chase; Cooper, Patrick
Subject: Re: FW: Wall 9C and D

Scott-

We will provide the estimated bedrock profile in tabular format for Wall 9D offset from LE. Julee and Chase also requested allowable bearing presures for Wall 9D on soil and rock on Nov. 2. The total estimated LOE for this work is 2 hrs. The work can be completed under TO DO, subtask 2.5.

Dave Walker, P.E.
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"Hendrickson, Chase"
<HendriC@wsdot.wa.gov>

11/01/2010 10:58 AM

To <Dave_Walker@URSCorp.com>
cc "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>
Subject FW: Wall 9C and D

Julee and Scott requested that I send you this information to provide us bedrock information for Lou. The wall limits are as shown on the profile W9D 9+93.36 to W9D 10+73.97. Hopefully you can provide us bedrock information based on the profile with station/elevation.

Thanks,

Chase

From: Hendrickson, Chase
Sent: Friday, October 29, 2010 2:12 PM
To: Tran, Luong

Cc: Trudeau, Julee (Consultant); Nash, Phil; Stoddard, Richard; Waldron, Glenn; Golbek, Scott
Subject: RE: Wall 9C and D

Lou,

Attached is a profile and plan view of what we have for Wall 9D. Please let me know if you have any questions.

Chase

From: Trudeau, Julee (Consultant)
Sent: Friday, October 29, 2010 9:07 AM
To: Hendrickson, Chase
Cc: Golbek, Scott
Subject: FW: Wall 9C and D

Can you create a profile here? Then we will probably need to get Dave Walker's recommendation on soil upslope of wall 9D.

JBT

Julee Trudeau, PE
Transportation Engineer
I-90 Snoqualmie Pass East Project
trudeaj@wsdot.wa.gov

WSDOT SCR
c/o I90 Project - Julee Trudeau
2809 Rudkin Road
Union Gap, WA 98903-1648

509-577-1890 (direct)

509-577-1869 (fax)

From: Tran, Luong
Sent: Friday, October 29, 2010 9:05 AM
To: Trudeau, Julee (Consultant)
Cc: Nash, Phil; Stoddard, Richard; Waldron, Glenn
Subject: Wall 9C and D

Hi Julee,

We would like to request for more information on wall 9D, what it the existing /proposed ground line of area beyond wall 9C into the hillside, where wall 9D sits on top? Can we excavate to bottom of wall 9D footing about 30' from top? Soil or Rock? If the footing is on rock we will have different design than on soil?

We need Geotech information/recommendation for this portion of the footing either on soil or rock? Bearing pressure.

Thanks,

Lou

[attachment "10-29-10 Wall 9D Profile.pdf" deleted by Dave Walker/Seattle/URSCorp] [attachment "10-29-10 Wall 9D Plan.pdf" deleted by Dave Walker/Seattle/URSCorp] [attachment "10-29-10 Wall 9D Profile.dgn" deleted by Dave Walker/Seattle/URSCorp]

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*** IMPORTANT: Do not open attachments from unrecognized senders ***

*** eSafe1 scanned this email for malicious content ***
*** IMPORTANT: Do not open attachments from unrecognized senders
***[attachment "11-05-10 Wall 9D Profile.pdf" deleted by Dave Walker/Seattle/URSCorp]

APPENDIX K.2
Resort Creek Bridges

Appendix K.2.1
Flow Failure Loads on Shafts

Chuck Vita/Seattle/URSCorp

03/06/2009 09:13 AM

Φιλεσ Ατταχηεδ: **0**
Τοταλ Εμαιλ Σιζε: **9 κβ**

To
JenkinD@WSDOT.WA.GOV
cc
GilesR@WSDOT.WA.GOV, GolbekS@WSDOT.WA.GOV, WoodJe@WSDOT.WA.GOV, "Cooper, Patrick" <CooperP@wsdot.wa.gov>, LewisR@WSDOT.WA.GOV, Dave Walker/Seattle/URSCorp@URSCORP, Cecil Urlich/Seattle/URSCorp@URSCORP, Vivianne Knight/Seattle/URSCorp@URSCORP, Chuck Vita/Seattle/URSCorp@URSCORP
bcc

Subject
RCB—Potential Lateral Loads at Abutments Due to Liquefaction-Induced Flow Sliding (Seismic Slope Failure)

Purpose of This Email: Brief Background Summary

his email responds to a WSDOT request made during GTM CP-4 (January 22, 2009) on Resort Creek Bridge and Wall, RCB and RCW. The GTM discussion points included URS' finding of a longitudinal residual factor of safety (rFS) of less than 1.0 and the potential for liquefaction flow sliding (or severe lateral spreading) at the RCB abutments, and the very real construction problems associated with remedial ground stabilization at the site that WSDOT wants to avoid.

As a result, URS was asked to provide lateral force estimates that WSDOT Bridge could use to determine if the bridge abutments could be made beefy enough to resist potential longitudinal lateral forces associated with liquefaction and flow sliding at the RCB abutments. The ideal was (and is) to see if the bridge can be made to resist the imposed forces so that the liquefied sands and overlying gravel embankment "crust" flows around and past the shafts and abutment structure. The lateral forces were to assume no ground stabilization.

Advancing the Discussion

This email provides very preliminary, simplified calculations of the potential lateral forces. The calculated forces also provide an opportunity for subsequent clarifying discussion, in light of updated GDM Section 6 on Seismic Design (Nov 2008), on how WSDOT wants to proceed on the project with analysis and evaluation of global stability for bridges and structural walls underlain by liquefiable soil -- significant problems at both GCB (Gold Creek Bridge) and RCW, besides RCB. This email only opens discussion.

Calculated Lateral Force Needed for rFS=1.1 by SLOPE/W: 800 kips per 8-ft Diameter Shaft

Following GDM 6.5.4.2, we ran SLOPE/W residual-strength (Sr) analyses at the RCB abutments with Sr in the liquefied sands based on Kramer's method (WA-RD 668.1, 2008). To achieve a target rFS=1.1 SLOPE/W calculated a needed lateral force of approximately 50 kpf (kips per ft) in the longitudinal direction, normal to the abutment.

The calculated 50 kpf results in a lateral force of 800 kps (kips per shaft) assuming a shaft diameter B=8 ft and a simple estimate of 2B for the width of influence of the flowing liquefied soils on the shafts (i.e., 50 kpf * 2*8-ft = 800 kps). The 2B estimate is preliminary.

The 800 kps results in a stress of 20 kips per vertical ft of shaft assuming the force is distributed as a uniform stress over the liquefied zone -- which extends about 40 ft between El 2480 and El 2440 at the west abutment.

Interpretation of the SLOPE/W results should consider that the limit equilibrium force-based analyses do not capture any dynamic or displacement effects of the flow sliding (or lateral spreading), including the likely displacement discontinuity at and near the interface between the liquefied sands and the overlying gravel crustal embankment. In particular, there is no explicit consideration of passive load imposed by the crustal layer, which is included in the following force-based analysis using the Japanese Road Association "Specification for Highway Bridges" (JRA 1996) as included in GDM 6.5.4.2.

Calculated Lateral Force by Japanese Highway Codes: 2,300 to 5,000 kips per 8-ft Diameter Shaft

Calculations were also made using the Japanese codes as a point of comparison with the SLOPE/W force of 800 kps. As GDM 6.5.4.2 states, Japanese codes specify the following lateral pressures on pile foundations due to liquefaction:

- 30% of total overburden pressure in the liquefied zone.
- Full passive pressure on the foundation system in the nonliquefied crustal layers.

The Japanese-code lateral pressures resulted in a calculated force of 2,300 kps assuming an 8-ft foundation width (shaft diameter), a 50-ft foundation height in the embankment between El 2480 and ~El 2530, and a passive pressure coefficient of Kp=1.0. A Kp=1.0 was reduced to account for a potential less-than ultimate passive pressure in the abutments. Recognizing that Kp=1.0 might not be appropriate, a more conservative Kp=3.0 resulted in a total lateral force of 5,000 kps. Note that these calculated forces do not include the full width of the abutment, but only the diameter of the shafts at B=8ft.

The Calculated Forces Do Not Consider Displacements

The above force-based approaches do not explicitly consider soil or foundation displacements. They implicitly assume that the foundation system is strong and stiff enough that the soil moves around and past the foundation system, and that in the absence of restraint from the foundation system the soil lateral displacements are relatively "very large."

Presumptive Free-Field Displacements for Displacement-Based Approaches

We have not calculated potential free-field lateral displacements, partly because such calculations are outside the current state of common practice for flow slides (URS is capable of making these calculations based on limit-equilibrium methods or FLAC analysis). At this time, we can, however, make very rough presumptive simplified estimates based on engineering judgment that may be of use for discussion purposes or for WSDOT to make exploratory calculations of potential foundation behavior.

Loosely following GDM 6.5.4.1 Option 3c, we would estimate on the order of 3 ft of displacement in the nonliquefied crust above El 2480. The lateral displacement would decrease linearly in the liquefied layer from 3 ft at El 2480 to zero at El 2440. These simple presumptive displacement estimates are idealized to occur longitudinally, normal to the abutments. Actual free field displacements would likely be spatially variable and complex, including both lateral and vertical movements both normal and oblique to the abutments.

Follow-up Discussion is Needed

While this email does not answer all the questions, it does provide some specifics for focused discussion during our upcoming Monday conference call on GCB scope and next Wednesday's GTM on RCB and RCW.

Regards,
Chuck

Chuck Vita, PhD, PE, GE
Principal Engineer
URS Corp, Century Square, 1501 4th Ave #1400, Seattle, WA 98101-1616
Office Phone: 206-438-2348. Cell: 206-390-9958.

APPENDIX K.3
Overburden Cut

Appendix K.3.1
Overburden Cut AW 112+70 to 118+15

Dave Walker/Seattle/URSCorp
10/04/2010 08:21 AM

To "Golbek, Scott" <GolbekS@wsdot.wa.gov>, TrudeaJ@consultant.wsdot.wa.gov, "Gipner, Mathew" <GipnerM@wsdot.wa.gov>
cc "norm norrish" <nnorrish@msn.com>, Mark Molinari/Seattle/URSCorp@URSCORP, I-90 SPE Project/App/URSCorp@URSCorp
bcc
Subject Re: I-90- AW 112+70 - AW 118+15 cut slopes

Julee-

WSDOT is proposing to make 1.5H:1V cuts on the north side of the road between AW 112+70 and 118+15, as shown in the attached files. This 545-foot long cut begins immediately east of the MP 59.7 Unnamed Creek crossing. The maximum cut height is approximately 48 feet at LW 116+50. The existing slopes above the proposed cuts are about 3H:1V or flatter. Cross sections of the proposed cut are attached.

URS reviewed the available information and inspected the existing slope during a site reconnaissance on September 29, 2010. Boring RKW-42-08 was drilled near the east end of the subject section. This boring encountered silty sand with gravel (alpine till) to 11 feet, on top of bedrock (moderately weak sandstone). The presence of till at the existing ground surface at the cut location was confirmed during the site reconnaissance. Bedrock was not seen in the existing cut.

A 1.5H:1V slope is an angle of 34 degrees. Assuming a friction angle of the till of 40 degrees and no cohesion, the factor of safety is 1.26 for an infinite slope. A factor of safety of 1.26 represents a probable lower bound because (1) the assumed strength parameters are probably conservative (till generally has some cohesion) and (2) the cut would be bounded by flatter slopes on the top and bottom. WSDOT's target factor of safety for cuts is 1.25.

Some considerations related to cuts in till are as follows:

- Till is generally difficult to revegetate. Lack of vegetation can result in increased erosion, including formation of rills. Jute mats should be considered to help promote revegetation.
- Till is susceptible to sloughing as a result of freeze-thaw action.
- Till may contain boulders. Any boulders more than half exposed in the cut face should be removed.

A cut with a slope no steeper than 1.5H:1V is considered to be acceptable at this location. Some erosion of the cut face may occur due to freeze-thaw action or runoff. It is anticipated that material eroded during these events could be contained in the ditch and removed as a maintenance item.

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"Gipner, Mathew" <GipnerM@wsdot.wa.gov>

"Gipner, Mathew"
<GipnerM@wsdot.wa.gov>
08/23/2010 01:16 PM

To <Dave_Walker@URSCorp.com>, "norm norrish" <nnorrish@msn.com>
cc "Trudeau, Julee (Consultant)" <TrudeaJ@consultant.wsdot.wa.gov>, "Golbek, Scott" <GolbekS@wsdot.wa.gov>
Subject I-90- AW 112+70 - AW 118+15 cut slopes

Hello Dave, as we discussed this morning we are looking to get official cut slope recommendations at the east end of the Phase 1C project. Currently we are proposing 1.5:1 cut slopes from AW 112+70 – AW 118+15 (that is unnamed creek @ MP 59.7 to the end of the main project). There are some small cuts along the AW west of this section but they are less than 10' tall and work with 2:1 or flatter slopes. Norm, the reason I am including you in this email is in case we encounter rock in these cuts. Please see the attached X-sections and plan views of the area. If you need any further information please let us know.

Thank You

Mathew Gipner
Transportation Engineer 2
I-90 Snoqualmie Pass East Project
Gipnerm@wsdot.wa.gov

2809 Rudkin Rd
Union Gap, WA 98903-1648

(509) 577-1883 (direct)

AW Cut Slopes 08-23-2010.pdf AW Cut Slopes Plan View Close 08-23-2010.pdf AW Cut Slopes Plan View Wide 08-23-2010.pdf

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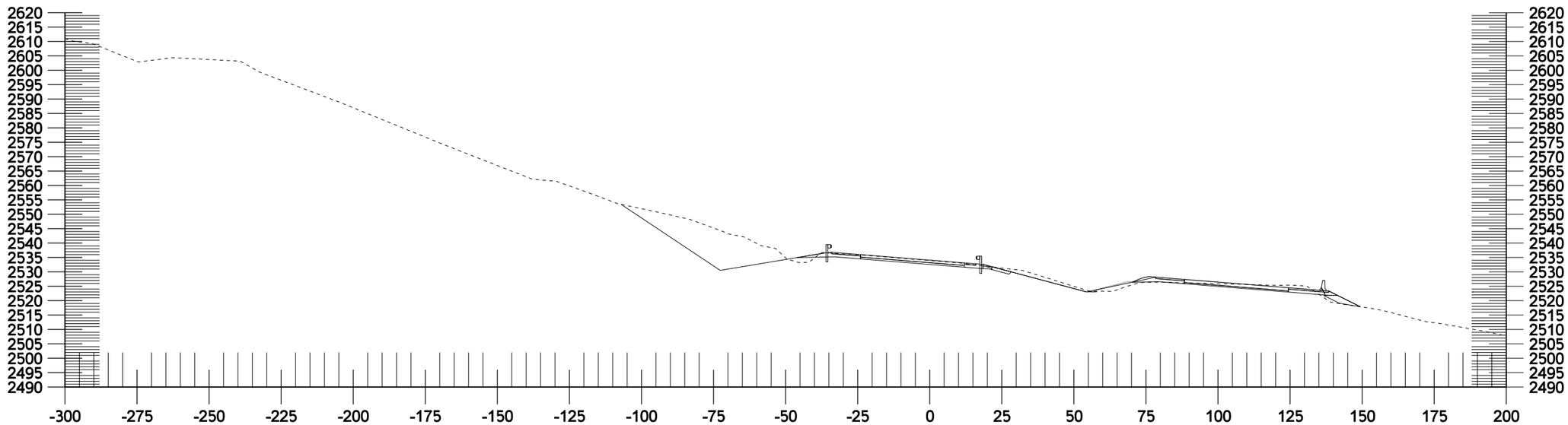
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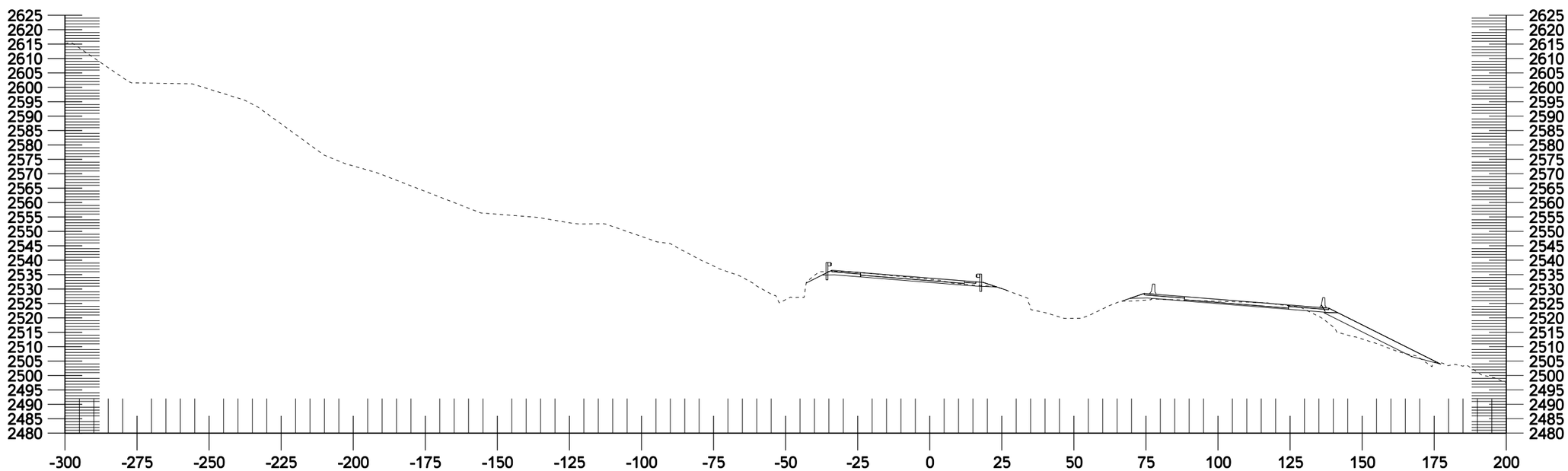
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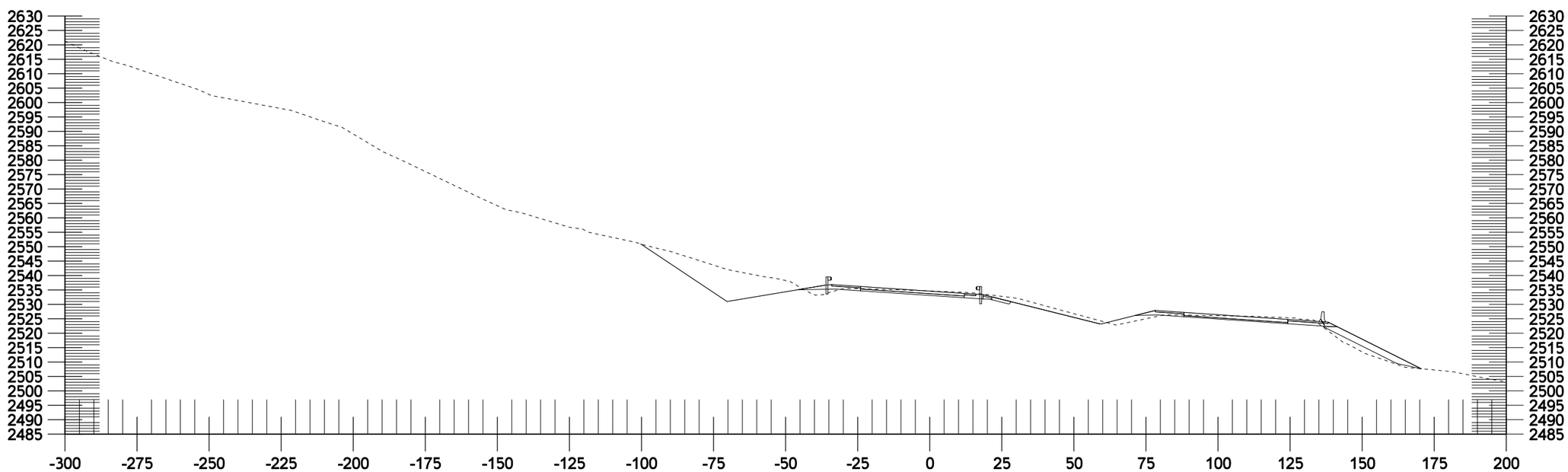
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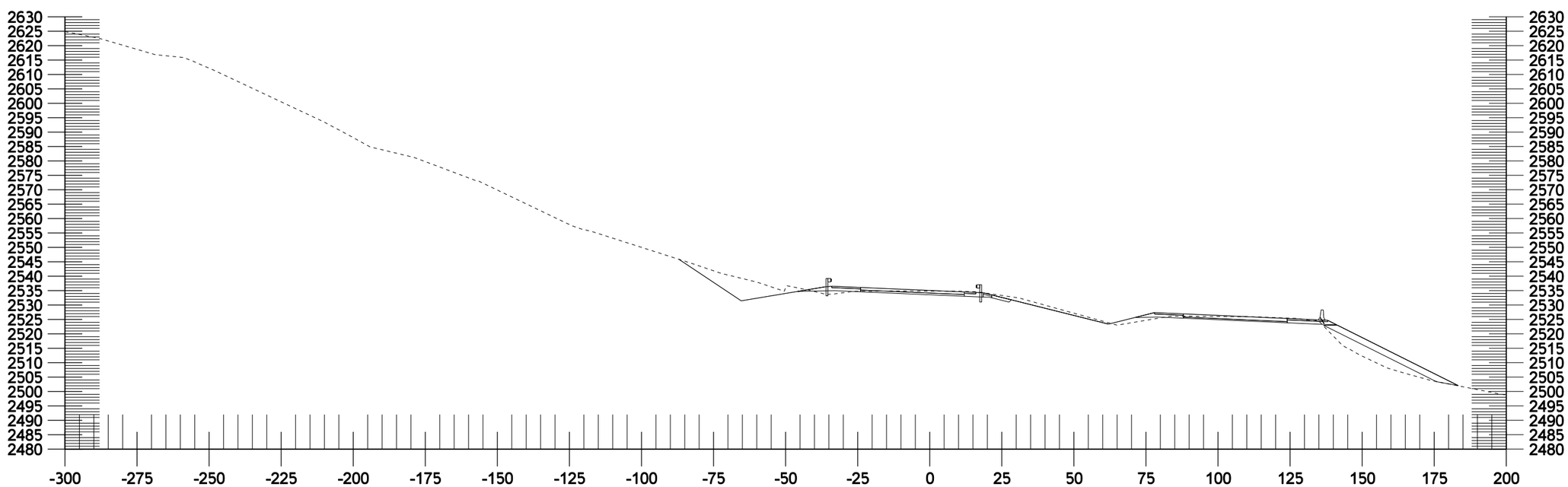
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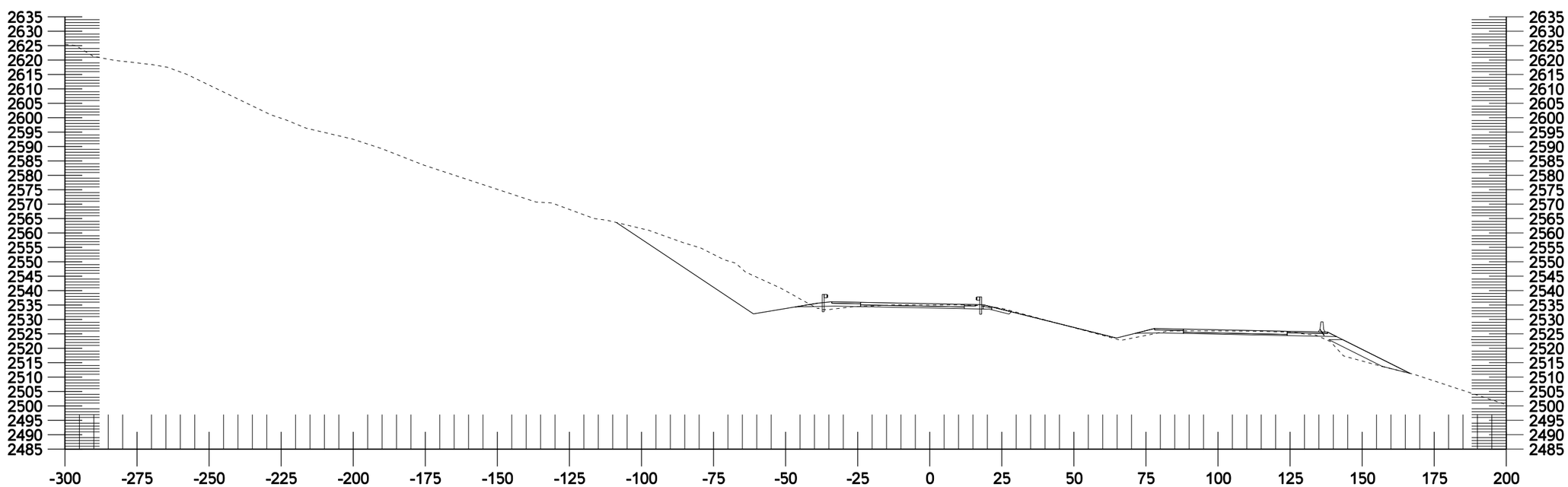
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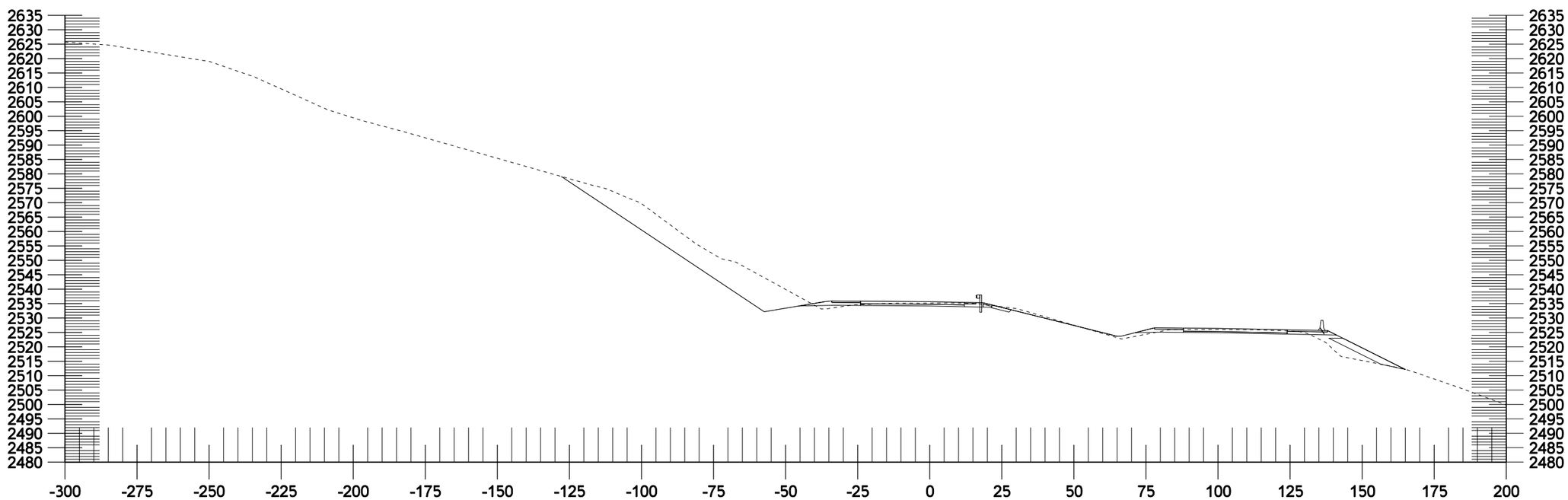
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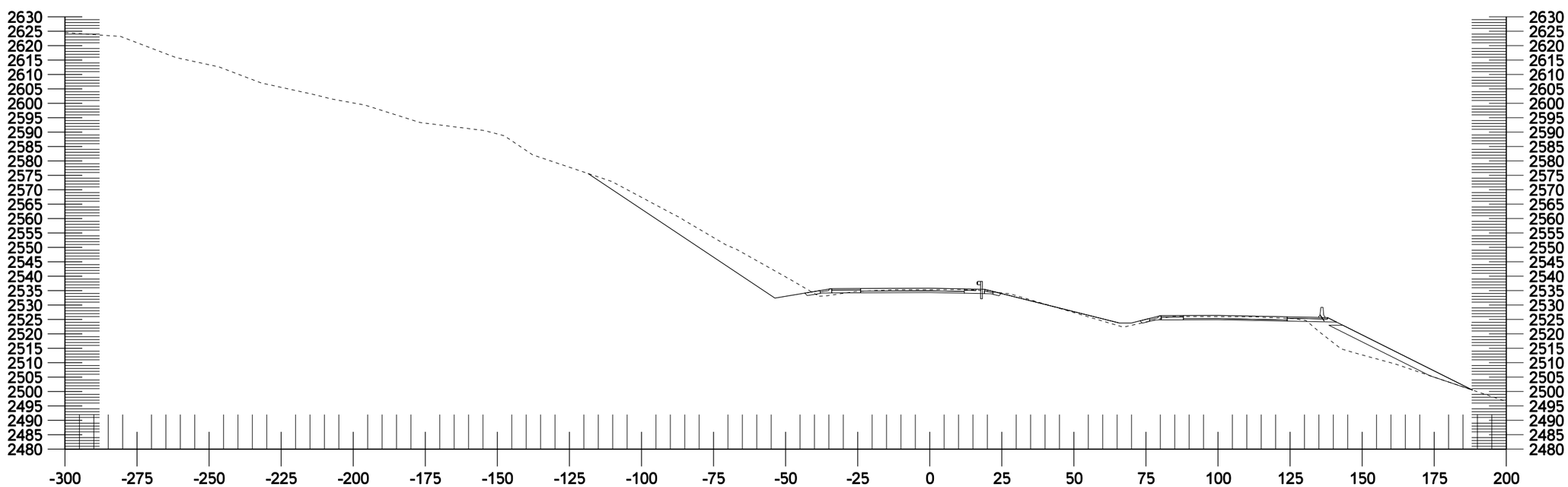
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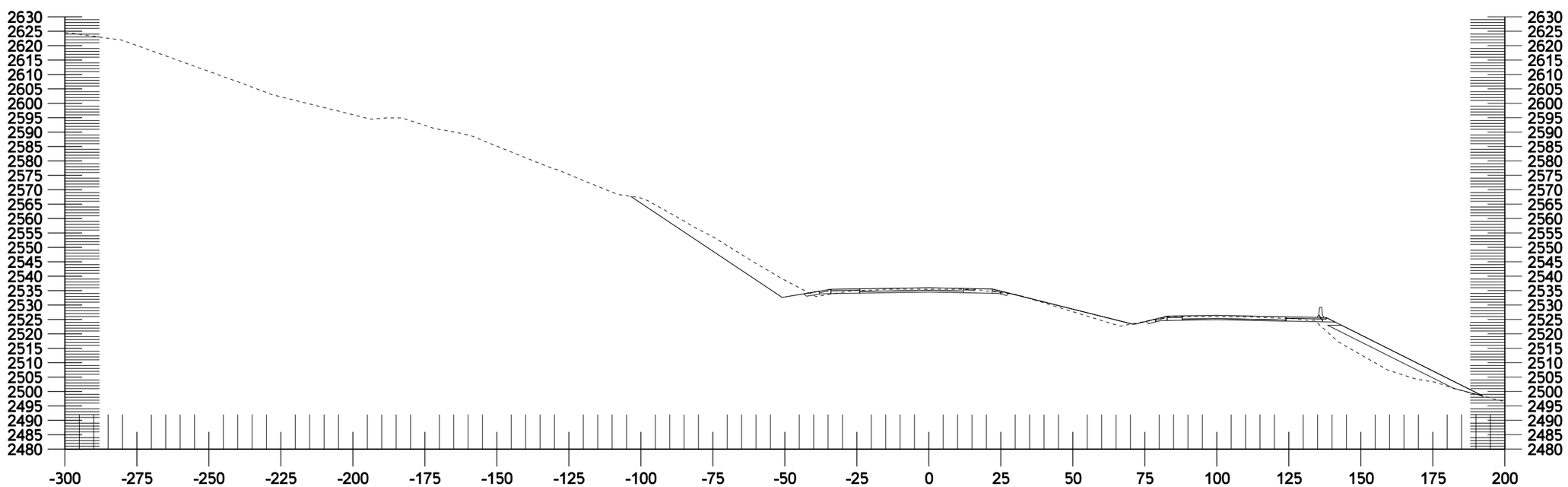
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