B-Branch Adult Fish Way - Erosion Repair
Design Document Report
October 2016

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B-Branch Adult Fish Way - Erosion Repair

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1. PURPOSE AND SCOPE

The purpose of this Design Document Report (DDR) is to document the erosion repair below B-Branch Adult Fishway located at Bonneville Lock and Dam. This product was prepared in accordance with the requirements in ER-1110-2-1150 Engineering and Design for Civil Works Projects. Plans and specifications for the repairs are being prepared concurrently.

2. AUTHORIZATION AND FUNDING

The project requires joint funding, 50% appropriated funds (non-routine O&M) and 50% BPA funding.

3. COORDINATION WITH OTHERS

Coordination is required for the following;

- Work B2 including but not limited to excavation in front of the fish units
- The project is constrained by Fish Passage Plan requirements, in-water work period requirements, operational requirements, and other contract work occurring at Bonneville Project
- Avian Wire removal and replacement before and after the BONN Erosion Repair work to facilitate Barge movement into the spillway
- PH 2 work which includes Lamprey work at BONN Washington Shores and Fish Guidance Efficiency at the gatewell slots and overall maintenance.
- PH1 work consists of Main Unit Breaker work and overall maintenance.
- PH2 – T11 and T12 work taking units out of service.
- Providing enough flow for Chum passage (125 Kcfs and 11.5')
- Extended Navigation Lock Maintenance Outage: 12/12/16 through 2/9/2017

4. INTRODUCTION

The Bonneville B-Branch Adult Fishway is one of the four fish ladder systems at the Bonneville Project that provide bypass routes for upstream migrating adult salmon, steelhead, lamprey and other fish species. It is also half of the Bradford Island fishway system, which includes the A-Branch. The B-Branch Fishway is located to the north of Bradford Island and south of the Spillway. The B Branch Fish ladder was constructed as part of the original Bonneville Project construction completed in 1938.

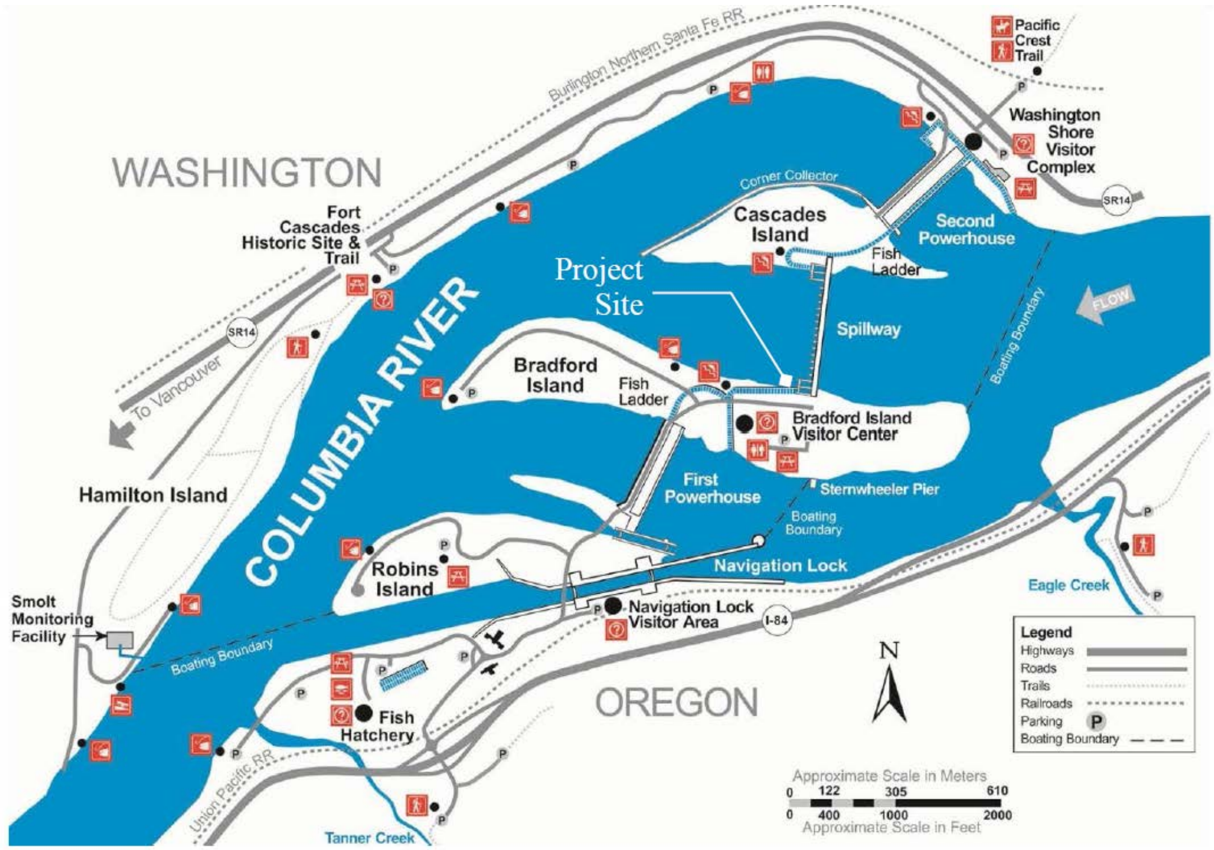


Figure 1 – Vicinity Map

In 2011, unusually high summer flows with extended duration resulted in erosion under the B-Branch Fishway at the south shore of the spillway. This erosion resulted in an emergency repair contract which required the B-Branch Fish Ladder to be dewatered to allow for repairs. The foundation of the fish ladder was compromised and the remedy involved grouting through holes in the bottom of the ladder. A photo of the eroded area during repair is shown below;

Figure 2 – Photo of 2011 Eroded Area during 2012 Repair

The voids under the fish ladder were filled with grout and the slope in front of the ladder was repaired with riprap, see paragraph on repair history for lessons learned.

Erosion was discovered again in June 2016 at the B-Branch Fishway at the south shore of the spillway.

Figure 3 – Photo in 2016 of Area Repaired in 2012

5. PROJECT REQUIREMENTS

The 2011 repairs were considered an emergency situation due to the requirement to support the foundation of the fish ladder and a desire not to compromise the Fish Passage Plan. Plans were quickly prepared, and a short selection contract was issued to a contractor already mobilized in the area. The fish ladder foundation was fixed with grouting and further protected with grouted rip-rap above water (See photo above) and larger riprap below water. The foundation grout is separate from the grout placed in the riprap which is limited to the exterior 3 feet. The 2011 repair was in retrospect sub-standard in that the riprap sizes selected were too small because they did not account for relatively steep slopes, and construction was plagued with unavoidable problems.

Note, the situation here in 2016 was not characterized as an “emergency” for NEPA compliance purposes because the conditions are more favorable than in 2011, that is the foundation support exists and the grouted portion of the repair under the ladder remains. However, anything could still happen and the foundation supporting the fish ladder is in jeopardy of being lost over the winter depending on flows and operations. Therefore, this was considered an engineering priority for the district with a focus on lessons learned from 2011.

During preparation of this design document, the PDT discovered historical repairs of the exact same area in 1965 lending a historical context. A summary is provided at the end of this report.

The B-Branch Adult Fishway requires a repair with a relatively long service life similar to that of the spillway, e.g. 100-years, and the repairs must not significantly change the hydraulics of the spillway.

The PDT is aware of the pending major rehabilitation of the Bonneville Project and its spillway. The final plans are unknown and the date of implementation, also unknown. Given that the embankment supporting the B-Branch Fish Ladder has eroded and required 3 repairs (1965, 2012, and 2016), the major rehab should consider extending the existing concrete apron downstream. This 2016 repair only considers a riprap style of design.

6. HYDROSURVEYS

Table 1 Chronology of Recent Hydrosurveys

20-Sep-2011	Survey of eroded surface performed by NWP-NWH
14-Mar-2012	Survey following 2nd Repair attempt performed by Solmar Hydro
7-Jul-2016	Survey following discovery of erosion on 12-Jun-2016, performed by NWP-NWH
8-Sep-2016	Topo/hydro survey and shutdown of the spillway in coordination with the Regional Fish Managers
TBD	QA survey by Government using ROV coordinated with Contract – focus on characterizing the materials under water
TBD	QC survey by Contractor following excavation and pre-placement of riprap for as-built documentation
TBD	QC survey by Contractor post placement of riprap for as-built documentation
	QA survey by Government using ROV coordinated with Contract for acceptance of completed works

7. REFERENCES

Basic data and criteria used in the design are contained in applicable engineer manuals and regulations, guide specifications, and other sources of criteria including those listed below.

- HDC 712-1 Hydraulic Design Criteria, Waterways Experiment Station (WES) dated September 1970
- EM 1110-2-1601 Hydraulic Design of Flood Control Channels USACE (1-Jul-1991)
- Technical Report HL-88-4 Stable Rip-Rap Size for Open Channel Flows, USACE Waterways Experiment Station (WES) dated September 1988
- Site Visit Trip Report, CENWP-EC 6-June-2016

- Columbia and Lower Willamette Rivers Below Vancouver WA & Portland OR - Comprehensive Evaluation of Project Datum: Compliance Report 2014
- Pacific Northwest National Laboratory—Rakowski, C. L., Serkowski, J.A., Richmond, M.C, and Guensch, G.R., “Development and Application of a 3D CFD Model for the Bonneville Project Tailrace for Proposed High Flow Outfall Structures”, September 2001
- Pacific Northwest National Laboratory—Rakowski, C. L., Serkowski, J.A., Richmond, M.C, and Perkins, W.A, “Computational Fluid Dynamics Modeling of the Bonneville Project: Tailrace Spill Patterns for Low Flows and Corner Collector Smolt Egress”, – 20056, 2010.
- Pacific Northwest National Laboratory—Rakowski, C. L., Serkowski, and J.A., Richmond, M.C, “Bonneville Project: CFD of the Spillway Tailrace”– June 12 2012.
- CENWP-EC-HD, Ebner, L.L. “ERDC Trip Report on Bonneville Spillway Stilling Basin”. March 03 2012.
- Bonneville Geological Report Final 31 January 1937

8. HYDRAULIC DESIGN

A calculation package for the hydraulic design is attached in Appendix A. A summary of the criteria and calculations follows.

8.1 Design Criteria

The design velocity in the river is used to size the proposed rip-rap repair. This velocity is adjusted upwards for side slope to determine the critical velocity for sizing rip-rap on flat grades. The layer thicknesses are determined as a function of rock size and whether the rock is installed above or below water. The section describes the methods used to determine the proposed rip-rap material to be applied.

- Design Velocity in River: The design velocity (V_S) in river is 15 fps. The process on how this velocity was determined is provided in the following paragraphs.
- Several hydraulic models were used to identify the design velocity. The first was the spillway physical model at ERDC (CENWP-EC-HD 2012). After the 2011 spill season, over a thousand cubic yards of material were found in the stilling basin. We used the physical model to investigate how the rocks moved into the stilling basin. What occurred was surprising – rocks moved upstream on the edges (bays 2 and 3 and bays 16 and 17). At bay 17, rocks would move up on the concrete apron downstream of the baffle blocks. The rocks then moved

towards the center of the spillway, they would work themselves to the center of the spillway and then jump into the stilling basin – upstream of the baffle blocks. Once there, they moved both north and south. Velocities were measured in the physical model and generally were 10 fps along the shore where the erosion is occurring. A CFD model was built to represent these conditions (PNNL 2012). The CFD model did show rocks moving upstream and into the stilling basin similar to that in the physical model thus our confidence that the CFD model was adequately representing the physics of deflector flow.

- The CFD results showed velocities in the 10 to 12 fps range (see example CFD plots in Figure 4 and Figure 5 (PNNL 2012). But the CFD runs were limited to total spill volumes of just over 300 Kcfs. Previous CFD model studies were conducted to develop design load conditions on the Bonneville 2nd Powerhouse Corner Collector Outfall (PNNL 2001). Flows in excess of the 100 year event (680 Kcfs) were pushed through the spillway. The results between the two models were compared for 100 Kcfs, 125 Kcfs, 200 Kcfs and 300 Kcfs through the spillway. In the area of repair the two models showed similar velocities. The 100 year event showed velocities less than 10 fps in the area of interest (See Figure 6 from PNNL 2001).

- Decreasing the design velocity from 15 fps is unwise due to the dynamic nature of the spillway environment and the 2012 repair was designed for a velocity of 12.5 fps and failed this year.

- Based on the above considerations, the design velocity was taken to be 15 fps in the river adjacent to the area under repair.

- The relatively small volume of stone being placed in the spillway will not negatively impact hydraulics off of the flow deflectors in the spillway.

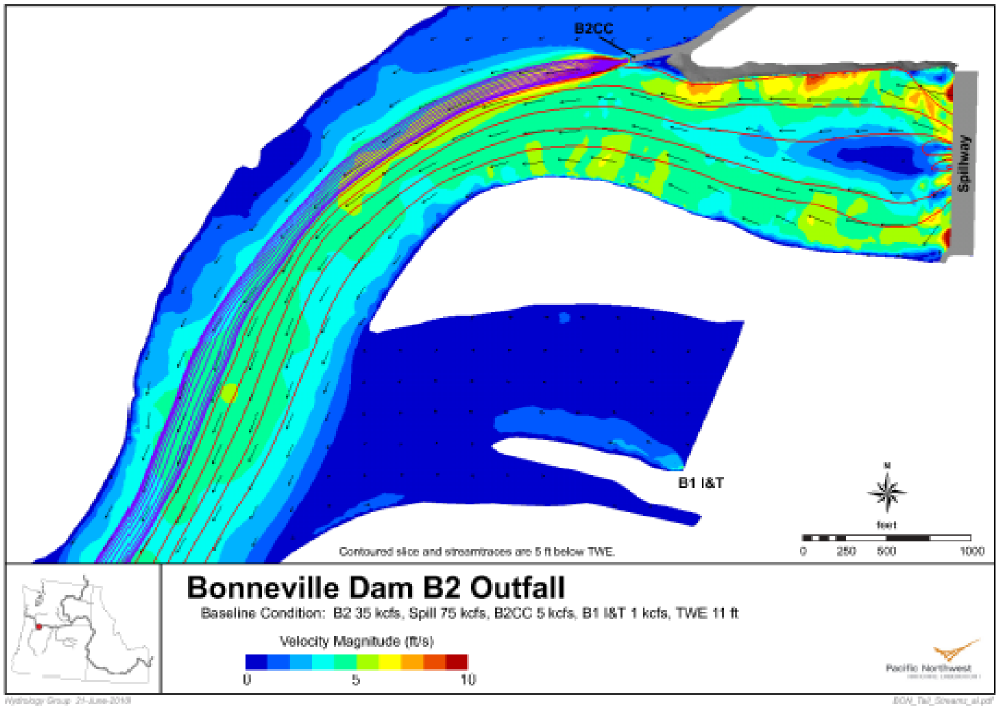


Figure 4 - CFD velocities in the Spillway Tailrace – Spill of 75 Kcfs

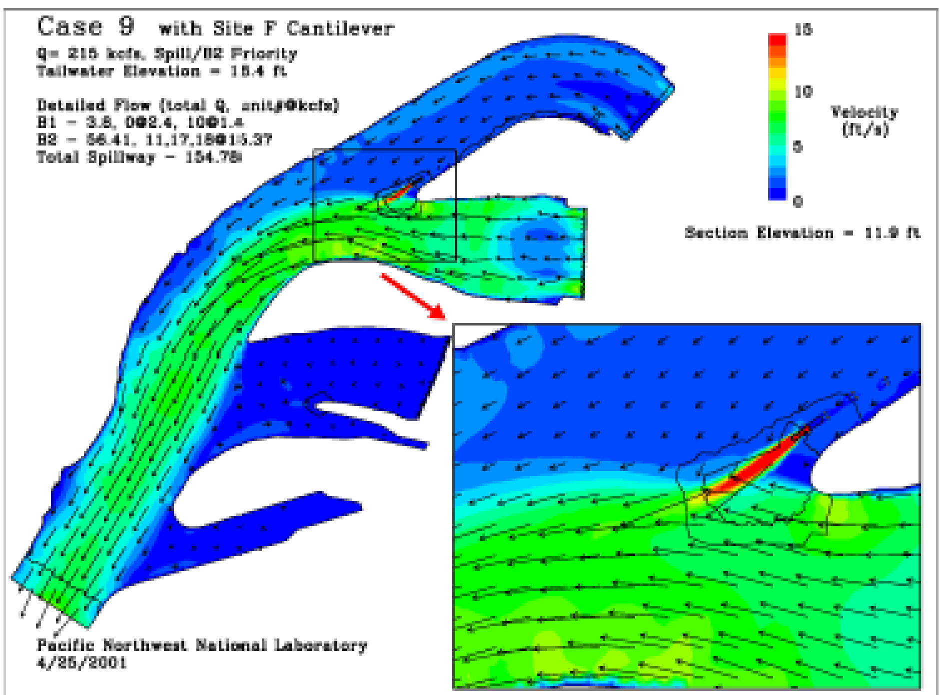


Figure 5 - CFD velocities in the Spillway Tailrace – Spill of 155 Kcfs

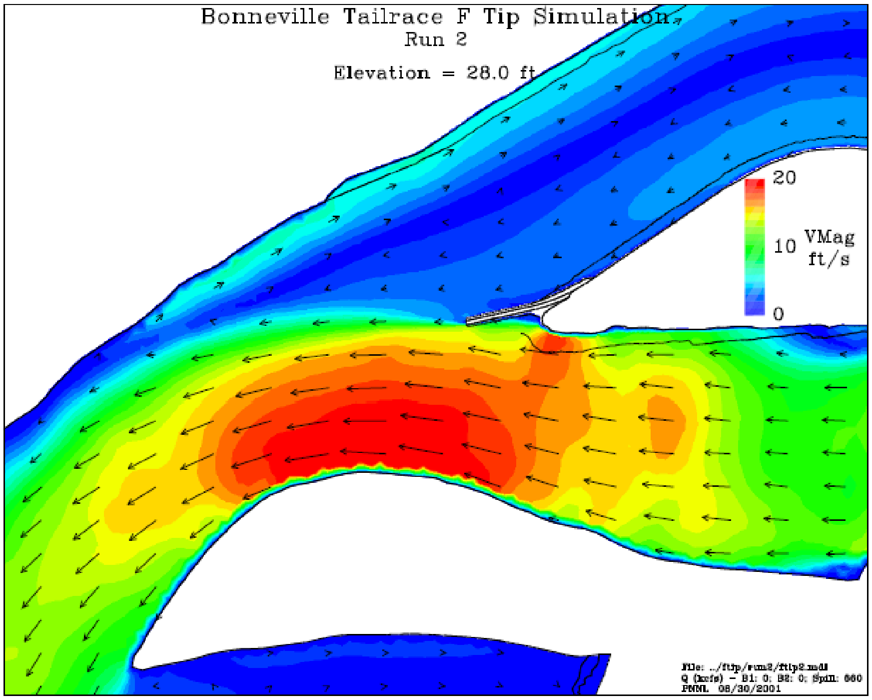


Figure 6 - CFD velocities in the Spillway Tailrace – Spill at 660 Kcfs

- Criteria affecting Rip-Rap Design: The following paragraphs show the criteria and procedures for slope correction, rip-rap sizing and layer thickness.
  - The top of the proposed revetment will vary between elevations 23 – 34 feet NAVD 88. The average top elevation is about 31 feet NAVD 88. The height of the revetment slopes will vary between 30 – 55 feet and averages approximately 43 feet.
  - The median tailwater elevation for December is 18.9 feet NAVD 88 (15.6 feet NGVD 29) and for the In-water work (IWW) period is 20.0 feet NAVD 88 (16.7 feet NGVD 29). Based in the median IWW tailwater, the average height of the underwater construction will be about 32 feet, with 11 feet above water.
  - Thickness of Rip-Rap Section should not be less than the spherical diameter of the upper limit W_{100} stone or less than 1.5 times the spherical diameter of the W_{50} stone, whichever results in greater thickness. (Increase by 50% for underwater installation per EM-1110-2-1601). [1]

[1] Rock gradations are often specified in terms of the percentile on weight or diameter for example the largest stones belong to the 100 percentile class (W_{100}) or ½ the stones belonging to the 50% Diameter Class (D_{50})

- Median size of rock determined by application of the Isbash Equation or Chart 712-1 of Hydraulic Design Criteria (1988):

$$Vc = C \cdot \sqrt{2g \cdot \left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right) \cdot D_{50}}$$

  - Vc = critical velocity for sizing rip-rap on flat grade
  - γ_s = Specific weight of stone = 165 lbs/ft^3
  - γ_w = Specific weight of water = 62.4 lbs/ft^3
  - D_{50} = Median size of rip-rap (feet)
  - C = Isbash Constant = 0.86 for high turbulence

- Slope Adjustments. The critical velocity (Vc) used to size the riprap is dependent on the following equation (Eq. 4.23 fromTR-HL-88-4, USACE 1988):

$$Vs = Vc\sqrt{K} \qquad \text{EQ 1}$$

Vs = critical velocity on sloped rock
Vc = critical velocity on flat slope from Isbash or Fig. 712-1
K = slope correction factor based on experimentation

Based on Figure 4-20 from TR-HL-88-4, the K values are the following for given side slopes:

K = 0.7 for 1V: 1.5H
K = 0.88 for 1V: 2H

Given a design velocity (15 ft/s) in the prototype, the velocity that would be applied in the Isbash Equation or Chart 712-1 must be adjusted upwards to account for side-slope in the rock sizing. Consequently Equation 1 is inverted to Equation 2 to estimate the critical flat slope velocities Vc required for sizing rock per Isbash Equation or Chart 712-1. In the Equation 2, Vs is the prototype velocity (15 ft/s) so that the critical velocity Vc can be adjusted to effectively size the rock on a side slope.

$$Vc = \frac{V_s}{\sqrt{K}} \qquad \text{EQ 2}$$

Table 2 - Required Critical Velocity Vc for Sizing Rock, Given Known Design Velocity Vs on Side Slope

Slope Alternative	1V: 1.5 H	1V: 2.0 H
K-Factor	K = 0.7	K = 0.88
Vs	Required Vc for Rock Sizing	
15 ft/s	17.9 ft/s	16.0 ft/s

- Required Rip-Rap Layer Thickness. Per EM 1601, the rip-pap layer thickness shall not be less than the minimum of 1 x D_{100} or 1.5 x D_{50}. For underwater placement, this dimension needs to be increased by 50% due to the uncertainty of the placement.

o A 150 foot section river length must be repaired based on engineering judgement. If a certain large section of broken concrete cannot be removed, then a transition section will need to be added. Assume that existing rip-rap material must be excavated to make room for new material.

8.2 Rock Sizes Alternatives

Two rock size alternatives are developed based on design slope. Alternative 1 would use a side slope of 1V: 2.0 H. Alternative 2 would use a side of 1V: 1.5 H. Note that the design slope used in 2011 repairs was between 1V: 1.5 H and 1V:2.0H. If a similar range of slopes is applied for design, the use of materials meeting criteria for Alternative 2 would be conservative but would not be the case for Alternative 1 materials.

8.2.1 Alternative 1: Presuming slopes 1V:2H

- Vc = 16 ft/s
- Isbash equation and Chart 712-1 require median rock W_{50} = 3,000 lbs and D_{50} = 3.3 foot diameter.
- Layer thickness equal to 5.0 feet above water, and 7.4 feet below water. Preliminary tonnage for budgeting is shown below.

Table 3 – Alternative 1 Rock Dimensions, Volumes and Weights

Alternative 1		Max Slope = 1V: 2 H	
Median IWW Tailwater Elevation =		20.0 ft NAVD 88	
Rip-rap Parameters	Units	Vc = 16.0 ft/s Above Water	Below Water
D_{50}	feet	3.3	3.3
W_{50}	lbs	3,000	3,000
Thickness	feet	4.9	7.4
Height	feet	11	32
Slope Length	feet	24.5	72.6
Length	feet	150	150
Known Volume	yd^3	4,220	
per 50% P&S	ft^3	113,940	
Void ratio		35%	
Solid Volume	ft^3	74,061	
Density	lbs/ft^3	165	
Total Weight	Tones	6,110	

8.2.2 Alternative 2: Presuming slopes of 1V:1.5H (or less)

- Using the relations in TR-HL-88-4 to account for side slope, the critical velocity used in the Isbash equation increases from 15 fps to about 18 fps resulting in a required W_{50} = 6,000 lbs and D_{50} = 4.1 foot diameter.
- Layer thickness of 6.2 feet is required above water and 9.3 feet below water. Preliminary tonnage for budgeting is shown below.

Table 4 – Alternative 2 Rock Dimensions, Volumes and Weights

Alternative 2		Max Slope = 1V: 1.5 H	
Median IWW Tailwater Elevation =		20.0	ft NAVD 88
Rip-rap Parameters	Units	Vc = 18.0 ft/s Above Water	Below Water
D_{50}	feet	4.1	4.1
W_{50}	lbs	6,100	6,100
Thickness	feet	6.2	9.3
Height	feet	11	32
Slope Length	feet	19.8	58.5
Length	feet	150	150
Est. Volume*	yd^3	4,306	
Est from Alt 1 Vol	ft^3	16,262	
Void ratio		35%	
Solid Volume	ft^3	75,570	
Density	lbs/ft^3	165	
Total Weight	Tones	6,235	

*Est Vol Alt 2 = Vol Alt 1 * Sqrt(1+S2)/Sqrt(1+S1) * TH(2)/TH(1)

The PDT recommends a 1V:2H slope for the following reasons;

- The repairs from 1965 used steeper slopes of 1V:1.5H and steeper as they transitioned to the existing concrete apron, therefore a lesser slope is likely to be more successful.
- The 1V:2H slope results in more placement of fill on top of existing grouted riprap which means there is less likelihood of loosening the foundation support to the fish ladder during construction.

9. RIP-RAP GRADATIONS

EM 1110-2-1601 Appendix F is derived from a MVN Report of Standardization of Rip-Rap Gradations. Presuming Alternative 1 for slopes of 1H:2V, the following gradations apply.

Table 5 – Riprap Gradations by Weight

Weight Class by Percent Finer ($W_{\%}$)	Lower Limits (lbs)	Upper Limit (lbs)
W_{15}	938 Note (4)	2,225 Note (5)
W_{50}	3,000 (Note 1)	4,445 Note (5)
W_{100}	6,000 (Note 2)	15,000 (Note 3)

1) The lower limit of W50 stone should not be less than the weight of stone required to withstand the design shear forces as determined by the procedure given in EM 1110-2-1601 and HDC 712-1.
2) The lower limit of W100 stone should not be less than two times the lower limit of W50 stone.
3) The upper limit of W100 stone should not exceed: five times the lower limit of W50 stone, that size which can be obtained economically from the quarry, or that size which will satisfy layer thickness requirements, i.e. Diameter less than the specified layer thickness. The layer thickness is 7.3 feet, however axis ratios will not allow any axis length greater than 6 feet
4) The lower limit of W15 stone should not be less than one-sixteenth the upper limit of W100 stone.
5) Computed using a trend line on data in Table 3-1 Gradation of Rip-Rap Placement in the dry, Low Turbulence Zones of EM 1110-2-1601, presuming a density of 165 pounds per cubic foot.

From the weights we can also compute the equivalent sphere diameter and specify tolerances on axis lengths.

Table 6 – Riprap Gradations by Dimension

Weight Class ($W_{\%}$)	Equivalent Spherical Diameter (ft)		Equivalent Spheroid Diameter (ft) [Maximum Axis Ratio: 3]			
	Lower Weight Limit	Upper Weight Limit	Lower Weight Limit		Upper Weight Limit	
			Short Axis	Long Axis	Short Axis	Long Axis
W_{15}	2.2	**3.0**	1.5	4.6	2.0	6.1
W_{50}	3.3	3.7	2.3	6.8	**2.6**	7.7
W_{100}	4.1	5.6	2.8	8.6	3.9	11.6
Weight Class ($W_{\%}$)	**Equivalent Spherical Diameter (ft)**		**Equivalent Spheroid Diameter (ft) [Maximum Axis Ratio: 2.5]**			
	Lower Weight Limit	Upper Weight Limit	Lower Weight Limit		Upper Weight Limit	
			Short Axis	Long Axis	Short Axis	Long Axis
W_{15}	2.2	3.0	1.6	4.0	2.2	5.4
W_{50}	3.3	3.7	2.4	6.0	2.7	6.8
W_{100}	4.1	5.6	3.0	7.6	4.1	10.2
Note: Specifications say maximum ratio of 3.0 on any stone and no more than 25% may be greater than 2.5:1.						

The tolerance on placement should be based on the size of riprap as follows;

- Above top neat line should be the greater of the short axis of an elongated W_{50} at the upper limit (2.6 feet) or the equivalent spherical diameter of upper limit of the W_{15} (3.0 feet). Excess placement above the neat line is generally not a revetment problem unless there are hydraulic issues.
- Below top neat line is about 1.0 foot based on the desire to maintain a minimum thickness of the layer. Based on this tolerance the practical remedy is to place some W_{50} and small material that would result in a position above the neat line at 3.7 feet (equivalent sphere) minus 1 foot equals 2.7 feet, which is about the same as the tolerance above the line.

The tolerance on excavation should also be based on the nature of the foundation and the desire to get a minimum layer thickness. Recommend + 0 feet and minus 1 foot.

Extreme limits of the tolerances given should not be continuous in any direction for more than five times the upper limit on W50 equivalent spherical diameter that is 3.7 feet times 5 equal to 18.5 feet. It is reasonable to round this down to 18 feet.

10. UNIT WEIGHTS

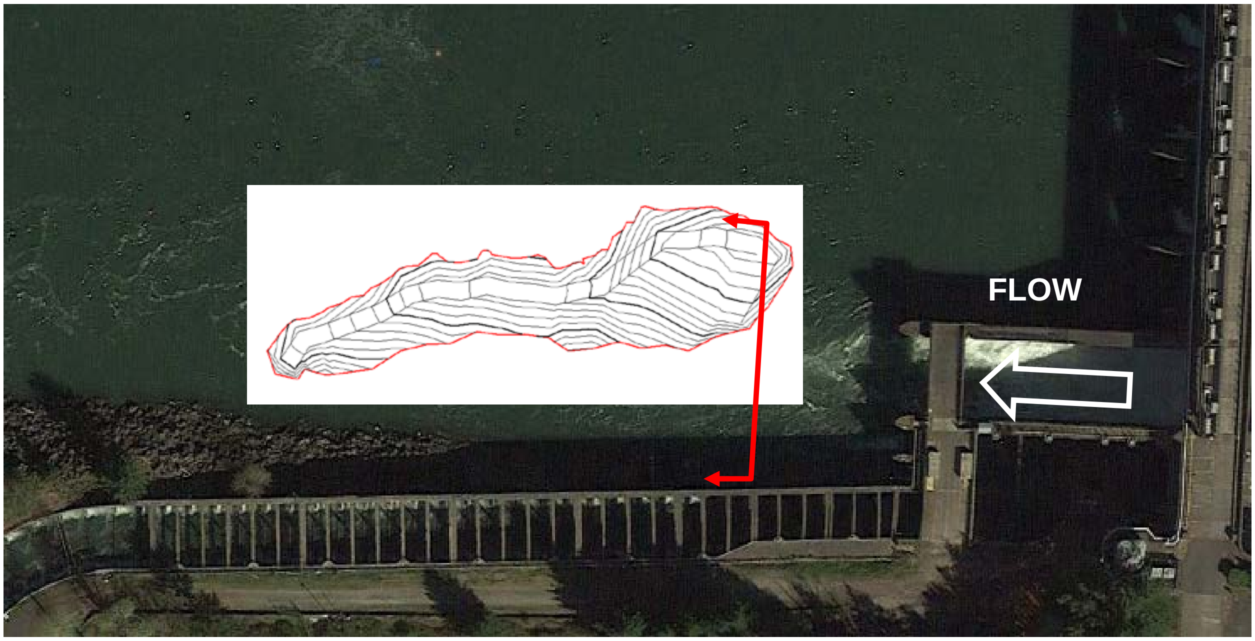
Table 7 – Unit Weights for Design and Specification of Riprap

	Specific Gravity	Unit Weight (pcf)	Comment
Lower limit for specs	2.60	162.2	Published average for sand
Presumed for design	2.64	165.0	Round number near the low end
Upper limit for specs	2.90	181.0	Round number nearing the high end of experience in the area

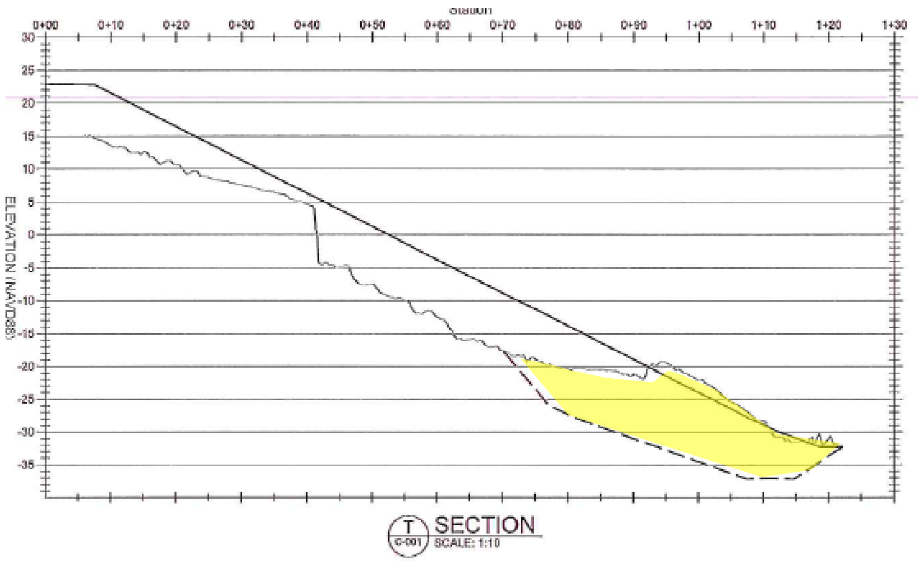
The upper limit also represents a threshold in the specifications for which the government should consider redesign of the riprap size and layer thickness.

11. EXACAVTION MATERIALS

Rip-rap material must be keyed into the toe of the slope. This requires the excavation of existing material as illustrated below.



a) Plan View Excavation Extents with Contours



b) Section View

Figure 7 - Plan and Section View of Excavation

The section shown is located immediately adjacent to the existing concrete apron.

The proposed excavate material is probably a combination of some alluvial material that is moving through the spillway, and rip-rap placed in 2012 that failed. The excavation shown is about 1,580 cubic yards. There is also a chance of encountering derrick stone placed in 1965. An example of alluvial material excavated from the spillway is shown below.

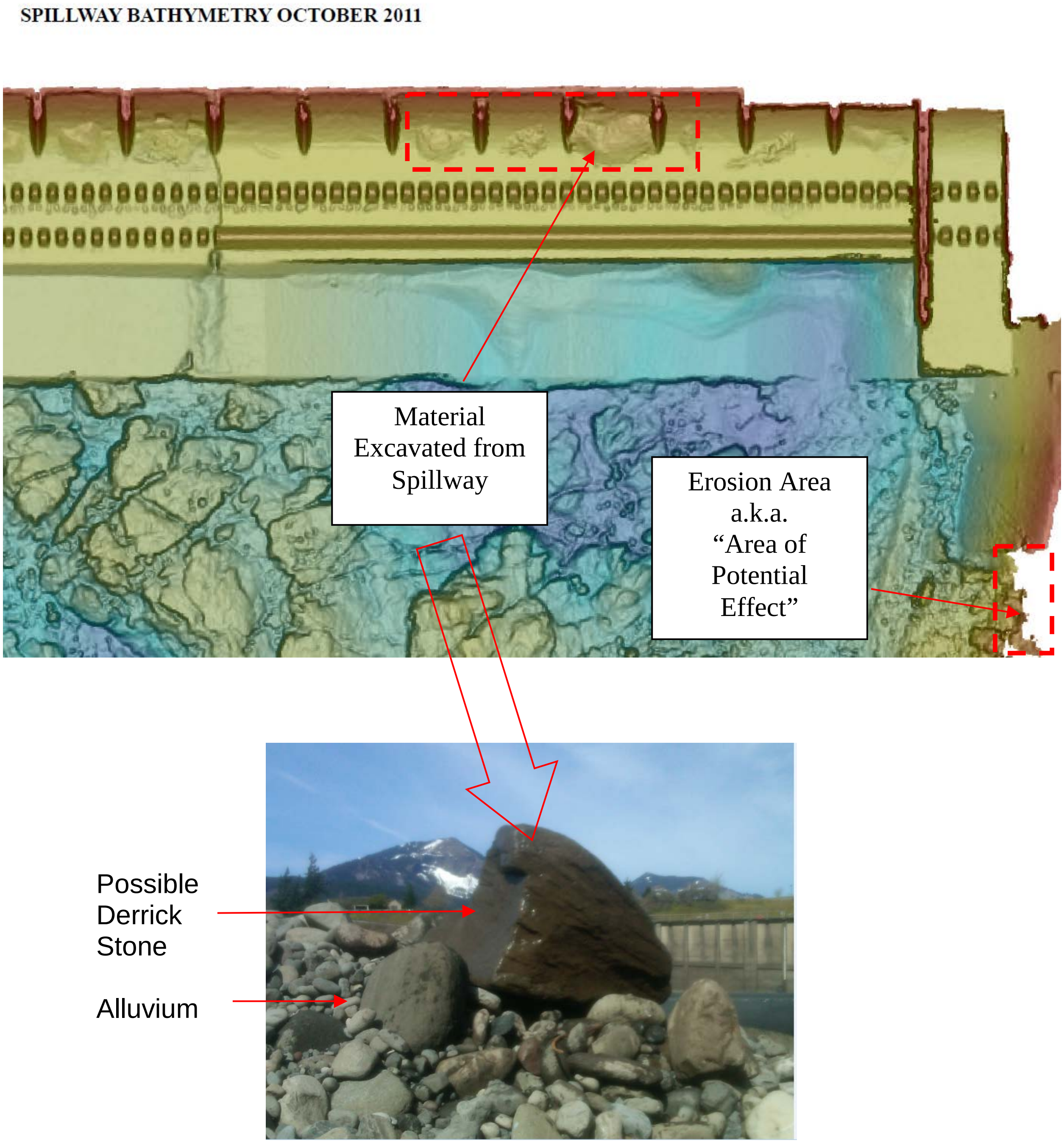


Figure 8 - Materials Excavated from the Spillway in 2011

The alluvium is probably loose boulders with some cobbles and gravels. The large angular stone may be a piece of derrick stone placed in 1965.

12. VOID RATIO

Void ratios are required to convert volumes to weights.

Table 8 Typical Void Ratios for Armor Rock

Table 1. Existing guidance for typical as-built properties of randomly placed irregular and rounded armour

Shape of rock	SPM (1984)[2] BS 6349 (1991)[3]	QMW/HR (1988)[4]	CIRIA/CUR (1991)[1]
Equant and irregular			
k_t	1.15 (rough)	0.75-0.85	0.75-1.20
n_v: %	40	37-39	38-40
Rounded			
k_t	1.02 (smooth)	0.73	0.8-1.2
n_v: %	38	35	35-37

Notes

1) CIRIA/CUR [1990] Manual on the use of rock in coastal and shoreline engineering. CURReport 154 Balkema, Rotterdam (ISBN 905410 1024).

2) US Army Corps of Engineers. The shore protection manual (SPM). US Army Corps of Engineers, Coastal Engineering Research Centre, US Government Printing Office, Washington DC, 1984, 4 edn. [This may not be the proper reference]

3) British Standards Institution. Code of practice for maritime structures: Part 1, General criteria. BSI, London, 1984, BS 6349.

4) CIRIA/CUR [1990] Manual on the use of rock in coastal and shoreline engineering. CURReport 154 Balkema, Rotterdam (ISBN 905410 1024).

5) This table and all collated data courtesy of Carlos Bosama MSc Thesis at Delft University Netherlands, paper titled Void Porosity Measurement in Coastal Structures

Table 9 Typical Values of Void Ratio for Various Soils

Type of soil	Void ratio, e	Natural moisture content in a saturated state (%)	Dry unit weight, γ_d kN/m^3
Loose uniform sand	0.8	30	14.5
Dense uniform sand	0.45	16	18
Loose angular-grained silty sand	0.65	25	16
Dense angular-grained silty sand	0.4	15	19
Stiff clay	0.6	21	17
Soft clay	0.9–1.4	30–50	11.5–14.5
Loess	0.9	25	13.5
Soft organic clay	2.5–3.2	90–120	6–8
Glacial till	0.3	10	21

Conversions from volumes to tonnage are typically placed in UFGS specifications. The recommended conversions are provided below.

Table 10 – Unit Weights for Design and Specification of Riprap

	Specific Gravity	Voids (%)	Cubic Feet per 2000 lbs
Riprap	2.64	35	16.3
Excavate Material	2.40	50	20.0

The actual conversions may vary.

13. TOE PROTECTION

The toe of the revetment is the most vulnerable portion. Therefore special measures for protection against scour are warranted. The first failure mode would be scour of erodible materials causing a loss of foundation support. A second failure mode could be the movement of alluvial boulders along the bottom, a process known as saltation.

An ODOT Manual on Bank Protection offers several good illustrations of toe protection reproduced below.

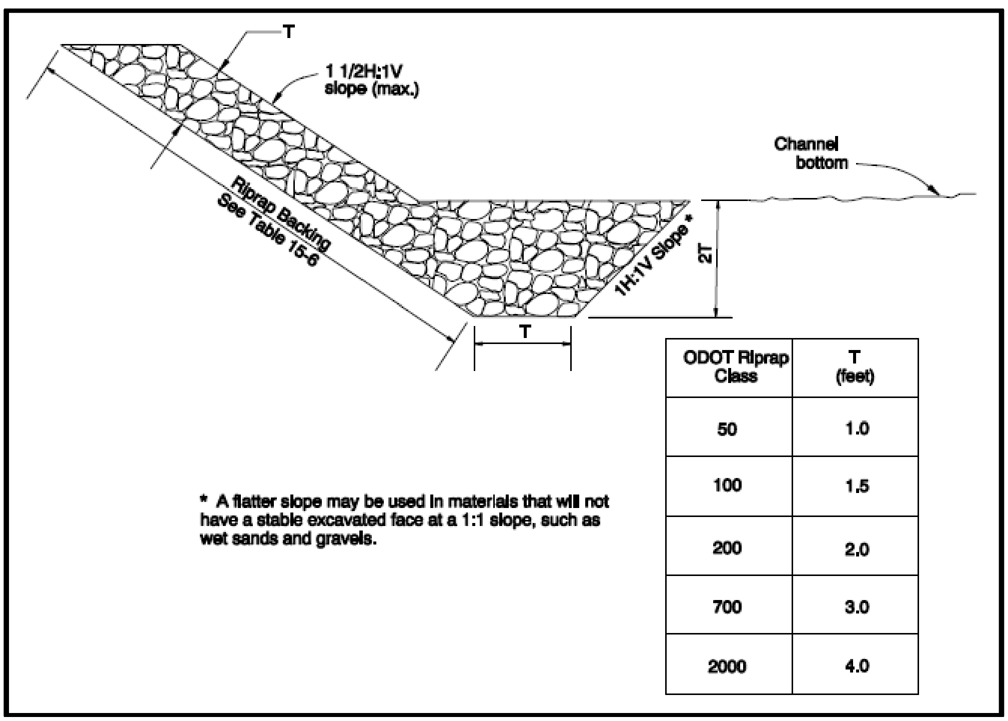


Figure 9 - Standard Riprap Section from ODOT

This will work well in areas that can be excavated, however interpretation of the geology suggest that bedrock may be encountered within the neat line for the excavation therefore some alternate method of toe protection may be required.

One alternative include a key excavated into the bedrock such as shown below;

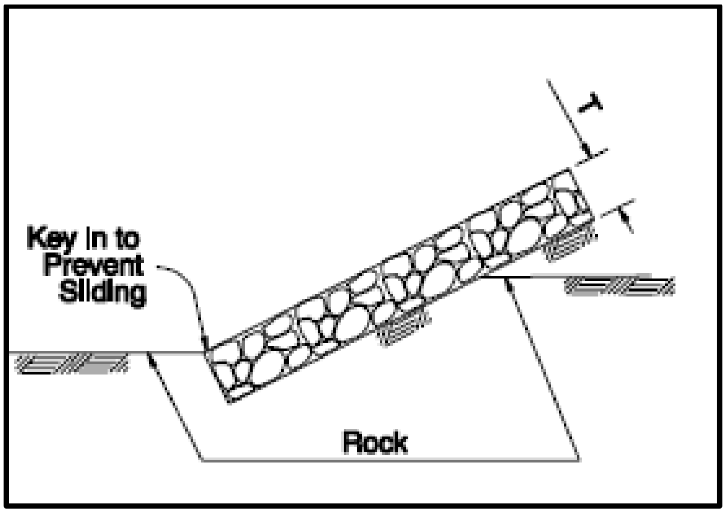


Figure 10 - Section View of Scour Protection Method No.4 from ODOT

The excavation of bedrock is a significant effort to be planned for.

Yet another alternative is employ the concept of “launched stone” which is placed in excess as a toe berm. The launch stone would naturally drop into the scoured hole as shown.

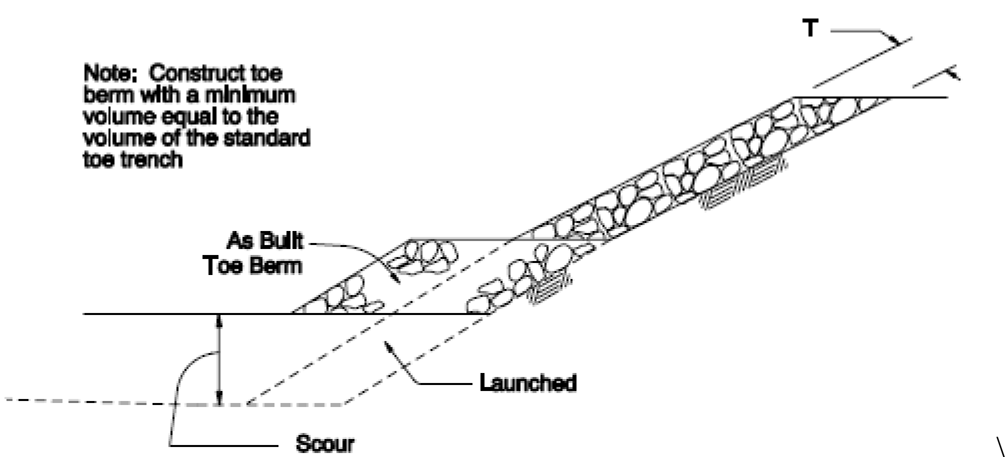


Figure 11 - Section View of Scour Protection Method No.5 from ODOT

PDT recommends that the contract base their work on the standard rip rap section presuming excavation with a clamshell bucket. If refusal is met, the contractor may elect to remedy this differing condition with Method 4 (minimal rock excavation) or Method 5 (balance of fill as toe berm).

14. GEOLOGY

The *Bonneville Geological Report- Final 31* was the original 1937 report[2]. Interpreted geologic sections that report are reproduced in part below.

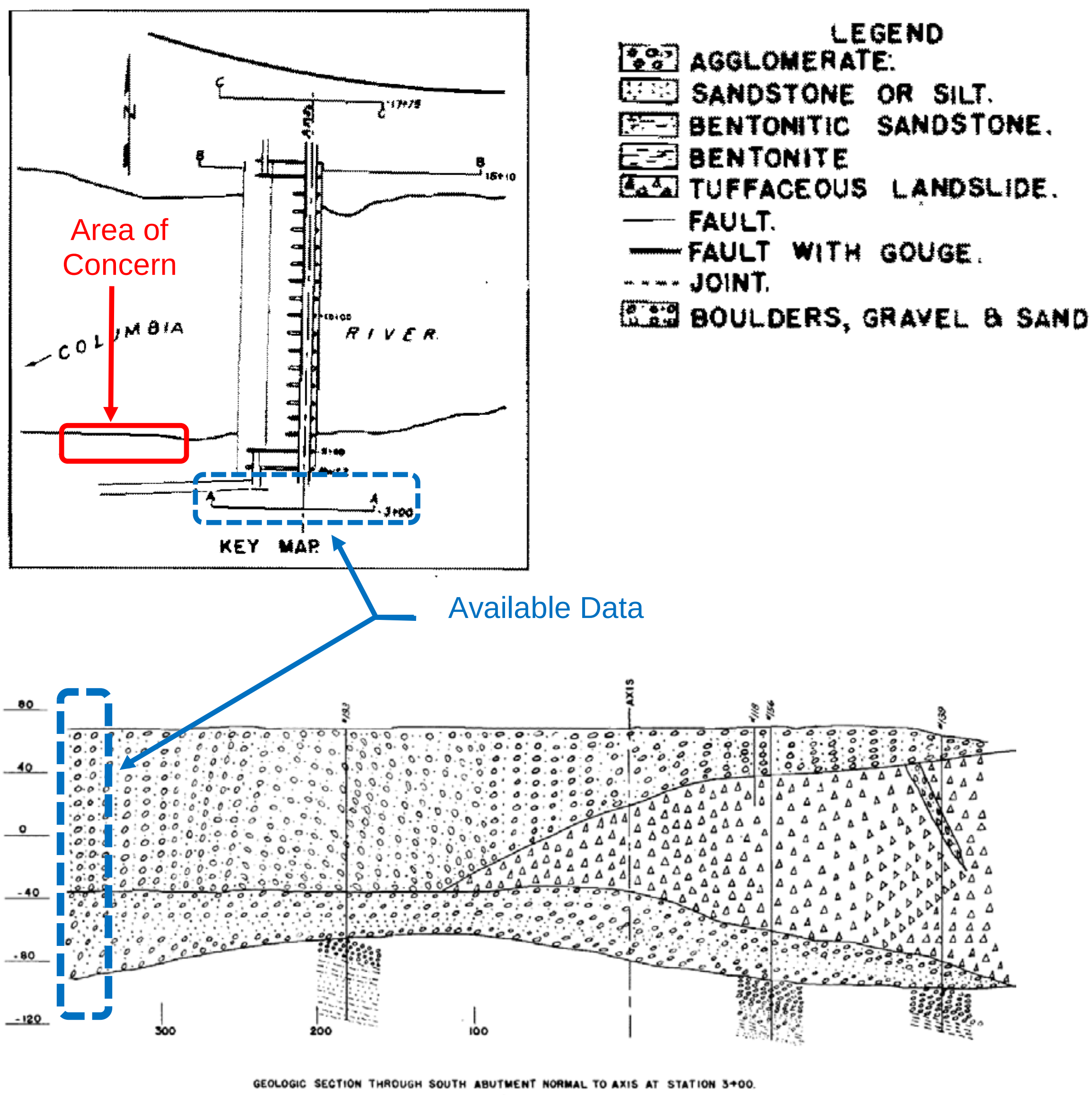


Figure 12 - Section View through South Abutment Reproduced from 1937 Final Report

[2] Report on file Z:\Misc_Resources\Dam_Safety_Routine_Tasks\PROJECTS\Bonneville\Reference_Material\Geotech Reports

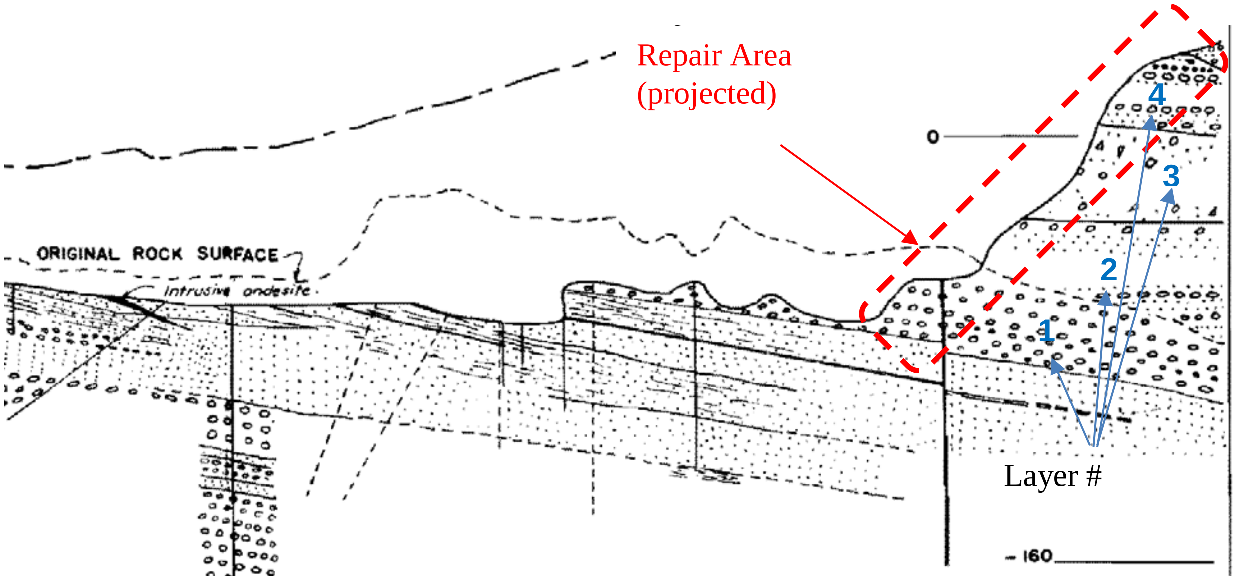


Figure 13 - Section View through Dam Axis from 1937 Final Report

A geologic description by stratigraphic layer is provided below.

Stratigraphic Layer	Geologic Description	Project Commentary
1	Agglomerates (a.k.a. volcanic sediments"	This layer is likely covered by recent alluvial deposits similar to that shown in above. However excavation into this material is likely required to meet the design grades. See more details below.
2	Boulders, Gravels and Sand and are pre-landslide alluvial materials.	These are overlain with historic revetments consisting of derrick stone. The existence of a filter layer is unknown at this time. Unlikely to encounter this material during excavation or riprap placement.
3	Landslide Deposit consisting of tuffaceous materials	Unlikely to encounter this material during excavation or riprap placement.
4	Younger layer of Boulders, Gravels and Sand	Foundation material for the fish ladder. A filter material of Class 100 Rip-Rap was placed in 2012. Some of this material should still be in place after demolition of the grouted riprap and excavating to the proposed grades.

The final report offers the following excerpt regarding the agglomerates expected in Layer #1;

> “Eagle Creek Formation. The oldest geological formation exposed in the Columbia Gorge is the Eagle. Locally this formation is composed entirely of volcanic sediments including agglomerates, conglomerates, sandstones, ashes, and even finer materials. Some of the beds, such as conglomerates and sandstones, contain materials that have been transported by and deposited in running water…
>
> The sedimentary rocks of the Eagle Creek Formation do not lend themselves very well to successful core drilling and considerable difficulty is experienced in getting good core recovery. These rocks are cut rapidly enough with its set with pieces of tungsten carbide alloy. Good core recovery is usually possible in drilling through the finer grained rock types, but if they are too badly altered to "bentonite" the material may be so soft that it is washed away by the water circulated through the drill stem. When less water is used, the cuttings are not removed fast enough and clog the core barrel. When drilling agglomerates or conglomerates, the drill is apt to recover only small pebbles and sections of larger pebbles while the matrix is all washed away. However, if a drill is used that will take a larger core than is commonly taken, better results are obtained. No drill cutting less than a 2 inch core should ever be used in making borings in rocks of this type and drills cutting from 3 inch to 4 inch cores obtain the best results.”

Based on the qualitative description of the core drilling in the eagle formation described above, the Government presumes the bedrock material at the toe of the slope should have a rock mass classification of weak (R2) to moderately strong (R3). Presumably the spillway was excavated down to a surface that would not simply wash away therefore the Government recommends the Contractor prepare to excavate R3 material. A more quantitative description is provided below.

Table 1: Field estimates of uniaxial compressive strength of intact rock.[3]

Grade*	Term	Uniaxial Comp. Strength (MPa)	Point Load Index (MPa)	Field estimate of strength	Examples
R6	Extremely Strong	> 250	>10	Specimen can only be chipped with a geological hammer	Fresh basalt, chert, diabase, gneiss, granite, quartzite
R5	Very strong	100 - 250	4 - 10	Specimen requires many blows of a geological hammer to fracture it	Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, peridotite , rhyolite, tuff
R4	Strong	50 - 100	2 - 4	Specimen requires more than one blow of a geological hammer to fracture it	Limestone, marble, sandstone, schist
R3	Medium strong	25 - 50	1 - 2	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer	Concrete, phyllite, schist, siltstone
R2	Weak	5 - 25	**	Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer	Chalk, claystone, potash, marl, siltstone, shale, rocksalt,
R1	Very weak	1 - 5	**	Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife	Highly weathered or altered rock, shale
R0	Extremely Weak	0.25 - 1	**	Indented by thumbnail	Stiff fault gouge

* Grade according to Brown (1981).
** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

15. DATUMS, TIDES, AND OTHER WATER LEVELS

The project as-built drawings show elevations in NGVD29 (MSL). However, the Corps initiated the Comprehensive Evaluation of Project Datum (CEPD) based on the lessons learned in Hurricane Katrina disaster. CEPD directs USACE to use the current national vertical reference system which is the North American Vertical Datum of 1988 (NAVD 88). The following is an excerpt from the CEPD on Bonneville Dam[3];

> "All of the Portland District dams including the Bonneville project were completed before this (NAVD88) vertical datum existed. Portland Dams were built using the previous elevation datum known as the National Geodetic Vertical Datum of 1929 (NGVD 29). NGVD 29 has become ingrained into the project. Various floors and decks are named by their NGVD elevations (the forebay decks at approximate NGVD elevation 90 on both powerhouse are referred to as the 90-deck, likewise the tailrace decks at approximate NGVD elevation 55 are known as the 55-decks). Project personnel have admitted that they some times determine an elevation by measuring up or down from the concrete floors…"

The horizontal coordinate system for hydro surveys is NAD 1983 HARN State Plane Oregon North FIPS 3601 Feet Intl.

Ordinary High Water is about 21 feet Columbia River Datum (CRD).

Datum Conversions

- MSL (+) 3.339 feet = NAVD88
- MSL (-) 8.88 feet = CRD
- OPUS Geoid o3 NAVD88 (-) 11.79 = CRD feet
- NGVD 29 was previously known as "Mean Sea Level Adjustment of 1929" (MSL). NGVD 29 datum was modified by the "Pacific Northwest Supplementary Adjustment of 1947" (NGVD 29/47). The elevation changes from NGVD 29 to NGVD 29/47 at Bonneville Dam were less than 0.2 feet.

16. WATER LEVEL FLUCTUATIONS

16.1 Tides

There is a minimal tidal impacts at Bonneville (a few inches) and it is accounted for in the tailwater data.

16.2 Storm Surge

A storm surge is not a concern at the Bonneville Project due to the inland location.

[3] \\Nwp-netapp2\staff_nwp\CenwP-OD\cenwp-od-nh\Admin\CEPD_Reports

16.3 Wave Action

A formal analysis of waves was not completed for this design. However, an interview with USACE Coastal Engineer, Rod Moritz P.E., resulted in the following assessment;

The rip-rap is designed for dealing with the maximum loading associated with:

- wind-wave action, or
- currents / standing waves from tailrace flow

Based on the limited fetch and direction from which waves could affect the revetment (wind and wave action would be associated with winds from W-SW over a distance of < 2 miles), the wave action affecting these revetments (3 ft max) would impose less "design load" than the currents associated with high flow tail-race.

17. TAILWATER ELEVATIONS

Tailwater elevations at Bonneville Dam are subject to seasonal and daily variations. River flow into the project varies seasonally and between years depending on climatic conditions and upriver flow management. Bonneville Dam is a run of the river project with a limited operational forebay range (70 to 77 feet NGVD 29) for temporary storage, so the project must on average discharge what comes into the project on a daily basis. Hourly variations in tailwater can occur daily by the fluctuating discharges needed to meet daily hydropower demands and possibly address Columbia River dam system adjustments. The ability of the Project or Columbia River dam system to adjust or restrict tailwater elevation at Bonneville Dam is highly limited.

The tailwater elevation is generally a function of total project discharge and tidal effects downstream. Project river inflow is largely a function of seasonal variations and upriver operations. Daily tailwater variations occur with either diurnal changes in inflow or changes in project discharge to meet fluctuating power demand. The rating curve for tailwater (or stage) elevation versus river discharge shown in Figure 14 for the In-Water Work period and in Figure 15 for the spring early summer freshet period. (In both rating curves, the datum for the tailwater elevations is NGVD 29. To convert to NAVD 88, add 3.34 feet.)

Elsewhere, the tailwater elevation data are shown in feet at two datum:

A. NGVD 29/47 (also known as Mean Sea Level).
   a. *Datum for informational drawings and historic hydrologic data*

B. NAVD 88 (Datum for plan sheets of proposed project)

Significant tailwater variation can occur in a single day due to variable hydropower discharges to meet power demand at this or other projects in the Columbia River system. Per project operating constraints, the allowable change in tailwater elevation are the following during the winter months (October – March) as follows:

Allowable Change in Tailwater Elevation	Time limit
3 feet (maximum)	60 minutes
7 feet (normal)	24 hours
10 feet (maximum)	24hours

During the 2016-17 in-water-work period, the priority powerhouse will be Bonneville First Powerhouse, which releases into a different discharge channel than the work site area. If power demands or river inflow rates exceed the capacity of the First Powerhouse, then either units in the Second Powerhouse or spillway gates must be operated. The hydraulic capacity of the First Powerhouse is approximately 110,000 cfs; however this capacity may be reduced by unit outages. During the in-water-work period, the remainder of the river flow must be first discharged through the Second Powerhouse. If the river inflow exceeds the hydraulic capacity of both powerhouses, the project must release discharge through the spillway and potentially disrupt the proposed work area.

Historic hourly tailwater elevation (or stage) duration data for the specific months of December - February at Bonneville Dam is provided in Table 1. All duration (or percent of time exceeded) data was compiled from an approximate record from 1973 - 1999. Historic mean daily discharge duration data for the specific months of December – February at Bonneville Dam is provided in Table 2.

17.1 Chum Salmon Tailwater Elevation Operations:

The chum tailwater elevation restriction is an operation that may affect the proposed work. This restriction pertains to maintaining a minimum tailwater throughout the in-water work period to protect Chum spawning beds downstream of Bonneville Dam. This minimum tailwater is established during November.

17.1.1 Chum Spawning Phase

In the first week of November or when fish arrive (as coordinated with the Technical Management Team (TMT), Bonneville Dam will start operating to provide a tailwater elevation (TWE) range of 14.8 - 16.3 feet NAVD 88 (11.5-13.0 feet NGVD 29) until spawning ends or December 31. An assumption of 14.8 NAVD 88 (11.5 feet NGVD 29) is prudent in most years. The official project TWE gauge is located 0.9 mile downstream of Bonneville Dam's powerhouse 1 on the Oregon shore, 50 feet upstream of Tanner Creek at river mile 144.5. Generally, the range of discharge from Bonneville Dam that is required to maintain tailwater elevation 14.8 feet NAVD 88 (11.5 feet NGBD 29) can vary from less than the project minimum discharge (80 kcfs) up to 135 kcfs. This range demonstrates the profound effect of natural conditions downstream of Bonneville Dam on the water elevation. Tides, wind, wave and unregulated inflows to the Columbia River all have an influence on the ability to regulate the tailwater elevation below Bonneville Dam with the outflow from Bonneville Dam.

In addition to the uncertainty and variability of downstream conditions that affect the tailwater elevation at Bonneville Dam, there are many upstream variables as well. Generally, the flow at Bonneville Dam is augmented by storage releases from Grand Coulee Dam which takes approximately 24 hours to arrive at Bonneville Dam and must pass through several nonfederal dams that can alter the shape and timing of the flow. Further, the volume of unregulated flow into the Columbia River upstream of Bonneville Dam is difficult to predict but is critical in meeting the spawning elevations. The ability to operate Bonneville Dam to a particular tailwater constraint is contingent on the ability of the hydrosystem to forecast and manage all of these variables and conditions. Reservoir operations upstream of Bonneville may provide additional water to help support the chum operation.

17.1.2 Chum Incubation and Egress Phase

Washington Department of Fish and Wildlife (WDFW) will inform TMT when they determine that chum spawning is complete at the Ives/Pierce Island area; this usually occurs in late December but will not extend past December 31. Following the completion of spawning, the operation is shifted to provide a tailwater elevation (to be determined by TMT) equal to or greater than the elevation of the highest redds that will be protected. This elevation is typically around 14.6 – 15.0 NAVD 88 (11.3 to 11.7 feet NGVD 29) during normal water years, which requires maintaining an approximate range of discharge between 95,000 – 145,000 cfs. However in 2012-2013 in-water work season, the required Chum tailwater elevation was over 16.8 feet NAVD 88 (13.5 feet NGVD 29) due to higher than normal river flows during the spawning period. The 16.8 foot NAVD 88 Chum operation tailwater elevation would require between 120,000 – 160,000 cfs to maintain. So an additional 20,000 – 25,000 cfs would be needed to maintain the higher 16.8-foot NAVD 88 TWE compared to a TWE of 15.3 feet NAVD 88. This extra required discharge would need to be discharged through the Second Powerhouse or spillway as the First Powerhouse will probably already be operating at capacity. Redds established due to conditions beyond the control of the action agency may not be protected. The end of the chum protection operation is coordinated at TMT after it is determined that completion of emergence and egress has occurred. The protection operation typically ends between mid-March and April 10. If the emergence period extends beyond April 10 and a decision is made to maintain the tailwater, TMT will need to discuss the impacts of TDG associated with spill and/or operation of the corner collector for fish passage at Bonneville Dam and its potential for negatively affecting fry in the gravel. However, typically spring flow augmentation volumes generally provide sufficient flows to sufficiently maintain the protection elevations.

Bonneville starts its spring spill around April 10, but a delay in the start of spill may be needed. The chum protection level decision will be revisited at least monthly through the TMT process to assure it is consistent with the need to provide spring flows for listed Columbia and Snake River stocks.

17.2 Definitions and Descriptions of Tables:

Discharge duration: The percent of time during the period of record that a specific total project discharge is exceeded (also reference to as 'percent exceedance'). The river discharge-duration curve for Bonneville is tabulated for the In-Water Work period and in for the spring/summer high flow freshet period in Table 11 and Table 12, respectively.

Note 'Discharge-Duration' is not the same as 'Probability of Annual Exceedance'—the probability that a specific total project discharge is exceeded at least once within a year. For example, the discharge at 1% probability of annual exceedance (also referred to as 100-year flood) is 680,000 cfs, whereas the discharge at 1% of time exceeded per discharge-duration data is considerably lower at 436,000 cfs for the calendar year.)

Stage Duration: The percent of time during the period of record that a specific stage is exceeded (also referenced to as percent exceedance). The stage-duration curves for the Bonneville Tailrace are provided in Table 13 for the in-water work period and in Table 14 for the highest flow period of the year (spring and early summer freshets). (Note that the respective datum for upper part (A) is NGVD 29 and NAVD 88 in both tables.) For example, referring to Table 12: during all the daily tailwater recordings (837 days) in the month of January for the years 1973 through 1999, the tailwater level exceeded 31.9 feet NAVD 88 (28.6 feet NGVD 29) only eight days, or 1% of the time, in January.

These days of exceedance can occur in consecutive days associated with a particular climatic or river wide event (such as a spring/summer freshet). Hence, it is likely that the number of years (or events) in which tailwater 31.9 feet NAVD 88 (28.6 feet NGVD 29) was exceeded was less than eight during 1973-1999.

The discharges and tailwater elevations for 1% annual chance exceedance and higher flood events are shown in Table 15.

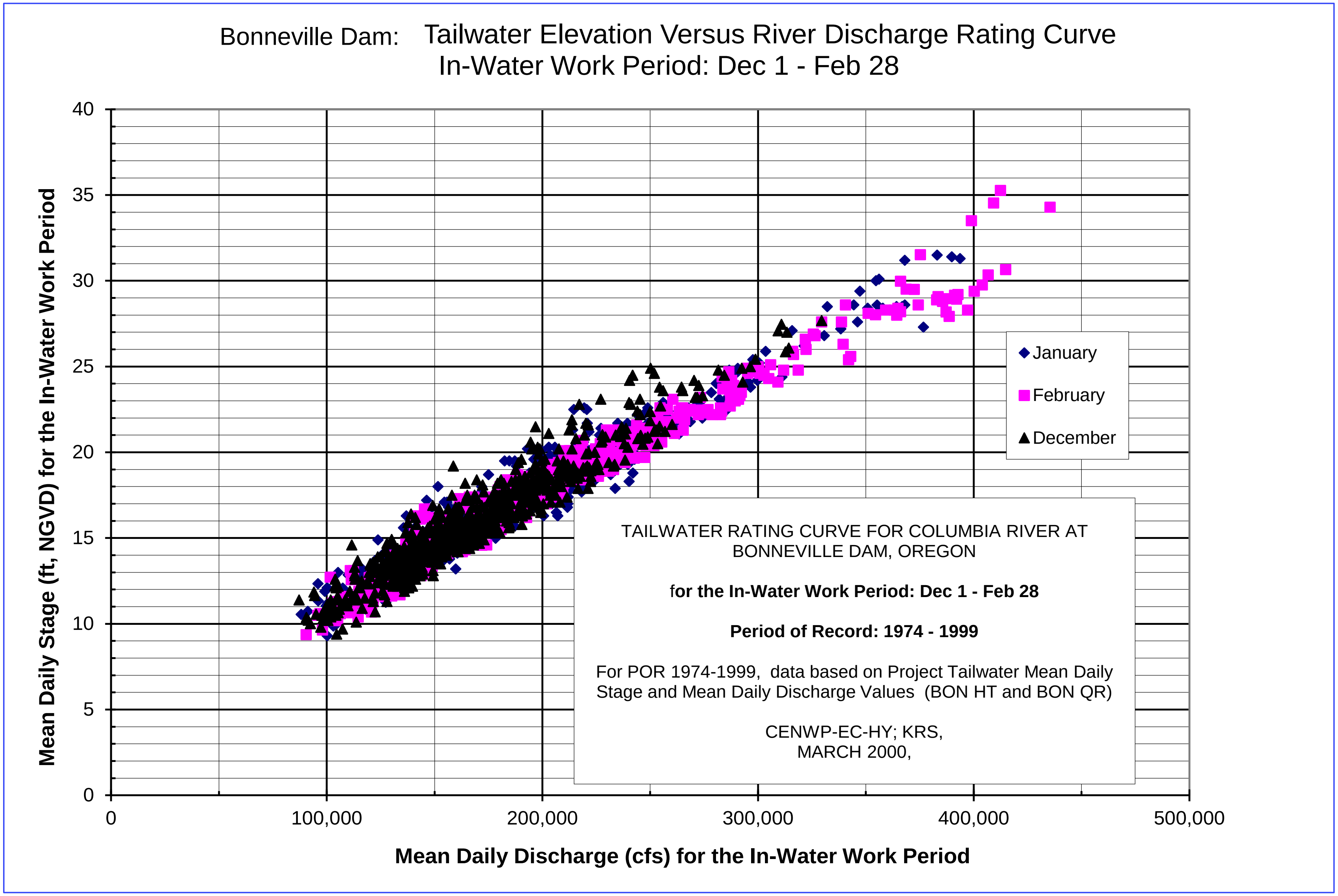


Figure 14 – Bonneville Tailwater Elevation versus Project Discharge Rating Curve during In-Water Work Period

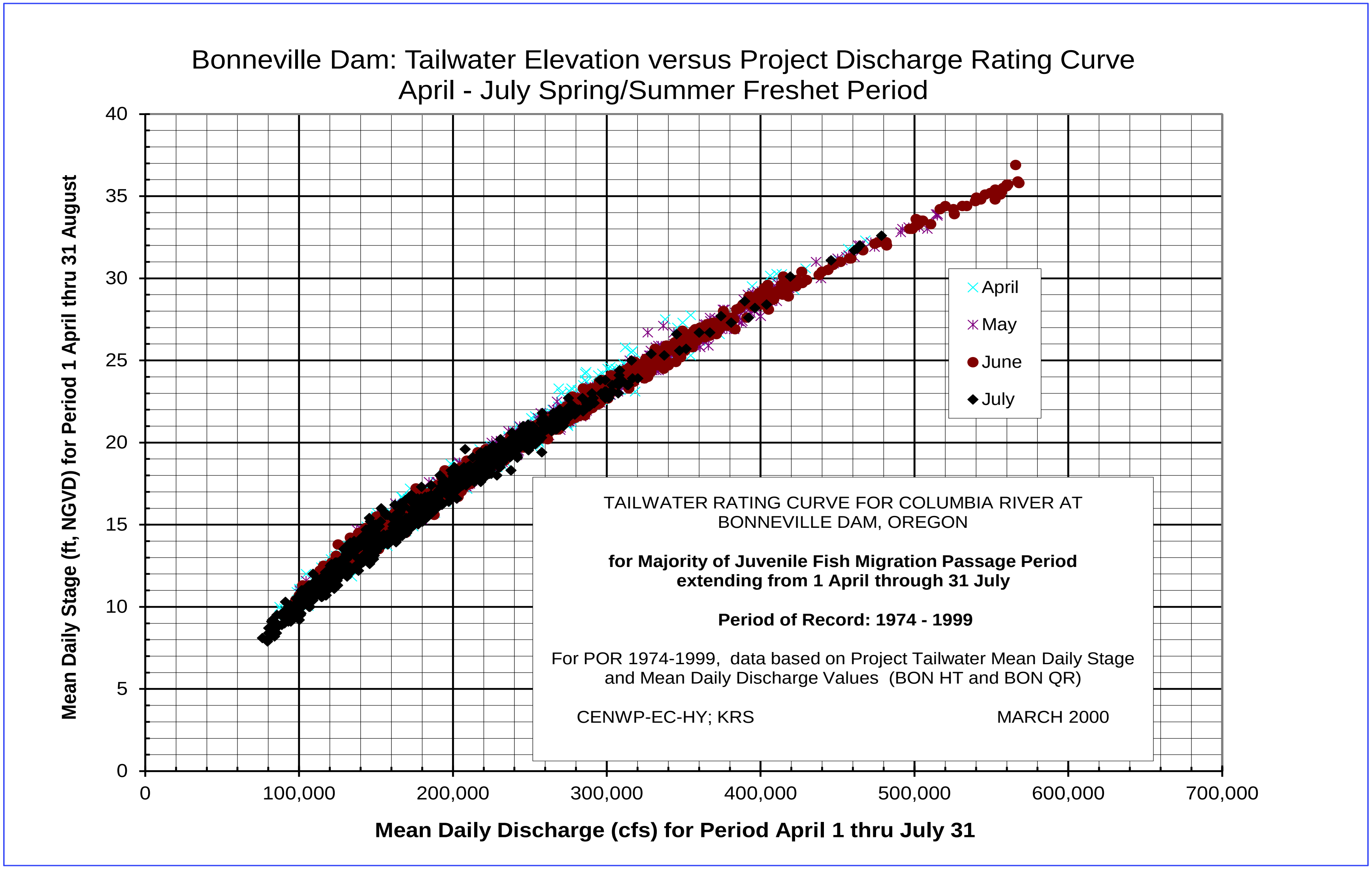


Figure 15 – Bonneville Tailwater Elevation versus Project Discharge Rating Curve during Spring/Summer Freshet Period

Table 11 – Bonneville Discharge Duration Table for In-Water Work Period

Bonneville Lock and Dam: Discharge Duration Data during Water Work Period
Mean Daily Discharge Rates (cfs) (Period of Record 1973-1999)

Percent of time exceeded	Annual	In-Water Work period			
		Dec	Jan	Feb	Mean Dec - Feb
Max	602,326	277,468	425,265	436,840	436,840
1	436,294	252,292	368,997	400,492	338,597
5	355,654	224,662	280,932	329,528	276,669
10	301,322	210,256	243,284	280,182	243,387
20	246,556	191,447	219,622	235,999	215,012
30	213,769	179,638	205,158	219,332	200,778
40	188,040	170,284	193,119	207,553	189,744
50	167,295	162,292	181,243	196,485	179,457
60	150,570	154,549	169,480	179,713	167,521
70	136,477	146,949	158,314	161,381	155,353
80	123,683	137,240	143,895	148,115	142,916
90	106,932	124,404	130,277	129,144	127,901
95	95,328	111,885	119,747	120,275	117,204
99	78,829	93,608	94,270	101,708	96,356
Min	55,787	79,283	77,569	80,358	77,569

Table 12 – Bonneville Discharge Duration Table for Spring/Summer Freshet Period

Bonneville Lock and Dam: Discharge Duration Data during Spring/Summer Freshet Period
Mean Daily Discharges April 1 Thru July 31
(Period of Record 1974-1999)

Percent of time exceeded	Annual	Spring/Summer Freshet Period				
		April	May	June	July	Mean Apr- July
Max	602,326	474,854	541,082	602,326	505,916	602,326
1%	436,294	423,210	504,459	584,071	410,084	480,076
5%	355,654	350,663	423,926	460,261	295,414	382,191
10%	301,322	319,623	395,455	418,880	272,204	351,250
20%	246,556	282,577	352,202	370,558	244,226	312,158
30%	213,769	259,373	323,666	337,165	223,309	285,675
40%	188,040	240,354	295,842	304,783	200,635	260,204
50%	167,295	214,264	273,743	280,484	179,796	236,902
60%	150,570	188,498	259,580	249,897	161,793	214,872
70%	142,957	177,778	250,662	231,966	151,298	202,894
80%	129,976	162,541	232,728	203,604	134,179	183,266
90%	116,206	149,467	216,329	172,618	116,727	163,830
95%	106,932	137,945	205,179	157,057	104,164	151,145
99%	95,328	123,060	174,391	135,827	91,759	131,289
Min	55,787	68,634	98,728	77,825	55,787	55,787

Table 13 – Bonneville Tailwater Stage Duration Table for In-Water Work Period

Bonneville Lock and Dam
Mean Daily Tailwater Elevations based on Hourly Data
(Period of Record 1973-1999)

A. DATUM: NGVD 29 (aka Mean Sea Level) (feet)					
		In-water Work period			
Percent of time exceeded	Annual	Dec	Jan	Feb	∑ Dec - Feb
Max	36.9	27.7	31.6	35.3	35.3
1%	30.1	24.9	28.6	29.9	27.7
5%	26.1	21.6	23.5	26.1	23.7
10%	23.8	20.2	21.5	22.7	21.4
20%	20.6	18.3	19.8	20.4	19.5
30%	18.7	17.2	18.5	19.3	18.3
40%	17.1	16.3	17.8	18.6	17.5
50%	15.6	15.6	16.8	17.7	16.7
60%	14.4	15.0	16.0	17.0	16.0
70%	13.3	14.3	15.1	15.5	14.9
80%	12.1	13.6	14.2	14.3	14.0
90%	10.7	12.5	13.2	12.9	12.9
95%	9.8	11.6	12.4	11.9	12.0
99%	8.5	10.4	11.1	10.6	10.7
Min	7.0	9.4	9.3	9.4	9.3

B. DATUM: NAVD 88 (feet)					
		In-water Work period			
Percent of time	Annual	Dec	Jan	Feb	∑ Dec - Feb
Max	40.2	31.0	34.9	38.6	38.6
1%	33.4	28.2	31.9	33.2	31.1
5%	29.4	24.9	26.8	29.4	27.0
10%	27.1	23.5	24.8	26.0	24.8
20%	23.9	21.6	23.1	23.7	22.8
30%	22.0	20.5	21.8	22.6	21.6
40%	20.4	19.6	21.1	21.9	20.9
50%	18.9	18.9	20.1	21.0	20.0
60%	17.7	18.3	19.3	20.3	19.3
70%	16.6	17.6	18.4	18.8	18.3
80%	15.4	16.9	17.5	17.6	17.4
90%	14.0	15.8	16.5	16.2	16.2
95%	13.1	14.9	15.7	15.2	15.3
99%	11.8	13.7	14.4	13.9	14.0
Min	10.3	12.7	12.6	12.7	12.6

Table 14 – Bonneville Tailwater Stage Duration Table for Spring/Summer Freshet Period

Bonneville Lock and Dam
Mean Daily Tailwater Elevations in feet based on Hourly Data
(Period of Record 1973-1999)

A. DATUM: NGVD 29 (aka Mean Sea Level)

Percent of time exceeded	Annual	Spring/Summer Freshet Period				
		April	May	June	July	Mean Apr- July
Max	36.9	32.3	33.9	36.9	32.6	36.9
1%	30.1	30.2	32.8	35.3	28.2	31.6
5%	26.1	25.8	29.1	31.0	23.0	27.2
10%	23.6	24.4	27.6	28.9	21.7	25.6
20%	20.6	22.3	25.6	26.6	20.1	23.6
30%	18.7	21.0	24.0	24.7	18.8	22.1
40%	17.1	19.8	22.7	23.5	17.7	20.9
50%	15.6	18.7	21.6	22.0	16.1	19.6
60%	14.4	17.0	20.8	20.3	14.7	18.2
70%	13.9	16.5	20.5	19.4	14.2	17.6
80%	12.7	15.4	19.4	17.3	12.8	16.2
90%	11.4	14.2	18.2	15.7	11.3	14.8
95%	10.7	13.6	17.5	14.6	10.6	14.1
99%	9.8	12.4	16.2	13.5	9.7	13.0
Min	7.0	9.5	10.8	10.4	7.9	7.9

B. DATUM: NAVD 88 (feet)

Percent of time exceeded	Annual	Spring/Summer Freshet Period				
		April	May	June	July	Mean Apr- July
Max	40.2	35.6	37.2	40.2	35.9	40.24
1%	33.4	33.5	36.1	38.7	31.5	34.9
5%	29.4	29.1	32.4	34.4	26.3	30.5
10%	26.9	27.7	30.9	32.2	25.0	29.0
20%	23.9	25.6	28.9	29.9	23.4	27.0
30%	22.0	24.4	27.3	28.0	22.1	25.5
40%	20.4	23.1	26.0	26.8	21.0	24.3
50%	18.9	22.0	24.9	25.3	19.4	22.9
60%	17.7	20.4	24.2	23.6	18.0	21.5
70%	17.2	19.8	23.8	22.7	17.5	21.0
80%	16.0	18.7	22.8	20.6	16.1	19.6
90%	14.7	17.5	21.5	19.0	14.6	18.2
95%	14.0	16.9	20.8	17.9	13.9	17.4
99%	13.1	15.7	19.5	16.8	13.0	16.3
Min	10.3	12.8	14.1	13.7	11.2	11.2

17.3 Flood Frequency

The project discharges and tailwater elevations for the 1% annual chance exceedance (i.e. 100 year flood) and higher events are shown below.

Table 15 - Flood Discharges and Tailwater Elevations at Bonneville Dam

Annual Chance Exceedance (ACE) or Design Event	**Project Discharge (cfs)**	**Tailwater Elevation (feet)**	
		NGVD 29/47	NAVD 88
1% ACE	680,000	40	43
Spillway Design Flow (Regulated)	850,000	45	48
Spillway Design Flow	1,600,000	68	71
Probable Maximum Flood	2,120,000	81	84

18. COST AND CONSTRUCTABILITY

Contractor will mobilize to the spillway via the Columbia River with multiple barges. Presumably, one barge for a crane, one for an excavator, and one for material (rock). The excavator will be used with a rock hammer to break up the grouted rip-rap. The crane will be used with a clamshell bucket to pick rocks from the barge and place them below the water line on the slope of the surface. A maximum of 1,580 cubic yards of alluvium may be excavated at the toe to form a key for the riprap slope. This is a high estimate because the excavation is likely to encounter bedrock and be significantly reduce. The sediment quality team assessed the sediment based on the photographic, topographic and context of the site and determined that no quality problems exists due to the course nature of the material and therefore prepared a “No Test” memorandum for record to attach to the specs. About 4,900 tons of riprap stone will be placed onto the slope after toe excavation. Once all work is completed a survey will be taken to determine acceptance of the work.

19. EROSION ANALYSIS 2011-2016

The erosion that was observed in summer 2011 and repaired in the spring of 2012 offers some lessons learned. Record drawings are in Appendix B.

Speculation in 2011 suggests that the existing riprap (circa 1965) may have eroded due to the Elevation 7 deflectors installed in 2001. Further, the 2011 water year was characterized by a low tailwater elevation and high spill rate.

The design, repair and contract was performed under “emergency conditions” because the foundation supporting the fish ladder was undermined. The foundation support issue was remedied with grout poured in through holes in the bottom of the fish ladder. Per the inspections in 2016 the grout under the fish ladder has held and there is no direct loss of support. Therefore, it is only the grouted riprap in the upper slope that failed to perform as planned.

An analysis of bathymetry data was performed with the following conclusions;

- Contrast of Mar-2012 with June-2016 survey indicates two erosion holes apparent at the water line. The upstream hole is coincident with the repair in 2011. The downstream hole in not apparent in the 2011 survey and, therefore, is new.
- The upstream hole is coincident with a large slab of concrete observed in the 20-Sep-2011 and 14-Mar-2012 surveys (See Tab “2011 - 2016 Slab Analysis Plan View”). Project personnel involved with construction at the time indicated there was some attempt to move the slab, but were unsuccessful. The PDT speculates this slab may have slid causing a loss of support to the surrounding armor because it is not observable in the June-2016 survey.
- The upstream hole had a slope of 1V:1.34H (74%) following the repair in 2012 as evidenced by the 14-Mar-2012 survey (See Tab “2011 - 2016 Slab Analysis Section View” and Tab). Note the contract called for slopes of 1V:1.5H to 1V:2H, therefore, the apparent as-built slopes were too steep in that area. Project personnel involved with construction at the time indicated that the Contractor was demobilized before the quality control survey could be completed. That QC survey was delayed due until 14-Mar-2012 because of the turbidity and debris in the water from an upstream dam removal project.
- Design documentation[4] from the 2011 repair indicated that
    - Predicted velocities of 10 fps with a recommendation for 15 fps to account for uncertainty. The Isbash equation was used to determine W_{50} = 2,000 lbs and D_{50} = 2.9 foot diameter. Unfortunately, the contractor was unable to obtain the required rock size and therefore a standard ODOT Class 2000 (i.e. W_{100}) gradation was used. This gradation only has a W_{50}

[4] pw://COE-NWPPW001POR.nwp.ds.usace.army.mil:pwnwp95/Documents/D{662642a4-bacf-495c-9faf-989ac3a8e87d}

= 700 lbs. A 2016 check on the rock used in construction with the Isbash equation again indicates it was only good for a velocity of 12.5 fps. The amount of uncertainty in the velocity was based only on judgment and the expedient choice of rip-rap was not necessarily bad in hind-sight.

- The Isbash equation does not account for slope, however research conducted by Maynord (1988) indicates the velocities used in the Isbash equation must be increased to account for slope on anything steeper than 1V:2H. Considering the design repair called for a 1V:1.5H slope the velocity should have been 18 fps resulting in W_{50} = 6,000 lbs and D_{50} = 4.0 foot diameter. Therefore, the rip-rap used in 2012 was grossly undersized. In hindsight a check on slope should have been made, however the conditions surrounding the repair would have still been the same.

- There is no indication as to why the rip-rap above the tail water elevation was reduced to Class 700. This class of riprap has a W_{50} = 200 lbs that meets the predicted velocity of 10 fps, but does not account for slope or the desired conservative velocity of 15 fps. The issue of riprap sizing may be moot given that it was grouted into position, because the Isbach equations no longer apply. The reduction in size may have been to reduce blanket thickness and tie into surrounding grades. The grouting was probably meant to account for the reduced riprap size.

- Contractor was James W. Fowler, 12775 Westview Drive Dallas, OR 97338-9632. Contract No. was W9127N-11-C-0034 NA.
- Interviews with the PDT from 2011/2012 indicates they wanted to place additional fill in 2012 but it never happened because the Contractor demobilized before a quality assurance survey could be completed See figure below. The situation was complicated by turbidity caused by the Condit dam removal upstream.
- Interviews with the PDT from 2011/2012 indicates the Contractor coordinated with Reservoir Control to limit sudden increases in tail water elevation on a daily basis. Photos of the 2016 erosion indicate the grouted layer thickness was not a full 36 inches thick near the top, however actual measurements could not be safely made. Therefore, the Contractor may have been rushed by rising tail water and the repair probably failed to meet spec.

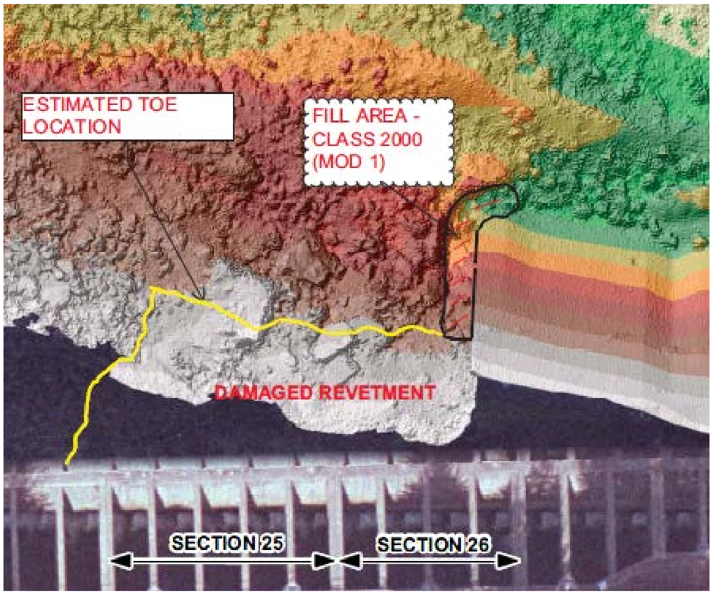


Figure 16 – As Built Hydrosurvey 2012 with Classified Elevation Part 1

- See Tabs “2012 As-Built - Repair Section” and “2012 As-Built - 3d Projection”. The files are titled as-builts because they captured what was completed under the contract. They also contain redlines for a design modification that never happened (pers. Comm Jerry Otto).

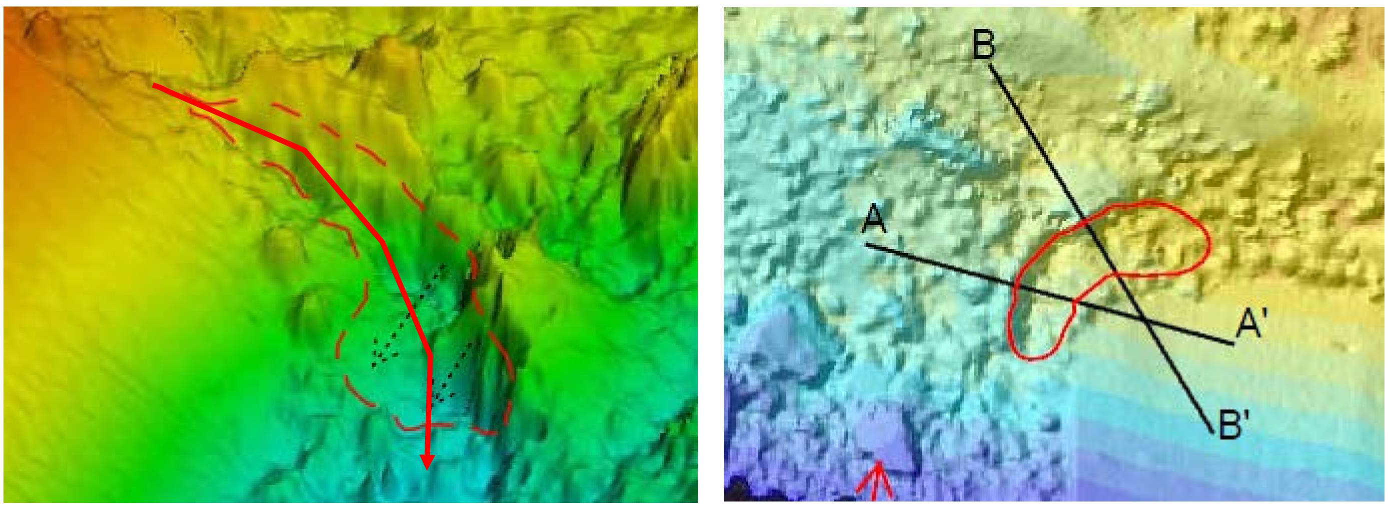


Figure 17 – As Built Hydrosurvey 2012 with Classified Elevation Data Part 2

20. REPAIRS FROM 1965

The PDT discovered some “As Constructed” drawings from 1965 in the exact same location as the 2011 and present day erosion events. Record drawings are reproduced in Appendix C and listed below.

- Revetment Restoration Plan As-Constructed M-1-62/1
- Revetment Restoration Sections As-Constructed M-1-62/2
- Revetment Inspection Sections As-Constructed M-1-61
- Revetment Restoration Section MD-1-59/2 (Design ?)

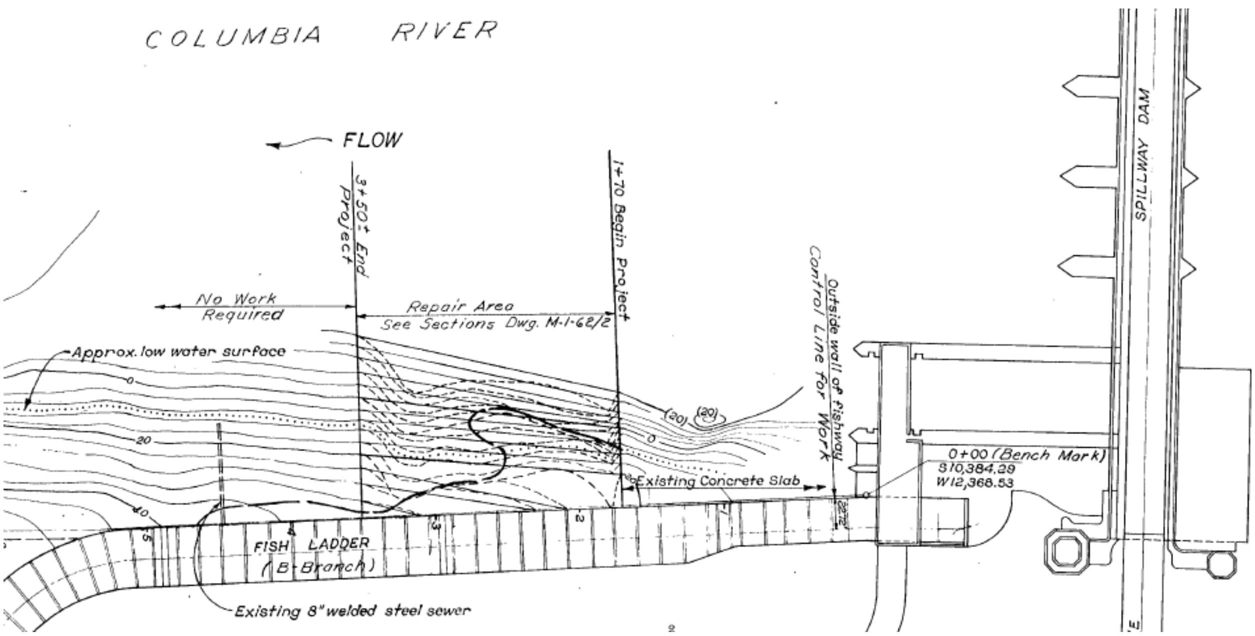


Figure 18 – Plan View of Repairs in 1965

Sections views stamped as as-built also exist. There are no specifications other than what is on the plans. The area was repaired with derrick stone- size not specified on the design drawings.

Notable observations from 1965 repair includes the following;

- Repaired slopes start at 1V:1.5H and increase gradually to meet the slope of the existing concrete apron 1V:1.25H.
- The size of the derrick stone is not specified on the drawings, however chances are the designers did not account for slope steepness explicitly because the research on that was not published by WES until 1988.

- The design drawings show that some grouted riprap was already in place prior to 1965 and that is what failed. The design drawings indicated they intended to grout in place the derrick stone above the water line and end dump below.
- The 1965 repair appeared to use 12-inch “spall bedding course” beneath the derrick stone. No indication that grouting was actually done on the as-constructed plans.

21. Product Development Team

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