Technical Memorandum

To:	Mort McMillen, PE	Project:	North Fishway Hydroelectric Project
From:	Bryan Duevel PE, GE, Jamie Schick, CEG	cc:	
Date:	March 5, 2015	Job No.:	5020.0
Subject:	Plunge Pool Condition Assessment and Scour Evaluation		

1.0 Introduction

1.1 Project background

The proposed Dalles North Fishway Project (project) involves construction of a new hydroelectric facility that will increase attraction flows and power generation capacity to the existing North Fish Ladder at the Dalles Dam. The Project owner is the Northern Wasco County People's Utility District (NWCPUD). The project is located on the North Fishway of the Dalles Dam, which spans the Columbia River at The Dalles, Oregon. The Dalles Dam is operated by the U.S. Army Corps of Engineers (USACE).

Field testing of the proposed attraction flows and associated impact on upstream migration patterns and timing is planned for this summer (anticipated to be May through September). The testing will utilize the existing AWSS channel and plunge pool at the site. The proposed test plan includes discharging 800 cubic feet per second (cfs) through the AWSS for one week durations every other week (that is, one week on, one week off). The AWSS currently sees flows only during NWCPUD powerhouse outages, which typically occur one day per month. The existing NWCPUD powerhouse currently provides water to the AWSS system during normal operation.

As part of the annual North Fishway outage, NWCPUD requested that McMillen Jacobs Associates (McMillen Jacobs) perform a plunge pool condition assessment and scour evaluation considering the proposed test program. This memorandum presents our field observations and the results of our analyses.

1.2 Summary of Existing Facility Configuration and Operations

The AWSS channel and plunge pool were originally constructed between 1954 and 1957. The system was composed of the upper plunge pool, upper channel and lower plunge pool. Each of these components were unlined (exposed rock) except for a reinforced concrete weir located at the end of the upper channel that directed flow into the lower plunge pool. The system was modified by NWCPUD between 1989 and 1991. The primary modifications were the additions of a new dewatering structure, intake screen and powerhouse in order to generate power from the AWSS flows. The new dewatering structure is a reinforced concrete structure located within the old upper plunge pool. The upper channel and lower plunge pool were not modified for the NWCPUD project (USACE, 1997). The general arrangement of the channel is shown in Photographs 1 and 3 (Appendix A).

Operational flows in the AWSS have consisted of the following:

- Prior to construction of the new powerhouse, the AWSS channel was operated on a continuous basis with flows up to 1,500 cfs.
- After construction of the powerhouse, the AWSS channel has operated only when the powerhouse is off line, which typically occurs 6 to 12 times per year for durations of about 12 hours. Flow in the channel is approximately 800 cfs during powerhouse outages (USACE, 1997).

We understand that USACE typically cleared debris from the plunge pool annually prior to the NWCPUD powerhouse construction. In the 5 years after the new powerhouse went online, an accelerated rate of erosion, scour and debris accumulation within the lower plunge pool were observed. USACE concluded that the increased erosion was the result of the following factors:

- Blast vibrations from the powerhouse construction degrading the adjacent rock.
- The age of the plunge pool.
- Short durations of high flows above design levels.
- A change in operational mode, resulting in occasional discharges with extended periods where rock exposed to freeze-thaw action, rather than constant discharge through the system.

In 1997, USACE identified that scour had reached potentially significant levels, possibly leading to larger scale rock block failures within the plunge pool vicinity. As a result of these observations, the plunge pool reinforcement project was initiated.

1.2.1 Plunge Pool Reinforcement

In the late 1990s, the plunge pool was reinforced in an effort to arrest further scour and erosion. Based on review of design drawings (USACE, 1998), the project included:

- Debris removal from the base of channel and pool.
- Scaling of sidewalls.
- Installation of rock bolts on a 6-foot by 6-foot pattern with additional spot bolts.

Based on observations of the existing plunge pool conditions, the following additional reinforcement elements were also installed:

- Irregular steel mine straps between rock bolts
- Cable netting
- Localized shotcrete
- Scour hole backfill

It is our understanding that this reinforcement has performed well. Hazardous scour and erosion has been arrested and plunge pool cleanout is now performed infrequently (once in the last 10 years). The observed conditions in the plunge pool are described in Sections 2.4 and 2.5.

2.0 Site Conditions

2.1 Previous Geologic Investigations

The subsurface conditions at the site have been well documented previously by USACE. Three boreholes were advanced in the AWSS channel and plunge pool as a part of the original exploration for the project. The additional boreholes were advanced adjacent to the pool for the NWCPUD powerhouse project. These boreholes are included on the Fish Ladder and AWS Repair design drawings (USACE, 1998).

2.2 Current Investigation

The current investigation included detailed mapping of the rock exposures in the existing plunge pool and fish ladder adjacent to the existing NWCPUD facility. Mapping was completed in accordance with our work plan dated February 2015 by a senior engineering geologist for McMillen Jacobs. Representative photographs of the plunge pool are included in Appendix A.

Rock observations on the west and east sides of the plunge pool were generally limited to the lower third as the upper portions were covered with shotcrete reinforcement. Shotcrete also covered the top and central portion of the north wall. The observations from this mapping are presented in the sections below.

Access to the plunge pool was obtained through a crane and man-basket. Communication and direction was maintained at all times with the crane operator through a NWCPUD representative that accompanied the McMillen Jacobs staff during work.

2.3 Plunge Pool Geology

The project area is underlain by basalt flows associated with the Miocene-aged Columbia River Basalt Group. These flows are part of a sequence of flood basalts that erupted from a series of vents and fissures located in eastern Oregon and Washington and Idaho from approximately 17 to 14 million years ago. Many of these flows traveled great distances, including flowing down the ancestral Columbia River to the Pacific Ocean.

In the vicinity of the Dalles Dam the specific Columbia River flows include the Priest Rapids and Roza members of the Wanapum Basalt. During site characterization and construction of the dam individual flows/units were identified using a letter designation. The three units located in the vicinity of the project include the Q, P, and P-1 flows (USACE, 1998). The following is a brief description of each unit from top to bottom.

- **Q-Unit –** This basalt is characterized as fresh to slightly weathered with wide to very widely-spaced fractures. It typically presents a “blocky” appearance in excavation due to the widely spaced columnar jointing. The contact with the underlying P unit is variable but generally dips gently to the east. Locally there is a paleosol horizon between these two units that can be a deterioration risk. Within the project area this flow is limited in extent. It is exposed in outcrops north and above the NWCPUD parking lot and within the upper channel.

- **P-Unit –** Basalt associated with this unit is typically fresh to slightly weathered and strong to very strong. Fracture spacing is variable from over 5 feet to less than 2 feet (moderately to widely spaced).The top 3 to 5 feet of this flow is vesicular and locally more weathered than the remainder of the unit. This unit is exposed from the ground surface to approximately 10 feet above the floor of the plunge pool (Figures 1 and 2, attached). It is also exposed in the fish ladder area.

- **P-1-Unit –** This unit is characterized as a subunit of the P unit that was placed in advance of the main flow. It is typically fresh to slightly weathered and moderately strong to strong. It is typically vesicular but the flow contact with the overlying P unit does not have a paleosol horizon due to the timing of emplacement. This unit has the closest fracture spacing of the three units, typically close to moderately spaced (2 inches to 2 feet). This unit is exposed in the lower 10 feet of the plunge pool (Figures 1 and 2, attached) and the fish ladder.

2.3.1 Discontinuities

Discontinuity data was collected at rock outcrops during the site reconnaissance using both scan lines and window mapping techniques. Fractures are typically steeply to moderately dipping, consistent with columnar jointing and blocky ground. Sub-horizontal fractures are also present, which can act as a release surface for block toppling. In general the joint orientations do not cluster into distinct groups. Fractures typically have low persistence (less than 10 feet) with some medium persistence (10 to 33 feet).

Fractures in the P-unit were typically widely spaced (2 feet to 6 feet) with some areas of moderately spaced (8 inches to 2 feet) fractures and generally columnar in form. On the north wall there is a zone with moderately spaced fractures that is approximately 10 to 15 feet thick and starts at approximately 20 feet above the plunge pool floor (Appendix A, Photograph 3). As discussed below this zone was treated with shotcrete during the 1998 plunge pool rehabilitation program.

The P-1 unit is typically moderately fractured but fracture spacing is more variable with zones of closely spaced (2 to 8 inches) fractures. This variability is most pronounced along the west wall within 15 to 25 feet of the plunge pool weir. An area of very closely (1 to 2 inches) to closely spaced joints was mapped at approximately 5 feet off the floor and 15 feet from the weir (Figure 2). The zone is approximately 6 feet in diameter.

2.3.2 Rock Quality Designation (RQD)

The RQD of a rock mass is evaluated by determining the percentage recovery of core in lengths greater than twice its diameter for each core run. RQD values for the different units were developed from historical boreholes completed for the project and estimated in the field using methods developed by Palstrom (2005). This method determines an RQD value for a cubic meter of rock that is independent of direction, thus eliminating the bias of rock coring. Table 1 summarizes the RQD values for rock at the site.

Table 1. Summary of RQD Values

Flow/Unit	Borehole Range[1] (Average)	Plunge Pool Measurements
Q-Unit	82 (30 to 95)	NA
P-Unit	75 (20 to 100)	78 (46 to 92)
P-1-Unit	83 (30 to 100)	75 (49-82)
P-1 Unit (Highly Fractured)	NA	30

Note 1: Data from USACE boreholes advanced in the vicinity of the plunge pool.

Overall the values reported in the USACE boring logs compare well with values developed during the plunge pool mapping. Field values for the P-1 unit are slightly lower than the values from boreholes. However, field measurements were limited to the upper 10 feet of the flow. Lower RQD values are common near flow tops and more representative of ground conditions exposed in the plunge pool.

2.4 Plunge Pool Reinforcement

A variety of reinforcement was installed during the 1998 plunge pool rehabilitation. Rock bolts were installed on the three exposed rock faces on approximate 6-foot spacing. Multiple additional spot bolts were also installed to secure key blocks. Specific details of the bolts were not identified on the drawings. Field observations and scaled design drawings indicate they were likely 10-ft long #8 bars. No information was available regarding the use of cement grout or resin for securing the anchors. Steel mine straps were installed on a ‘X’ pattern between the pattern bolts on both the west and east walls (Figures 1 and 2). These straps were rusted on their surfaces but intact.

Rockfall cable netting was installed on both the west and east plunge pool walls. Based on visual observations, we infer that this mesh was originally installed down the full length of the cut face. The lower third of the cable netting is no longer present. We infer that the lower portion of the netting has been worn away by channel flows and degradation due to regular wetting. The cable netting on the west side is missing the lower 8 feet and is severely rusted where below the static water level.

Several large scour holes were noted in the USACE plunge pool repair report (USACE, 1997). Several of these were apparently repaired by bolting several H-beams to the walls on approximately 3 to 4 foot centers and backfilled with concrete. This repair was completed on the west wall approximately mid-depth, adjacent to the intersection with the north wall (Photograph 7) and above the plunge pool weir. On the east wall, this mitigation was completed approximately mid-depth adjacent to the intersection with the north wall (Photograph 8).

The final mitigation element included the application of a thin layer of fiber-reinforced shotcrete to both the west and east walls. Shotcrete was also applied to the back wall in the area of moderately fractured rock noted in Section 2.3.1. The shotcrete may have been originally applied down the entire west and east walls, however there is very little remaining below approximately 30 to 40 feet from the surface. We infer this lower shotcrete has been eroded off the rock face. In many areas the shotcrete layer is less than 1 inch thick. It is unclear whether shotcrete was used to backfill select areas or as an attempt to cover the entire face.

2.5 Plunge Pool Scour Observations

No significant scour within the plunge pool was observed. The majority of ground improvement within the pool was in good condition with the exception of the lower slope netting and possible erosion of shotcrete on the lower west and east walls. One area of apparent scour was identified near the bottom the west wall within the P-1-Unit. As discussed above, this area is more closely fractured and includes an area of very closely to closely spaced fractures (Figure 2). Several blocks appear to have been plucked from this area and a minor scour hole approximately 6 foot in diameter and 3 to 4 feet deep has developed (Photograph 5). This scour hole coincides with the very closely to closely fractured rock.

The plunge pool floor has a ‘subrounded’ surface associated with grinding down of the exposed rock by loose debris within the pool (Photograph 9). The surface has been scoured down 3-5 feet since initial construction in the 1950s. Based on the smooth surfaces, ‘plucking’ of rock blocks is not actively occurring. The observed scour may be associated with grinding of rock blocks on the floor during spilling events.

Very little debris (less than 10 yards) was present on the plunge pool floor, and was primarily limited to small deposits near the north end of the pool. Most material is less than 6 inches in diameter. Several larger blocks (up to 3 feet in diameter) are present on the floor. These blocks may have been produced by active free-thaw action that regularly occurs during the winter months. Several possible ‘source holes’ were noted in the rock walls during the inspection. These blocks may have also been produced upstream of the plunge pool.

3.0 Scour Evaluation

3.1 Scour Analysis

Scour analyses were performed for the plunge pool following the Erodability Index Method described by Annandale (1995, 2006). Using this method, an erodability index value is calculated based on rock mass properties. Based on field and laboratory data, an erosion threshold relationship has been established, which is compared with the calculated erodability index.

Stream power was calculated for the lower plunge pool based on the channel characteristics, geometry, and channel discharge. For the proposed AWSS channel discharge of 800 cfs during the test period, the stream power was calculated to be approximately 25 kilowatt per meter squared (kW/m^2). Stream power calculations are included in Appendix B.

Rock mass properties for the plunge pool have been developed for this evaluation based on our review of the subsurface data and our field observations. Three erodability index values were developed for the range of rock mass conditions observed in the inspection (average P-Unit value, average P-1-Unit, and highly fractured P-1-Unit). The input values and erodability indices are presented in Table 2. Calculations are included in Appendix C.

Table 2. Recommended Values for Scour Analyses

Parameter[1]	P-Unit (Average)	P-1-Unit (Average)	P-1-Unit (Low RQD)
Mass Strength Number (M_s)	140	100	80
Joint Set Number (J_n)	3.3 (three joint sets+random)		
Joint Roughness Number (J_r)	1.5 (rough, planar)		
Relative Ground Structure Number (J_s)	0.5		
Block Size Number ($K_b = RQD/J_n$)	25	18	9
Discontinuity Bond Shear Strength Number ($K_d = J_r/J_a$)	1.5	0.75	0.75
Erodability Index (K)	2,600	680	270

Note 1. Parameters defined by Annandale (1995, 2006).

Figure 3 shows the erodability index values plotted against the stream power. Each value falls below the erodability threshold. Based on this analysis, the available stream power generated by the test flows are not large enough to initiate scour.

3.2 Discussion of Results

The analyses indicate that the available stream power at a channel discharge of 800 cfs is not sufficient to initiate scour within the plunge pool considering the rock mass conditions. This is consistent with the observations made in the pool during the current assessment, and with the overall performance since implementation of the plunge pool reinforcement. We note that localized highly fractured zones in the rock approach the erodability threshold, which is consistent with the small scour hole observed on the west wall.

Note that the analyses performed do not consider that the rock mass is now heavily reinforced with rock bolts, mine straps and shotcrete and thus overstate the potential erodability. Even if minor scour were to occur in isolated areas, the surrounding rock mass is sufficiently reinforced to limit its extent.

The observations and analyses indicate that the degraded rock mass conditions identified in the mid-1990's responsible for the accelerated scour are not currently exposed. The reinforcement of the plunge pool and scaling of loose rock in the late 1990s has arrested the scour that occurred previously.

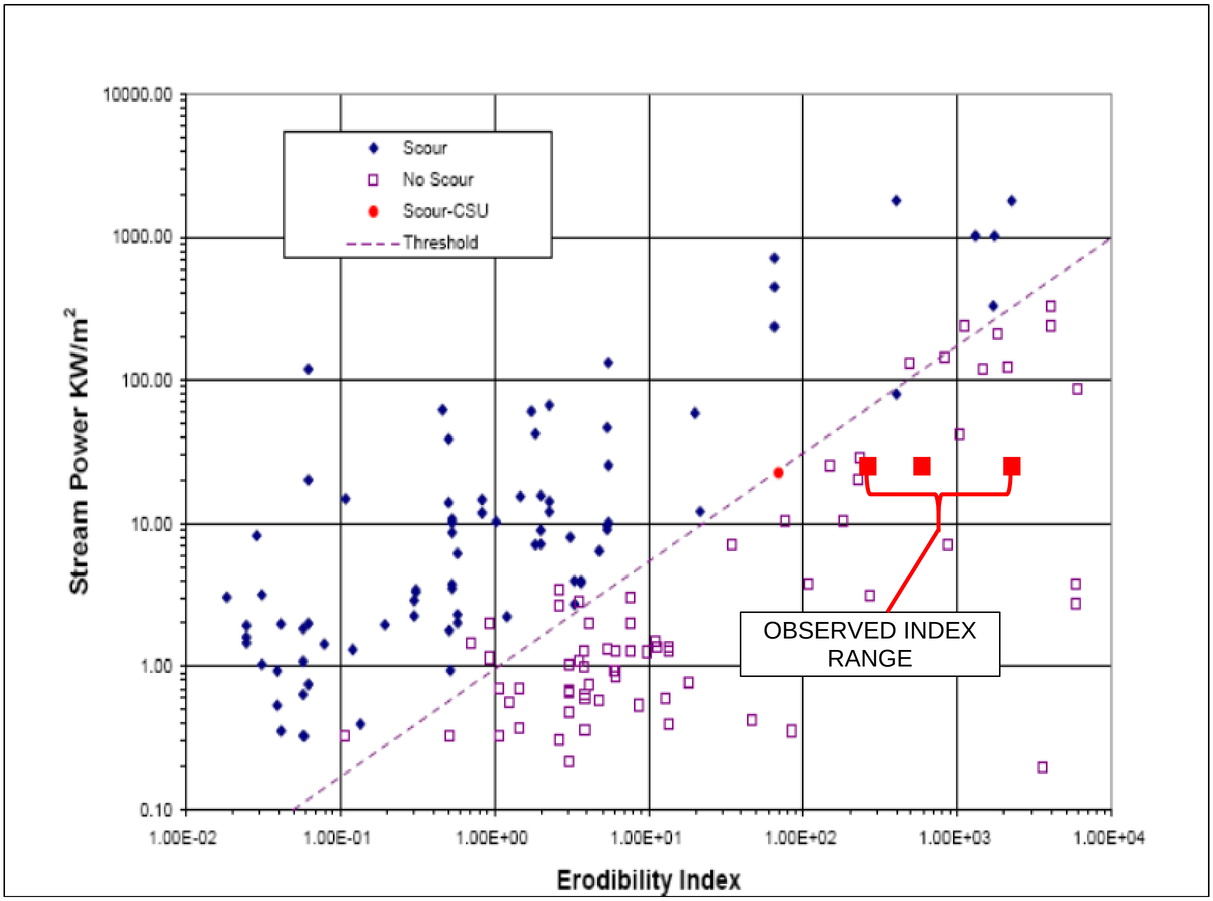


Figure 3. Erodability Index Plot

4.0 Summary and Conclusions

Our observations in the lower plunge pool indicate that in general, the reinforcement placed in the late 1990s is in good condition and performing well. The accelerated scour that was observed in the first few years after the NWCPUD powerhouse has been arrested. Evidence of recent scour in the lower plunge pool was limited to a small area on the west wall located within an isolated zone of highly fractured P-1-Unit basalt.

Analytical scour analyses were performed that indicate that the stream power of the proposed flows are below the erosion threshold of the rock mass except for zones of highly to extremely fractured rock, which present only in isolated areas. Even if minor scour were to occur in isolated areas, the surrounding rock mass is sufficiently reinforced to limit its extent.

The proposed test period will last on the order of 5 months, and will occur during the summer season. Thus, further freeze-thaw action will not be problematic during the test.

Based on the results of our observations and calculations, it is our opinion that the proposed flows will not adversely affect the AWSS channel and lower plunge pool.

We recommend that the lower plunge pool be actively monitored for evidence of scour during after the test program. The monitoring should include the following:

- Systematic photographic documentation of the of the exposed plunge pool once during each week that the AWSS is not in use by operations personnel. Photographs should be conveyed to the design team.
- Site visits by a representative of McMillen Jacobs midway through and at the conclusion of the test program to perform a more detailed inspection of the exposed plunge pool during a period that the AWSS is not in use.

In the event that unusual conditions are observed during the test period, the test program can be modified or remedial actions can be taken, if required.

5.0 References

Annandale, G.W., 1995. Erodability. *J. Hydraulic Research*, 33(4), 471-93.

Annandale, G.W., 2006. “Current Technology to Predict Scour of Rock”, in *Golden Rocks 2006, The 41st U.S. Symposium on Rock Mechanics*, 17-21 June, 2006, Golden Colorado.

Deere, D. U, Deere, D.W., 1989. “Rock quality designation (RQD) after 20 years”. *U.S. Army Corps of Engineers Contract Report GL-89-1*, Waterways Experiment Station, Vicksburg, MS (67).

Palmstrom, A. 2005, “Measurements of and Correlations between Block Size and Rock Quality Designation (RQD)”, *Tunnels and Underground Space Technology 20*, pp 362-377.

USACE, 1997. *Letter Report: The Dalles Lock and Dam, Repair of North Fishway Auxiliary Water Supply System (AWSS)*, Prepared for Chief, Construction, Operations and Readiness Division, Dated November 19, 1997.

USACE, 1998. The Dalles Lock and Dam, North Fish Ladder AWS Repair, 80% Technical Review Drawings.

<u>ATTACHMENTS</u>

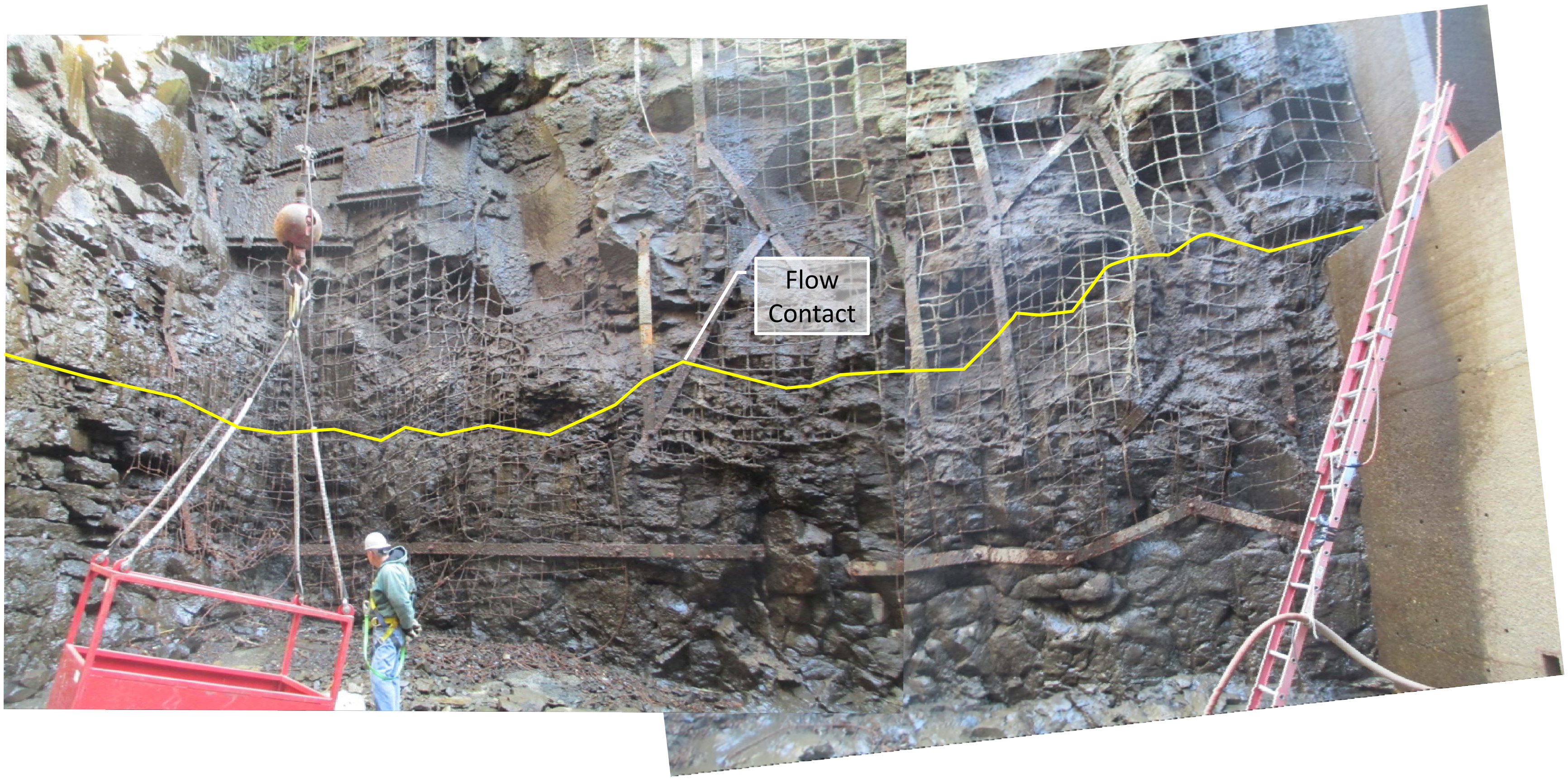
Figure 1

Figure 2

Appendix A: Site Photographs

Appendix B: Energy Calculations

Appendix C: Erodability Index Worksheets



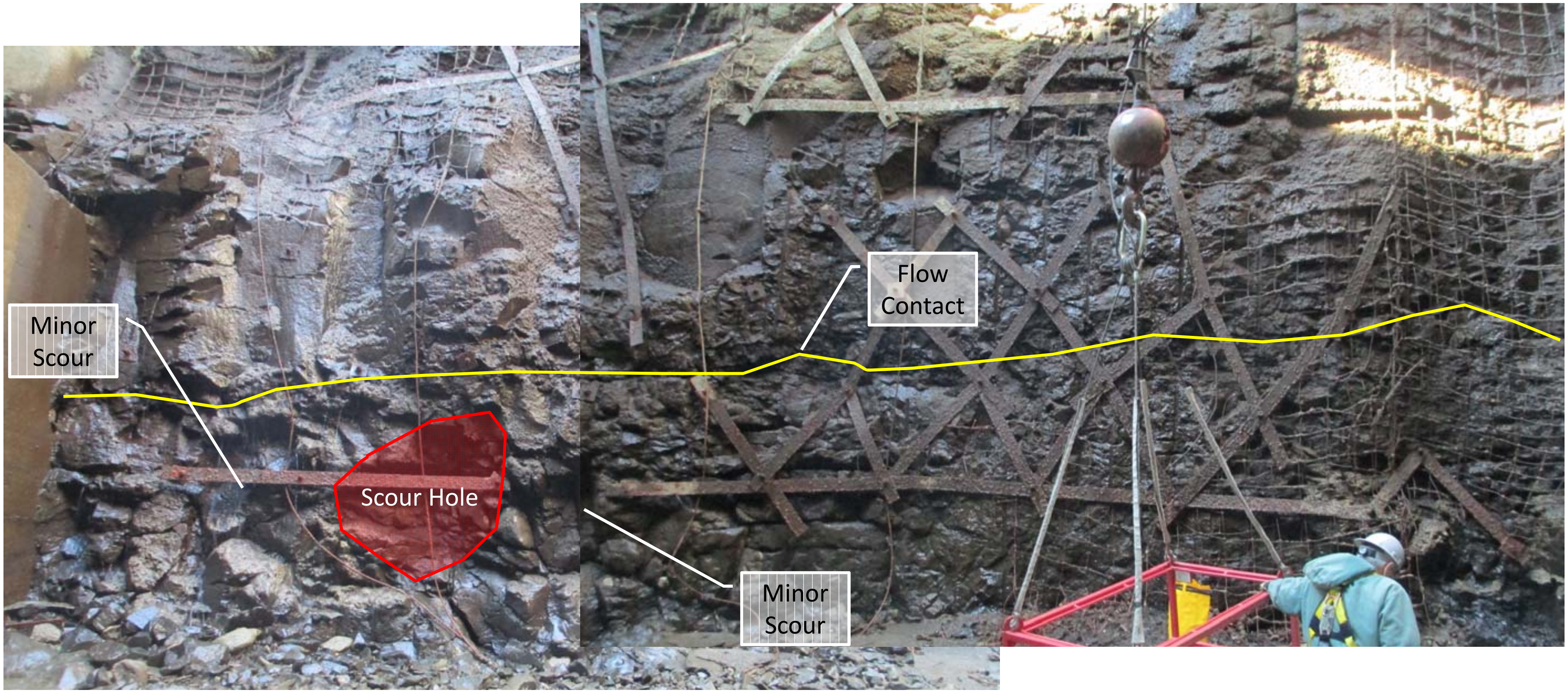
McMILLEN JACOBS ASSOCIATES

FREEDOM HYDROELECTRIC PROJECT
THE DALLES, OREGON

LOWER EAST PLUNGE POOL WALL

FIGURE
1

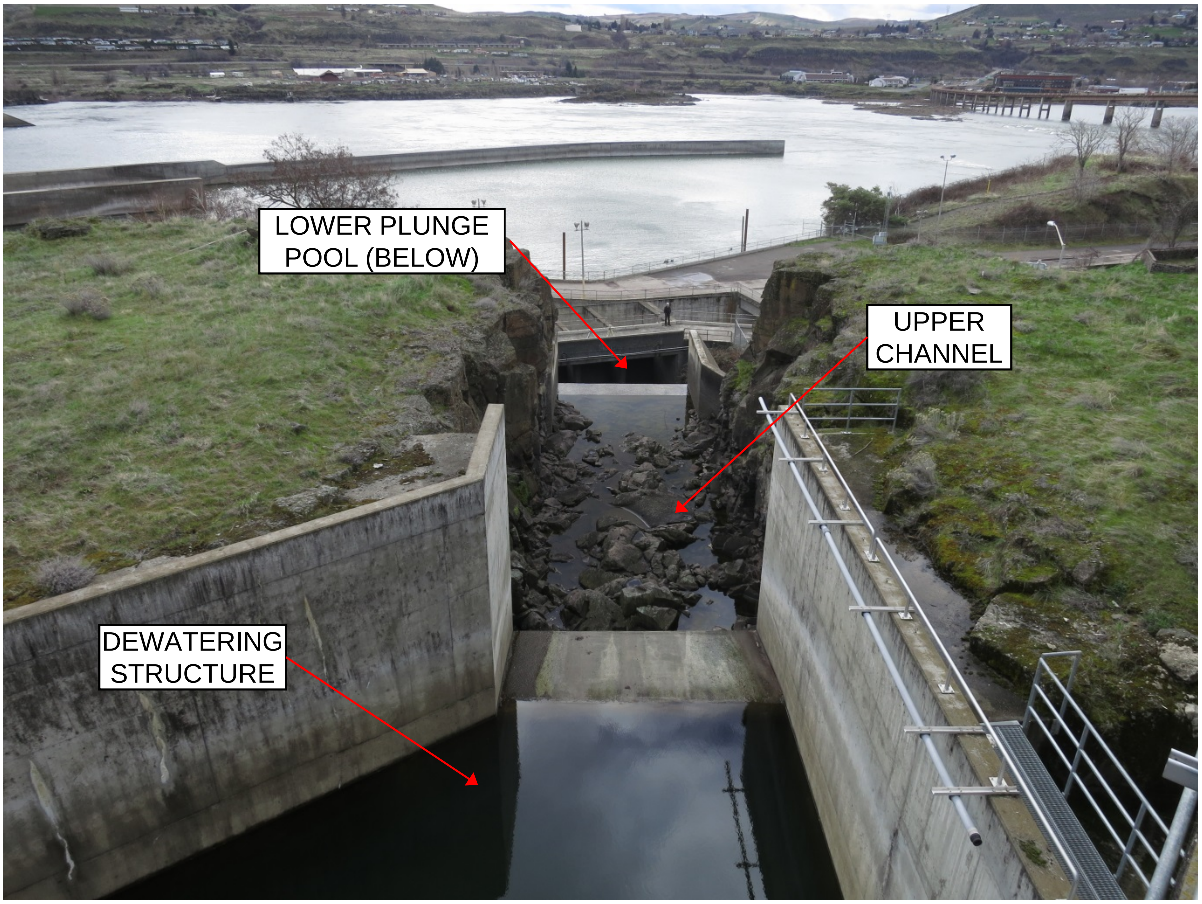
MARCH 2015



McMILLEN JACOBS ASSOCIATES	FREEDOM HYDROELECTRIC PROJECT THE DALLES, OREGON	FIGURE 2
	LOWER WEST PLUNGE POOL WALL	MARCH 2015

APPENDIX A

SITE PHOTOGRAPHS



Photograph 1. View of the AWSS channel from upstream end. Lower plunge pool cannot be seen in photograph.

Photograph 2. View of the west wall of the lower plunge pool. Ground support on this wall includes shotcrete, cable mesh, rock bolts and metal straps. The shotcrete and cable mesh are missing from most of the lower portion of the lower plunge pool.

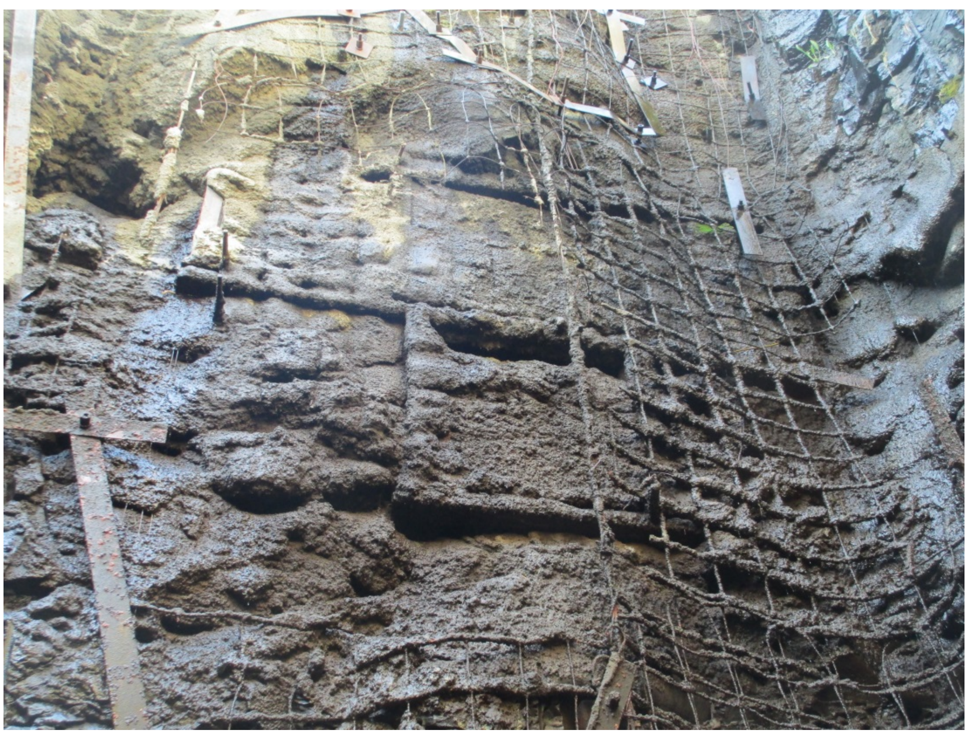
Photograph 3. View looking north at the back of the lower plunge pool. This face was supported primarily with pattern bolts and some shotcrete in the middle and immediately below the spillway..

Photograph 4. View of the east wall of the lower plunge pool. Ground support on this wall includes shotcrete, cable mesh, rock bolts and metal straps. The shotcrete and cable mesh are mostly intact down the cut face.

Photograph 5. View of scour area on the west wall. Scoured area is ~ 6 feet wide. 4 feet high and 3-4 feet deep. Exposed tape is ~ 2 feet in length.

Photograph 6. Photograph of the ground above the scoured area presented in Photograph 4. Note the pattern bolting and relatively massive basalt with crudely developed columnar jointing.

Photograph 7. West wall scour holes covered by steel beams and concrete fills.



Photograph 8. East wall scour holes covered by beams and concrete fills.

Photograph 9. Lower plunge pool floor

ATTACHMENT B

STREAM POWER CALCULATIONS

McMILLEN JACOBS ASSOCIATES	Project: The Dalles Dam North Shore Fishway System		Sheets: 3
	Subject: Plunge Pool Erodibility		Date: 2/25/2015
	By: N.Cox	Checked:	Project No. 13-045

Purpose:

The purpose of this calculation sheet is to analyze the stream power in the plunge pool and upper channel of the North Shore Fishway on the Dalles Dam

References:

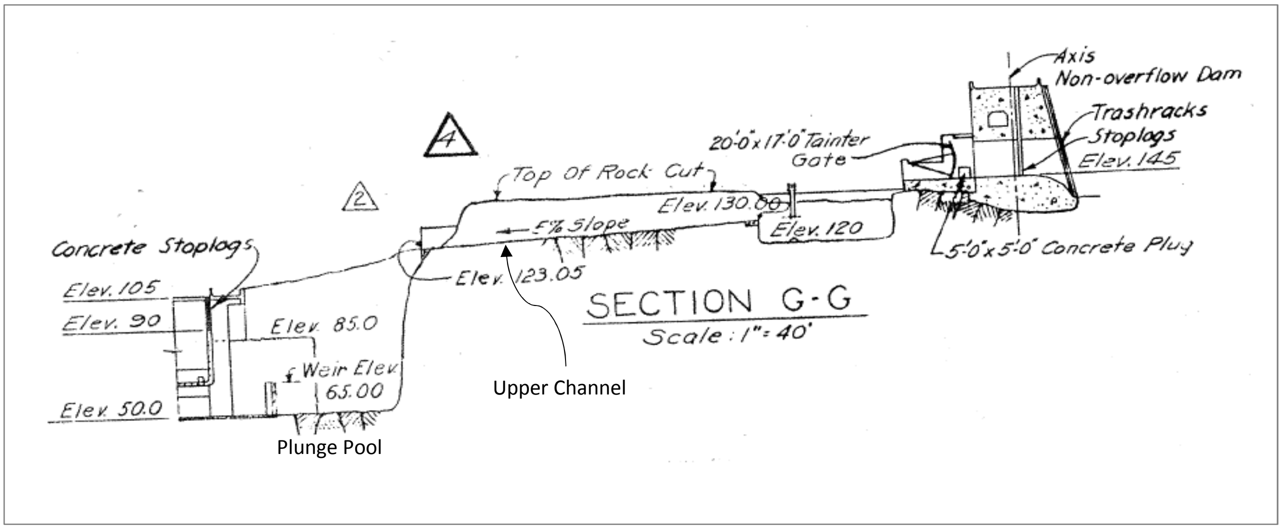
Annandale, G.W. "Erodibility". Journal of Hydraulic Research, Vol. 33, 1995, No. 4: 471-494.

George, Michael F., and Sitar, Nicholas. "Block Theory Application to Scour Assessment of Unlined Rock Spillways". Department of Civil and Environmental Engineering, University of California Berkeley, California. Report No. UCB GT 12-02, May 2012.

CENWP-PE-GG. "Memorandum, The Dalles Lock and Dam, Repair of North Fishway Auxiliary Water Supply System (AWSS), Letter Report". USACE, Portland District, November 19, 19997.

McMillen, LLC, Draft Technical Memorandum 001, The Dalles Dam North Shore fishway System, Preliminary Hydraulic Analysis. May 16, 2013.

DDF-1-4-2/1 The Dalles Dam, General Plan & Sections, 2-25-54



Calculation:

Stream Power

$$P_{sp} = \frac{\gamma Q \Delta E}{A}$$

Where:

P_{sp} = Unit Stream Power (ft lb/s/ft^2 or W/m^2)

γ = Unit Weight of Water (lb/ft^3)

Q = Discharge (ft^3/s)

A = Flow Area (ft^2)

ΔE = Energy Dissipated over the flow area (ft)

Upper Channel

Discharge =	800	ft^3/s	
Upper Channel Width =	20	ft	(Channel width as defined in drawing, DDF-1-4-2/1)
Upper Channel Slope =	0.05		(CENWP-PE-GG, 1997)
Invert El. at Fall =	123.05	ft	(Channel invert at where fall starts as defined in drawing, DDF-1-4-2/1)
Channel Length =	65	ft	(Approximate Channel Length, CENWP-PE-GG, 1997)
Low Manning's n =	0.035		(For Velocity Considerations, CENWP-PE-GG, 1997)
Est. Manning's n =	0.055		(For Depth Considerations, CENWP-PE-GG, 1997)
g =	32.2	ft/s^2	(Gravitational Acceleration)
γ =	62.37	lb/ft^3	(Specific Weight of Water)

$$\Delta E = s_f L$$

Where:
ΔE = Energy Dissipated over the flow area (ft)
s_f = Energy Slope
L = Unit Length (ft)

Low y_n =	2.60	ft	(Normal Depth for Low Manning's n)
Estimated y_n =	3.51	ft	(Normal Depth for Estimated Manning's n)
y_c =	3.68	ft	(Critical Depth at Fall)
s_f =	0.0500		(Energy Slope)

Stream Power, P_{sp} =	3,119.03	ft lb/s/ft^2	(Low Manning's n)
	45.52	kW/m^2	
Stream Power, P_{sp} =	2,313.04	ft lb/s/ft^2	(Estimated Manning's n)
	33.76	kW/m^2	

Sensitivity: Low Manning's n

Manning's n	Flow Depth (ft)	Energy Slope s_f	P_{sp} (ft lb/s/ft^2)	P_{sp} (kW/m^2)
0.035	2.08	0.0994	7,754.65	113.17
0.035	2.34	0.0691	4,788.84	69.89
0.035	2.60	0.0500	3,119.03	45.52
0.035	2.86	0.0374	2,120.69	30.95
0.035	3.12	0.0287	1,493.88	21.80

Sensitivity: Estimated Manning's n

Manning's n	Flow Depth (ft)	Energy Slope s_f	P_{sp} (ft lb/s/ft^2)	P_{sp} (kW/m^2)
0.055	2.80	0.0980	5,665.76	82.69
0.055	3.15	0.0686	3,525.63	51.45
0.055	3.51	0.0500	2,313.04	33.76
0.055	3.86	0.0377	1,583.62	23.11
0.055	4.21	0.0291	1,122.97	16.39

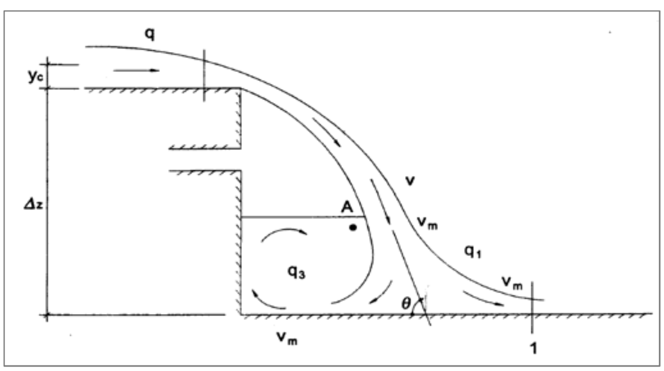
Plunge Pool

$$\Delta E = \Delta z + \frac{3}{2} y_c - y_1 - \frac{v_m^2}{2g}$$

$$v_m = \frac{v}{2}\left(1 + \cos\theta\right)$$

$$\cos\theta = \frac{1.06}{\sqrt{\frac{\Delta z}{y_c} + \frac{3}{2}}}$$

Where:
ΔE = Energy Dissipated over the flow area (ft)
Δz = Ventilated Drop Height (ft)
y_c = Critical Depth (ft)
y_1 = Downstream Flow Depth (ft)
v_m = Velocity at Moment of Entrainment (ft/s)
v = Velocity just above point A (ft/s)
θ = Jet Impingement at Base of Drop



Discharge =	800	ft^3/s	
Plunge Pool Width =	35	ft	(Estimated Plunge Pool width as defined in drawing, DDF-1-4-2/21.1)
Plunge Pool Invert El. =	50	ft	(Invert Elevation, Scour Depth as much as 5 feet below, CENWP-PE-GG, 1997)
Δz =	73.05	ft	(Difference from Channel Invert at Fall and Plunge Pool Invert)
Minimum Tailwater =	72.7		(Minimum Tailwater Elevation at Fish Ladder Entrance, TM-001, section 2.6)

H_L Fish Entrance =	1.5	ft	(Head Loss at Fish Entrance, TM-001, section 3.9)
H_L Diffuser System =	3.0	ft	(Head Loss in AWSS Diffuser System, TM-001, section 3.8)
y_1 =	27.2	ft	(Downstream Flow Depth based on Minimum Tailwater Elevation & Head Losses)
g =	32.2	ft/s^2	(Gravitational Acceleration)
γ =	62.37	lb/ft^3	(Specific Weight of Water)

Kinematics

$$v_f^2 = v_i^2 + 2ad$$

Where:

v_f = Final Velocity
v_i = Initial Velocity
a = Acceleration
d = Distance

v_i =	11.41	ft/s	(Velocity in Upper Channel at Fall)
Upper Channel Slope =	0.05	Radians	(Arc Tangent of slope)
v_{ix} =	11.40	ft/s	(Horizontal Component of Velocity, based on Slope)
v_{iy} =	0.57	ft/s	(Vertical Component of Velocity, based on Slope)
d =	45.85	ft	(Vertical Distance to Water Surface)
v_{fx} =	11.40	ft/s	(Horizontal Component of Velocity, before jet enters Tailwater)
v_{fy} =	54.34	ft/s	(Vertical Component of Velocity, before jet enters Tailwater)
v_f =	55.52	ft/s	(Final Velocity before jet enter Tailwater)

Δz =	73.05	ft	(Difference from Channel Invert at Fall and Plunge Pool Invert)
y_c =	3.51	ft	(Normal Depth is being used instead of Critical Depth Because Supercritical Flow)
y_1 =	27.2	ft	(Downstream Flow Depth based on Minimum Tailwater Elevation & Head Losses)
v =	55.52	ft/s	(Velocity just above point A)
Cos θ =	0.224		(Based on Equation above)
v_m =	33.99	ft/s	(Velocity at Moment of Entrainment)
ΔE =	33.17	ft	(Approximated Energy Dissipated)

Stream Power, P_{sp} =	1,738.50	ft $lb/s/ft^2$	(Estimated Manning's n)
	25.37	kW/m^2	

Sensitivity: Flow Depth over Free Fall

Depth at Fall y_c (ft)	Velocity at Fall v_i (ft/s)	Vel just above point A (ft/s)	ΔE (ft)	P_{sp} (ft $lb/s/ft^2$)	P_{sp} (kW/m^2)
2.80	14.26	56.18	32.36	1,695.82	24.75
3.15	12.68	55.80	32.78	1,718.29	25.08
3.51	11.41	55.52	33.17	1,738.50	25.37
3.86	10.37	55.32	33.53	1,757.40	25.65
4.21	9.51	55.16	33.88	1,775.53	25.91

Sensitivity: Plunge Pool Width

Plunge Pool Width (ft)	P_{sp} (ft $lb/s/ft^2$)	P_{sp} (kW/m^2)
28.00	2,173.13	31.71
31.50	1,931.67	28.19
35.00	1,738.50	25.37
38.50	1,580.46	23.07
42.00	1,448.75	21.14

ATTACHMENT C

ERODABILITY INDEX CALCULATIONS

Rock Mass Erodibility Index

Based on Annandale, G., *1995. Erodibility.* Journal of Hydraulic Research, Vol 33. No. 4. pp 471-494

JACOBS ASSOCIATES

1109 First Ave. Suite 501, Seattle, WA 98101

Project: **Freedom**
Date: **2/27/2015**
Location: **Average of P unit**
Latitude
Longitude
Rock Type: **Basalt**
Elevation, ft: **Not Recorded**
Geologist/Engineer: **Schick**

M_s; Mass Strength Number

Hardness	Identification in Profile	USC, Mpa
Very soft rock	A. Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife; B. is too hard to cut triaxial sample by hand	A. Less than 1.7 B. 1.7 - 3.3
Soft rock	A. Can just be scraped and peeled with a knife; B. indentations 1 mm to 3 mm show in the specimen with firm (moderate) blows of the pick point.	A. 3.3 - 6.6 B. 6.6 - 13.2
Hard Rock	Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow.	13.2 - 26.4
Very Hard Rock	Hand-held specimen breaks with hammer end of pick under more that on blow.	26.4 - 53.0 53.0 - 106.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material	Larger than 212.0

J_a; Joint Alteration Number

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	1.0[1]	1.0 - 5.0[2]	5.0[3]
Tightly healed, hard, non-softening impermeable filling	0.75	---	---
Unaltered joint walls, surface staining only	1.0	---	---
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	2.0	4.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0*	10.0*
Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock	3.0*	6.0**	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0*	13.0*
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0*	8.0**	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0*	10.0**	18.0

Note: 1. Joint walls effectively in contact.
2. Joint wall come into contact after approximately 100 mm shear.
3. Joint wall come into contact at all upon shear.
4. *Values added to Barton et el's data.
5. **Also applies when crushed rock occurs in clay gouge without rock wall contact.

K_b; Block Size Number

J_n; Joint Set Number

Number of Joint Sets	Joint Set Number, J_n
Intact, no or few joints/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

Erodibility Index

UCS, Mpa	140
J_v; Joint Volumetric Count	8
RQD	82.00
Jn	3.34
Ja	1.00
Jr	1.50

Annandale's M_s number	140
Annandale's K_b number	24.55
Annandale's K_d number	1.50
Annandale's J_s number	0.5

Erodibility Index (K) 2578

Required Stream Power (RSP) = 1(K)^0.75 362.38 **kW/m^2**

$$K = M_s K_b K_d J_s$$

Where:

M_s is the mass strength number

K_b is the block size number

K_d is the discontinuity bond strength number

J_s is the relative ground structure number

RQD = 115-3.3(Jv)

Jv; Joint Volumetric Count

J_v	x	y	z
	2	3	3

J_s; Relative Ground Structure Number

Dip Direction of Closer Spaced Joint Set (degrees)	Dip Angle of Closer Spaced Joint Set (degrees)	Ration of Joint Spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.14	1.20	1.24	1.26
In direction of flow	89	0.78	0.71	0.65	0.61
	85	0.73	0.66	0.61	0.57
	80	0.67	0.60	0.55	0.52
	70	0.56	0.50	0.46	0.43
	60	0.50	0.46	0.42	0.40
	50	0.49	0.46	0.43	0.41
	40	0.53	0.49	0.46	0.45
	30	0.63	0.59	0.55	0.53
	20	0.84	0.77	0.71	0.67
	10	1.25	1.10	0.98	0.90
	5	1.39	1.23	1.09	1.01
	1	1.50	1.33	1.19	1.10
0/180	0	1.14	1.09	1.05	1.02
Against direction of flow	-1	0.78	0.85	0.90	0.94
	-5	0.73	0.79	0.84	0.88
	-10	0.67	0.72	0.78	0.81
	-20	0.56	0.62	0.66	0.69
	-30	0.50	0.55	0.58	0.60
	-40	0.49	0.52	0.55	0.57
	-50	0.53	0.56	0.59	0.61
	-60	0.63	0.68	0.71	0.73
	-70	0.84	0.91	0.97	1.01
	-80	1.26	1.41	1.53	1.61
	-85	1.39	1.55	1.69	1.77
	-89	1.50	1.68	1.82	1.91
180/0	-90	1.14	1.20	1.24	1.26

Notes: 1. For intact material take J_s = 1.0
2. For values of r greater than 8 take Js as for r = 8

J_r; Joint Roughness Number

Joint Separation	Condition of Joint	Joint Roughness Number
Joints/fissures tight or closing during excavation	Discontinuous joints/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Slickensided undulating	1.5
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation	1.0
	Shattered or micro-shattered clays	1.0

Rock Mass Erodibility Index

Based on Annandale, G., *1995. Erodibility.* Journal of Hydraulic Research, Vol 33. No. 4. pp 471-494

JACOBS ASSOCIATES

1109 First Ave. Suite 501, Seattle, WA 98101

Project:	**Freedom**	Location:	**Average P-I unit**	Rock Type:	**Basalt**		
Date:	**2/27/2015**	Latitude		Elevation, ft:	**Not Recorded**	Geologist/Engineer:	**Schick**
		Longitude					

M_s; Mass Strength Number

Hardness	Identification in Profile	USC, Mpa
Very soft rock	A. Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife; B. is too hard to cut triaxial sample by hand	A. Less than 1.7 B. 1.7 - 3.3
Soft rock	A. Can just be scraped and peeled with a knife; B. indentations 1 mm to 3 mm show in the specimen with firm (moderate) blows of the pick point.	A. 3.3 - 6.6 B. 6.6 - 13.2
Hard Rock	Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow.	13.2 - 26.4
Very Hard Rock	Hand-held specimen breaks with hammer end of pick under more that on blow.	26.4 - 53.0 53.0 - 106.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material	Larger than 212.0

J_a; Joint Alteration Number

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	1.0[1]	1.0 - 5.0[2]	5.0[3]
Tightly healed, hard, non-softening impermeable filling	0.75	---	---
Unaltered joint walls, surface staining only	1.0	---	---
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	2.0	4.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0*	10.0*
Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock	3.0*	6.0**	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0*	13.0*
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0*	8.0**	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0*	10.0**	18.0

Note: 1. Joint walls effectively in contact.
2. Joint wall come into contact after approximately 100 mm shear.
3. Joint wall come into contact at all upon shear.
4. *Values added to Barton et el's data.
5. **Also applies when crushed rock occurs in clay gouge without rock wall contact.

K_b; Block Size Number

J_n; Joint Set Number

Number of Joint Sets	Joint Set Number, J_n
Intact, no or few joints/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

Erodibility Index

UCS, Mpa	**100**
J_v; Joint Volumetric Count	**8**
RQD	**60.00**
Jn	**3.34**
Ja	**2.00**
Jr	**1.50**
Annandale's M_s number	**100**
Annandale's K_b number	**17.96**
Annandale's K_d number	**0.75**
Annandale's J_s number	**0.5**
Erodibility Index (K)	**674**

Required Stream Power (RSP) = 1(K)^0.75 **131.83** **kW/m^2**

$K = M_s K_b K_d J_s$

Where:

M_s is the mass strength number

K_b is the block size number

K_d is the discontinuity bond strength number

J_s is the relative ground structure number

RQD = 115-3.3(Jv)

Jv; Joint Volumetric Count

J_v	x	y	z
	2	3	3

J_s; Relative Ground Structure Number

Dip Direction of Closer Spaced Joint Set (degrees)	Dip Angle of Closer Spaced Joint Set (degrees)	Ration of Joint Spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.14	1.20	1.24	1.26
In direction of flow	89	0.78	0.71	0.65	0.61
	85	0.73	0.66	0.61	0.57
	80	0.67	0.60	0.55	0.52
	70	0.56	0.50	0.46	0.43
	60	0.50	0.46	0.42	0.40
	50	0.49	0.46	0.43	0.41
	40	0.53	0.49	0.46	0.45
	30	0.63	0.59	0.55	0.53
	20	0.84	0.77	0.71	0.67
	10	1.25	1.10	0.98	0.90
	5	1.39	1.23	1.09	1.01
	1	1.50	1.33	1.19	1.10
0/180	0	1.14	1.09	1.05	1.02
Against direction of flow	-1	0.78	0.85	0.90	0.94
	-5	0.73	0.79	0.84	0.88
	-10	0.67	0.72	0.78	0.81
	-20	0.56	0.62	0.66	0.69
	-30	0.50	0.55	0.58	0.60
	-40	0.49	0.52	0.55	0.57
	-50	0.53	0.56	0.59	0.61
	-60	0.63	0.68	0.71	0.73
	-70	0.84	0.91	0.97	1.01
	-80	1.26	1.41	1.53	1.61
	-85	1.39	1.55	1.69	1.77
	-89	1.50	1.68	1.82	1.91
180/0	-90	1.14	1.20	1.24	1.26

Notes: 1. For intact material take J_s = 1.0
2. For values of r greater than 8 take Js as for r = 8

J_r; Joint Roughness Number

Joint Separation	Condition of Joint	Joint Roughness Number
Joints/fissures tight or closing during excavation	Discontinuous joints/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Slickensided undulating	1.5
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation	1.0
	Shattered or micro-shattered clays	1.0

Rock Mass Erodibility Index

Based on Annandale, G., *1995. Erodibility.* Journal of Hydraulic Research, Vol 33. No. 4. pp 471-494

JACOBS ASSOCIATES

1109 First Ave. Suite 501, Seattle, WA 98101

Project: **Freedom**
Date: **2/27/2015**
Location: **Highly Fractured P-1 unit**
Latitude
Longitude
Rock Type: **Basalt**
Elevation, ft: **Not Recorded**
Geologist/Engineer: **Schick**

M_s; Mass Strength Number

Hardness	Identification in Profile	USC, Mpa
Very soft rock	A. Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife; B. is too hard to cut triaxial sample by hand	A. Less than 1.7 B. 1.7 - 3.3
Soft rock	A. Can just be scraped and peeled with a knife; B. indentations 1 mm to 3 mm show in the specimen with firm (moderate) blows of the pick point.	A. 3.3 - 6.6 B. 6.6 - 13.2
Hard Rock	Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow.	13.2 - 26.4
Very Hard Rock	Hand-held specimen breaks with hammer end of pick under more that on blow.	26.4 - 53.0 53.0 - 106.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material	Larger than 212.0

J_a; Joint Alteration Number

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	1.0[1]	1.0 - 5.0[2]	5.0[3]
Tightly healed, hard, non-softening impermeable filling	0.75	---	---
Unaltered joint walls, surface staining only	1.0	---	---
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	2.0	4.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0*	10.0*
Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock	3.0*	6.0**	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0*	13.0*
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0*	8.0**	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0*	10.0**	18.0

Note: 1. Joint walls effectively in contact.
2. Joint wall come into contact after approximately 100 mm shear.
3. Joint wall come into contact at all upon shear.
4. *Values added to Barton et el's data.
5. **Also applies when crushed rock occurs in clay gouge without rock wall contact.

K_b; Block Size Number

J_n; Joint Set Number

Number of Joint Sets	Joint Set Number, J_n
Intact, no or few joints/fissures	1.00
One joint/fissure set	1.22
One joint/fissure set plus random	1.50
Two joint/fissure sets	1.83
Two joint/fissure sets plus random	2.24
Three joint/fissure sets	2.73
Three joint/fissure sets plus random	3.34
Four joint/fissure sets	4.09
Multiple joint/fissure sets	5.00

Erodibility Index

UCS, Mpa	80
J_v; Joint Volumetric Count	8
RQD	30.00
Jn	3.34
Ja	2.00
Jr	1.50
Annandale's M_s number	80
Annandale's K_b number	8.98
Annandale's K_d number	0.75
Annandale's J_s number	0.5
Erodibility Index (K)	269

Required Stream Power (RSP) = 1(K)^0.75 **66.09** **kW/m^2**

$K = M_s K_b K_d J_s$

Where:

M_s is the mass strength number
K_b is the block size number
K_d is the discontinuity bond strength number
J_s is the relative ground structure number

RQD = 115-3.3(Jv)
Jv; Joint Volumetric Count

J_v	x	y	z
	2	3	3

J_s; Relative Ground Structure Number

Dip Direction of Closer Spaced Joint Set (degrees)	Dip Angle of Closer Spaced Joint Set (degrees)	Ration of Joint Spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.14	1.20	1.24	1.26
In direction of flow	89	0.78	0.71	0.65	0.61
	85	0.73	0.66	0.61	0.57
	80	0.67	0.60	0.55	0.52
	70	0.56	0.50	0.46	0.43
	60	0.50	0.46	0.42	0.40
	50	0.49	0.46	0.43	0.41
	40	0.53	0.49	0.46	0.45
	30	0.63	0.59	0.55	0.53
	20	0.84	0.77	0.71	0.67
	10	1.25	1.10	0.98	0.90
	5	1.39	1.23	1.09	1.01
	1	1.50	1.33	1.19	1.10
0/180	0	1.14	1.09	1.05	1.02
Against direction of flow	-1	0.78	0.85	0.90	0.94
	-5	0.73	0.79	0.84	0.88
	-10	0.67	0.72	0.78	0.81
	-20	0.56	0.62	0.66	0.69
	-30	0.50	0.55	0.58	0.60
	-40	0.49	0.52	0.55	0.57
	-50	0.53	0.56	0.59	0.61
	-60	0.63	0.68	0.71	0.73
	-70	0.84	0.91	0.97	1.01
	-80	1.26	1.41	1.53	1.61
	-85	1.39	1.55	1.69	1.77
	-89	1.50	1.68	1.82	1.91
180/0	-90	1.14	1.20	1.24	1.26

Notes: 1. For intact material take J_s = 1.0
2. For values of r greater than 8 take Js as for r = 8

J_r; Joint Roughness Number

Joint Separation	Condition of Joint	Joint Roughness Number
Joints/fissures tight or closing during excavation	Discontinuous joints/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Slickensided undulating	1.5
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation	1.0
	Shattered or micro-shattered clays	1.0