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AN INTEGRATED STRATEGY FOR SEEPAGE CONTROL FROM BULL RUN DAM # 2 SPILLWAY APPROACH CANAL [(*)]

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1. INTRODUCTION

Bull Run Dam # 2 is located approximately 48 kilometers (30 miles) east of Portland, Oregon, USA, on the western slope of Mount Hood. The zoned-earthfill dam is part of a large watershed facility, and is one of two main dams which supply water to the City of Portland. Bull Run Dam # 2 and its appurtenant structures were completed in 1961. Initial reservoir filling occurred from December 21, 1961, until January 10, 1962, at which time the reservoir reached its full capacity of 26 million m^3 (21,000 acre-feet). A site plan (Fig.1) shows the project layout including the embankment dam, spillway approach canal, spillway, headworks, and the central area (between the dam, canal and river).

The dam site, approach canal, and spillway were built on a massive ancient landslide. The slide debris consists predominantly of blocky volcanic breccia, with occasional basalt boulders and stiff clay zones. The majority of this material originated from the Rhododendron Formation - an indurated volcanic mudflow

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[(*)]Une strategie pour le control de suintement du barrage “Bull Run” #2 le canal d’ entree de la voie deroutement.

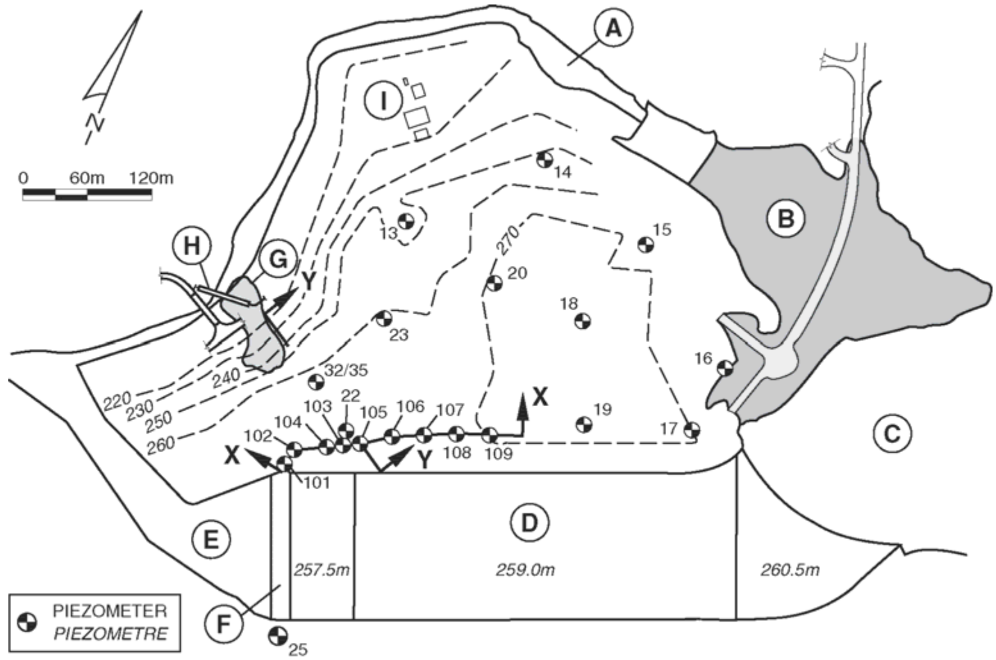

Fig. 1

Site Plan

Vue en plan

(A) Bull Run River	*(A) Rivière Bull Run*
(B) Embankment Dam	*(B) Barrage remblai*
(C) Reservoir, Normal Water Level 262m	*(C) Retenue (niveau d´eau normale 262m)*
(D) Spillway Approach Canal	*(D) Canal approche d´évacuateur de crue*
(E) Spillway Chute	*(E) Coursier d´évacuateur de crue*
(F) Spillway Crest	*(F) Crête d´évacuateur de crue*
(G) 1995 Landslide	*(G) 1995 Glissement de terrain*
(H) Conduit 2 and 4 Crossing	*(H) Traversement des conduites 2 et 4*
(I) Headworks	*(I) Bâtiments des operations*

breccia that slid from the high cliffs above the right abutment of the dam. Due to unusual geologic conditions and the presence of ancient landslide terrain over most of the site, Bull Run Dam # 2 was constructed with a spillway located approximately 610 meters (2,000 feet) downstream from the main embankment. The dam is connected to the reservoir via a 427-meter (1,400-foot) approach canal. The majority of the approach canal is unlined. The original design plans called for complete lining of the approach canal with compacted clay. However, due to adverse weather and soft, saturated soil in the canal floor, only 46 meters (150 feet) in front of the spillway and portions of the north bank were lined with compacted impervious soil. The remainder of the canal was thought to contain sufficient surficial clay to control seepage into the underlying slide debris.

Since completion of the dam and initial reservoir filling during the winter of 1961-62, piezometric groundwater levels north of the spillway have steadily risen. The largest rise occurred in piezometer P-32 (and its replacement, P-35), located 76 meters (250 feet) north of the spillway crest, which has risen 12.5 meters (41 feet) from elevation 245 meters (804 feet) in 1964 to elevation 257.5 meters (845 feet) in 1998. Piezometer P-22, located about 27 meters (90 feet) north of the canal, has experienced a 4-meter (13-foot) rise from elevation 256 meters (840 feet) in 1966 to elevation 260 meters (853 feet) in 1998.

Continuous monitoring and evaluation of 35 years of piezometric data indicates that the increased groundwater levels in the slide debris are related to seepage from the approach canal. On November 28, 1995, during heavy winter storms and elevated reservoir levels, a flow slide occurred on the steep bank above the Bull Run River approximately 30 meters (100 feet) west of P-35 and 91 meters (300 feet) northwest of the spillway crest after a period of heavy rainfall. The location of the slide is shown in Figure 1. The slide debris crossed the headworks access road and hit the truss bridge that carries two of the three water supply pipelines. The truss bridge failed and both pipelines had to be taken temporarily out of service. Remediation included infilling the eroded headscarp and outer slope with rockfill, and replacing the bridge and pipeline.

1.1. THE PROBLEM AND ITS SETTING

The steady rise in groundwater levels in the central area north of the spillway, and the November 1995 landslide of the slope below indicated that the stability of the area was degrading with time. If left untreated, seepage through the heterogeneous landslide debris could continue to erode and pipe the fine-grained soils, leading to larger erosion channels. This in turn would lead to more seepage and potentially additional threats to the stability of the 46-meter (150-foot) slope. Given time, a progressive series of slope failures could regress towards the spillway and threaten to breach the side of the canal. Additionally, personnel safety was of concern, since a gravel road at the base of the slope is driven daily for inspection purposes.

To minimize the potential for future landslides in this area and to identify subsurface conditions and high permeability zones, a comprehensive field exploration program was conducted to evaluate the slide debris north of the approach canal. Nine exploratory borings were drilled at 15- to 30-meter (50- to 100-foot) spacing along the access road immediately north of the spillway approach canal.

The results of the drilling resulted in a clearer definition of the seepage location and enabled a cost-effective design that could minimize seepage. The main objective of the exploration program was to couple this new information with the historical piezometric readings in order to identify and isolate zones of seepage and to refine preliminary conceptual mitigation options.

As a result of the detailed field program, earlier conceptual mitigation options are now being refined in order to target high seepage areas. A clearer definition of the seepage areas has minimized uncertainties associated with the mitigation measures, resulting in a more cost-effective solution. The recommended option will be part of an integrated approach in long-term seepage control and will be designed with flexibility for expansion if additional seepage reduction is desired.

2. REVIEW OF HISTORICAL PIEZOMETER DATA

In the early 1960s approximately 25 piezometers were installed in various locations around the project site to monitor groundwater levels. Starting in 1987, the water level in the approach canal was also recorded (the spillway approach canal becomes isolated from the reservoir when the water level in the reservoir drops below elevation 260.5 meters (855 feet) due to a narrow sill at the entrance to the canal). Normal pool elevation is 262 meters (860 feet). The floor of the canal is at elevation 259 meters (850 feet) for a length of 335 meters (1,100 feet) and drops to elevation 257.5 meters (845 feet) for a distance of 46 meters (150 feet) in front of the spillway.

Of the 25 original piezometers, 12 are located in the central area or near the spillway. These piezometers are P-13, P-14, P-15, P-16, P-17, P-18, P-19, P-20, P-22, P-23, P-25, and P-32/35 (Fig. 1). Additional piezometers were installed in the 1990s to provide more information on groundwater levels north of the spillway. These recent piezometers are P-46 through P-52, and P-101 through P-109. Piezometers P-101 to P-109 were installed in 1998-99. Due to the short historical time span, these piezometers were not included in our review of historical piezometer data.

Yearly plots of groundwater levels for each piezometer from 1964 to 1998 were reviewed, and the maximum and minimum levels were recorded for each piezometer. The decade summary is tabulated in Table 1. Based on recommendations from the Federal Emergency Regulatory Commission (FERC) and independent consultants, about half the piezometers from 1976 through 1990 were not read. Plotted at each piezometer on Figure 2 is the change in peak groundwater level from the 1960s to the 1990s. Only three piezometers (P-22, P-23, and P-32/35) show a groundwater increase greater than 1.5 meters (5 feet). These three piezometers, located north of the spillway, show a moderate to strong correlation with the canal/reservoir level. Peak groundwater levels have risen in these piezometers 4.9, 3.4, and 10.7 meters, respectively (16, 11 and 35

Table 1
Decade Summary: Minimum and Maximum Piezometer Readings
Résumé de décade: Mesures des piezometres minimum et maximum

Piezo No.	1960s		1970s		1980s		1990s	
	HIGH	LOW	HIGH	LOW	HIGH	LOW	HIGH	LOW
P-13	226.2	222.5	225.2	222.5	-	-	227.1	222.5
P-14	241.4	229.8	241.1	231.6	-	-	241.7	231.0
P-15	248.4	246.9	247.5	246.9	-	-	249.3	247.2
P-16	259.1	255.4	259.1	254.4	258.5	254.2	259.4	255.1
P-17	260.0	256.0	260.3	255.4	260.0	256.3	260.9	256.3
P-18	256.9	254.5	255.1	254.5	-	-	260.0	254.8
P-19	265.5	256.9	265.8	256.9	-	-	266.1	256.9
P-20	246.3	243.8	248.4	243.5	-	-	247.8	243.2
P-22	256.6	253.3	257.9	253.3	259.1	253.0	260.0	253.6
P-23	250.5	248.4	251.2	249.0	-	-	255.4	252.4
P-25	259.7	257.9	260.6	258.2	260.0	259.1	260.0	258.5
P-32/35	246.9	243.5	253.0	244.8	256.9	244.8	257.6	251.2

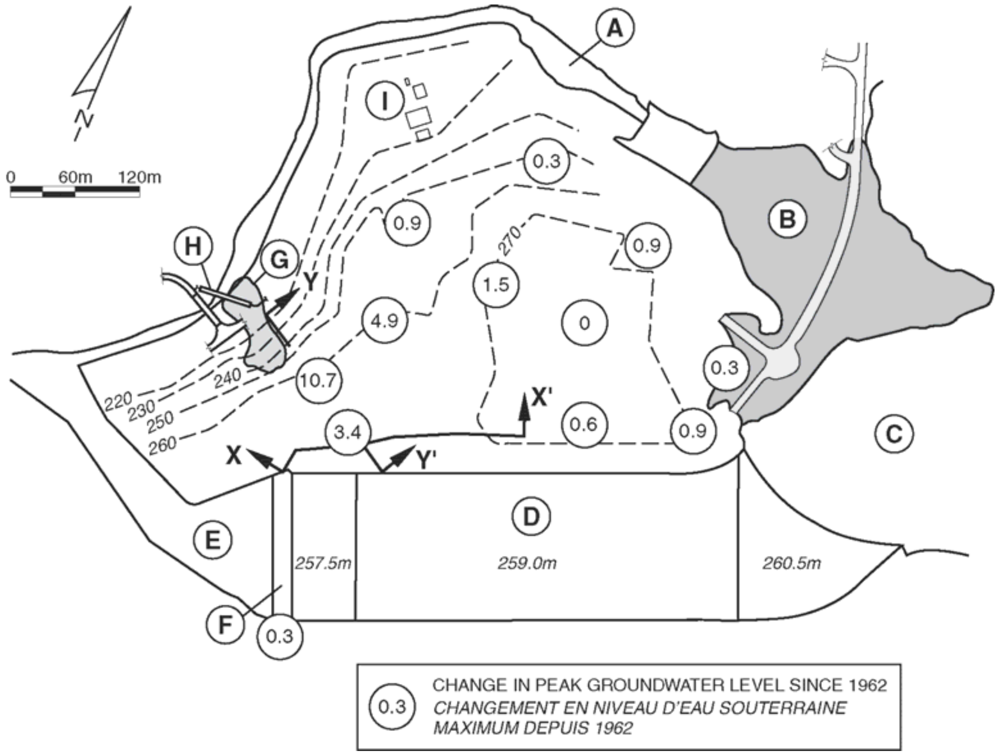

Fig. 2

Piezometer Plan

Plan des piézometres

(A) Bull Run River	*(A) Rivière Bull Run*
(B) Embankment Dam	*(B) Barrage remblai*
(C) Reservoir, Normal Water Level 262m	*(C) Retenue (niveau d´eau normale262m)*
(D) Spillway Approach Canal	*(D) Canal approche d´évacuateur de crue*
(E) Spillway Chute	*(E) Coursier d´évacuateur de crue*
(F) Spillway Crest	*(F) Crête d´évacuateur de crue*
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(I) Headworks	*(I) Bâtiments des operations*

feet), since initial reservoir filling and operation of the spillway. The remainder of the piezometers in the central area has not shown a significant increase in groundwater levels since 1964. Piezometers P-16, P-17 and P-19, located along the north side of the canal closer to the reservoir, have shown only minor increases of 0.3 to 0.9 meters (1 to 3 feet) over the 35 year-period. Similarly, P-25 on the south side of the spillway has shown virtually no change in groundwater level.

3. FIELD EXPLORATIONS

The field-drilling program consisted of nine exploratory borings. The purpose of the field explorations program was to:

1. investigate subsurface soil and rock conditions;
2. perform in-situ permeability tests of the soil and rock formations,

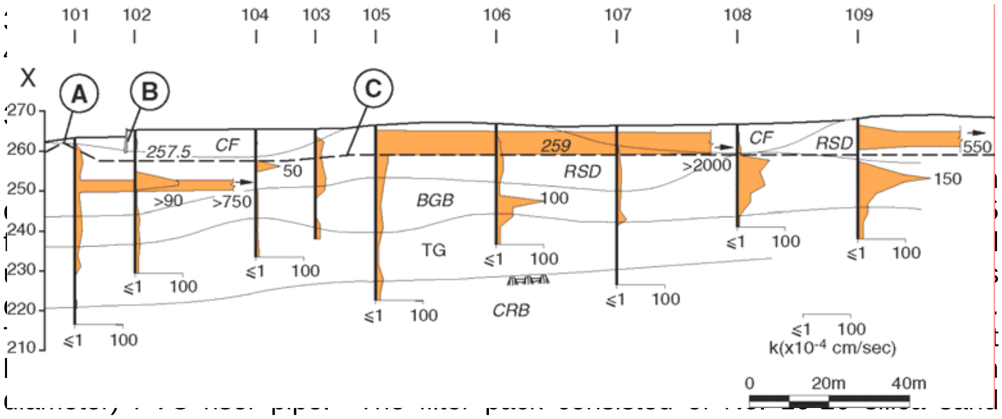


Fig. 3

Cross Section X-X'

Coupe transversale X-X'

(A) Spillway Crest	*(A) Crête d´évacuateur de crue*
(B) Spillway Wing Wall	*(B) Mur aile d´évacuateur de crue*
(C) Spillway Approach Canal Bottom	*(C) Fond du canal approche d´évacuateur de crue*
CF Gravelly Clay (Fill)	*CF Argile Graveleux (Remplisseur)*
RSD Rhododendron Slide Debris	*RSD Débris Glissade Rhododendron*
BGB Basalt Gravels and Boulders	*BGB Graviers et Roches Arrondies Basalte*
TG Terrace Gravels and Boulders	*TG Graviers et Roches Arrondies Terrasse*
CRB Columbia River Basalt Bedrock	*CRB Bedrock Basalte Rivière Columbia*

locally raise the grade in low areas. The remainder of the borings, P-105, P-106 and P-109, encountered landslide debris immediately below the base rock of the access road. The fill consists of stiff, slightly sandy to sandy, clayey silt to silty clay with numerous gravel-sized and occasional cobble-sized rock fragments. Fill was encountered to a maximum depth of 7 meters (23 feet) in borings P-104 and P-108.

Landslide debris was encountered in all of the borings except for P-108. The slide debris consists predominantly of nested blocks and boulders of very soft (R1) to soft (R2), decomposed to highly weathered volcanic breccia (Rhododendron Formation). The matrix between the nested Rhododendron blocks consists of medium dense, slightly clayey, gravelly, silty sand. The majority of the slide material originated from the bluffs above the right abutment of the dam. The thickness of the landslide debris ranges from 8.2 meters (P-104) to 18.5 meters (P-101) (27 feet to 61 feet). The slide debris apparently pinched-out in the area of P-108 during emplacement, or was subsequently eroded.

The alluvial deposits beneath the slide debris consist of soft (R2) to hard (R4) gravel, cobbles, and boulders in a matrix of slightly clayey, sandy silt. The deposit, encountered in all borings, ranges in thickness from 4.3 meters in P-101 to 15.8 meters in P-108 (14 feet to 52 feet). The material is subangular to subrounded, implying limited transportation before deposition. Therefore, the origin of the deposit is uncertain. It could be a locally-derived alluvial deposit of the old Bull Run River, or it could be an ancient landslide deposit which predates the overlying Rhododendron slide debris.

The cemented terrace deposits consist of gravels, cobbles, and occasional boulders in a slightly to moderately cemented, silty sand matrix. The matrix is orange-brown, reflecting previous oxidation. This unit was also encountered in all the borings, with a maximum thickness of 18.6 meters (61 feet) in P-101.

The underlying bedrock is Columbia River Basalt. Three borings (P-101, P-105, and P-107) were drilled to bedrock. The material is dark gray, slightly weathered, hard (R4), and highly to moderately jointed.

3.3. PERMEABILITY TESTING

The permeability of the soil and rock layers was determined using single-packer, constant head, and falling head tests. A total of 146 field permeability tests were performed during the field exploration program. The permeabilities calculated from the in-situ tests were highly variable, ranging from 0.02 x 10^{-4} cm/sec. in basalt bedrock to over 2,000 x 10^{-4} cm/sec. in the landslide debris. Permeabilities in the clay fill were low, ranging from 0.2 x 10^{-4} cm/sec. to 18 x 10^{-4} cm/sec. The test results indicated that the Rhododendron slide debris had the widest variation and largest permeabilities, ranging from 0.02 x 10^{-4} cm/sec. to at least 2,000 x 10^{-4} cm/sec. This largest permeability was encountered from 1.5 to 7.5 meters (5 to 25 feet) in P-105. This zone took the maximum flow rate of 265 liters per minute (70 gpm) without ever filling the drill rods to the surface.

The alluvial deposits beneath the slide debris had lower permeabilities than the slide debris, ranging between 0.15 x 10^{-4} cm/sec. to 150 x 10^{-4} cm/sec. Permeabilities for the cemented terrace deposits were even lower, ranging from 0.09 x 10^{-4} cm/sec. to 20 x 10^{-4} cm/sec. Likewise, the permeabilities for the basalt were low, ranging from 0.02 x 10^{-4} cm/sec. to 10 x 10^{-4} cm/sec. The permeability results are plotted graphically on cross-section X-X' (Fig. 3).

4. EVALUATION OF SEEPAGE

The results of the piezometer data review indicate that groundwater levels have risen steadily in the area immediately north of the spillway crest. The other piezometers in the central area and P-25 (south of spillway) have shown insignificant groundwater increases.

Three possible sources of water leakage were considered during the course of this study. They include: (i) leakage from the canal, (ii) leakage from the reservoir, and (iii) deep-seated seepage under the canal from the southside. A discussion of each possible leakage source is provided below.

4.1. LEAKAGE FROM CANAL

The three piezometers which have shown a steady increase in groundwater levels over the years are located close to the steep bank above the Bull Run River where the 1995 landslide occurred. This area has the highest hydraulic gradient from reservoir/canal levels to the south bank of the Bull Run River and, therefore, has the highest potential for piping erosion. The subsurface borings

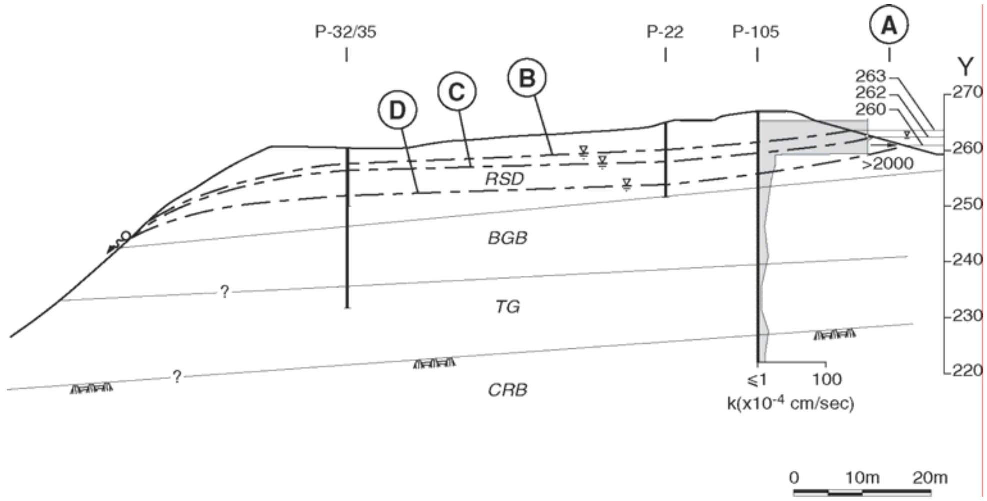


Fig. 4

Cross Section Y-Y'

Coupe transversale Y-Y'

(A) Spillway Approach Canal
(B) Groundwater Level at Canal El. 263
(C) Groundwater Level at Canal El. 262
(D) Groundwater Level at Canal El. 260

(A) Canal approche d´évacuateur de crue
(B) Niveau d´eau souterraine au elev. 263 du canal
(C) Niveau d´eau souterraine au elev. 262 du canal
(D) Niveau d´eau souterraine au elev. 260 du canal

indicate that the landslide debris north of the canal consists of nested blocks of volcanic breccia within a matrix of silty, gravelly sand. Over time, piping erosion can occur when water seeping from the canal under a high gradient carries the finer fraction of the slide matrix into larger voids within the nested breccia blocks.

Cross-section Y-Y' (Fig. 4) shows how P-22 and P-32/35 respond to varying canal water levels. During winter storms, the water level in the canal can reach elevation 263 meters (864 feet) or higher; water levels in P-22 spike 1.8 meters (6 feet) above normal levels, and P-32/35 spikes 1.2 meters (4 feet). Conversely, when the canal is lowered 1.5 meters (5 feet) from normal pool to elevation 260.5 meters (855 feet) in the summer, a significant groundwater drop of 3.0 to 4.3 meters (10 to 14 feet) occurs in P-22 and P-32/35. It appears that the response of the piezometers is the result of one or more high permeability zones in the canal near boring P-105. One such zone was encountered in P-105 from 1.5 to 7.6 meters (5 to 25 feet). Groundwater has been measured at the base of this zone when the canal is at elevation 262 meters (860 feet), the normal reservoir pool level. One explanation for the piezometer response is that the highly permeable zone in P-105 acts likes an underground weir. As water in the canal rises above 262 meters (860 feet) during storms, groundwater rises into the high permeability zone and causes the observed spikes in P-22 and P-32/35. Similarly, when the canal is lowered in the summer, groundwater in the area of P-105 drops below the high permeability zone and effectively cuts off the flow of water towards P-22 and P-32/35, resulting in the significant drops in these piezometer water levels.

The leakage appears to be coming from the side of the canal, not the floor. A review of the canal levels during summer drawdown indicates that the canal remains at elevation 260.5 meters (855 feet) for weeks or months without a significant drop in water level. As mentioned earlier in the report, a sill at the entrance to the canal at elevation 260.5 meters (855 feet) isolates the canal from the reservoir when the reservoir drops below this elevation. If a high seepage zone existed below elevation 260.5 meters (855 feet) within the canal, the canal would not be able to hold water for an extended period of time.

4.2. LEAKAGE FROM RESERVOIR

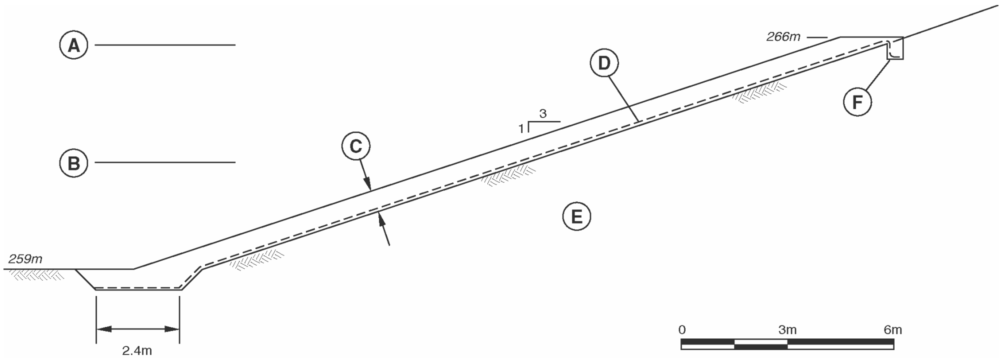
Seepage from the canal closer to the reservoir or from the reservoir itself follows a longer seepage path through the central area. This seepage has a lower hydraulic gradient and less potential for piping erosion than the shorter path near the spillway. The piezometers closer to the reservoir have not shown a significant increase over the last 35 years. P-19, located near the entrance of the approach canal, shows a strong correlation with rainfall and has winter groundwater levels which are higher than the reservoir. This would result in a flow of groundwater towards the canal, not the other way around. For these reasons, it appears that the main reservoir is not the source of leakage which has resulted in elevated groundwater levels north of the spillway.

4.3. DEEP-SEATED SEEPAGE UNDER THE CANAL FROM SOUTH SIDE

The possibility of deep-seated seepage under the canal from the south side was addressed. Water could seep into higher ground south of the canal and flow through pervious zones (slide debris, fractured basalt) towards the north side of the spillway. If pervious zones were present and connected to this higher ground, there should be an artesian pressure north of the canal which feeds upward into piezometers P-22 and P-32/35 to account for their rises during floods. As part of the field exploration and instrumentation program, we installed a deep piezometer in P-107 to measure groundwater pressures at the basalt/terrace gravel interface. This piezometer is not measuring artesian pressures. Likewise, the other piezometers installed at depth (P-46 and P-47) have not detected artesian pressures. It is also noted that P-25, located on the south side of the spillway, has not shown any increase in groundwater levels since 1964. Therefore, it appears that the area south of the canal is not the source of leakage resulting in elevated groundwater levels north of the spillway.

5. SEEPAGE REDUCTION MEASURES

Several remedial options were evaluated to reduce seepage from the side of the canal into the central area. For reasons discussed in the previous section, these options target the north side of the canal in the vicinity of P-105. There is a chance that leakage is occurring through the side of the canal closer to the reservoir, however, this scenario is considered unlikely and the remedial treatment was not extended into this area. There are two general methods of seepage reduction: line the north bank of the canal, or construct an underground cutoff wall along the access road north of the canal.



5.1. CANAL SLOPE LINERS

Various liner materials were evaluated including synthetic geomembranes with a protective cover, roller-compacted concrete, conventional concrete and shotcrete. Impervious slope liners would be placed on the north bank of the canal over an area 152 meters (500 feet) wide and extending from the canal floor up to elevation 266 meters (872 feet) (see Fig. 5). This area was determined to be the minimum coverage needed to provide a high degree of probability of success in reducing seepage. The existing piezometers will be monitored following construction of the liner system. The liner system has the flexibility to be expanded in future years if piezometer monitoring indicates limited groundwater drawdowns during peak winter storms when the spillway is passing water.

Several criteria were used to evaluate various liner options including: constructibility; construction impacts on reservoir and water supply operations; effectiveness in reducing seepage; durability; maintenance; stability on 1V (vertical); 3H (horizontal) slopes; and construction cost. A subjective rating system for the options, including the underground walls, is provided in Table 2.

The results of the evaluation indicated that a geomembrane with a crushed rock cover would provide the optimal performance for price. This option would

Fig. 5

Typical Section of Composite Geomembrane Liner

Section typique de revêtement géomembrane composite

(A) Probable Maximum Flood El. 265.5	*(A) Niveau d´eau maximum p6robable elev. 265,5*
(B) Normal Pool El. 262	*(B) Etang normale elev. 262*
(C) 0.6m Rockfill Cover	*(C) Couvre-enrochement 0,6m*
(D) 60-mil Geomembrane	*(D) Géomembrane 60-mil*
(E) Existing Ground	*(E) Terrain existant*
(F) Anchor Trench	*(F) Tranchée d´ancrage*

consist of placing a geotextile and 60-mil geomembrane liner covered with 610 millimeters (24 inches) of rockfill. A typical section is shown on Figure 5. All of the other liner options had significant concerns of one form or another. Clay covers are susceptible to desiccation cracking, and damage from burrowing animals and tree roots. Concrete, shotcrete and RCC have potential leakage problems due to cracking during curing. This could also lead to extensive and expensive maintenance to keep the cracks sealed long-term.

5.2. UNDERGROUND CUTOFF WALLS

A slurry trench cut-off wall and a grout curtain were also considered during the evaluation. The slurry trench would consist of constructing a 183-meter (600-foot) long, 15.2-meter (50-foot) deep cement-bentonite barrier wall north of the canal.

The intent of the cutoff wall would be to stop seepage through the highly pervious zones in the upper slide debris unit. Some seepage could still occur under the wall through the basalt gravels and boulders, but the amount of seepage should be small and would allow lower groundwater levels to be achieved in the central area.

The grout curtain option would consist of installing a grout curtain wall in the same location as the cement-bentonite slurry trench. The depth would also be approximately 15.2 meters (50 feet). Like the slurry trench, the intent of the grout curtain would be to stop major seepage through the upper slide debris unit, with some leakage still occurring under the wall. The grout curtain would be installed using three rows of grout holes. Sleeve-port grout pipes would be installed in drillholes at 1.5-meter (5-foot) spacing along each row. The sleeve-ports allow grouting to occur sequentially over discreet zones from the bottom of the hole to the ground surface. The outer two rows would be installed before the middle row. This would allow the outer rows to harden and create a relatively impermeable zone for higher grout pressures in the middle row.

The slurry trench and grout curtain options were both expensive, involved relatively difficult construction conditions, and, in the case of the grout curtain, presented some uncertainty regarding seepage reduction effectiveness. For these reasons, the underground cut-off walls were not selected.

Table 2
Criteria Ratings: Seepage Reduction Options
Evaluation du criterion: Options de réduction de la infiltration

Evaluation Criteria	Remedial Option						
	GM with Clay	GM with Rockfill	RCC	Concrete Panels	Shotcrete	Slurry Trench	Grout Curtain
Constructibility	VG	VG	G	G	G	P/G	U
Construction Impacts on Operations	L	L	L	L	L	VL	VL
Effectiveness	G	VG	VG	G	VG	VG	U
Durability	P	G	E	VG	VG	E	VG
Maintenance	M	L	M	M	M	VL	L
Stability	G	VG	VG	VG	VG	E	VG
Cost	VL	VL	L	L	M	H	H

VL = Very Low P = Poor U = Uncertain
L = Low G = Good
M = Moderate VG = Very Good
H = High E = Excellent

6. CONCLUSIONS

An extensive field exploration program was undertaken to evaluate subsurface conditions and permeabilities in the area north of the spillway and approach canal at Bull Run Dam # 2. The results of the permeability tests identified a localized, highly permeable zone. This zone is hydraulically aligned with piezometers that have shown increased groundwater levels since dam construction. A review of historical piezometer data and canal drawdown levels led to the conclusion that the most probable source of seepage is from the north side of the canal within 152 meters (500 feet) of the spillway. It does not appear that the main reservoir or the area south of the canal are significant sources of seepage.

Several options were presented to line the canal bank with an impervious blanket or to construct an underground cutoff wall north of the canal. The liner options include a geomembrane with a clay or rockfill cover, an RCC liner, a shotcrete liner, or conventional concrete panels. Liner options involve simple construction techniques and would minimally impact reservoir operations during summer drawdown. They are relatively inexpensive and offer*ed* a high degree of confidence in reducing groundwater levels in the central area. However, there were a few concerns including potential desiccation cracking, burrowing animals and root damage to clay covers; and potential cracking during curing of RCC, shotcrete and conventional concrete liners.

The geomembrane liner with a rockfill cover provided the best solution for reducing seepage. The 60-mil textured geomembrane will provide a good seepage cutoff, while the rockfill provides good resistance to scour. Furthermore, the rockfill cover is not susceptible to cracking, rapid drawdown pressures, burrowing animals or root growth.

REFERENCES

[1] Cornforth Consultants Inc., "Phase 1 Preliminary Engineering Seepage Evaluation, Spillway Approval Canal, Bull Run Dam No. 2, April 1999.

SUMMARY

The steady rise in groundwater levels in the central area north of the Bull Run Dam # 2 spillway, and the November 1995 landslide of the slope below indicated that the stability of the area was degrading with time. If left untreated, seepage through the heterogeneous landslide debris could continue to erode and pipe the fine-grained soils, leading to larger erosion channels. To minimize the potential for future landslides in this area, and to identify subsurface conditions and high permeability zones, a comprehensive field exploration program was conducted to evaluate the slide debris north of the approach canal. The main objective of the exploration program was to couple this new information with the historical piezometric readings in order to identify and isolate zones of seepage and to refine preliminary conceptual mitigation options. As a result of the detailed field program, earlier conceptual mitigation options are now being refined to target high seepage areas. A clearer definition of the seepage areas has minimized uncertainties associated with the mitigation measures, resulting in a more cost-effective solution. The recommended option will be part of an integrated approach in long-term seepage control and will be designed with the flexibility for expansion if additional seepage reduction is desired.

RESUME

Un programme d' exploration tres vaste a ete entrepris pour evaluer les conditions et les permeabilites des sous surfaces des regions/du nord de la voie de deroutement et du canal d' entree. Les resultats des tests de la permeabilite a identifie une zone hautement permeable et localisee. Du point de vue hydraulique, cette zone est alignee avec des piezometres lesquels ont montre une augmentation des niveaux des eaux sous terrain de puis la construction du barrage.

Une revue des donnees historique de piezometre et des niveaux du canal de “drawdown” a conduit a la conclusion que la source, la plus proba_ble, du suintement est du cote’ nord du canal dans les 152 metres (500 feet) de la voie de deroutement. Il ne parait pas que le reservoir principal ou bien la region du sud du canal soient les sources responsables du suintement.

Plusieures options ont ete presentees pour aligner la rive avec une couverture imprevue ou pour construire un mur de coupure sous terrain au niveau du nord du canal. Les options d’ alignement comprennent une “geomembrane” avec une couverture d’ argille ou un remplissage de roche, un paquebot “RCC” ou bien un panneau conventionnel du beton. Les options d’ alignement font participer des techniques simples de construction et peuvent influencer de facon mineure les operations du reservoir pendant le “drawdown” de l’ ete. Ils sont relativement peu couteaux et ont offert une haute degree de confidence pour reduire les eaux sous terrain dans la region du centre. Cependant, il y a eu quelques inquietudes, y incluses le craquement potentiel de dessication, les empreintes des animaux et les degats causes par les racines aux couvertures d’ argille; et le potentiel craquement pendant la gurison de “RCC” et du paquebot conventionnel de beton.

La “Geomembrane” avec remplissage de roche a fournit la meilleure solution pour deduire le suintement. La “60-mil” geomembrane fournirra une bonne coupure du suintement, tandis que le remplissage de roche fournit une bonne resistance au parcours. De plus, le remplissage de roche n’ est pas susceptible de craquer, une pression rapide de “drawdown”, des empeintes des animaux ou la poussee des racines.

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