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DAM SAFETY UPGRADE OF THE RUSKIN DAM RIGHT ABUTMENT

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ABSTRACT:

Ruskin Dam is located near Mission, British Columbia and is one of three BC Hydro facilities in the Alouette-Stave-Ruskin Hydroelectric System. The dam was constructed between 1929 and 1930 and is a concrete gravity structure founded primarily on bedrock. At the right abutment, which consists mainly of glacially deposited sands and silts, the dam connects to a cut-off system consisting of sloping concrete slabs, founded on retaining walls and sheet piles, which extend upstream of the dam. Significant seepage and piping issues occurred at the right abutment after first filling of the reservoir in 1930, and a number of remedial actions were carried out in an attempt to address the problems. Through extensive investigations it was determined that the re-occurring abutment seepage and piping issues, as well as the low seismic withstand of the concrete slab cut-off, posed significant dam safety risks to the facility. A seepage control upgrade project was initiated to address the deficiencies, which resulted in the construction of a new seepage cut-off wall with a special tie-in to the concrete dam, and a reverse filter blanket and drainage system on the downstream slope to collect and measure seepage. Analyses were carried out to evaluate the performance of the proposed upgrade during the design earthquake and to model the seepage regime. The identification, assessment and mitigation of dam safety risks during construction were a key part of the planning and design of the upgrades.

RÉSUMÉ:

Le barrage Ruskin est situé près de Mission en Colombie-Britannique et est l'un des trois aménagements que possède BC Hydro dans le système hydroélectrique Alouette-Stave-Ruskin. Le barrage fut construit entre 1929 et 1930 et consiste d'un barrage-poids en béton fondé principalement sur le roc. À l'appui droit, qui consiste principalement de dépôts de sable et de limon d'origine glaciaire, le barrage se joint à un système limitant les infiltrations d'eau qui consiste, en partie, de dalles en béton inclinées vers l'amont du barrage. Des problèmes d'infiltration d'eau et d'érosion interne ont étés observés à l'appui droit lorsque le réservoir fut rempli en 1930, et de nombreuses mesures de remédiations ont étés mises en place depuis. Après plusieurs études, il a été déterminé que l'infiltration d'eau répétée dans l'appui droit ainsi que la modique résistance sismique des dalles en béton posaient un risque à la sécurité du barrage. Un projet fut initié afin de contrôler les infiltrations d'eau et a résulté en la construction d'une nouvelle tranchée d'isolation avec un raccordement spécial au barrage, ainsi qu'un système de drainage et de perré en aval du barrage afin de collecter et mesurer les infiltrations d'eau. Des analyses ont été effectuées afin d'évaluer la performance des améliorations proposées durant le séisme type et afin de modéliser le régime des infiltrations d'eau. L'identification, l'évaluation, et l'atténuation des risques à la sécurité des barrages durant la construction furent une partie intégrale de la planification et de la conception des améliorations.

1 INTRODUCTION

Ruskin Dam is located on the Stave River and is the most downstream facility in the Alouette-Stave-Ruskin hydroelectric system. Water discharged from the 90-MW Stave Falls Generating Station and Dam flows into Hayward Lake; a 6-km-long reservoir impounded by Ruskin Dam. The 105-MW Ruskin Generating Station is located below the left (east) abutment of Ruskin Dam, and water discharged from the Ruskin powerhouse and spillway flows into the lower Stave River. The confluence of the lower Stave River and the Fraser River is located approximately 3 km further downstream. Ruskin is classified as an Extreme Consequence dam, according to the current BC Hydro Dam Safety Management Manual. The location of Ruskin Dam is shown in Figure 1, and photos of the dam and right (west) abutment are shown in Figure 2.

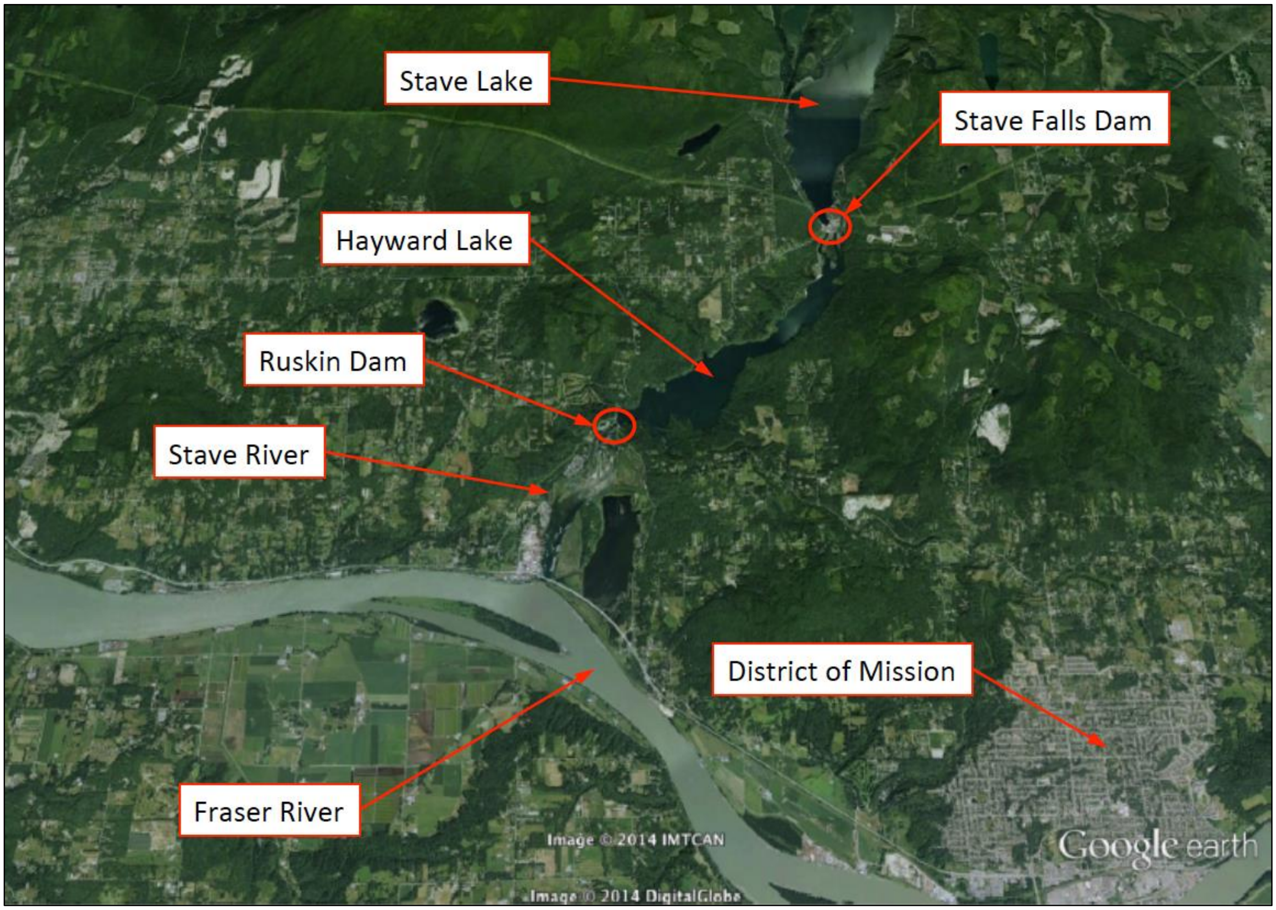

Figure 1: Location of Ruskin Dam

The dam is a 130-m-long concrete gravity structure founded predominantly on bedrock, and consists of an 85-m-long, seven-bay radial-gated spillway. The dam is 58 m high from its deepest foundation to the road deck on the dam crest at El. 45.77 m. The right abutment of Ruskin Dam is founded on a bedrock ridge with a maximum height of about El. 30 m, or about 15 m below the road deck. Above bedrock, the abutment consists mainly of glacially deposited silts and highly erodible sands, which are part of the Quadra Sand deposits. The Quadra deposits are very dense, but are susceptible to piping and were called the "weakest part of the dam" by Victor Dolmage (Stewart, Batten and Associates, 1929), prior to construction of the dam. A sloping concrete slab cut-off wall system was constructed on the right abutment soils to provide seepage cut-off; however, piping issues occurred in the abutment sands immediately after the first filling of the reservoir. In addition, the dam is located in a seismically active area resulting in additional dam safety risks to the facility. Due to these deficiencies and the ageing condition of the powerhouse, the dam and generating station are currently being upgraded. The upgrades include improvements to the right abutment, the replacement of the seven-bay spillway and single lane bridge on top of the dam with a new 5-bay spillway and two-lane bridge, water-to-wire replacement of the generating equipment, and the modernization and seismic upgrade of the powerhouse and switchyard.

The upgrades to the right abutment were initiated to address the following deficiencies:
- Seepage and piping of the right abutment soils during normal operation and post-earthquake
- Earthquake-caused deformations to the concrete slab cut-off system, leading to seepage and piping

The right abutment improvements were completed in 2013, and included a new cut-off wall with a special tie-in to the concrete dam, and a reverse filter blanket and drainage system on the downstream abutment slope. Groundwater levels between the new and existing cut-off walls are maintained by a horizontal drain to ensure that the concrete slabs do not fail during a reservoir drawdown. The upgrades are designed to survive a maximum design earthquake (MDE) with an annual exceedence frequency (AEF) of 1 in 10,000 years without catastrophic collapse, which is a magnitude 7.5 event with a peak ground acceleration (PGA) of 0.71g. The upgrades are also designed to be operational following an earthquake with an AEF of 1 in 475 years, which corresponds to a PGA of 0.26g.

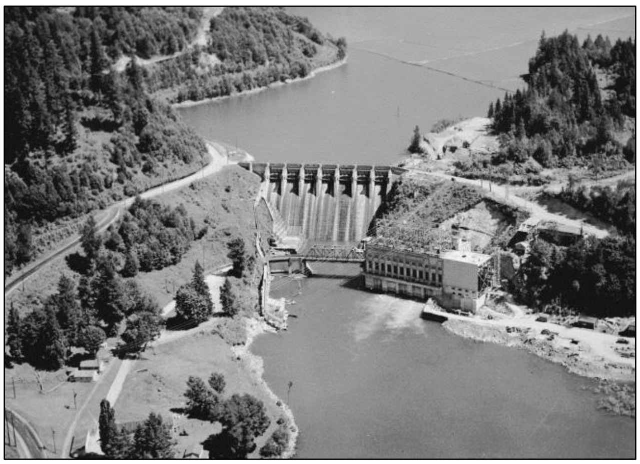

Figure 2: Photos of Ruskin Dam and generating station in 1950 (left) and after the right abutment upgrades in 2010 (right)

2 PERFORMANCE HISTORY

The original seepage cut-off system for the right abutment extends about 130 m upstream from the concrete dam, and is composed of inclined concrete slabs founded on three structures: a concrete gravity section (16.2 m long), a concrete core wall section (14.5 m long), and a steel sheet-pile section (99.3 m long). Each foundation section supports a concrete plinth that connects to a concrete slab which is sloped at 1.25 horizontal to 1 vertical and extends up to El. 44.1 m (maximum height of 16 m), except at the intersection with the concrete dam, where the top of the slabs is at about El. 45.6 m. The connection of the dam at the right abutment, west of the concrete slabs, required a deep excavation in the foundation soils, which was backfilled with rockfill on the upstream side and loose, uncompacted fills on the downstream side. The rockfill was placed against the native silts and sands without any filters. The general arrangement of the original cut-off system is shown in Figure 3 and the construction of the concrete slab cut-off system is shown in Figure 4.

Performance issues occurred immediately in the right abutment during first filling of the reservoir in 1930, when sand-laden seepage flows began flowing from the finger drains into the gallery drain and exiting into the spillway (see Figure 3). Flows through the gallery drain reached a maximum of 566 l/min, and about 68 m^3 of sand was measured over a period of 5 months, once the reservoir elevation had reached El. 42.6 m (near the normal maximum level). The constant loss of sand through the rockfill resulted in settlement of the concrete slabs in 1931, and an inclined shaft was constructed to inspect the underside of the slab. Cavities were backfilled with coarse gravel and grout was injected via pipes installed through holes in the face of the slabs in an attempt to seal the voids in the rockfill. A number of remedial measures were attempted to stop the piping, including driving additional sheet piles and the placement of clay at the upstream end of the slabs, plugging the finger drains with sand and clay, and dumping clay at the joint between the gravity wall and core wall. In 1932, a 1.1 m wide by 1.5 m high adit was constructed to provide an alternate drainage path to the finger drains (see Figure 3). The concrete-lined-adit was connected to the rockfill via a gravel filled screen, which ensured that the rockfill remained dry above about El. 30 to 32 m. Flows were measured at the downstream end of the adit, and voids continued to be monitored beneath the slabs by soundings through the grout holes. Ground settlement and sinkholes continued to occur on the bank above the top of the concrete slabs.

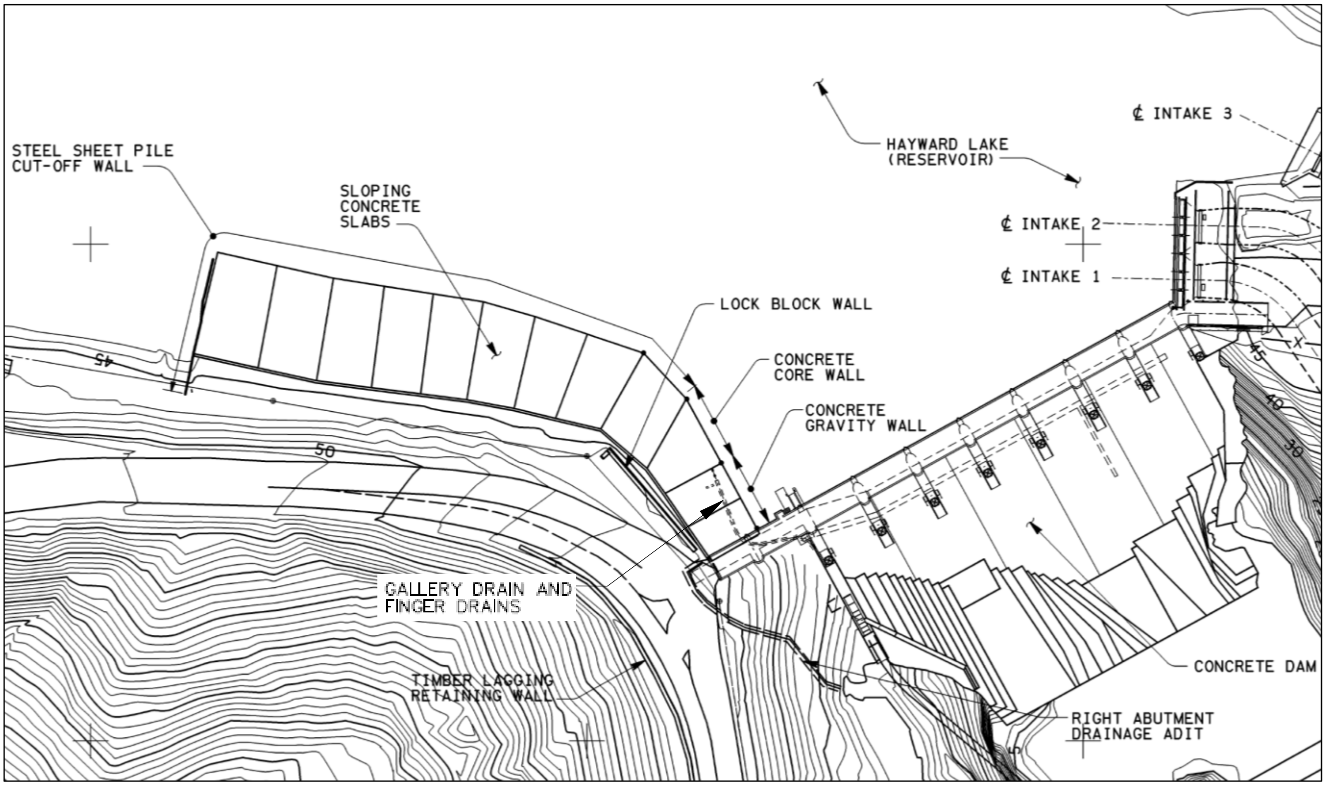

Figure 3: General arrangement of Ruskin Dam and right abutment – prior to upgrades

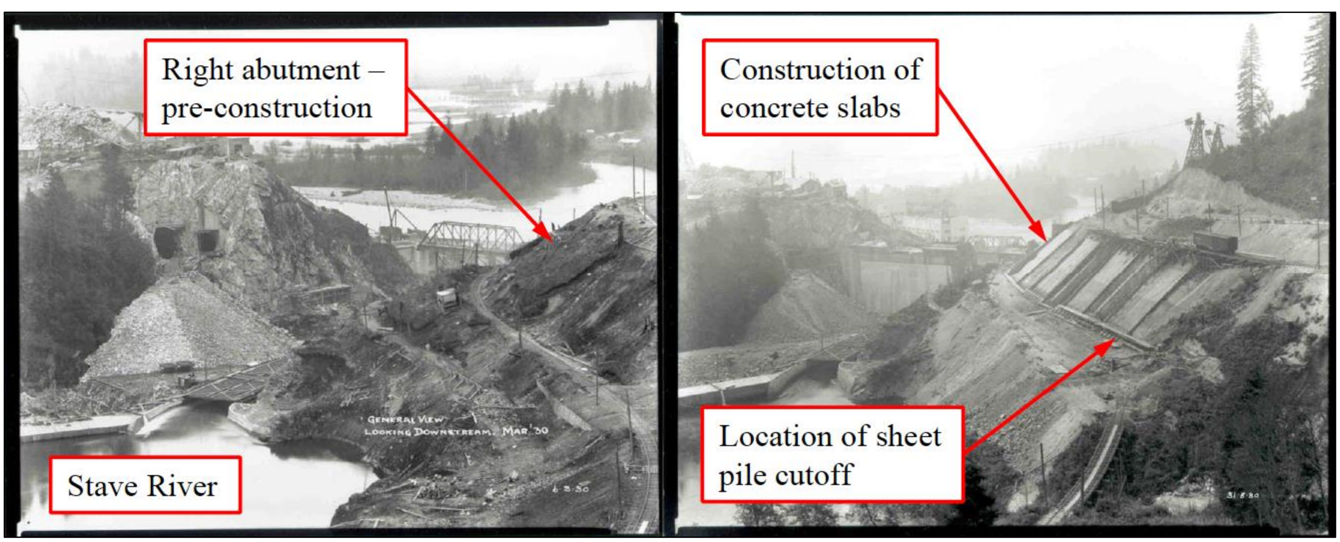

Figure 4: Right abutment prior to construction of original cut-off (left) and during construction of concrete slabs (right)

In 1998, increasing flows in the downstream end of the adit resulted in dam safety concerns about the performance of the right abutment. Remedial work was carried out during 1998 and 1999 to seal three potential leakage sources, including the gap between the slab and sheet pile located at the upstream end of the cut-off system, the open grout pipes in the slabs, and a crack in the concrete slabs immediately upstream of the dam. In the late 1990s, a deficiency investigation was initiated to assess the dam safety risks associated with the right abutment, and it was identified that deficiencies existed for normal loading (seepage) and seismic loading. The study concluded that the concrete slabs would experience significant deformation during the MDE, potentially leading to saturation of the downstream slope, retrogressive failure of the abutment and uncontrolled release of the reservoir. Failure of the cut-off system was predicated to occur for an earthquake with an AEF of 1 in 475 years, which at the time coincided with a PGA of 0.23g. Several options were proposed for improving the abutment, including upgrades to the existing slabs, construction of a new cut-off, and various options for improving the abutment piping resistance and seepage collection. In 2007, a capital project was initiated to determine and implement long-term upgrades for the right abutment, based on the findings of the deficiency investigation.

3 SITE CHARACATERIZATION

Various investigations and characterizations of the right abutment have been carried out, including drilling and/or instrument installations in 1986, 1988, 1995, 1996, 1998, 1999, 2007, 2008, 2009, and 2012. The investigations were focused on understanding the nature of the dam safety issues at the site, and characterizing the geologic history and engineering properties of the foundation soils to enable stability and deformation analyses to be carried out. The investigations initiated in 2007 for the capital program consisted of mud rotary drilling, diamond drilling, sonic drilling, standard penetration testing (SPT), cone penetration testing (CPT), cross-hole and down-hole shear wave velocity measurements, non-intrusive geophysical measurements, and pressuremeter testing. The following sections on geologic history, site characterization and determination of engineering soil parameters focus mostly on the Quadra Sand (Unit 4) due to its prevalence within the right abutment and its association with dam safety issues.

3.1 Characterization of geologic history and foundation soil units

The site has been classified into eight foundation soil units above bedrock, based on the geologic history and engineering properties. The Stave River valley experienced significant glacial and fluvial process, particularly during the Fraser Glaciation, which occurred from about 26,000 years before present (BP) to 11,000 years BP (Armstrong, 1981). Below the foundation soils, the diorite bedrock was likely shaped by fluvial processes, which resulted in the formation of the deepest channel where the concrete dam is located. The lowest soil deposit (Unit 1), consists of silty sand and gravel, and is typically found in channels or gullies at low points in the bedrock. Based on the unconformable contact with the overlying soil, Unit 1 was likely eroded prior to the deposition of Unit 2. Unit 2 consists of glaciomarine silty clay that was likely deposited during the late Olympia Nonglacial Interval. Both Units 1 and 2 were deposited prior to the onset of the Fraser Glaciation.

Global cooling caused the valley glaciers to advance from the Coast Mountains, resulting in increased erosion and periglacial activity. During the onset of the glaciers, proglacial deposition from the advancing ice resulted in the deposition of Units 3 and 4. Unit 3 is a grey, massive, non-plastic silt layer whose deposition likely occurred in standing water, perhaps when the channel was blocked. Unit 4 consists of Quadra Sand deposits, which are an outwash deposit from the Fraser Glaciation, placed sub-aerially on flood plains by braided rivers (Clague, 1976). Quadra Sand was deposited from north to south progressively down the Georgia-Puget Lowland, and evidence from the mineralogy indicates that the sand was derived from the granitic rocks of the mainland of British Columbia (Clague, 1976). Quadra Sand at Ruskin Dam is observed to be locally brown to grey, fine to medium grained with fines content ranging from 5 to 40%. Some coarse sand and fine gravel are present at the contact with the overlying glacial till/drift (Unit 6). A relatively steep contact between Unit 4 and Unit 6 is observed in surface outcrops, which tend to dip towards the river. The steep incline of the erosional contact between the sand and glacial till likely indicates fluvial down-cutting prior to glaciation. Within Unit 4 there are several silt interbeds (Unit 5), with the most prominent and continuous layer located at about El. 29-30 m.

Once the glaciers reached the site during the Vashon Stade, the foundation soils were overridden with ice, culminating in a maximum thickness of about 1800 m (Armstrong et al., 1965). Unit 6, consisting of silty sand and gravel (glacial drift or till), was deposited at the base of the glaciers. Following the retreat of the main glaciers at the end of the Fraser Glaciation, several smaller ice advances occurred during the Fort Langley Time Interval between 13,000 years BP to 11,400 years BP (Armstrong, 1981), which resulted in the local erosion of Unit 6 and the deposition of a thick succession of interbedded glaciomarine, deltaic, littoral and ice-contact sediments, labelled as Unit 7. These deposits are only present above Unit 4 on the hillside above the dam. Unit 8 consists of slope-wash and other colluvial deposits deposited in small quantities at the base of slopes. The typical arrangement of the foundation units in the right abutment is shown in Figure 5.

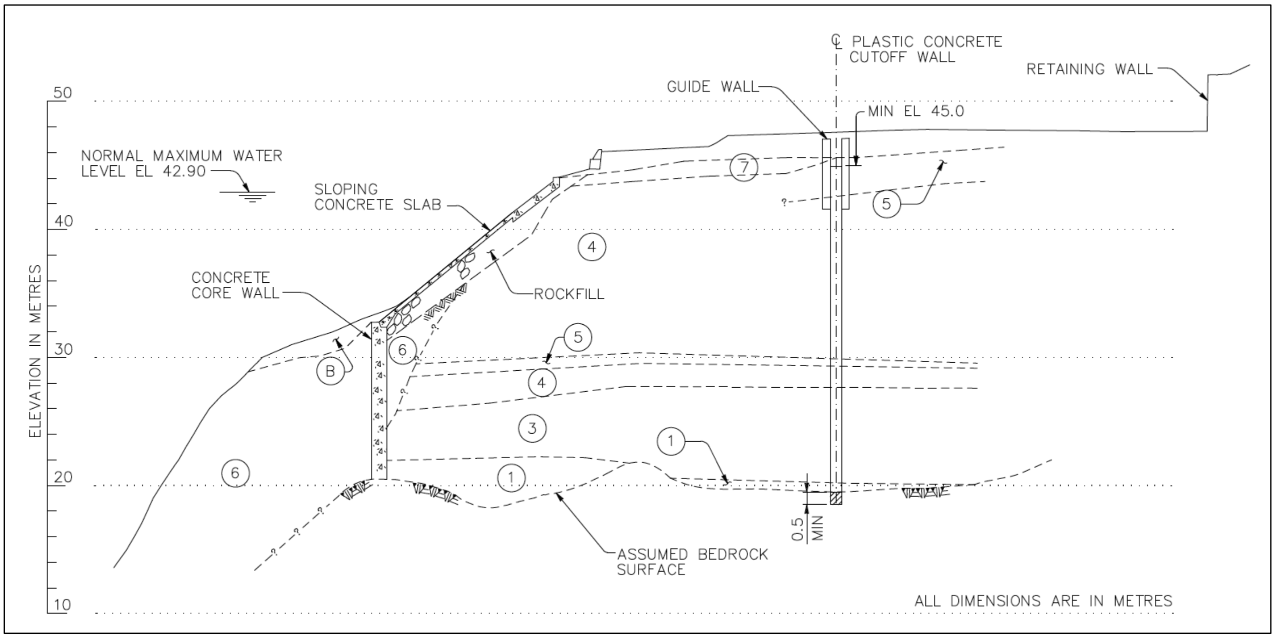

Figure 5: Configuration of foundation soil units for section through concrete core wall and slabs

Most of the dam safety issues at the right abutment are related to the presence of Quadra Sand (Unit 4). The sands, although dense, are highly susceptible to the seepage-caused migration of fine material, which can result in the formation of pipes and sinkholes or other forms of localized loosening. Quadra Sand is present on both the right and left abutments, but is most prevalent on the right abutment, where it ranges in thickness from about 15 to 20 m below the top of the dam (see Figure 6). The sand is also located on the hill above the dam with a variable thickness of up to about 20 m.

3.2 Characterization of disturbed soils

Due to the susceptibility of the Quadra Sand to become loosened from seepage and piping, it was necessary to determine locations where the sand was disturbed. Investigations carried out during 2007-2008 to identify these locations consisted of penetration testing, geophysical methods and visual observations. Non-intrusive geophysical investigations, including the use of impact echo and ground penetrating radar, were carried out from the surface of the slabs during a reservoir drawdown to assess the potential for voids or loosened sand beneath the concrete. These methods were used to detect differences or anomalies from undisturbed sand beneath the concrete slabs. The anomalies were classified as follows:

- Primary anomalies – regions that are most likely to be voids.
- Secondary anomalies – regions that differ significantly from the background reflections. These may be related to small voids at the base of the slab, but could also be changes in sub-grade material, backfill or related to infrastructure such as drainpipes.

- Tertiary anomalies – small subsurface regions that are significantly different from the background. These may be subtle diffractions, possibly related to small changes in the concrete subsurface, small voids, or boulders beneath the slab.

Figure 6: Exposure of Quadra Sand on the right abutment downstream of the dam

These data were combined with the results from cross-hole geophysical testing and penetration data to determine areas of loosened sand. Figure 7 shows penetration data plotted by station from the upstream end of the slab and the locations of primary, secondary and tertiary anomalies from the results of the non-intrusive geophysical investigations. The results indicate that large voids are not likely to exist beneath the concrete slabs, but suggest that localized zones of disturbed sands are present, which appear to correlate with the locations of sinkholes and depressions, geophysical anomalies, variations in measured shear wave velocities and reduced in situ densities. In order to evaluate the in situ soil density of the disturbed sands, a 20 m deep CPT was conducted under the slabs near the rockfill, at an inclination angle of 70°. The results of this test indicated that the disturbed sands have a measured cone tip resistance (q_t) of 18 to 21 MPa, from which an equivalent $(N_1)_{60}$ of about 34 blows/ft was derived.

Based on this information, the following locations of disturbed or loosened soil zones were identified:

- Near the upstream end of the concrete slabs (disturbance in this area may have been caused by seepage around the end of the cut-off or reverse gradients during reservoir drawdowns).
- In the area around the rockfill (likely caused by piping of fines or sand into the rockfill).
- In the area around the adit (possibly caused by seepage flowing along the back or sides of the adit). The extent of this loosened zone appears to be limited to less than about 5 m from the adit.

In addition to these locations, several very large features have been found downstream of the dam, which may be related to subterranean seepage flows in the sands. A buried sinkhole discovered in this area in 2009 was measured to be about 1.8 m in diameter and about 4 m in depth from the top of the exposure (see Figure 8). No significant seepage was noted to be present in the sinkhole. In 2012, another large void was encountered near the same area during sonic drilling and cone penetration testing. These sinkholes are believed to be associated mainly with seepage from the hillside, not the reservoir, but highlight the propensity of the Quadra Sand to become loosened through seepage and fines migration.

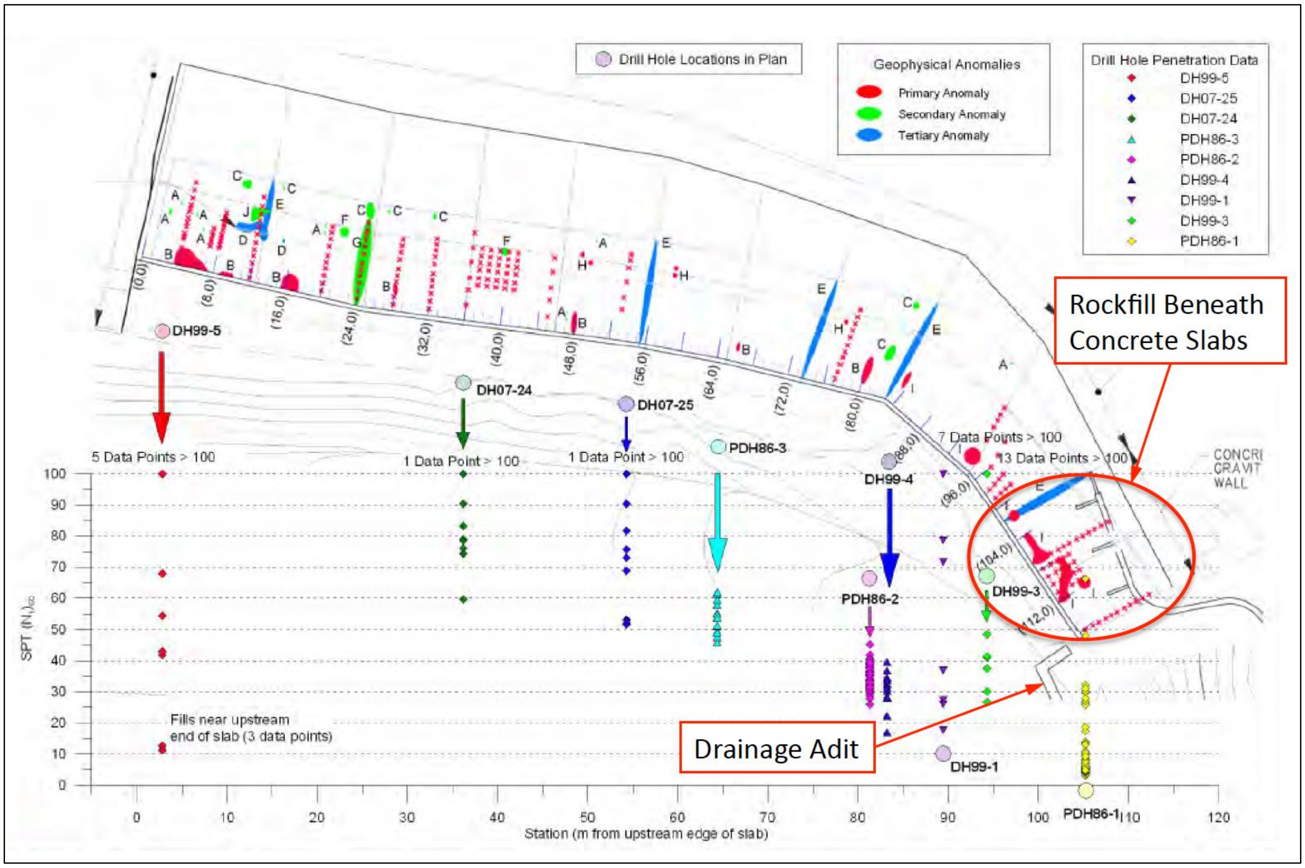

Figure 7: Location of potential voids and loosened soil

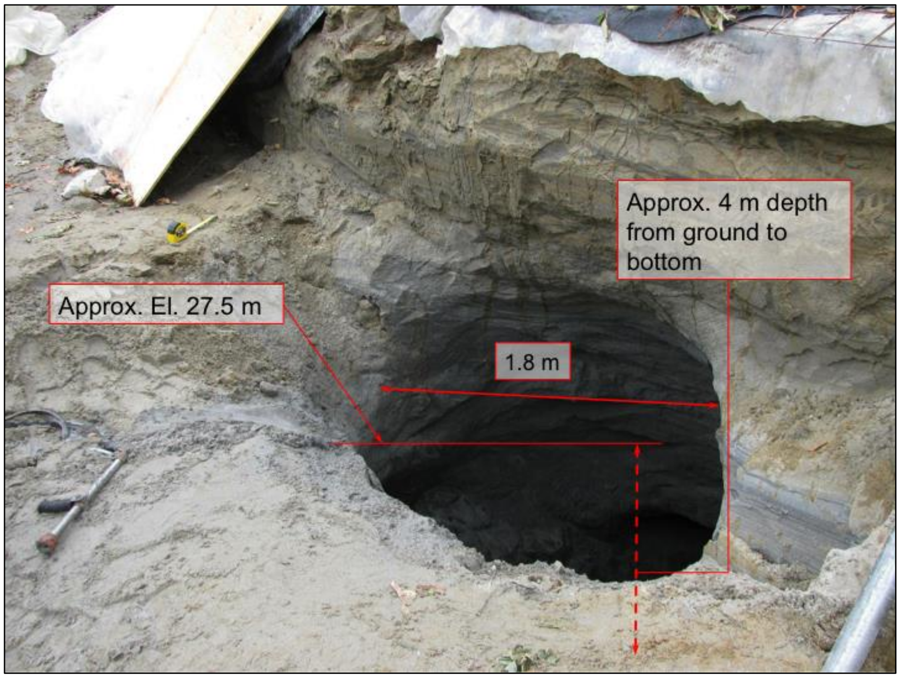

Figure 8: Sinkhole discovered in the Quadra Sand unit downstream of Ruskin Dam

3.3 Engineering properties of foundation soils

Engineering properties were obtained for the foundation soil units at Ruskin Dam to allow stability and deformation analyses to be carried out. Standard penetration and CPT results confirm that the Quadra Sand is very dense if undisturbed, with many SPTs terminated prior to completion and drill-outs required in most CPT holes. Standard penetration results from the Quadra Sand are shown in Figure 9. As discussed in Section 3.2, SPT blow counts of less than an $(N_1)_{60}$ of about 34 blows/ft or less were inferred to indicate disturbed sands. Shear wave velocity measurements were carried out in drill holes located both upstream and downstream of the dam. In general, the shear wave velocities in the sand are quite high, ranging from about 400 to 550 m/s.

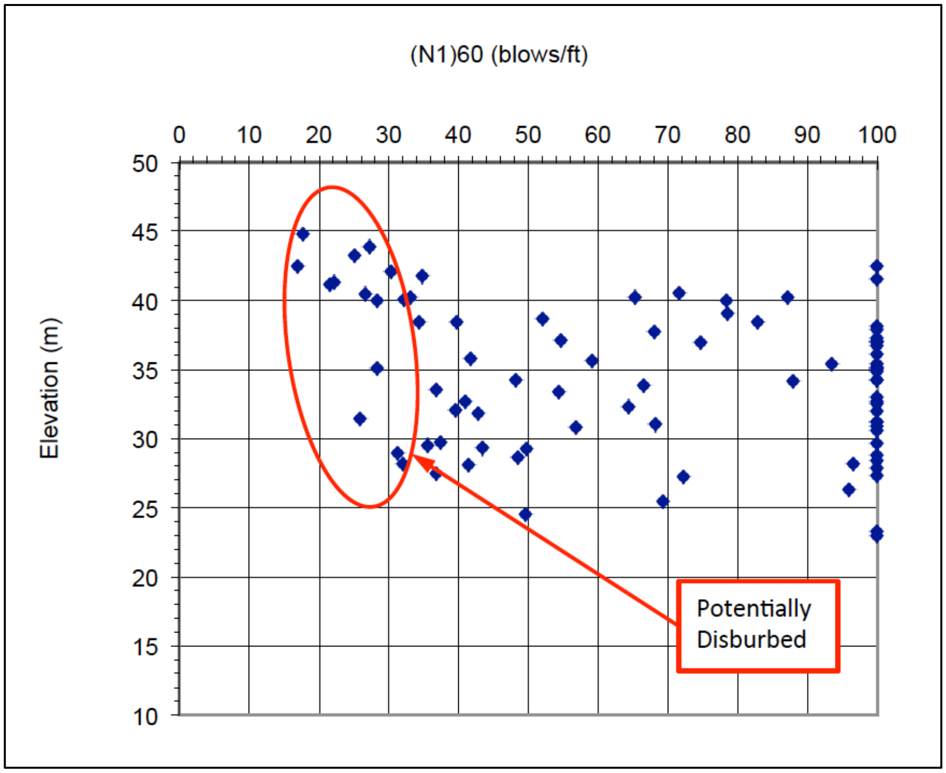

Figure 9: Summary of SPT results from Unit 4

Six pressuremeter tests were conducted in the Quadra Sand unit in one drill hole - located upstream of the dam - to obtain strength and deformation properties of the undisturbed sand. The test results indicate that the Quadra Sand has a high shear modulus and high horizontal stresses. Analyses of the test using the Non-associated Mohr Coulomb model resulted in G_{max} values between about 200 and 600 MPa (based on the unload/reload paths). Horizontal stresses were calculated between 100 and 300 kPa, depending on the depth, which is equivalent to $k_0 \approx 0.8$. The calculated friction angle from the upper five tests ranged from 42 to 44°, with a corresponding dilation angle of about 15°. The lowest test yielded a slightly lower peak friction angle of 37°, and a dilation angle of about 7°.

4 DESIGN OF UPGRADES

From the results of the characterization, it was concluded that although the Quadra Sand in the right abutment is very dense when undisturbed, large deformations are likely to occur during the MDE where the sand is disturbed near the concrete slabs and drainage adit. Thus, an upgrade option was selected to either place the new structures far enough from the damaged zones to minimize deformations or to strengthen the loosened soils by jet grouting. In addition, the upgrade option was also required to minimize the occurrence of piping and fines migration of the sands during normal operation.

The selected upgrade option for the right abutment consists of the following components:

- A new cut-off wall located inland from the top of the existing concrete slab cut-off wall. Deformations at the wall are minimized by placing it a sufficient distance away from the sloping concrete slabs and potentially loosened soils.
- Jet grouting adjacent to and below the right abutment end of the concrete dam and around the drainage adit to strengthen the soil and allow for the construction of the slot for the tie-in of the new cut-off wall with the concrete dam.
- Tie-in connecting the cut-off wall with the concrete dam constructed with overlapping drill holes and backfilled with a highly flexible, watertight material. The flexible tie-in is required to be highly robust, capable of surviving the differential deformations between the dam and right abutment soils expected during the MDE.
- Jet grouting of the soil immediately downstream of the tie-in to strengthen any potentially disturbed soils and loose fill, and thus minimizing seismic deformations. The improved soil also provides support for the new two-lane bridge above the dam.
- A seepage training wall to direct seepage away from the loose fills downstream of the dam and into a downstream filter blanket.
- A reverse filter blanket, sheet pile erosion barrier and drainage system on the downstream slope to prevent piping and collect seepage.
- A horizontal drain to replace the drainage adit and maintain the groundwater between the two cut-off walls near the existing level, preventing failure of the slabs during reservoir drawdowns.
- New instrumentation, including weirs and piezometers.

The general arrangement of the right abutment upgrades is shown in Figure 10.

The new cut-off wall is a backup to the existing seepage cut-off system, as the existing concrete slab cut-off system will be utilized as the primary seepage barrier until it fails during a seismic event or from long-term deterioration. When that happens, flows through the new horizontal drain are likely to increase, possibly requiring closure of the drain and transferring full reservoir loading to the new cut-off wall and tie-in.

4.1 Cut-off wall and training wall

The cut-off wall extends about 100 m upstream from the dam and is located between 18 and 20 m inland from the top of the existing concrete slab to minimize the impact of ground deformations during a seismic event. The cut-off wall is designed to function with full reservoir levels against the abutment. The training wall is connected to the cut-off wall near the dam, and extends downstream by about 40 m. The purpose of the training wall is to direct seepage away from the loose fills downstream of the dam and into the filter blanket on the right abutment slope. Plastic concrete was selected as the wall material to accommodate the deformations expected during the MDE, and the walls were designed with a width of 1 m to accommodate localized strain concentrations.

Non-linear seismic deformation analyses were carried out during the design phase with the finite element program VERSAT (Wu, 2009) to calculate probable deformations and strains, and to model the stiffness of the plastic concrete material. The performance of the cut-off wall and training wall was analyzed for both the MDE and the 1 in 475-year earthquake. Eight time histories were used, which were linearly scaled to the uniform hazard response spectrum at the period of interest. For the cut-off wall, the average displacements calculated for the MDE at the maximum reservoir level (El. 42.9 m) were generally less than 350 mm and the computed strains were generally lower than the 15% shear strain (10% axial strain) limit of the plastic concrete. Parametric analyses were also conducted to investigate the impact of the wall stiffness on the stresses and deformations. The shear strength of the wall was varied between 0.1 MPa to 1 MPa and subjected to the MDE ground motions. Generally, large strains were computed in the wall for the low strength plastic concrete, and higher tensile stresses developed when the plastic concrete was assigned higher shear strengths, especially at the elevation of the Unit 3 silt. Based on these results, an envelope of shear strengths between 0.1 MPa and 0.75 MPa, approximately corresponding to unconfined compressive strength (UCS) values of 200 kPa and 1.5 MPa, were used for the plastic concrete. Example strain and tensile stress results from a typical deformation analysis at the cut-off wall are shown in Figure 11. The analysis also indicated that cracking is not likely to occur for the 1 in 475-year earthquake ground motions.

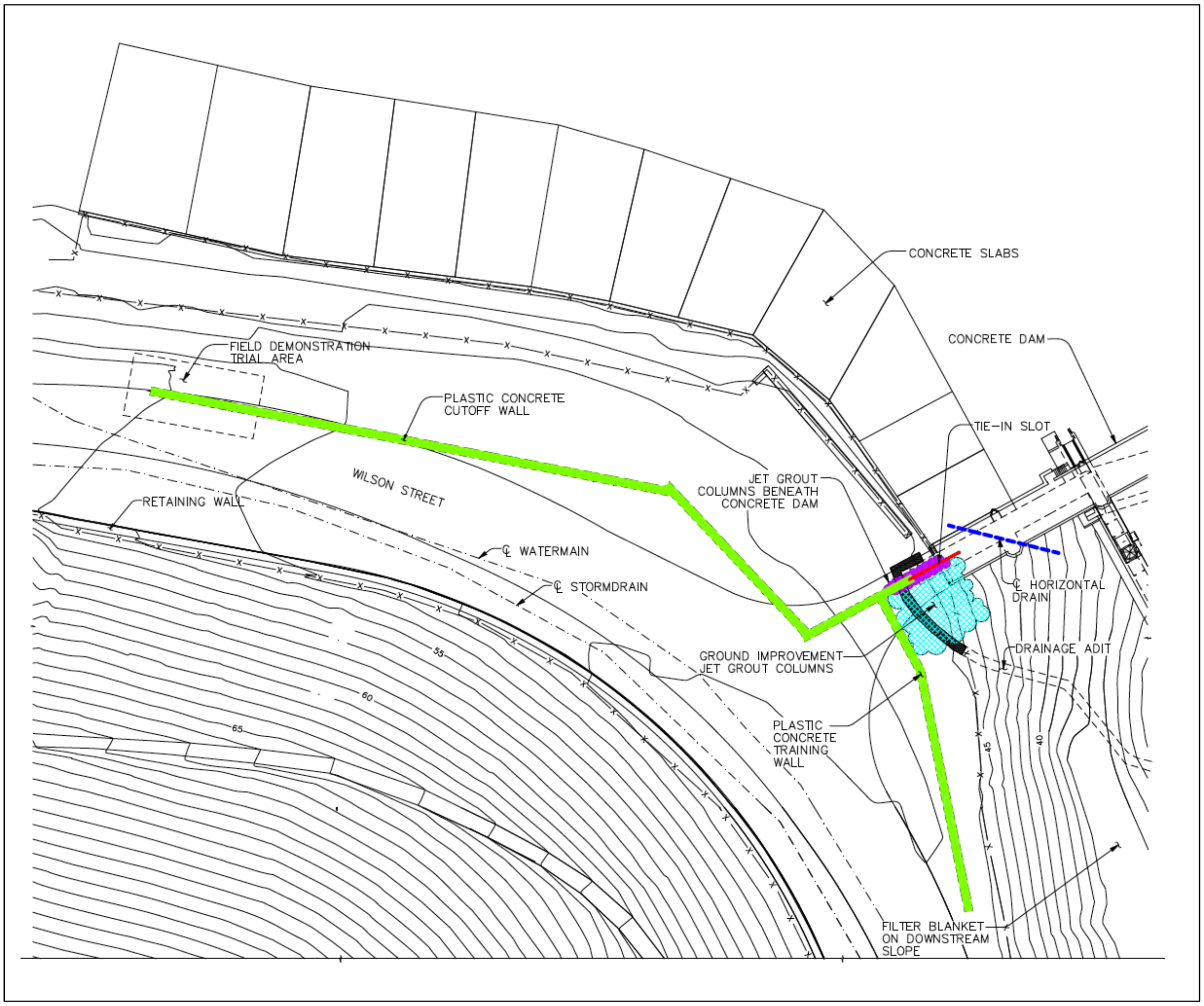

Figure 10: General arrangement of right abutment upgrades

Seepage modeling was carried out with the finite difference program MODFLOW (Waterloo Hydrogeologic, 2006) to simulate the performance of the new seepage control system for the right abutment. The results were used to determine:

- the required length of the cut-off wall,
- post-construction and post-earthquake gradients and heads for various cases,
- the effect of higher strains (i.e. deformation or cracking) of the cut-off wall in Unit 3 silt between El. 27 and 29 m,
- the effect of a reservoir drawdown on the concrete slabs following construction of the cut-off wall,
- the effect of the horizontal drain on the water levels and flows, and
- the effect of planned and emergency reservoir drawdowns on the existing concrete slabs.

The seepage analyses confirmed that the cut-off and training walls would be sufficient to control the reservoir seepage gradients in the right abutment for both normal operation and post MDE loading conditions. The analyses also confirmed that any seepage resulting from localized higher strains (or failures) in the new cut-off wall caused by the MDE would be contained within the downstream filter blanket. Finally, the requirement for a horizontal drain was identified to maintain low heads beneath the concrete slabs, thereby preventing failure during a reservoir drawdown.

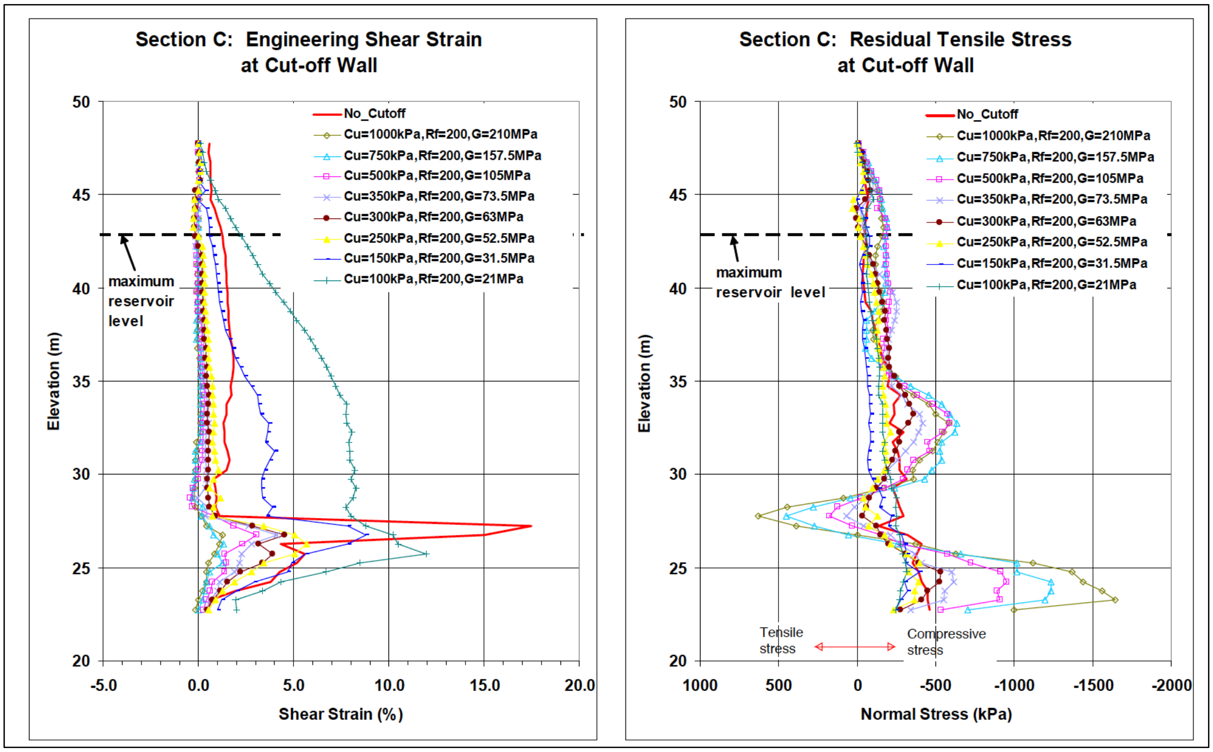

Figure 11: Earthquake-induced strains and tensile stresses at cut-off wall

The cut-off wall and training wall panels were constructed by a hydromill using the slurry panel method. The hydromill was required to excavate through soil Units 6, 5, 4, 3, 2 and 1 of the right abutment to a maximum depth of about 34.5 m. All panels were keyed into bedrock by at least 0.5 m to ensure an adequate seal. The panels were constructed with plastic concrete (cement, bentonite and coarse and fine aggregate), which is able to accommodate up to 15% shear strain. The top of the walls terminate at El. 45 m, which is approximately 1 m higher than the top of the existing concrete slabs. A section along the cut-off wall is shown in Figure 12.

4.2 Tie-in of cut-off wall to concrete dam

The tie-in of the cut-off wall to the concrete dam is the most critical component of the right abutment upgrade, as leakage through this portion of the wall would result in a very short seepage path around the end of the concrete dam. The tie-in is also complicated by the geometry of the concrete dam at this location. The dam, west of its connection to the sloping concrete slabs, is a narrow 2.4 m wide section founded partially on bedrock and partially on soil, where concrete was placed against steeply excavated silts and sands during the original construction. A very flexible connection is required to accommodate the potential differential deformations between the concrete dam (founded primarily on bedrock) and the foundation soils of the abutment, while still providing a watertight connection.

As part of the design for the tie-in, the soils beneath the overhanging portion of the concrete dam were jet grouted to create a block of soilcrete with a minimum UCS of 6 MPa, and the first panel of the plastic concrete wall was excavated into the soilcrete to form a key. A slot was then constructed by a series of overlapping drill holes in the dam, soilcrete, and plastic concrete, and extended a minimum of 0.5 m into bedrock. To construct the slot, very accurate NQ primary pilot holes were initially drilled along the slot length. These holes were then opened to 0.3 m diameter and a special guide system was used to drill the 0.3 m diameter secondary holes by following the primary holes on either side, resulting in a slot with a minimum width of 168 mm. The slot was backfilled with highly flexible asphalt-based mastic capable of withstanding 50 mm of concentrated deformation. To ensure that the end of the tie-in does not crack during a seismic event, the slot was extended past the soil/bedrock/concrete interface by about 2 m and into the gravity portion of the dam. A section through the tie-in is shown on the right side of Figure 12. Downstream of the dam, the loose fill soils were strengthened by jet grouting to provide additional stability for the tie-in during a seismic event while also acting as the foundation for the downstream footing for the new bridge deck. The jet grouted block extends approximately 8.5 m downstream of the dam centreline and about 10 m east from the training wall (see Figure 10). The vertical extents are from El. 43.5 m to bedrock or the Unit1/2 soils located immediately above bedrock.

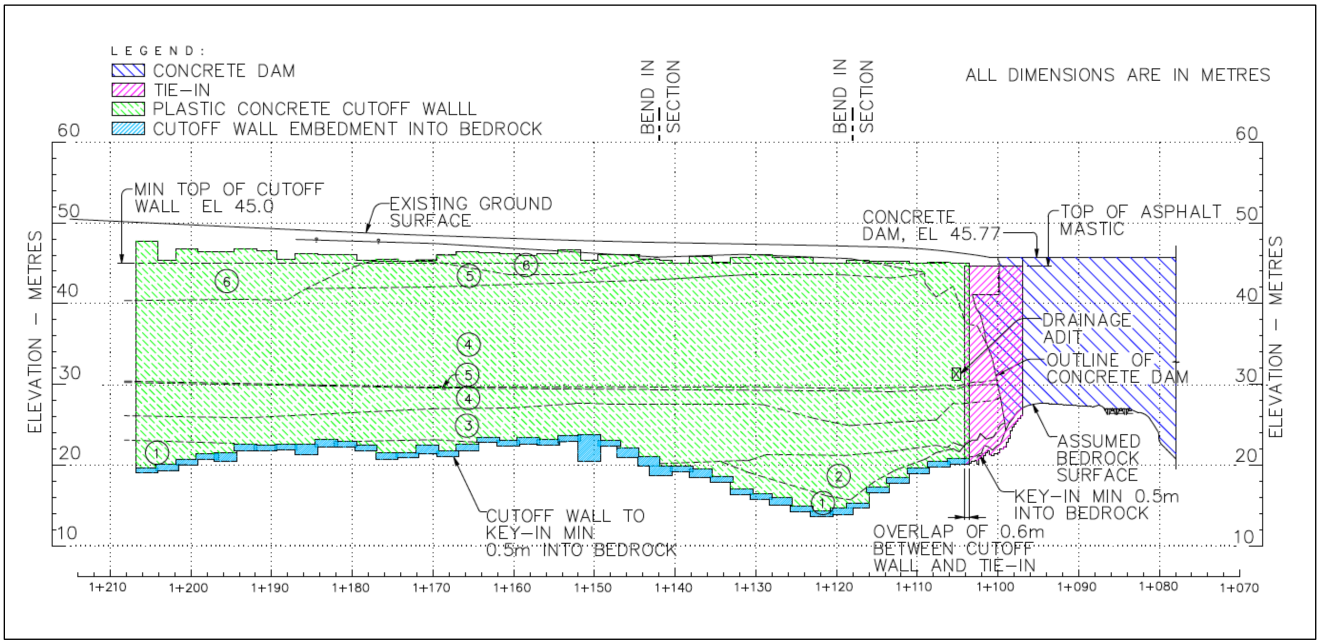

Figure 12: Section through cut-off wall and tie-in

Slope stability through the tie-in area was difficult to analyze due to the 3-D nature of the geometry and the complicated intersection of materials (soil, concrete, and soilcrete). Soil is present on both sides of the tie-in, sloping down parallel with the dam (concrete slabs on the upstream side and filter blanket slope on the downstream side). Critical deformations through the tie-in were estimated by considering the lateral displacement of the concrete dam, which is expected to move with the bedrock during the MDE relative to the abutment soils. Based on 3-D modeling carried out for the concrete dam, the maximum lateral movement of the dam is estimated to be about 18 mm. A minimum of 50 mm deformation through the flexible tie-in membrane in the upstream/downstream direction (without cracking) was conservatively adopted for the tie-in design. In addition to the differential movement between the dam and soil, another concern was that large deformations in the slopes upstream or downstream of the dam could potentially expose the tie-in, resulting in cracking or deformation. Upstream of the dam, the worst-case deformations were estimated to be about 0.5 m in the horizontal direction (towards the reservoir) and 0.2 m in the vertical direction based on the 2-D non-linear seismic deformation analyses, leaving the tie-in to remain supported by soils. On the downstream side, smaller deformations are expected due to the ground improvement provided by the jet grout columns. Horizontal and vertical displacements at the top of the soilcrete are expected to be about 0.03 m and 0.01 m, respectively, which are unlikely to cause damage to the tie-in.

4.3 Downstream improvements

On the downstream slope, the upgrade consists of a two-zone filter blanket at a slope of 2 horizontal to 1 vertical, which is designed to keep the native soils from piping and allow seepage to be collected. The filter blanket consists of a 2.5 m wide zone of concrete sand, sized appropriately to prevent the migration of fines from the Quadra Sand (but allow seepage), and a 2.5 m wide zone of fine rockfill to provide stability and protection of the filter. At the toe of the blanket an erosion barrier was installed in the Unit 3 and 4 soils above bedrock by the installation of a sheet pile wall. On the slope side of the sheet pile wall and below the blanket filter sand, a French drain was constructed consisting of slotted high-density polyethylene (HDPE) pipe, surrounded by drain gravel and filter sand. A separate French drain was constructed perpendicular to the portal of the drainage adit to collect potential seepage flows around the outside of the adit and from the area between the adit and the concrete dam pier. Both drains are connected to a weir house located about 32 m downstream of the dam, which was constructed on bedrock. Weir 13 (Blanket French Drain) and Weir 14 (Adit French Drain) currently monitor flows from the abutment, and Weir 15 will monitor flows from the horizontal drain once it has been completed. The location of the drains and the seepage collection areas on the downstream slope are shown in Figure 13.

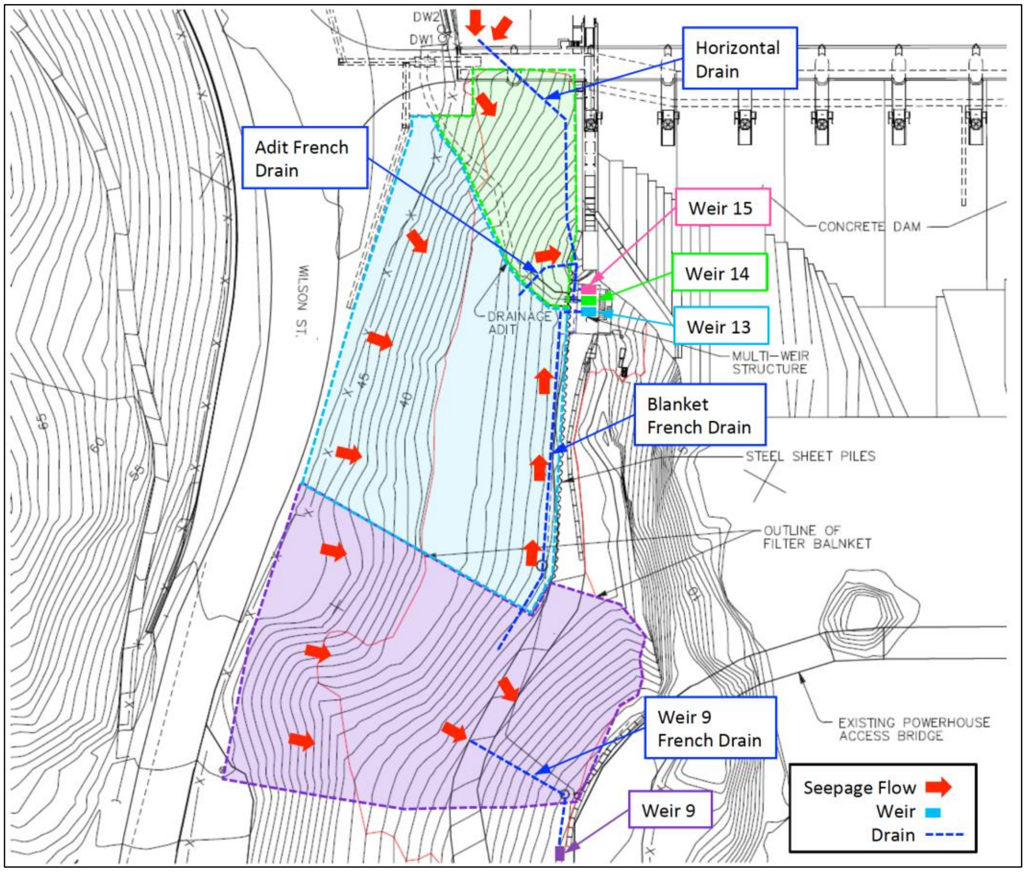

Figure 13: Seepage collection areas on the downstream right abutment

4.4 Horizontal drain

A drilled-in, horizontal drain was designed to replace the drainage adit, which had been plugged with concrete prior to jet grouting. The purpose of the drain is to collect water from the rockfill beneath the concrete slabs and transport it through the dam and into Weir 15. The horizontal drain contains a 2.75 m long screened section, pre-packed with filter sand to prevent the ingress of fines into the drain. The invert of the screen is at about El. 30.8 m and is designed to maintain water levels at about El. 31 to 32 m in the rockfill. The location of the drain is shown in Figures 10 and 13. At the downstream end, the drain will terminate in a vault in the new adit gallery, which will be constructed in the widened concrete dam pier. Two valves will control the flow. The main valve (downstream valve) is to be located directly upstream of the weir house and will allow for flow regulation during normal operation and shut-off of the flow post-earthquake. A secondary valve (upstream valve) will be located in the vault and will allow maintenance or replacement to be carried out on the downstream valve.

5 CONSTRUCTION

The upgrades were constructed in three stages. Stage 1 consisted of improvements to the downstream abutment, as well as the realignment of Wilson Street to create space for construction of the cut-off wall. This work was completed between April 2009 and June 2010 with a direct construction cost of $6.9 million. The Stage 2 improvements consisted of construction of the cut-off wall, training wall and tie-in, which were completed between January 2012 and July 2013 with a direct construction cost of $24.3 million. Finally, the drilling and installation portion of the horizontal drain was carried out as part of the concrete dam upgrade work in June 2014 (construction of the vault and valves has not yet been completed). Due to the complex nature of the site and the significant dam safety risks associated with the Stage 2 work, a unique procurement model was used to award the construction contract. An early contractor involvement (ECI) model was adopted for the project, which allowed the risks and complexities to be communicated to the two proponents involved in the process. The model enabled the proponents to develop and refine their approach to construction, leading to the award of the contract to the best approach.

Two deep reservoir drawdowns were required to mitigate dam safety risks during the Stage 2 construction (one in 2012 and one in 2013). The first drawdown was required to mitigate the risks of jet grouting in potentially disturbed sands beneath the dam, and the second was required to mitigate risks associated with drilling and backfilling of the open slot in the dam during construction of the tie-in. For jet grouting, the performance of nearby instruments was calibrated in a field trial to understand the behaviour of pore water pressures in the abutment soils. The results of the field trials allowed instrument warnings and alarms to be set for instruments near the production jet grouting area. Strain-gauge type transducers were installed in these piezometers to allow high frequency data acquisition and display at a rate of about one reading per second. If an alarm was triggered during jet grouting, the high pressure was immediately reduced to low pressure and the pore water pressures observed during dissipation. If the pressures dissipated in a way that was consistent with the performance during the field trial, jet grouting was continued. Other potential risks that were identified during jet grouting consisted of jetting through loose zones or voids in the Quadra Sand (Unit 4), as well as in the loose fills downstream of the dam. The loss of reflow in the Quadra Sand was not significant and any voids that were encountered were quickly filled with grout. However, loss of reflow occurred during the jet grouting trial in the loose fills (downstream and adjacent to the concrete dam), resulting in the requirement to pre-grout the soils with a soft cement-bentonite mix prior to jet grouting. Post-construction monitoring of instruments has generally indicated that the new cut-off system is providing adequate seepage control with head differences observed across the cut-off wall, and a decrease in seepage flows from about 16 l/min to 2 l/min in Weir 14 and from 5 l/min to 1 l/min in Weir 13.

6 CONCLUSION

The right abutment of Ruskin Dam was identified to have deficiencies under normal operation (seepage and piping risks) and for seismic conditions (failure of the cut-off system). The presence of Quadra Sand on the right abutment contributed to a history of piping, ground settlement, and other dam safety risks. A new seepage cut-off system has been constructed to remediate these deficiencies. The implementation of the upgrades required careful understanding of the dam safety issues, characterization of the site and design of a new cut-off system which meets the site constraints and project requirements. The design and construction of the new cut-off and training walls, tie-in to the dam, horizontal drain, and downstream seepage collection system resulted in a robust solution that is capable of withstanding the MDE, as well as addressing the concerns with seepage and piping during normal operation. The use of flexible components such as plastic concrete for the cut-off and training walls and asphalt mastic for the tie-in ensures that the displacements and deformations expected during the MDE can be accommodated without significant increases to post-earthquake seepage flows.

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