TUNNELS AND PENSTOCKS OF THE NAM THEUN 2 HYDROELECTRIC PROJECT

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ABSTRACT:

The 1,070 MW Nam Theun 2 Hydroelectric Project is located in central Lao PDR. The tunnels and penstocks convey the power flow from the reservoir impounded by the Nakai Dam, down to the surface powerhouse located at the base of the escarpment on the Nam Kathang River. The gross head of the project is 360 m. The maximum power flow is conveyed through a concrete-lined headrace tunnel, down a concrete-lined pressure shaft, through a steel-lined high pressure tunnel to steel-lined bifurcations where the water is conveyed to four Francis turbines, each 250 MW, and two Pelton turbines, each 35 MW. The headrace tunnel, 1,500 m long, was excavated through near horizontally bedded sandstone and siltstone rock at the upstream end which then gradually transformed to steeply dipping sandstone and siltstone rocks. The 270 m deep vertical pressure shaft and the 1,000 m long near horizontal, high pressure steel-lined tunnel were excavated through these steeply dipping sandstone and siltstone rocks. The bifurcations and penstocks are designed for a peak transient pressure of 4.9 MPa. A concrete-lined surge shaft at the downstream end of the headrace tunnel limits the maximum transient pressures in the high pressure tunnel. The project presented some unique design challenges for the design of the steel-lined high pressure tunnels, bifurcations, tunnel plugs, hydraulic throttle in the surge shaft and an access door for maintenance in the downstream end of the headrace tunnel.

RÉSUMÉ:

Le projet hydroélectrique de Nam Theun 2 se situe au centre de la République Démocratique du Laos. Les tunnels et les conduites forcées prennent leurs eaux dans la retenue artificielle crée par le barrage du Nakai. La centrale hydroélectrique, d'une puissance de 1,070 MW, est située en contrebas des escarpements du plateau, au bord de la rivière Nam Kathang. La hauteur de chute est de 360 m et le débit maximum est acheminé par un tunnel de tête et une cheminée d'équilibre verticale dans sa partie amont. Une conduite forcée et un réseau de bifurcations en acier acheminent enfin l'eau vers quatre turbines Francis de 250 MW, et deux Peltons de 35 MW. Le tunnel de tête de 1,500 m a été percé dans des grés au litage sub-horizontaux à l'amont, et dans une formation argilo-gréseuse et de grès d'un litage graduellement très incliné dans sa partie aval. La cheminée d'équilibre verticale, d'une hauteur de 270 m et le tunnel haute pression horizontal renforcé en acier de 1,000 m de long ont été percés à travers des litages de grès et d'argiles gréseux très inclinés. Le dimensionnement des bifurcations et des conduites forcées est basé sur des efforts maxima transitoires de 4.9 MPa. La cheminée d'équilibre, revêtue de béton armé située dans la partie aval du tunnel de tête, limite l'intensité des effort transitoires dans le tunnel haute pression. Le projet présentait des cas uniques et complexes de calcul, en particulier pour la partie de tunnel renforcé en acier, des bifurcations, des bouchons d'étanchéité, l'orifice hydraulique de la cheminée d'équilibre, et la porte d'accès pour la maintenance de la partie aval du tunnel de tête.

1 THE PROJECT

The Nam Theun 2 Hydroelectric Project is located in highlands of Lao PDR. The Nakai River is impounded by the Nakai Dam and flows are diverted from the reservoir though a headrace canal to the power intake at the edge of the plateau. The power flow is conveyed through a single headrace tunnel, pressure shaft and high pressure tunnel to the surface powerhouse at the base of the escarpment. The high pressure tunnel bifurcates into two steel-lined tunnels just upstream of the powerhouse and then each of these two tunnels bifurcate again into four steel penstocks which convey the water to four Francis turbines. In addition to the Francis units, there are two Pelton units located in the same powerhouse. These units are supplied with water from a branch-off in the single steel-lined high pressure tunnel just upstream of the first bifurcation.

The power flow from the powerhouse is discharged into the Nam Kathang River and a regulating dam is located some distance downstream of the powerhouse to regulate the downstream releases. Water is released from the regulating dam into a downstream channel and then to the Xe Bang Fai River which flows into the Mekong River. Figure 1 shows the project location and the overall layout.

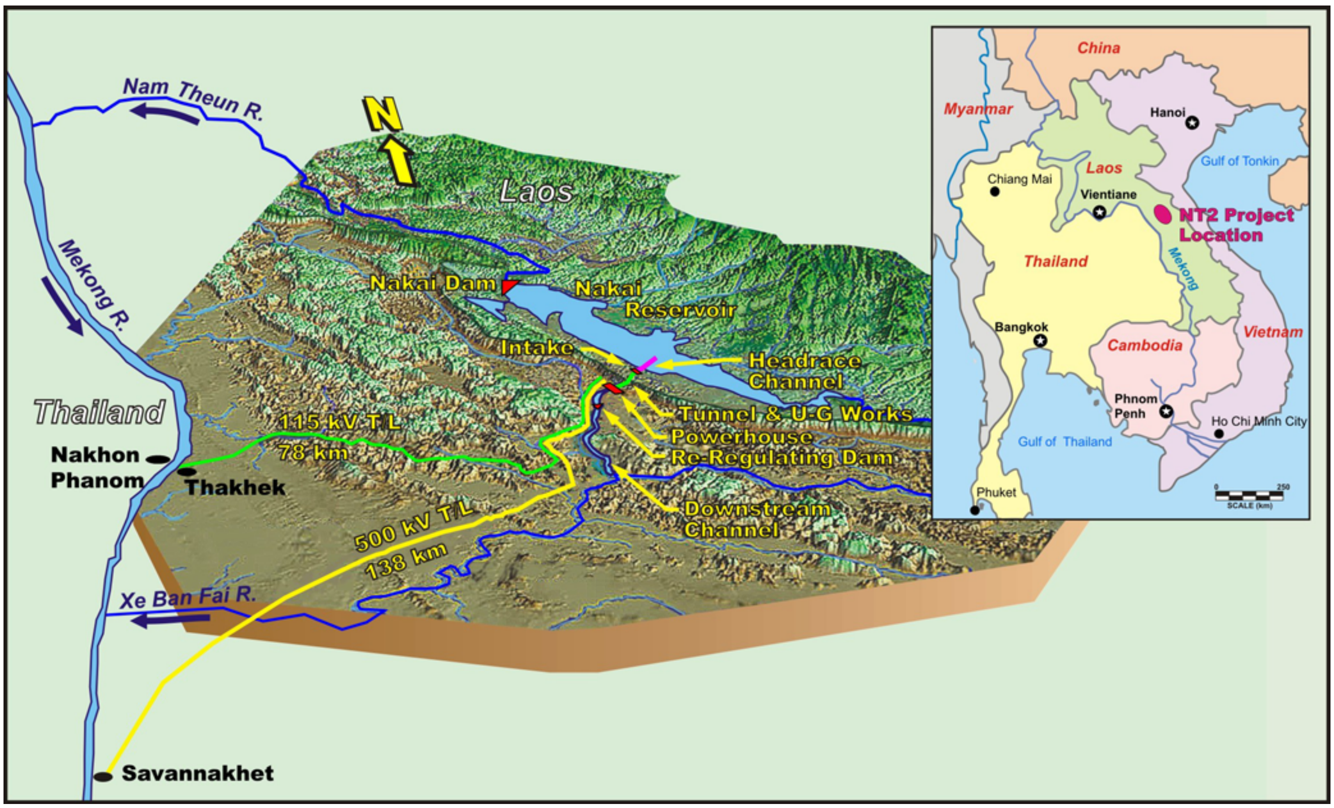

Figure 1: Layout of the Nam Theun 2 Hydroelectric Project

The total installed capacity of the Nam Theun 2 powerhouse is 1,070 MW comprising four 250 MW Francis turbines and two 35 MW Pelton turbines. The Francis units will export power to Thailand and the Pelton units will provide power into the local Lao gird. The gross head of the project is 360 m. The plan of the underground works is presented in Figure 2 and a profile through the waterway is presented in Figure 3. The underground works comprise an intake, a 9.2 m internal diameter headrace tunnel, an 8.8 m internal diameter vertical pressure shaft, an 8.8 m internal diameter concrete-lined high pressure tunnel, a 7.15 m internal diameter steel-lined pressure tunnel bifurcating into two 5.08 m internal diameter steel-lined tunnels and then into four 3.6 m internal diameter unit penstocks. The waterway for the Pelton units branches-off from the main 7.15 m diameter steel-lined high pressure tunnel into a 2.55 m internal diameter steel-lined tunnel which bifurcates into two 1.8 m internal diameter unit penstocks.

The maximum power flow in the tunnel is 344 m^3/s and the maximum transient design pressure for the manifold (bifurcations and unit penstocks) is 4.9 MPa.

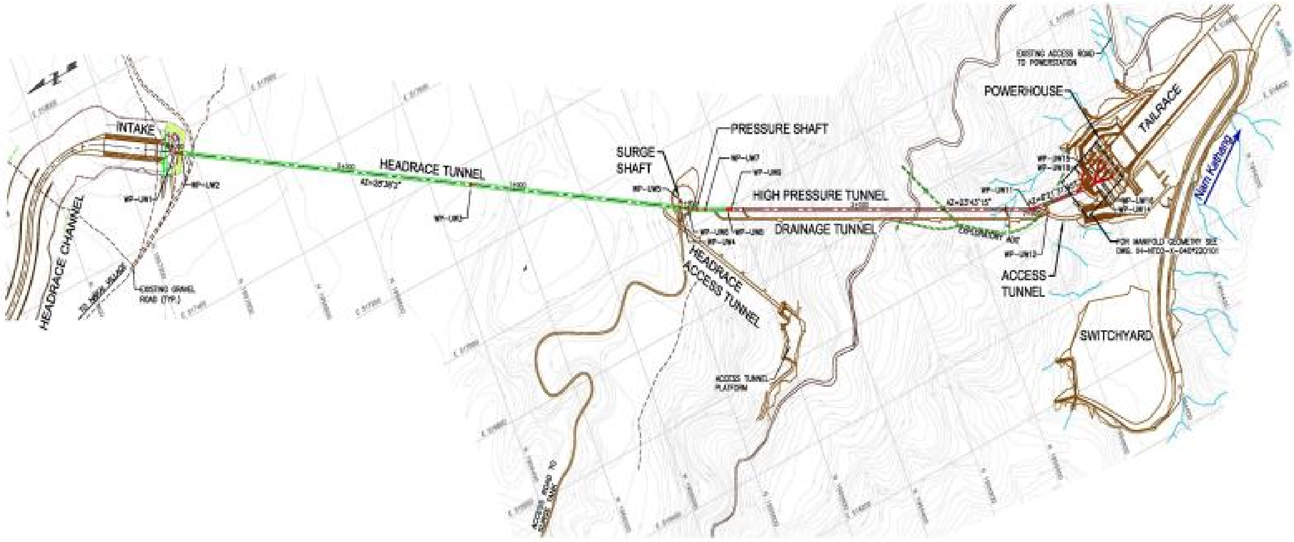

Figure 2: Plan of the Underground Works

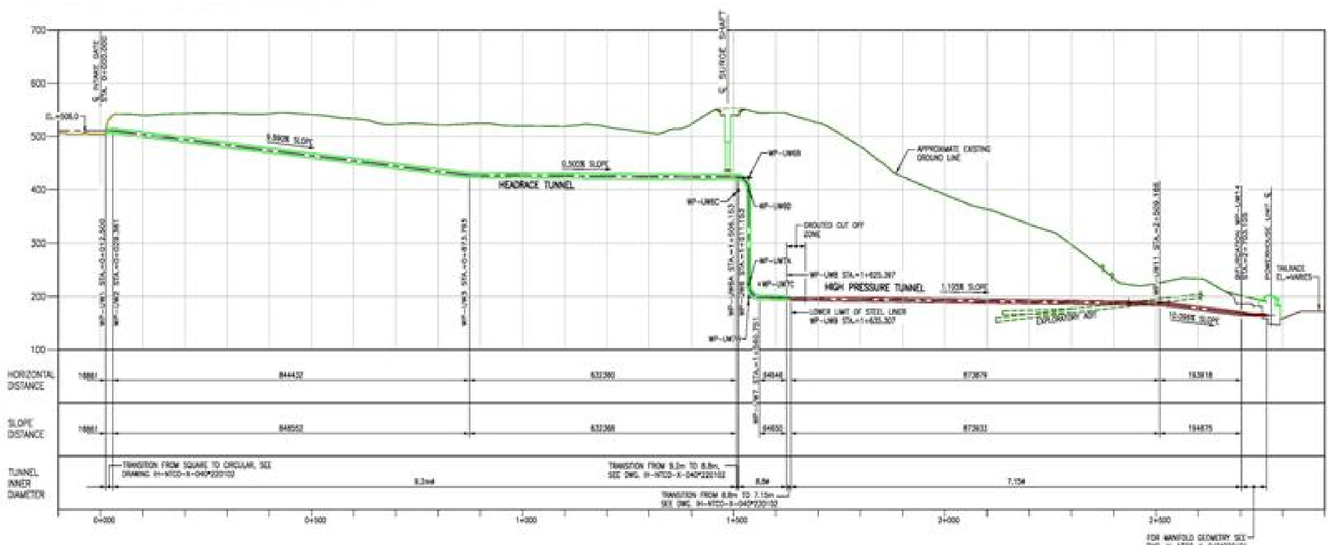

Figure 3: Profile of the Waterway

2 THE GEOLOGY ON THE ALIGNMENT OF THE TUNNELS

The tunnels are located in sedimentary Khok Kruat and Phu Kradung formations of the Khorat Group. The bedrock is sedimentary interbedded sandstone, siltstone and mudstone and the stratigraphy includes lengths of fractured mudstone or shear zones. The headrace tunnel is mostly located within the Khok Kruat formation with near horizontal bedding. The downstream portion of the headrace tunnel and the remainder of the underground works are located within the lower Phu Kradung formation which dips at approximately 70°. The dip and strike of these bedding planes were found to be favourable for the excavation of the tunnels as the strike relative to the tunnel alignment was approximately at 45° to the axis of the tunnel and the bedding planes dip in the upstream direction.

The rock conditions at the transition between the two formations in the headrace tunnel was initially a concern as the available borehole cores were not able to identify if this contact was highly fractured. However, it was found that during the advance of the tunnel excavation through this region the bedding planes gradually changed from a dip of 70° to near horizontal and no difficult tunnelling conditions were encountered.

All the underground excavations were mapped along their entire length. Rock conditions after each blast were evaluated using the Barton Q system (Hoek, et al, 1995).

The rocks through which the tunnels were excavated are relatively weak for the size and internal pressure of the project. The deformation modulus of the rock surrounding the tunnel is an important parameter needed for the design of the tunnel linings. Various methods were used to evaluate this parameter, both in the field and analytically. Plate jacking tests were originally carried out in an adit in the exploratory tunnel in the early stages of the project. Deformation modulus values derived from these tests were used as the basis for the conceptual design of the project. During construction, additional plate jacking tests were done in adits off the drainage tunnel, one in the region with lower rock cover over the tunnel and the other near the upstream end of the steel-lined portion of the high-pressure tunnel.

In addition to these plate jacking tests, seismic refraction tests (Scarabee Method) were done at selected locations in the headrace tunnel, surge shaft, pressure shaft and high pressure tunnel and also at the plate jacking test adits. These tests provided estimates of the approximate widths of the zone of rock surrounding the tunnel that had been disturbed by blasting and which has a lower deformation modulus than the surrounding undisturbed rock.

Utilizing the continuous record of the rock conditions along the entire length of the tunnels and the evaluation of the Barton Q' rock quality index along the entire length, the deformation modulus was estimated analytically using the equations presented in Hoek, et, al, 2002.

For the concrete-lined tunnels, "good", "intermediate" and "poor" rock conditions used design deformation modulus values for the undisturbed rock surrounding the tunnel of 12 GPa, 7 GPa and 3 GPa, respectively. For the design of the steel-lined portion of the high pressure tunnel, a maximum value of the deformation modulus used in the final design was specified as 5 GPa and all the rock along the length of this portion of the tunnel, met or exceeded this specified design value.

The deformation modulus values of the disturbed rock surrounding the tunnel excavation was generally assumed to be half of the deformation modulus of the undisturbed rock for the "good" rock conditions and one quarter of the deformation modulus of the undisturbed rock for "intermediate" and "poor" rock conditions. The width of this zone of disturbed rock was obtained from the seismic refraction tests and design values of 1 m, 2 m and 2.5 m were used for "good", "intermediate" and "poor" rock conditions, respectively.

3 DESIGN OF THE TUNNEL LININGS

3.1 Concrete-Lined Tunnels

The tunnel linings are designed for rock loads, net internal pressure, external groundwater pressure when dewatered, grout pressures during construction, temperature differential and seismic motion. The thickness of the headrace tunnel concrete lining is 425 mm in the upper section of the tunnel and 475 mm in the lower section. Generally good rock conditions were encountered in the headrace tunnel which resulted in 78% of the length of the headrace tunnel not requiring reinforcement in the lining. The remainder of the headrace tunnel generally had one layer of hoop reinforcement, typically 20 mm diameter bars at 200 mm centres, mainly to limit the crack width in the concrete lining to meet the specified leakage criteria.

The thickness of the surge shaft concrete lining varied from 390 mm in the lower portion, 8.8 m internal diameter, to 550 mm in the upper portion, 13.9 m internal diameter. The lining is reinforced throughout the height of the surge shaft, mainly as a result of the lower deformation modulus of the rock in these areas and the larger diameter of the upper shaft.

The pressure shaft concrete lining varied from 600 mm to 800 mm thick and is reinforced along its entire height. The concrete-lined portion of the high pressure tunnel is 750 mm thick and is reinforced along it length, typically with 2 layer of hoop steel.

3.2 Steel-Lined Tunnels

The steel-lined portion of the high pressure tunnel extended approximately 1,000 m upstream of the manifold. Of that distance, only the lower 50 m upstream of the manifold, had to be designed for the full internal pressure to be carried by the steel lining alone as a result of the low rock cover. The remaining length of the steel lining is designed for composite action with the surrounding rock carrying a portion of the internal pressure, with the exception where the lining passed through the drainage chambers.

The criteria used to determine whether the rock cover was sufficient to permit the steel-lined high pressure tunnel to be designed with composite action with the surrounding rock is that given in the ASCE/EPRI Guides 1989. The steel lining is designed for internal pressure, external pressure when dewatered, grouting pressure, temperature differentials and seismic motion. The standard used for the design of the steel lining is the CECT Recommendations, 1984. For the external groundwater load case, a maximum design external pressure of 0.3 MPa is used for lining downstream of the grout cut-off zone. This is the maximum groundwater pressure surrounding the lining with the drainage system in place around the high pressure tunnel. The buckling resistance of the steel liner used the methodology of Jacobsen, 1974 and the design value of the annular gap between the outside of the steel lining and the surrounding concrete is 4.5×10^{-4} times the internal radius of the steel liner.

The steel used for the steel lining is Sumiten 610F-TMC with a yield stress of 490 MPa and an ultimate strength of 610 MPa. For the bifurcations, the thickness of the steel using the Sumiten 610F-TMC steel was found to be too thick to fabricate and a high strength, quench-hardened steel is used, HT80 steel with a yield stress of 685 MPa and an ultimate strength of 780 MPa. The thickness of the steel lining in the cylindrical portions of the high pressure tunnel varied 29 mm to 50 mm and the maximum thickness of the steel shell in the main 7.15 m/5.08 m/5.08 m bifurcation is 75 mm and the thickness of the internal stiffener is 170 mm.

4 POWER CONDUIT COMPONENTS

There are a number of interesting features in the power conduits and these are presented in this paper. They include:

- An intake with two identical intake gates;
- An access door into the lower end of the headrace tunnel;
- The three dimensional analysis and design of the intersection of the surge shaft and the headrace tunnel;
- The orifice in the surge shaft;
- The grout curtain and drainage curtain at the upstream end of the steel-lined portion of the high pressure tunnel;
- The analysis and design of the steel bifurcations; and
- The analysis and design of the Pelton branch-off from the main steel-lined high pressure tunnel.

4.1 Power Intake

The intake is located at the end of the headrace channel and comprises a traditional horizontal intake with trash racks, trash rake, and intake gates. Based on safety considerations, the Owner specified that the intake must contain two fully functional intake gates instead of the usual arrangement of an intake gate with maintenance stoplogs upstream of the intake gate.

The intake gates are raised and lowered by way of hydraulic actuators mounted above the intake deck. The gates can be raised and dogged just below the underside of the intake deck. Maintenance platforms are provided inside of the intake for personnel access to both sides of the gates.

Only one of the gates is designated as the operational intake gate at any one time. This is usually the downstream gate. The other gate is the stand-by gate. In the event of a failure of the designated intake gate to close during an emergency closure sequence, the stand-by gate will automatically take over and close, thus providing redundancy against a failure of the gate to close in an emergency. If required, the upstream gate can be closed to facilitate the dewatering of the downstream gate for inspection and maintenance. The upstream gate cannot be dewatered.

Two identical fully functional intake gates also provide double device isolation when the power conduit to the powerhouse is dewatered for inspection and maintenance. This double device isolation feature is becoming increasing important today in the design of hydropower plants in terms of worker safety.

4.2 Headrace Tunnel Access Door

To facilitate construction of the 1.5 km long headrace tunnel, a headrace access tunnel was constructed from a terrace located approximately halfway down the escarpment just off the access road that runs up the escarpment from the powerhouse to the village of Nakai. At the completion of construction, this access tunnel was retained and a watertight door is located at the end of this tunnel where it joins the headrace tunnel. A clear opening of 2 m square was specified for this door. In addition, the door is required to open inwards and be able to be opened by a single person.

It was decided to design the door as a circular door with a hemispherical dome as the main water retaining structure. The arrangement of the door is such that the inscribing circle of the inner flange of the door and door frame contains the minimum opening size of 2 m square. Figure 4 is a photograph of the concrete plug in the headrace access tunnel with the door opening prior to installation of the door itself. The door is hinged on one side and when closed, is bolted from the headrace access tunnel side. Figure 5 is a photograph of the door being installed. "O" ring seals on machined flanges of the door and embedded door frame provide the watertight sealing arrangement. A concrete ramp on the headrace access tunnel side provides access to the 2 m square opening through the door and a portable ramp provides similar access on the headrace tunnel side.

Figure 4: Headrace Access Tunnel Opening

Figure 5: Headrace Access Door

The door was analyzed by finite element analysis and as the hemispherical dome is in compression and therefore sensitive to potential buckling, a buckling analysis was also performed. For this analysis, the first buckling mode shape of the dome was computed, and then the geometry of the dome in the finite element model was altered to include the maximum specified fabrication out-of-tolerance dimensions conforming to the first buckling mode. The analysis was then continued by increasing the applied external pressure until yield or buckling of the dome occurred.

4.3 Headrace Tunnel/Surge Shaft Junction

The surge shaft is connected to the headrace tunnel just upstream of the headrace tunnel/pressure shaft junction. The surge shaft comes off the side of the headrace tunnel and immediately rotates upward through a 90° bend. The surge shaft orifice is located a short distance above this bend. This intersection also includes a smaller branch access tunnel from the main headrace access tunnel that was required during construction and which was closed at the end of construction.

The design of the concrete lining of this junction was done by finite element analysis which is presented in Figure 6. The concrete lining was modelled and the rock was simulated by support conditions being applied to the concrete lining where the loads are transferred to the rock from the lining.

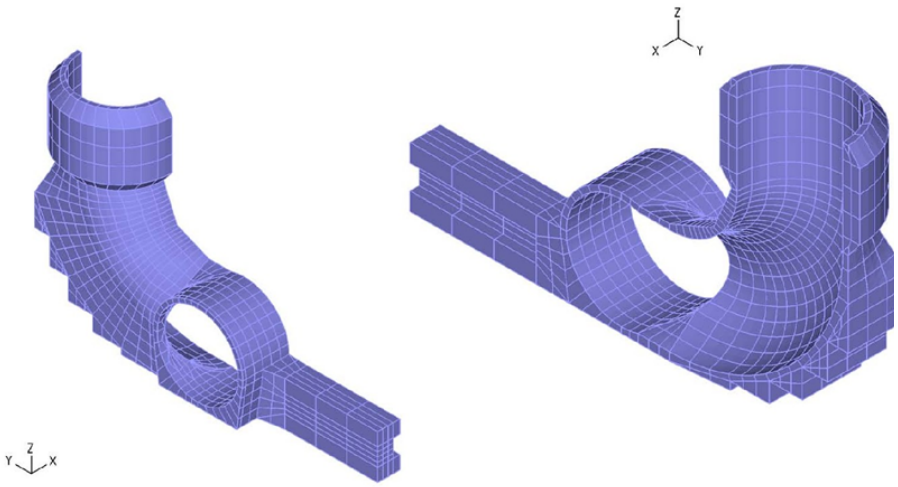

Figure 6: Finite Element Model of the Headrace Tunnel/Surge Shaft/Branch Access Tunnel Junction

The concrete lining is designed for the internal pressure, consolidation grouting pressures during construction, external water pressures when dewatered, seismic and thermal loads. For the internal water pressure load case, it was assumed that the external groundwater pressure acts on the outside of the lining at the same time as the internal pressure acts, and thus the lining only carries the net internal pressure.

The reinforced concrete design was done using the results of the finite element analysis and the reinforcing steel designed to limit the crack width in the concrete under the net internal water pressure to a maximum of 0.2 mm. Minimizing the leakage of water out of the concrete-lined tunnels was required as the Owner specified a maximum leakage from the concrete-lined waterways which was measured during commissioning.

The smaller branch access tunnel at this junction was required to be open for the passage of concrete trucks until the very end of construction. This required that the consolidation grouting of the headrace tunnel and the concrete lining of this junction had to be done before this tunnel was closed. In order to achieve these construction requirements, the concrete plug portion of the branch access tunnel was lined with reinforced concrete. Couplers were installed in the opening so that the main reinforcing steel in the hoop and longitudinal directions in the headrace tunnel lining could be re-established once this access was no longer required.

Consolidation grouting of the main headrace tunnel, surge shaft, pressure shaft and the junction of the headrace tunnel/surge shaft was completed with the access opening in place. The opening was propped during the consolidation grouting of the junction. Once all the construction plant and material had been removed from the lower portion of the headrace tunnel through this opening, a concrete plug was cast inside the concrete lining of this branch access tunnel. Shear keys were provided in the lining of this branch adit to key the plug into the lining and the contact between the lining and plug was grouted using the *tube-a-manchette* system.

4.4 Surge Shaft Orifice

The transient hydraulic design of the power conduit required an orifice at the base of the surge shaft to limit the upsurge and down surge in the surge shaft under maximum load rejection and load acceptance, respectively. The specified values of the headloss coefficients for the orifice are 0.00018 m/(m^3/s)^2 for the upsurge and 0.00044 m/(m^3/s)^2 for the down surge. The hydraulic shape to the orifice was determined by hydraulic model studies performed by Northwest Hydraulic Consultants Ltd. under the direction of Klohn Crippen Berger Ltd.

An additional requirement specified by the Owner was that the orifice must be constructed in such a way that it could be removed and replaced without major demolition and reconstruction work being required. This requirement precluded the construction of the orifice using conventional concrete. The solution was a steel orifice that was bolted onto flanges embedded into the surge shaft lining. As the outer diameter of the orifice was the same as the surge shaft diameter and as the construction tolerances for the concrete construction of the surge shaft were not as tight as those for the steel orifice construction, it was no possible to fabricate the orifice in one piece and lower it down the shaft into place.

The orifice was fabricated in three pieces and each piece lowered separately into the shaft and bolted to the embedded flanges. Once these individual segments of the orifice were in place, closure plates between these segments were then welded in place to complete the orifice. A manhole is provided into the orifice to complete the welding, bolting and corrosion protection. The finite element model used for the design of the orifice is presented in Figure 7, showing the 60° segment of the orifice that was analysed.

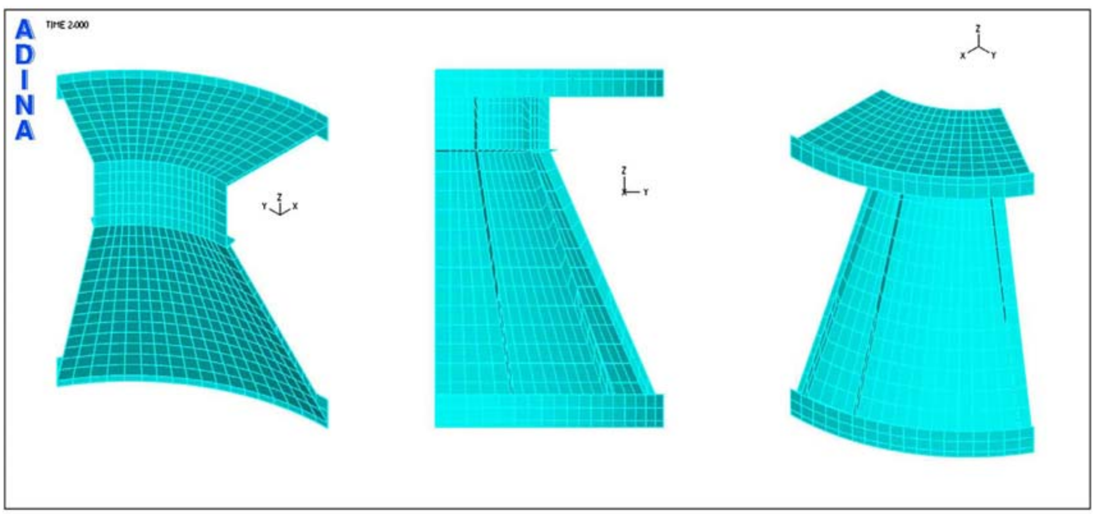

Figure 7: Finite Element Model of the 60° Segment of the Surge Shaft Orifice

4.5 High Pressure Tunnel Grout Cut-off and Drainage Curtain

The design of the steel-lined high pressure tunnel limited the maximum external water pressure acting on the outside of the steel-lining when the power conduits were dewatered to 0.3 MPa. This was achieved by providing a drainage tunnel running parallel to the high pressure tunnel and a fan of drain holes drilled into the rock from the drainage tunnel towards the high pressure tunnel. These drain holes were drilled above and below the high pressure tunnel to reduce the external groundwater pressure at the high pressure tunnel to a value below the maximum design external pressure.

Although the concrete-lined portion of the high pressure tunnel located upstream of the steel-lined portion as well as the pressure shaft are reinforced to limit crack widths and thus minimize leakage out of the tunnel, these sections of the tunnel are still a potential source of high pressure water that could be injected into the surrounding rock at reservoir pressure. In the absence of a grout cut-off at the upstream end of the steel-lined portion of the high pressure tunnel, this high pressure groundwater could migrate through the rock to the back of the steel-lining and overload the lining when dewatered. This could result in the buckling failure of the lining.

To prevent migration of the potential high pressure groundwater upstream of the steel-lined portion of the high pressure tunnel, a grout cut-off was formed in the rock surrounding the first 33 m of the steel-lined high pressure tunnel as shown in Figure 8. Immediately downstream of this grout cut-off zone, a drainage curtain was formed by drilling a fan of drain holes from a chamber in the high pressure tunnel. The purpose of the drainage curtain is to intersect any high pressure groundwater that manages to get through the grout cut-off before it can act on the outside of the steel lining downstream of this drainage chamber.

The grout cut-off consists of 16 rings of grout holes at 2 m centres drilled through the steel lining. Figure 9 presents a typical cross section through the grout cut-off zone. Void grouting to fill any voids in the crown of the tunnel that occurred during the concreting of the liner was initially done. Then each ground consolidation hole was drilled and grouted in a sequence that progressively grouted the rock further out from the tunnel. The original grouting sequence was developed for the worst case scenario for the rock permeability. During construction, it was found that most grout takes were very low and that the rock was generally tight in this area. Consequently some grouting stages were reduced or eliminated in areas of very low grout take.

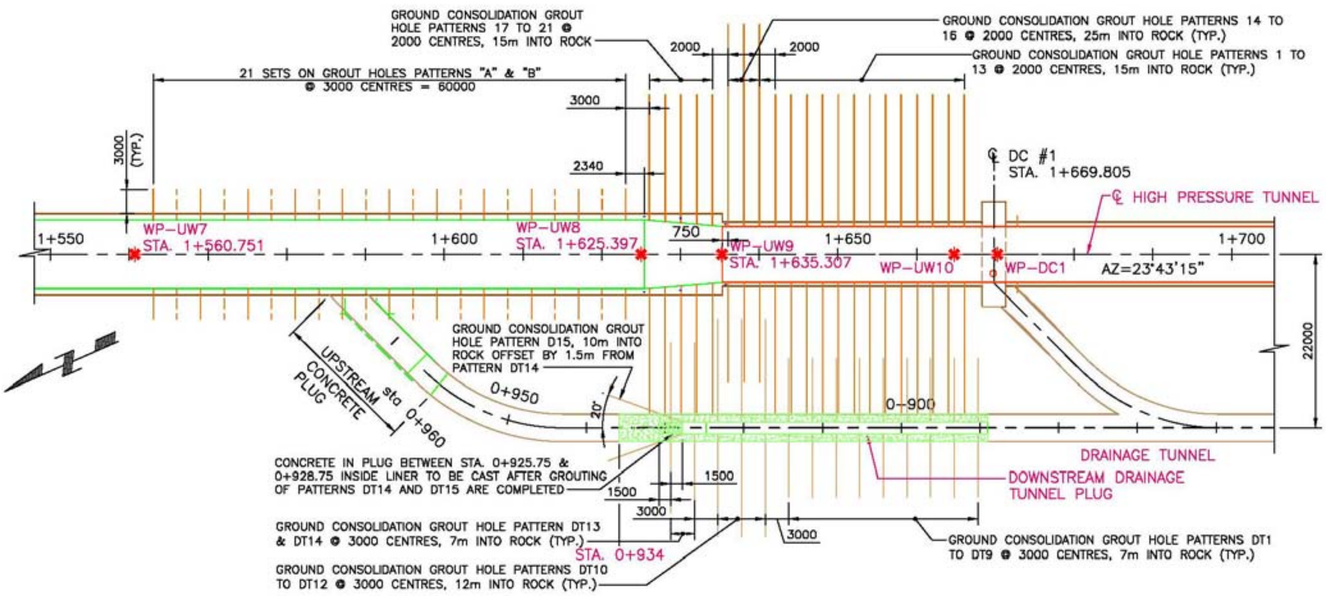

Figure 8: Plan of the Upstream End of the Steel Lining in the High Pressure Tunnel

In order to excavate the pressure shaft concurrent with the on-going construction in the high pressure tunnel, the drainage tunnel was extended to a point approximately midway between the upstream end of the steel lining and the pressure shaft. This extension was plugged at the end of construction. To match the grout cut-off around the high pressure tunnel, a similar grout cut-off was installed around the drainage tunnel where it is alongside the upstream end of the steel lining.

The concrete plug in the drainage tunnel had to correspond to the full length of the grout cut-off. The length of the plug is much longer than that required to resist the water pressure acting on the upstream face of the plug. Therefore only the first 5.2 m of the plug in the drainage tunnel was solid concrete. Behind this solid portion, the tunnel was lined with reinforced concrete to withstand the external groundwater and grouting pressures. The space inside the gallery in the concrete plug is sufficient to use a drilling machine to drill and grout the ground consolidation grout holes required for the grout cut-off.

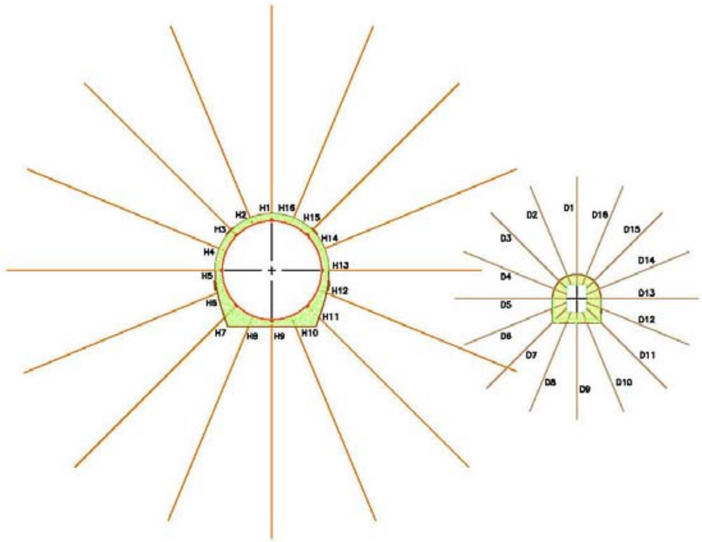

Figure 9: Cross Section through the High Pressure Tunnel and Drainage Tunnel at the Grout Cut-off

4.6 Penstock Bifurcations

The 7.15 m internal diameter steel-lined pressure tunnel bifurcates into two 5.08 m internal diameter steel-lined tunnels just upstream of the powerhouse. Each of the two 5.08 m diameter steel-lined tunnels then bifurcates into two 3.6 m internal diameter unit penstocks that convey power flow to each of the four Francis turbines. The height of the rock above these bifurcations is not sufficient for a portion of the surrounding rock to carry the internal pressure. Therefore even although these bifurcations are embedded in the concrete backfilled tunnels, they are designed to carry the full internal pressure without any composite action with the surrounding rock.

The design type/shape selected for these bifurcations utilized an internal stiffener and was based on the design and shape of the type of bifurcation that was originally patented by Escher Wyss and is presented in Figure 10. Plotting the internal design pressure vs. diameter indicates that these bifurcations are at the upper limit of the current state-of-practice for large bifurcations.

The steel chosen was high tensile quench-hardened HT80 steel manufactured in Japan. The analysis and design of these bifurcations was done using the ADINA finite element program. The finite element model used for the analysis is presented in Figure 11. The final arrangement resulted in the internal stiffener (or sickle plate) having a thickness of 170 mm and the thickest portion of the shell of the bifurcation being 75 mm.

Plates for the bifurcation were rolled and fabricated in the on-site shop and then lowered down a shaft just upstream of the powerhouse into an enlargement in the high pressure tunnel. The steel plates for the two downstream bifurcations were then welded in this enlargement in the tunnel to form the bifurcations and when the fabrication of these bifurcations was completed, they were slid down the tunnels into their final positions. The main bifurcation was then assembled in the same enlarged portion of the high pressure tunnel and welded in situ. The enlarged portion of the high pressure tunnel and the lower portion of the access shaft were then backfilled with concrete on completion.

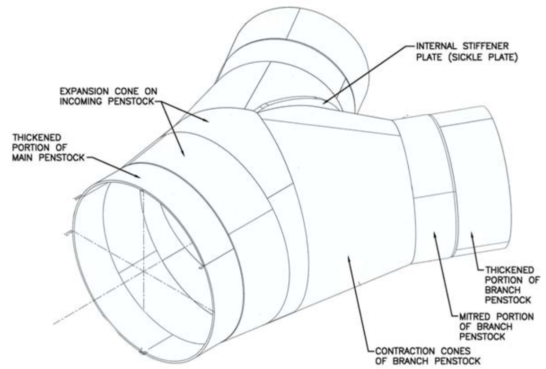

Figure 10: Typical Bifurcation Arrangement

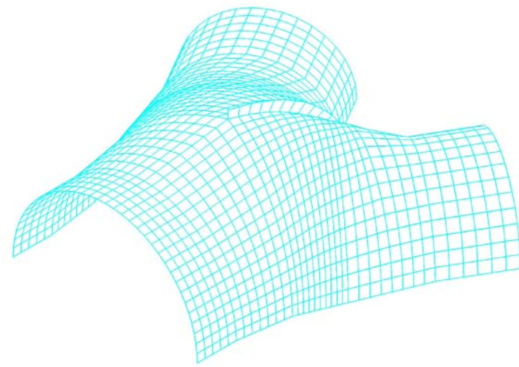

Figure 11: Finite Element Model of the Bifurcation

4.7 Pelton Branch-off

The power flow for the 2 x 35 MW Pelton units is drawn off the main 7.15 m high pressure tunnel upstream of the main bifurcation. The 2.55 m internal diameter Pelton conduit branches off from the main 7.15 m diameter steel lining on a 20 m radius curve, rotating through 68°. This curved branch-off resulted in a significant length of the 7.15 m steel lining being discontinuous with respect to the ability to carry the internal pressure in hoop tension. Due to the high internal pressure and relatively large diameter of the main steel conduit, conventional arrangements of stiffeners at this junction did not work.

Numerous solutions for the stiffeners were attempted using finite element analysis to obtain an acceptable combination of plate thickness and level of stress. The maximum thickness of plate that could be rolled to the 2.55 m diameter was 50 mm and this thickness limited the solution options. In addition, as the branch-off occurred on a bend in the 2.55 m diameter conduit, the resulting geometry of the intersection of the two steel conduits is complex and a conventional stiffener arrangement at the junction of the two conduits would have been very difficult to fabricate.

The final arrangement adopted for this junction was to provide a stiffening ring to carry the unbalanced hoop tension from the 7.15 m diameter steel lining, on the 2.55 m diameter branch pipe, some distance from the actual joint of these two pipes. This stiffener was aligned parallel with the centreline of the main 7.15 m diameter steel lining. Circumferential stiffeners are welded to the 7.15 m diameter liner and then welded to this stiffening ring. These circumferential stiffeners transfer the unbalanced forces from the hoop tension in the 7.15 m steel liner to the stiffening ring. These unbalanced forces act equally on the top and bottom of the stiffening ring. The finite element model used for the design of this branch-off is presented in Figure 12.

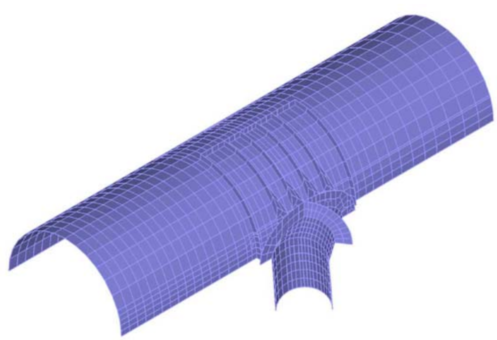

Figure 12: Finite Element Model of the Pelton Branch-off

5 CONCLUSIONS

The underground waterway of the Nam Theun 2 Hydroelectric Project was successfully watered-up and commissioned in 2008. The maximum water loss specified from the underground waterway was 50 litres/sec. During the tunnel filling test in July 2008, the initial peak water leakage from the tunnels was measured as 25 litres/sec, reducing over the period of the test and stabilizing at a measured value of 6 litres/sec that is well within the specified maximum leakage rate.

During the tunnel filling test, all the piezometers installed in boreholes surrounding the steel-lined portion of the high pressure tunnel measured groundwater pressures less than the 0.3 MPa maximum allowable design external water pressure, indicating satisfactory performance of the grout cut-off curtain at the upstream end of the steel lining and the drainage system surrounding the steel-lined high pressure tunnel.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Hoek, E., Kaiser, P.K. and Bawden, 1995, W.F., Support of Underground Excavations in Hard Rock, Balkema.

Hoek, E., C. Carranza-Torres and B. Corkum, 2002,"Hoek-Brown Failure Criterion – 2002 Edition"; accompanying the computer program ROCLAB version 1.021, Rocscience Inc., Ontario, Canada.

ASCE/EPRI Civil Engineering Guidelines for the Planning and Designing of Hydroelectric Developments, 1989.

CECT, Recommendations for the Design, Manufacture and Erection of Steel Penstocks of Welded Construction for Hydroelectric Installations (1979, Rev 1984).

Jacobsen, S., 1974, Buckling of Circular Rings and Cylindrical Tubes under External Pressure, Water Power and Dam Construction, December.

RED RIVER FLOODWAY EXPANSION PROJECT - DESIGN AND CONSTRUCTION OF THE OUTLET STRUCTURE

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ABSTRACT:

The City of Winnipeg is protected from flooding by the Red River Floodway. Originally constructed in the 1960s, the Floodway has since prevented more than an estimated $8 billion in damages. In 1997, the largest flood since the 1800s occurred, and this "flood of the century" taxed the existing Floodway to its limits. It was clear following the 1997 flood that there was a need to prepare for a larger future flood.

The Outlet Structure is located at the downstream end of the Floodway, where the diverted water re-enters into the Red River north of the City of Winnipeg. It is a large concrete drop structure used to dissipate energy in the water before it is returned to the Red River. Both physical and numerical models were set up and used to develop an efficient and cost effective design of the expanded Outlet Structure.

Construction of the expanded Outlet Structure began in the spring of 2007 and was completed in the spring of 2009. This paper discusses both the numerical and physical modelling activities, and the design and the construction of the Outlet Structure.

RÉSUMÉ:

La Ville de Winnipeg est protégée des inondations par le canal de dérivation de la rivière Rouge. Depuis sa construction dans les années 60, le canal a empêché plus de $8 milliards de dommages. En 1997, la plus grande inondation depuis les 1800s s'est produite, et cette "inondation du siècle" a presque causé le canal de dérivation d'atteindre sa limite de capacité. Il était évident en observant l'inondation de 1997 qu'il y avait un besoin de se préparer pour une inondation plus grande dans la future.

La structure de contrôle de la sortie est située aux nord de la ville de Winnipeg où l'eau est déversée du canal de dérivation dans la rivière Rouge. L'ouvrage de sortie est une structure de béton qui dissipe l'énergie de l'eau avant qu'elle soit retournée à la rivière Rouge. Des modèles physiques et numériques ont été préparés et utilisés pour développer une conception efficace et rentable afin d'augmenter la structure de contrôle de la sortie.

Les travaux sur la structure de contrôle de la sortie ont débutés aux printemps 2007 et se sont complétés aux printemps 2009. Cet article discute de l'analyse des modèles physiques et numériques, de la conception et de la construction de la structure de contrôle de la sortie.

1 INTRODUCTION

The Winnipeg Floodway Infrastructure has protected the City of Winnipeg against the devastating effects of floods on the Red River for over 40 years. However, the 1997 flood event exceeded the secure design capacity of the existing infrastructure. Following this flood, the Province of Manitoba studied a variety of options for improving Winnipeg's flood protection capacity. Expansion of the 46.7 km long Floodway channel was selected as the preferred option to provide the city with a much needed increase in its protection level (KGS Group, 1999).

The expansion has resulted in a 130% increase in design flow capacity through the Floodway Channel. The Outlet Structure is an important component of this channel, as it serves to dissipate the energy of Floodway flows prior to releasing them back into the Red River downstream of the city. The structure will require significant modification to allow it to safely pass these larger flows. Both physical and numerical models were set up and used to develop an efficient and cost effective design of the expanded Outlet Structure. This paper discusses both the numerical and physical modelling activities, and the design and construction of the Outlet Structure

2 FLOODWAY HISTORY

The Red River Valley is located in central North America and is drained by the Red River, which flows north into Lake Winnipeg. The valley is known for being extremely flat with very fertile farmland. The City of Winnipeg is located at the confluence of the Assiniboine River and the Red River in southern Manitoba. Spring flooding within the Red River valley is fairly common and occurs when a number of conditions are met: high soil moisture content in the fall, early deep frost, substantial snowfall in the winter, and a late and rapid snowmelt in the spring. Heavy spring rains and ice jamming can also contribute to the magnitude and impacts of a flood event.

The original Floodway was constructed following the 1950 Manitoba flood, which at that time was the largest flood the City had seen. Construction of the Floodway started in 1962 and was completed in 1968. The total cost of the Floodway was approximately $63 million. The Floodway has operated more than 20 times since it was built, and it is estimated that it saved Manitoba more than $8 billion in flood damages.

In 1997, Manitoba experienced the "Flood of the Century". This flood came close to reaching the Floodway capacity and threatening the protection of Winnipeg. After the "Flood of the Century" it was clear that additional flood protection measures would be required to provide an increased level of protection to the City of Winnipeg. A series of flood protection upgrade studies were undertaken immediately following the flood, and in 2002 the Province of Manitoba selected the Floodway Expansion project as the measure to further improve the City of Winnipeg's flood protection. The Manitoba Floodway Authority was established in 2003 to oversee the design and construction of the Floodway Expansion project.

The Floodway expansion will provide a 130% increase in the design capacity of the Floodway channel, increasing Winnipeg's protection during major floods. By increasing the design capacity of the Floodway channel from 1,700 m^3/s (60,000 cfs) to 3,960 m^3/s (140,000 cfs), the level of protection from floods will increase from a 1 in 90 year to a 1 in 700 year probability of occurrence. The project will provide protection to more than 450,000 Manitobans, over 140,000 homes, over 8,000 businesses, and prevent more than $12 billion in damages to the provincial economy in the event of a 1 in 700 year flood event.

The Floodway Expansion project involves extending and raising the 45 km long West Dyke, widening the Floodway channel by 60%, modifying the Inlet Control Structure to improve reliability, replacing/modifying eight drainage structures, reconstruction of a portion of Winnipeg's aqueduct, replacing and upgrading 8 bridge crossings, modifications to utilities and services, expanding the Outlet Structure, and improving erosion protection downstream of the Floodway Outlet.
Construction began in 2005 with most of the works completed by the fall of 2009. Work on some bridges, as well as at the Inlet Structure will continue beyond the fall of 2009.

3 ORIGINAL OUTLET STRUCTURE

The original Floodway Outlet Structure consisted of a 49.4 m (162 ft) long concrete ogee overflow with a crest elevation of 222.5 m (730 ft), and a concrete horizontal apron slab extending 24.3 m (80 ft) downstream from the rollway. The side walls adjacent to the apron slab sloped downward in the direction of flow to allow the return flow to re-circulate and assist energy dissipation. The structure was originally designed for a flow of 1,700 m^3/s (60,000 cfs) but was expected by the original designers to be capable of successfully passing 2,832 m^3/s (100,000 cfs) under emergency conditions. The structure was equipped with two 1,070 mm (42 inch) diameter conduits that were capable of passing 3.4 m^3/s (120 cfs) from the low flow channel without overtopping the ogee crest.

The concrete ogee overflow portion of the existing Floodway Outlet Structure was designed as a gravity structure. The stability of the concrete apron slab was achieved through passive rock anchors as well as half round pressure relief drains that extended through the slab at the construction joints. The retaining walls were designed as cantileveral retaining walls to resist the applied soil and hydrostatic loads. The Floodway Outlet Structure was founded on bedrock, and bedrock formed the base of the channel downstream of the structure. East and west embankments connected the structure to the channel. The side slopes of the downstream channel had rip rap erosion protection from the base of the channel to approximately el. 225.0 m (738.0 ft).

The original Floodway Outlet Structure was able to pass the 1997 flood with some temporary topping up of the east and west embankments. The structure was able to pass approximately 1,870 m^3/s (66,000 cfs) during the flood (the reported flow varied between 65,000 and 70,000 cfs for the 1997 flood). Erosion of the channel may have occurred after passage of the flood, as a series of sand/gravel bars were observed downstream near the confluence of the Red River.

Flow conditions downstream of the structure were very turbulent, and exhibited large waves and high velocities. This indicated that a considerable amount of energy remained in the flow after passing through the structure. The waves and currents created a concern for erosion of the west bank of the Red River, and that concern needed to be addressed as a part of the expansion design studies.

4 DESIGN STRATEGY

The design for the modified structure was accomplished through a combination of empirical analysis, numerical modelling, and physical modelling. The design process involved three very distinct steps. In the conceptual design phase, or Project Description and Environmental Assessment Phase 1 (PDEA1), empirical design charts were utilized to provide an initial design for the expanded structure. In the preliminary design phase (PDEA2), the initial concepts were refined using both numerical analysis and a physical model study. In the final design phase, the numerical model was again used to help further refine the hydraulic design of the structure, leading to a more cost effective design.

5 CONCEPTUAL DESIGN (PDEA1)

The PDEA1 design began with development of general design criteria or a basis of design for the expanded Outlet Structure. Major design requirements for the project included the following:

- The new Floodway Outlet Structure was to consist of the following features:
  - Ogee crest with sloping upstream face
  - Concrete apron slab
  - Vertical side-walls that were to be provided over the full length of the apron slab. These walls would help to eliminate recirculation currents which formed in the original structure, which was equipped with sloping side walls.
  - Appurtenances could be provided as required to enhance energy dissipation within the basin.
- Provision of low level water passages to permit the release of low flows in the Floodway Channel without incurring significant upstream ponding. The capacity of these low level water passages was not to be less than the original structure capacity.
- The design discharge for the Floodway Outlet Structure was to be the 1 in 700 year flow, estimated to be 3,960 m^3/s (140,000 cfs). The passage of this design flow was to be accomplished with an energy level of el. 231.65 m upstream of the structure.
- The unit discharge released by the Floodway Outlet Structure was not to be greater than that of the existing structure during the 1997 flood unless energy dissipating appurtenances were used.
- The discharge channel leading from the Floodway Outlet Structure to the Red River was to be oriented and configured such that flow velocities along the adjacent west bank of the Red River would not exceed the magnitude that would occur under existing conditions.
- The average velocity in the Floodway Outlet Channel immediately below the structure was not to exceed 5 m/s over the bedrock for the design flood condition. This restriction was included to protect the Limestone bedrock in the downstream channel.

Once the general design guidelines had been established, work began on the engineering design of the structure. This design was initially undertaken based on the use of standard empirical nomographs and charts provided by the USACE for these types of low head structures.

6 PRELIMINARY DESIGN (PDEA2)

The PDEA2 design built off of the earlier work conducted during the PDEA1 study phase. The PDEA2 preliminary design process involved seven distinct work parcels that were to be completed simultaneously over an approximately 7 month time frame. The design of the expanded Outlet Structure formed one of these work parcels. Other parcels included the design of the Floodway Channel, the West Dyke, modifications to the Inlet Control Structure, and design of the Floodway bridge crossings.

The continuous interaction between parcel consultants was required in order to successfully complete the work within the given time frame. The preliminary design at the PDEA2 stage involved an iterative approach, such that all seven project parcels proceeded simultaneously. During Iteration 1 the initial designs were developed based on the PDEA1 information. At the end of Stage 1, the design progressed and issues were resolved, more and more of the assumptions were replaced with factual information. This information was exchanged between all work parcels, and the designs were then revisited and updated during Iteration 2. Three iterations were conducted in total, with the preliminary design established at the end of Iteration 3.

During the first iteration of design, the proposed configuration for the expanded Outlet Structure was refined using a sophisticated numerical model. The results of the first iteration were then used to plan a physical model study, which was conducted during the second iteration of design. The purpose of the physical model was to verify the design concept that was established using the numerical model.

6.1 Numerical Model

With the advancements in computing power made since the 1980s, Computational Fluid Dynamics (CFD) analysis has emerged as a powerful alternative design tool. CFD analysis involves the solution of the governing equations for fluid flow at thousands of discrete points on a computational grid, giving the analyst a full three dimensional representation of the fluid flow domain. For these design studies, the commercially developed model "FLOW-3D" was utilized to provide advanced design support.

The FLOW-3D model, developed by Flow Science Incorporated of Los Alamos, New Mexico, USA was selected for use in this study. FLOW-3D is a well-tested, reliable CFD software product developed and supported by Flow Science, Inc. It is designed to assist in investigation of the dynamic behaviour of liquids and gases in a very broad range of applications. FLOW -3D has been designed for the treatment of time-dependent (transient) problems in one, two and three dimensions, and is based on a finite difference solution of the complete Navier-Stokes equations. Because the program is based on the fundamental laws of mass, momentum, and energy conservation it can be applied to almost any type of flow process. One of the major strengths of the FLOW-3D program for hydraulic analysis is its ability to accurately model problems involving free surface flows. It is robust, handling transitions between sub-critical and super-critical flow within a single model set up. These capabilities make the model well suited for simulating the varied and complex flow conditions, which will occur at the Floodway Outlet Structure during operation. This powerful tool was applied during all stages of the design of the Outlet Structure and Channel to provide timely insight into critical design issues related to the hydraulic performance of the structure, and downstream erosion.

Two basic models representing "numerical flumes" were used to assist in the design of the Outlet Structure and Channel, each varying in detail and size. The first model concentrated on providing a "near field" analysis of the Floodway Outlet Structure, and utilized a very fine mesh to capture the specific flow characteristics of the various Floodway Outlet Structure alternatives analysed. The second model involved the simulation of a much greater area, including portions of the reaches downstream and upstream of the Floodway Outlet Channel confluence with the Red River. This model, dubbed a "far field" model, utilized a coarser mesh and was used to evaluate erosion potential along the west riverbank, both with the existing Floodway and an expanded Floodway.

6.1.1 Outlet Structure Model (Near Field Model)

Numerical analyses were initially undertaken to evaluate the performance of a number of possible design alternatives for the Outlet Structure. The developed models required a relatively fine mesh to ensure all hydraulic processes were being simulated to an appropriate level of detail.

Prior to utilizing the CFD model in this important design role, it was necessary to calibrate and verify the models to ensure they were adequately simulating the complex hydraulic phenomena associated with structure operation. For the near field model used to assist in the design of the Outlet Structure, two calibration/validation runs were undertaken. In the first, the model was setup to replicate operation of the existing structure during the passage of the 1997 flood. Numerical model parameters were adjusted as necessary during this exercise so as to best replicate observed flows and patterns during the 1997 flood. To facilitate the calibration, the model geometry was first set up based on construction and excavation drawings for the existing structure. Appropriate roughness values were selected for the model based on earlier analyses of the channels, and an appropriate turbulence model was selected. The model was configured to pass the peak flow of 1,870 m^3/s (66,000 cfs) experienced during the passage of the 1997 flood.

Once run, the results of the model were compared with available anecdotal observations of hydraulic performance taken in 1997. The comparison was quite favourable, as described below:

- Flow conditions over the ogee crest, and the general location of the hydraulic jump were very similar to that observed in available photographs.
- Water levels upstream of the structure compared very well to those determined from existing rating curves. Simulation results were within 0.1 m (0.3 ft) of anticipated values.
- During the operation of the structure in 1997, large waves were generated by the hydraulic jump downstream of the structure. These waves were estimated to be some 2 m (6.5 ft) in amplitude. Waves generated by the hydraulic jump within FLOW-3D were similar in size and frequency to those observed in 1997.
- Numerical model study results indicated the presence of a high velocity jet that formed along the bed downstream of the structure, as shown in Figure 1. This jet exposed downstream sections of the basin and rock channel to velocities of between 8 to 10 m/s (26 to 33 ft/s). The jet was undulating, periodically detaching and reattaching to the channel bottom. This pulsating action could potentially cause erosion in the downstream channel, and also is the primary cause of the large waves discussed in the previous point.

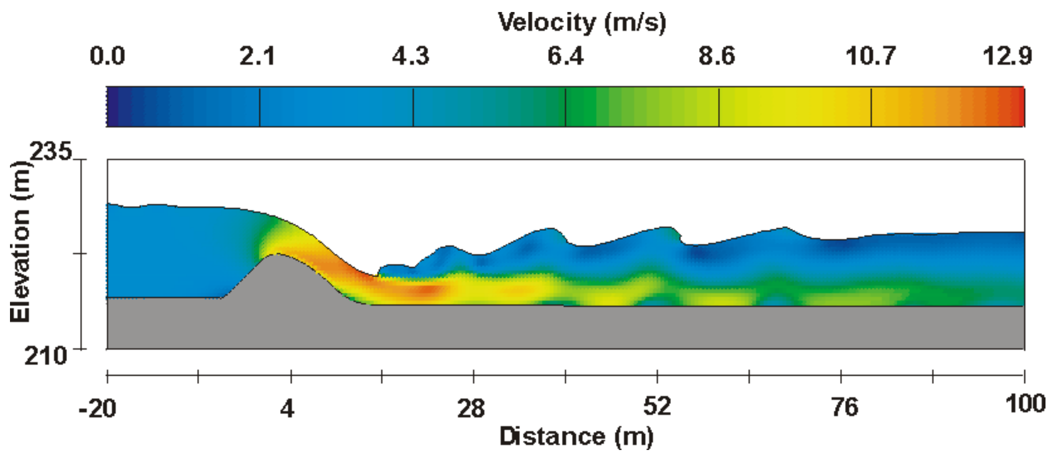

Figure 1: Velocity profile through the original Outlet Structure for the 1997 flood simulation

Following this calibration exercise, the model was applied in a verification mode to see if it would show continued good response without adjustment of the calibration parameters.

Section 6.2 below describes how a physical model study was ultimately performed at the University of Manitoba Hydraulic Research and Testing Facility (HRTF). The objective of the study was to review the performance of a near-to-final design concept for the Outlet Structure under a range of flow conditions. The data collected during this series of tests also provided an excellent source of data with which to verify the performance of the numerical model.

To facilitate the verification test, the numerical model was set up to replicate one of the test cases of the physical modelling program. Initially, the selected design alternative for the Outlet Structure – a 100 m (328 ft) wide structure, complete with energy dissipation appurtenances - was setup within the “numerical flume”. Care was taken in the setup of this model to exactly replicate all structural details associated with the 100 m wide Outlet Structure design. Following this, the model was run to simulate the same test case in the physical model study, which involved passage of the project design flow, the 1:700 year flood. The results from the numerical model were then compared against those of the physical model. This included a comparison of water levels, velocity patterns and magnitudes throughout the reach.

The comparison obtained between the two test results was excellent, both in terms of velocity measurements and water surface profiles. Figure 2, for example, illustrates a plan view of velocity magnitudes and patterns at a plane cut near the surface of the numerical model. Superimposed on the plot are spot measurements of velocities taken in the physical model for this test. As shown, the comparison between computed and observed velocities is very good. Figure 3 compares the water surface profile computed within the numerical model to measurements taken in the physical model. Again, the match between the two is very good. The close match verifies the numerical model's ability to replicate real world hydraulic phenomena.

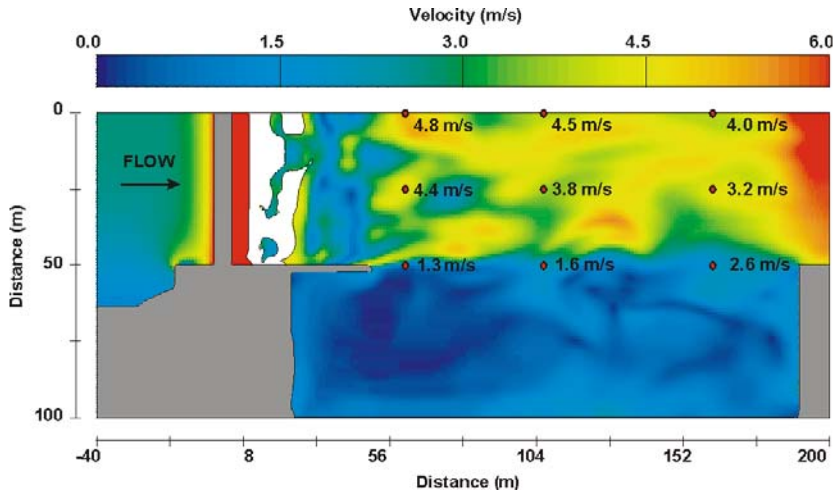

Figure 2: Plan view of velocity magnitudes and patterns downstream of Outlet Structure

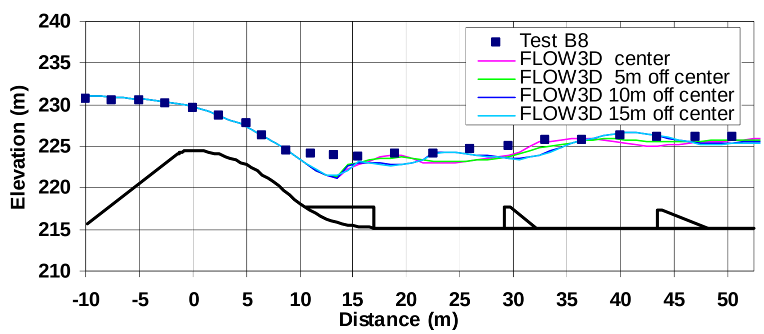

Figure 3: Comparison of numerical and physical model water surface profiles

Following its set up and calibration, the numerical model was used to test a variety of Outlet Structure configurations. The model was used to evaluate the overall effect of variations in the proposed structure width, height, approach channel depth, sidewall configuration, and apron design (i.e. with or without basin appurtenances).

After design Iteration 1, the preferred design concept for the Floodway Outlet Structure was identified as a 100 m (328.1 ft) wide structure, complete with energy dissipation appurtenances. This concept was selected early in the second iteration of design in order to provide sufficient time for the construction and testing of the physical model. Numerical analyses showed this option to perform the best, and the physical model also performed well, verifying the selected design.

6.1.2 Outlet Channel Model (Far Field Model)

The Red River Floodway discharges into the Red River downstream of the St. Andrews Lock and Dam. The purpose of the Floodway Outlet Channel is to convey Floodway Channel flows back to the Red River after they have passed over the Floodway Outlet Structure. Immediately downstream of the structure, the Floodway Outlet Channel invert is at el. 215.65 m (707.5 ft), rising to elevation 217.63 m (714 ft) before entering the Red River. With the proposed expansion, the Floodway Outlet Channel required a larger conveyance area to carry the higher design discharge and to ensure that the velocities within the channel as they enter the Red River are not excessively high. Through the proper hydraulic design of the channel geometry, undesirable flow velocities and conditions can be minimized. The FLOW-3D model was set up and used to ensure the selected design for this channel would perform satisfactorily.

The developed numerical model for the "far field" runs included a reach of the Red River and the existing/expanded Floodway Outlet Channel. This model covers a significantly greater area than for the Outlet Structure model described above. It extends from just downstream of the St. Andrews Lock and Dam to a point approximately 3.5 km (2.2 miles) downstream of the Floodway confluence. Care was taken in selecting the upstream and downstream boundaries of the model to ensure that flow patterns at the confluence were accurately simulated.

The geometry for the model was based on data gathered from a number of sources and drawings, including 1951 hydrographic surveys, Lidar surveys and Floodway construction drawings. A comparison of aerial photographs taken over the past 50 years also showed little change in the overall channel width, indicating the channel to be relatively stable. The upstream boundaries were set as velocity boundaries at the St. Andrews Lock and Dam and at the Floodway Outlet Structure. The downstream boundary for the model was set as an elevation boundary based on the results of previous HEC-RAS analyses. Flow conditions through the reach were then simulated based on the prescribed boundaries and bathymetric data.

Following its setup, the numerical model was first configured to replicate the natural river under pre-Floodway conditions. The pre-Floodway condition was selected for initial calibration because, under these conditions, flows are primarily one dimensional in nature. This allowed for a direct comparison with the results of earlier HEC-RAS water profiles. To verify the model's ability to reproduce existing Floodway conditions, the model was set up to simulate operation of the existing Floodway during passage of the 1997 flood. This case closely resembles the design condition for the Floodway - a condition that was tested using a physical model in 1962. The results of the FLOW-3D simulation were then compared against general observations taken during the 1997 flood (primarily photographs) and quantitative data taken during the original model study tests to verify the performance of the model. The match between model results and observed data was good in both cases. Overall flow patterns, as observed in photographs taken in 1997, matched well with FLOW-3D simulations. As well, the overall velocity profiles computed within the model were comparable with limited data obtained in the 1962 physical model tests, giving confidence in the model's ability to simulate hydraulic conditions in this relatively complex reach.

Following its set up and calibration, the numerical model was used to test a variety of Outlet Channel designs. These designs varied in shape and depth. In the PDEA1 studies, both flared and unflared channel shapes were tested. However, the results indicated that there was little difference between the downstream velocity patterns associated with the two. Therefore in the PDEA2 studies, the numerical model study runs concentrated on the unflared option since it was considerably cheaper to construct.

The model results were also used to assess the length and size of riprap required to protect the west bank from erosion during large flood events. As indicated earlier, during the 1997 flood this bank was subject to erosive forces arising from the aggressive wave action caused by the undulating jet, and high river channel velocities. The placement of rip rap protection along this bank will prevent this from occurring in the future.

6.2 Physical Model

The numerical analysis helped to identify a cost effective design concept (utilizing baffle blocks as primary energy dissipators) for the Floodway Outlet Structure design. Because of the importance of this structure to the overall Floodway infrastructure, a physical model of the concept was also constructed and tested. The design concept for testing was selected early during Iteration 2 so that sufficient time would be available for this testing. At that time, the numerical analyses showed that the 100 m option functioned well up to the design event. It was recognized that the adoption of a narrower structure may also be possible but the acceptability of such a design could not be confirmed at the time construction of the model was initiated. The primary objective of the testing program was to verify the performance of the selected alternative and also to verify the numerical modelling results. This would increase overall confidence in the use of numerical modelling techniques to further enhance the hydraulic performance of the design. Although numerical modelling techniques and tools have advanced significantly, they still require careful model development and selection of design parameters in order to achieve representative results. The ability to calibrate the numerical model against the physical model results provides increased confidence in the numerical model, and allows the designer to use this tool effectively to advance the refinement of the design, and take advantage of potential cost savings options. The model study was undertaken at the Hydraulic Research and Testing Facility (HRTF) of the University of Manitoba.

6.2.1 Model Description

The hydraulic model included a 100 m (328 ft) length of the Floodway Channel, the proposed Floodway Outlet Structure (with all hydraulic features included) and a 200 m (656 ft) length of the Floodway Outlet Channel. Since the hydraulic conditions which influence the operation of this structure are determined predominantly by gravitational and inertial forces, modelling of the Floodway Outlet Structure and Channel followed Froude scaling laws. An undistorted model scale of 1:50 was selected to ensure that the complex hydraulic conditions for the structure would be completely simulated. Figure 4 illustrates the constructed model.

Water was pumped from a storage sump thru a V-notched measuring weir before entering the model through a headtank that uniformly distributed the flow across the upstream end of the model. Tailwater levels were set with a tilting gate at the downstream end of the model. The tailwater stage discharge curve utilized reflects the total discharge of the Red River downstream of the confluence with the Floodway Outlet Channel. This discharge includes the outflow from the Floodway, the Red River flow that is passed thru the City of Winnipeg including the contribution from the Assiniboine River.

(a)

(b)

Figure 4: Physical Model (a) Dry (b) Passing design flow

6.2.2 Test Program and Results

A number of test series were developed to assess the operation of the proposed design. The primary objective of this test series was to assess/confirm the general hydraulic performance of the selected Floodway Outlet Structure design. The test series would represent the operation of the Floodway Outlet Structure over a range of flows up to the design discharge. For a given Floodway discharge, the tailwater conditions can vary depending on the mode of operation and the Assiniboine River contribution. The test program was developed to assess how the Floodway Outlet Structure would operate over a range of tailwater conditions.

Each of the runs was well documented through a series of still photographs, videotaping, and the gathering of instrumentation data.

Generally the test results showed that the 100 m wide option would perform quite adequately. The hydraulic jump within the basin was stable and any wave action generated by the jump downstream appeared to dissipate before reaching the end of the model. The velocities measured in the model along the channel bed were likely less than that required to induce scouring of the bedrock. The downstream return currents were slow, lessening concerns of bank erosion. The one exception to the performance was the presence of a relatively poor flow condition around each of the upstream abutment walls. The corner radii appeared to be too small for the 90 degree change in direction that flow makes at this location. This resulted in considerable flow separation along the abutment wall. This flow separation resulted in an acceleration of the water to fill the resulting void. This resulted in a very high velocity along the upstream channel bed that was capable of lifting 1.0 m (3.3 ft) diameter riprap and depositing it in the stilling basin. It was noted that the shape of the abutments in this area would need to be modified in order to prevent the lifting of the riprap and the flow separation. It is important to note that although high velocity zones existed around the abutments in the numerical model, the movement of stone could not be simulated.

The final overall comparison between the numerical model and physical model was very good, as discussed in Section 6.1.1. This close match allowed the design team to more confidently analyze more aggressive alternative designs using the developed numerical model. This additional analysis was undertaken during the final design phase, described below.

7 FINAL DESIGN

The preferred design concept identified at the end of the PDEA2 studies was a 100 m wide rollway, complete with a USBR low Froude number stilling basin which incorporates chute blocks, baffle blocks, and a dentated end sill. Numerical analyses showed this option to perform the best, which was further verified by the physical model. However, additional numerical and cost analyses undertaken after the PDEA2 design cycle indicated further economies in structure design could be possible. These economies were realized by building a smaller structure, and accepting that the reduced hydraulic performance of the structure may lead to a greater risk of channel degradation and long term maintenance expenses. For example, by further narrowing the structure, cost savings of approximately $400,000/m width reduction were possible. After the Iteration 3 design cycle, the design team was given the opportunity to more rigorously test the performance of the numerical model against actual data collected during the physical model tests. The very good match obtained provided additional confidence that the numerical model could be used for this advanced design support.

With this gain in confidence, it was possible to use the numerical model to assess more aggressive Floodway Outlet Structure designs, which may result in decreased energy dissipation and therefore increased flow velocities along the downstream bedrock channel, but offer significant cost savings. Additional runs were commissioned during the final design phase that ultimately allowed:

- The final structure width to be reduced by 10% to 90 m
- Pptimization of the curvature of the structure abutment walls to reduce localized flow velocities

The design of the Floodway Outlet structure was finalized based on the results of both the physical and numerical model studies described above. The final structure configuration, shown in Figure 5, includes the following features:

- 90 m wide rollway with a crest elevation of 223.85 m (734.4 ft),
- USBR low Froude number stilling basin design, complete with chute blocks on the downstream side of rollway, baffle blocks on the apron slab and a dentated end sill at the end of the apron slab. Use of these appurtenances allowed the structure to be shortened significantly.
- Full height vertical sidewalls to eliminate any recirculation over the sidewalls
- Revised abutment wall design based on a radius of curvature equal to 1.5 times the design head
- The concrete volume in the apron slab was minimized by the use of post-tensioned anchors.
- The baffle blocks are anchored to bedrock to resist hydrodynamic and impact forces
- The new training walls are sized and designed based on stability and structural requirements of the design criteria.
- The embankments have been designed with a semi-pervious core, filter zones, rockfill shell, and riprap cover. The semi-pervious core consists of a silty sand.
- The Floodway Outlet Channel slopes have been designed to meet the design criteria, and to minimize the amount of excavation.

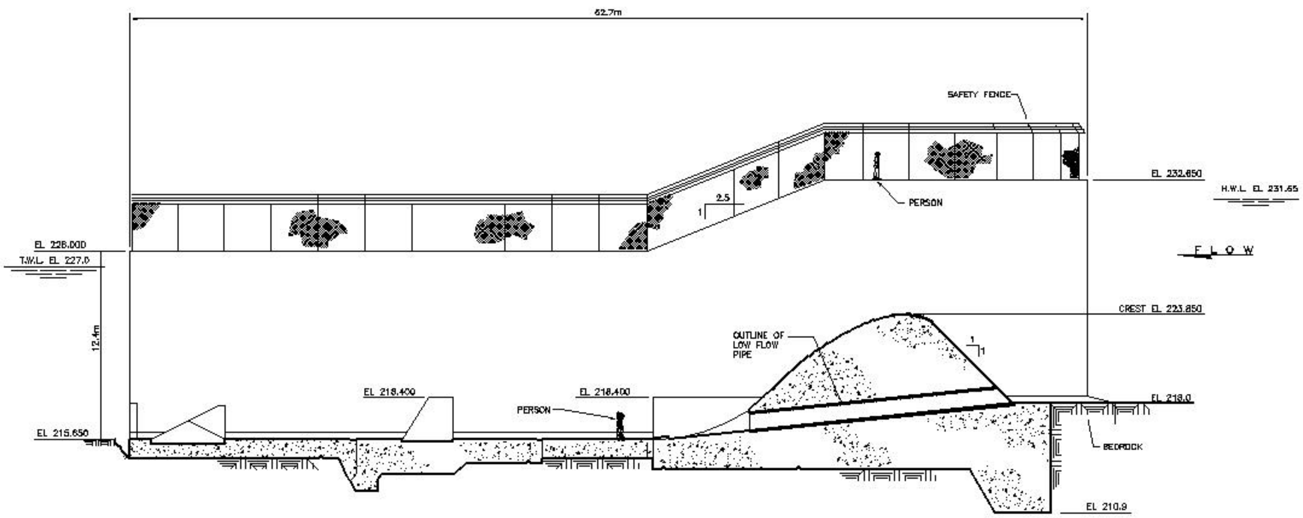

Figure 5: Final Outlet Structure

8 CONSTRUCTION

Construction of the Outlet Structure began in August of 2007, and was essentially completed in early April of 2009. Some cleanup and minor work was required in the summer of 2009.

An important aspect of construction was the need to maintain flows within the Floodway throughout the year, and in particular, the ability to pass a significant flood event in the spring. As construction involved building on the existing structure, a detailed water diversion scheme and construction schedule was required in order to meet these requirements. The tender documents provided a water diversion and construction sequence plan, however, it also allowed the contractor the flexibility to develop their own plan provided these requirements were met. The contractor, PCL Constructors Inc., developed their own water diversion and construction sequence plan.

The following outlines the approximate material quantities associated with the construction of the Outlet Structure:

Concrete	22,500 m^3
Earthwork excavation	250,000 m^3
Bedrock excavation	50,000 m^3
Rock Anchors	2,650 linear metres
Abutment fill	49,000 m^3
Erosion protection (riprap)	80,000 tonnes

9 SPRING FLOOD - 2009

The Outlet Structure was scheduled to be functionally complete by March 31, 2009, with some cleanup work to take place in the summer of 2009. This was timely, as early flood forecasts for 2009 predicted spring runoff to be well above average. However, weather conditions did not cooperate resulting in the spring melt occurring sooner than normal. Additional precipitation further exacerbated runoff conditions and increased the magnitude of the flood.

These circumstances resulted in a flood that was somewhat unique in that river ice had not had a chance to degenerate and weaken before the flood waters arrived. Significant ice jamming resulted, both within the City of Winnipeg and downstream of the Outlet Structure. Water levels rose quickly in late March, and water began naturally spilling into the Floodway Channel on March 25th. In addition, the release of a large ice jam on the Red River at St. Andrews and its subsequent re jamming immediately downstream of Lockport caused water levels downstream of the Outlet Structure to rise both significantly and suddenly on the morning of March 26th. Fortunately, the contractor was able to prevent flooding of the construction site, and completed the required works by early April.

The 2009 flood ended up being the 2nd largest flood in the last 100 years. The following table compares the 2009 flood with previous floods of significance.

Table 1: Top Six Red River Floods in Last 100 Years

Year	Approximate Natural Spring Peak Discharge*† m^3/s (cfs) (Red River at James Ave. Winnipeg)	Approximate Peak Floodway Channel Discharge† m^3/s (cfs)
1950	3,060 (108,000)	Not Applicable
1979	3,060 (108,000)	1,150 (40,500)
1996	3,060 (108,000)	1,075 (38,000)
1997	4,615 (163,000)	1,870 (66,000)
2006	2,800 (99,000)	905 (32,000)
2009	3,540 (125,000)	1,200 (42,500)

* Computed natural discharge without the use of Red River Floodway, Portage Diversion, Shellmouth Dam.
† Values from Manitoba Water Stewardship documents.

The 2009 flood provided an opportunity to evaluate the effectiveness of the new Outlet Structure's ability to dissipate energy. Figures 6 to 8 show photos of downstream flow conditions taken near the peaks of the 1997, 2006, and 2009 floods respectively. Energy dissipation associated with the original Outlet Structure resulted in significant return currents and wave action being generated. These turbulent flow conditions persisted far beyond the confines of the stilling basin and at times had been observed to impact upon the west bank of the Red River opposite the Outlet Structure. In comparison, the downstream flow conditions observed during the 2009 flood can be seen to be significantly calmer with the majority of the energy dissipation taking place within the limits of the Outlet Structure wing walls. These full height walls were also seen to effectively eliminate the generation of downstream return currents. It should be pointed out that the peak Floodway flow during the 2006 flood (Figure 7) was 10,500 cfs lower than the peak flow experienced during the 2009 flood, yet the downstream flow conditions are distinctly less chaotic, which demonstrates the effectiveness of the new structure.

Figure 6: Flow conditions – 1997

Figure 7: Flow conditions - 2006

Figure 8: Flow conditions – 2009

10 SUMMARY

The final design for the expanded Floodway Outlet Structure was accomplished through a combination of empirical analysis, numerical modelling, and physical modelling. The approach utilized in designing this important structure worked well, and the overall match obtained between the physical and numerical model results was excellent. Construction of the project was completed this past spring, and the structure operated flawlessly in passing its first runoff event, the 2009 flood, the second highest flood in the past 100 years of record.

11 ACKNOWLEDGEMENTS

The authors are grateful for the support of all who contributed to the success of this project. In particular, we would like to acknowledge the Manitoba Floodway Authority for all of their support, and their willingness to share the results of these studies.

12 REFERENCES

Acres, KGS Group and UMA. 2004. Floodway Expansion Project Definition & Environmental Assessment Preliminary Engineering Report Appendix D Outlet Local Drainage Structures and Syphons Pre-Design.

KGS Group. 1999. International Joint Commission Flood Protection for Winnipeg.

McGREGOR RESERVOIR REHABILITATION PROJECT

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ABSTRACT:

The Carseland-Bow River Headworks (CBRH) is a major multi-purpose water delivery system, located in southern Alberta, that supplies water from the Bow River to 87 000 ha of agricultural land and numerous municipalities. The system was originally constructed starting in 1909 by the Southern Alberta Land Company. Major upgrades were carried out in the 1950's by the Prairie Farm Rehabilitation Administration and between 2004 and 2009 by Alberta Transportation. The CBRH system is comprised of three major reservoirs, including the McGregor, Travers and Little Bow reservoirs, and associated infrastructure including 65 km of main canals, 6 km of interconnecting canals, and numerous water control structures.

McGregor Reservoir is the largest of the three reservoirs, and has a surface area of 5300 ha and a live storage of 365 800 dam^3 at its full supply level. The rehabilitation project included raising the McGregor north and south dams and incorporating an auxiliary spillway to accommodate the Probable Maximum Flood, replacing the irrigation outlet structure at the south dam, providing a new McGregor inlet structure and Travers inlet structure, and upgrading sections of the CBRH main canal and connecting canal.

This paper discusses the rehabilitation of the McGregor Reservoir components and a number of unique challenges that had to be overcome, particularly since the reservoir and canal operations had to be maintained throughout the five year construction period.

RÉSUMÉ:

Le système de Carseland-Bow River Headworks (CBRH) est un système d'alimentation en eau important du sud de l'Alberta qui fournit 87 000 hectares d'eau aux terres agricoles et à de nombreuses municipalités. Le système a été construit en 1909 par la Southern Alberta Land Company. Des travaux majeurs ont été entrepris dans les années 1950 par la Prairie Farm Rehabilitation Administration, et entre 2004 et 2009 par le ministère des Transports de l'Alberta (Alberta Transportation). Le CBRH comprend trois réservoirs principaux, dont les réservoirs McGregor, Travers et Little Bow ainsi que des infrastructures connexes incluant des canaux principaux de 65 km de long, des affluents de 6 km de long et de nombreuses structures de contrôle des eaux.

Le réservoir McGregor est le plus grand des réservoirs et affiche une superficie de 5 300 hectares ainsi qu'un stockage dynamique de 365 800 dam^3 à pleine capacité. Le projet de réfection comprend le rehaussement des barrages sud et nord du réservoir McGregor et l'intégration d'un déversoir auxiliaire en vue d'une crue maximale probable, le remplacement de la structure d'irrigation de sortie au barrage sud, l'installation d'une nouvelle structure d'entrée aux réservoirs McGregor et Travers, et l'amélioration du canal principal et des affluents du système CBRH.

Ce document traite de la réfection des systèmes du réservoir McGregor et des nombreux défis qui doivent être relevés, particulièrement ceux touchant l'exploitation du réservoir et du canal pendant les cinq années que durera la construction.

1. INTRODUCTION

The Carseland-Bow River Headworks (CBRH) system is comprised of a series of canals, storage reservoirs and water control structures that transmit flows diverted from the Bow River, near the Town of Carseland in Southern Alberta, and convey it into the Bow River Irrigation District (BRID) as shown on Figure 1. The system includes three major storage reservoirs, namely in sequential order from upstream to downstream, the McGregor, Travers and Little Bow Reservoirs. The CBRH system, which is owned and operated by Alberta Environment, supplies water to 87 000 ha of agricultural land and numerous municipalities. The reservoirs also provide important recreational opportunities to the region.

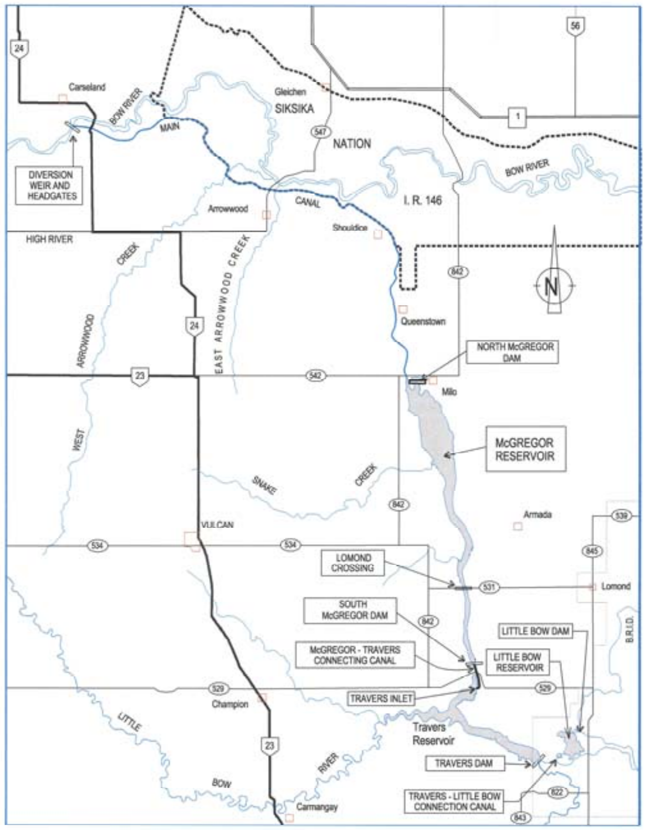

Figure 1: Plan of the CBRH system.

McGregor Reservoir is situated at the downstream end of the 65 km long CBRH main canal and is the largest of the three reservoirs. At its full supply level (FSL) of 874.38 m, McGregor Reservoir is approximately 33 km in length, has a surface area of approximately 5290 ha, and a live storage capacity of 365 800 dam^3.

The reservoir is contained between two earthfill dams referred to as the North and South McGregor Dams. The dams were originally constructed by the Southern Alberta Land Company Ltd. commencing in 1910. Significant upgrades were subsequently carried out by the Prairie Farm Rehabilitation Administration (PFRA) in the 1950's and additional minor upgrades by Alberta Environment (AENV) in the 1980's and 1990's. In addition to the dams, key components of the McGregor Reservoir system include the McGregor inlet structure, an irrigation outlet structure at the South McGregor Dam, and the Travers inlet structure. No spillway facilities were originally provided for the McGregor Reservoir.

Within the reservoir, there is one roadway crossing referred to as the Lomond Crossing or Highway 531 Crossing. The crossing is located on a natural drainage divide where runoff water flowed either north along Snake Creek (part of the McGregor valley) into the Bow River or south into the Little Bow River valley (now Travers Reservoir). As part of reservoir construction, a channel was excavated through this topographical high point and a culvert crossing was installed to allow water to flow from the north portion of the reservoir into south portion. The north portion of the reservoir contains about 90 percent of the storage and the south only 10 percent.

Starting in 2001, Alberta Transportation (formerly Alberta Infrastructure and Transportation and hereafter referred to as TRANS) began a major program to rehabilitate the entire CBRH System, including the McGregor Reservoir system components. Rehabilitation of the McGregor Reservoir system included raising the McGregor north and south dams and incorporating an auxiliary spillway to accommodate the Probable Maximum Flood, replacing the irrigation outlet structure at the south dam, providing a new McGregor inlet structure and Travers inlet structure, and upgrading sections of the CBRH main canal and connecting canal.

Rehabilitation of the McGregor Reservoir components commenced in 2004 and was completed by 2009. The rehabilitation work was carried out utilizing multiple construction contracts. The major contracts included the North Dam Inlet System (2004-2005), South Dam Outlet System (2004-2007), McGregor-Travers Connecting Canal Realignment (2007-2008) and the Travers Inlet Structure (2008-2009). One remaining contract is required to abandon and demolish the original Travers inlet spillway structure and is scheduled sometime after the 2009 irrigation season.

As part of other rehabilitation work by TRANS, Highway 531 was upgraded and the culvert crossing was replaced in 2005 to 2006 with a new bridge structure, thereby more freely permitting flows and recreational boating between the two portions of the McGregor Reservoir.

The design and construction of the rehabilitation work involved a number of unique challenges. One of the primary challenges was the requirement to maintain the operation of the reservoir and canals and the supply of water to downstream users throughout the construction duration. This paper discusses the rehabilitation work, and the design and construction challenges that had to be addressed.

2. RESERVOIR FLOOD HANDLING UPGRADES

Under normal operating conditions, the majority of inflows into McGregor Reservoir occur via the CBRH main canal which enters the reservoir on the west side of the north dam. In addition to diversion flows from the Bow River entering the reservoir, inflows can also occur due to precipitation and snowmelt runoff from the catchment areas adjacent to the main canal and from the McGregor Reservoir catchment area, including Snake Creek. The gross catchment area for the McGregor Reservoir, including the CBRH main canal, is approximately 1509 km^2.

As previously noted, the existing McGregor Reservoir does not have any spillway facilities as part of the original project and the irrigation outlet structure at the south McGregor dam represented the only means of releasing water. The south McGregor dam is considered to be a very high consequence category dam in accordance with 1999 Canadian Dam Association's (CDA) Dam Safety Guidelines whereas the north McGregor dam is a high to very high consequence dam. Consequently, the inflow design flood (IDF) for the project is the Probable Maximum Flood (PMF) with a peak inflow of 1620 m^3/s.

It was observed that the topography on the west side of the north dam was actually lower than either of the dam crests and therefore during a major flood, uncontrolled overflow would occur in this area before overtopping of the dams. Any such overflow would drain into the Snake Creek valley and ultimately flow northward to the Bow River. It was also expected that during a major flood event in the McGregor basin, similar flooding would also likely be occurring in the Little Bow River basin which drains into Travers Reservoir. Consequently, the flood handling strategy for McGregor Reservoir included storing and releasing excess flood water at the north dam without having to make any significant releases via the south dam outlet structure into Travers Reservoir. Since it was recognized that some overflow would already occur towards the north, an auxiliary spillway was incorporated into the design at the north end of the reservoir.

The auxiliary spillway consists of a 300 m wide low overflow section on the canal bank located approximately 1200 m to 1500 m upstream of the reservoir. The invert of the auxiliary spillway was set at the level of the 1:1000 year flood level at 876.50 m and the spillway is designed for a peak discharge of 224 m^3/s during the PMF. It was determined that this discharge would be less than the natural flow that would have occurred had McGregor Reservoir not be constructed.

In order to accommodate backflooding of the canal and allow flows to travel upstream from the reservoir to the auxiliary spillway, a 50 m wide bypass channel was provided around the McGregor inlet (Drop No. 4) structure and a pre-existing Highway 542 culvert crossing was replaced with a new bridge. Between the north dam and auxiliary spillway, dykes and canal banks were constructed to the PMF level, to direct any spill into Snake Creek at a safe distance away from the North Dam. Additional freeboard was provided above the PMF level for each of the North and South Dams. Due to a potential for more severe impacts on the downstream Travers Dam, the final crest elevation of the South Dam was set 300 mm higher than that of the North Dam.

3. NORTH DAM INLET SYSTEM

The North Dam is situated in the NE ¼ Sec. 35-18-22-W4M and the NW ¼ Sec. 36-18-22-W4M. The dam is oriented in a northwest-southeast direction across the McGregor valley (also referred to as the Snake Creek valley at this location) and is situated approximately 1 km east of the Drop No. 4 structure at the downstream end of the CBRH main canal. The town of Milo, Alberta is located immediately east of the North Dam site on Highway 542. At the dam site, the valley width varies from approximately 700 m at the top of slope to approximately 400 m across the valley bottom. The dam extends beyond the top of valley slope to join with a containment dyke and access road on the west side of dam. In total, the dam crest is approximately 1170 m in length between the east abutment and the intersection of the dam, containment dyke and west access road. The location of the dam relative to the other project components is shown on Figure 1. The North Dam site plan is shown on Figure 2.

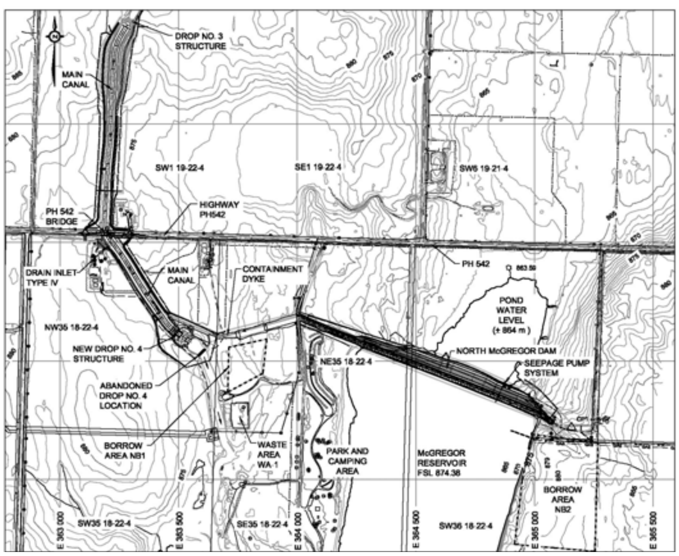

Figure 2: Plan of the North Dam site.

The rehabilitation and upgrading of the North Dam Inlet System included the following main items of work:

- Raising the North Dam embankment by 0.6 m to 1 m to a crest elevation of 878.05 m which results in a maximum dam height of 14.6 m above the original valley bottom. Raising the dam consisted of extending the upstream slopes and riprap, and widening the downstream side which also included incorporating drains which extended to the downstream toe of the dam.

- Constructing a containment dyke from the west abutment of the North Dam to the intersection of the inlet canal and Highway 542 with a maximum height of approximately 2.6 m.
- Constructing a bypass channel on the right (west) canal bank around Drop No. 4 with an approximate width of approximately 50 m and an invert elevation of 875.00 m.
- Rehabilitating and upgrading the inlet canal between Drop Nos. 3 and 4 for an increased design discharge 51.0 m^3/s.
- Constructing an auxiliary overflow spillway on the left (east) bank of the inlet canal with an approximate width of 300 m and an invert corresponding to the 1:1000 year flood (i.e. Spillway Design Flood) level at elevation 876.5 m.
- Constructing a new Drop No. 4 inlet structure with a design discharge 51.0 m^3/s.
- Demolishing and abandoning portions of the original Drop No. 4 structure.
- Installing a new seepage pump system to pump seepage flows that occur under the North Dam back into McGregor Reservoir.
- Installing two new standpipe piezometers and extending several existing pneumatic and standpipe piezometers as well as two slope inclinometers for foundation and embankment monitoring.

Since the placement of earthfill for the North Dam was only carried out on the downstream side and at the crest of the dam, there were no significant impacts on the reservoir operation and work was scheduled during the summer and fall months of 2004 and then completed in the spring of 2005. Similarly the construction of the containment dyke along the canal also had little impact on water supply operations. However, due to operation of the CBRH main canal during the irrigation season, work related to the inlet canal rehabilitation and construction of the new Drop No. 4 structure had to be scheduled during the winter months after the canal system was shut down. Although this is normally done for most canal rehabilitation work, this section of canal is somewhat different due to the backflooding that occurs when the reservoir is at or near FSL.

Since McGregor Reservoir provides the most storage within the CBRH system, it was maintained at its normal winter operating level which is only 0.8 m below FSL. Consequently, this meant that dewatering of the canal downstream of the Drop No. 3 structure was necessary to upgrade the canal and construct the new Drop No. 4 structure. An earthfill cofferdam was therefore constructed across the inlet canal downstream of the original Drop No. 4 structure and the section of canal was pumped out. The presence of silty sand soils at the canal invert level required that a wellpoint dewatering system be installed surrounding the deep excavation for the new Drop No. 4 structure which extended up to 9 m below reservoir level. Since the wellpoints are typically only effective to a depth of approximately 5 m, the wellpoints were installed at the base of canal. Heating and hoarding was required for both the dewatering system and the concrete work.

4. SOUTH DAM OUTLET SYSTEM

4.1 General

The South Dam is situated in the SW ¼ Sec. 21-15-21-W4M and is oriented in an east-west direction across the McGregor valley at a location approximately 1 km north of Highway 529. At the dam site location, the valley width varies from approximately 300 m to 400 m at the base of valley to approximately 1200 m at the top of valley slope. The valley slopes are up to 50 m above the valley bottom of which 35 m to 40 m extend above the dam crest. The dam embankment extends across the lower portion of valley and is approximately 640 m in length. The location of the dam relative to the other project components is shown on Figure 1. The South Dam site plan is shown on Figure 3 and a typical section of the new outlet structure is shown on Figure 4.

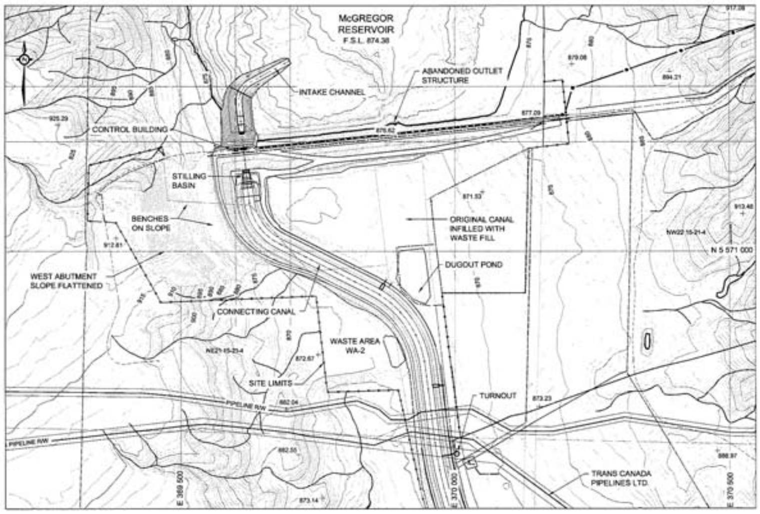

Figure 3: Plan of the South Dam site.

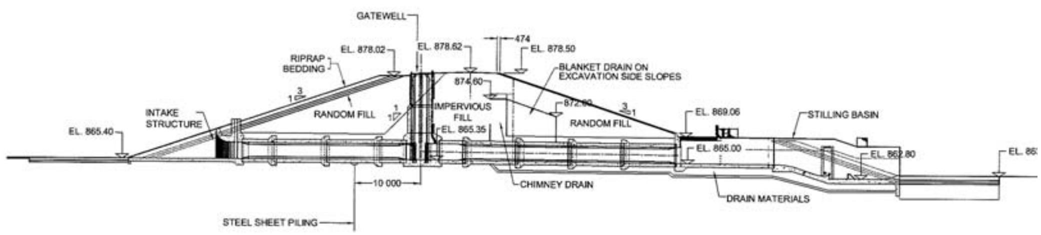

Figure 4: McGregor South Dam outlet structure.

The rehabilitation and upgrading of the South Dam Outlet System included the following items of work:

- Raising the South Dam embankment by 1.3 m to 1.5 m to a crest elevation of 878.5 m which results in a maximum dam height of 13.3 m above the original valley bottom. Similar to the North Dam, the upstream slopes and riprap were extended, the downstream side widened, and drains were incorporated at the downstream toe of the dam.
- Constructing a new outlet structure including an inlet channel, intake structure, gatewell, conduit, stilling basin, and outlet canal with an increased design discharge of 79.3 m^3/s.
- Installing a SCADA system for monitoring and operating the facility.
- Constructing a new section of outlet canal between the McGregor South Dam and the Travers Inlet and rehabilitating the remaining section.
- Installing geotechnical instruments for foundation, dam embankment and abutment slope monitoring.
- Installing structure instruments for measuring lateral earth pressures, and survey plates and pins for monitoring structure displacements.
- Demolishing portions of the original outlet structure, and abandoning others in place.

After reviewing various options for replacing the original outlet structure, the west dam abutment was identified as the most suitable location for the new structure because it offered more favourable foundation conditions. It was determined that replacement of the structure at its original structure location in the middle of valley could not be accomplished without interrupting the supply of irrigation flows. In addition the foundation conditions at the middle and east side of the valley were very soft. The proposed location on the west abutment also allowed isolation of the structure excavation with a cofferdam while the original outlet structure could continue to be operated.

4.2 Cofferdams

A work plan was developed whereby two primary cofferdams would be used to manage reservoir elevations such that the majority of storage in the reservoir could be maintained and adequate releases could be made to meet downstream water demands. One cofferdam was located at the upstream toe of the South Dam near the new outlet structure location, and the second at the Highway 531 Lomond Crossing approximately 8 km north of the South Dam.

In order to construct the earth cofferdam at the South Dam, it was proposed that the reservoir level be lowered by approximately 4 m. Since lowering of the entire reservoir would represent a significant loss of storage (approximately 185,000 dam3), the Lomond cofferdam was constructed to separate the reservoir into two parts (north and south portion) and thereby allow each portion to be operated at different water levels. As described previously, the Lomond Crossing was originally constructed at a topographical high and therefore reservoir depths at this location are typically shallow in the order of 4 m or less. A relatively deep, narrow channel that extends up to 9 m below FSL, was originally excavated through the high ground to permit flows to reach the south dam outlet structure. A benefit of constructing the Lomond cofferdam was that 90 percent of the reservoir storage is located in the north portion of reservoir. Consequently, any lowering of the south portion would only have a relatively small affect on the total available storage. Therefore the work plan evolved whereby the Lomond cofferdam would essentially be used to maintain adequate storage within the north portion of the reservoir and permit the water level in the south portion to be raised or lowered by about 4 m to facilitate construction at the South Dam. Lowering of more than 4 m was not carried out since a minimum head was required to release the required flows via the original outlet structure and there were additional concerns regarding overwintering of fish in the reservoir. Water balance modelling was performed to verify that adequate storage could be provided and downstream water demands could be met during construction.

Construction of the cofferdams and management of reservoir levels was carried out in the following manner:

- Construction of the Lomond cofferdam was carried out first and began in late August of 2004 as the reservoir level approached its seasonal low level of approximately 872 m (i.e. 2.4 m below FSL) due to high irrigation demands during July and August. After completion of the cofferdam in mid September, the reservoir level in the north portion of the reservoir was refilled up to its winter operating level of 873.6 m and the south portion lowered and operated at approximately 870 m.
- Construction of the Lomond cofferdam was undertaken in stages. The initial stage involved dozing earthfill material, derived from local clay till soils, into the reservoir to construct the submerged base of the cofferdam. Even with the seasonal low reservoir level, the depth of fill below water was up to 6 m, and therefore a very wide base with flat slopes was constructed. It was also necessary to reinforce the downstream (south) section of the cofferdam with a gravel berm to permit drawdown of the south portion of the reservoir. The section of the cofferdam above water was constructed in a normal manner with compacted lifts. Riprap was then provided from slightly below reservoir level to the top of cofferdam.
- To provide water from the north portion of the reservoir into the south portion, three gated 1.8 m diameter corrugated steel pipe (CSP) conduits were installed through the cofferdam. It was impractical to size the gated conduits to provide the maximum flow rate required during the peak irrigation demand periods in the following 2005 irrigation season. Consequently, provisions were included in the design for breaching the cofferdam prior to the start of the 2005 irrigation season and thereby equalizing the water levels in both portions of the reservoir.
- After completion of the Lomond cofferdam, the south portion of the reservoir was lowered to 870 m to permit construction of the South Dam cofferdam. The position and alignment of the cofferdam allowed almost all of it to be constructed in the dry. The reservoir side of cofferdam slopes were also protected with riprap. Dewatering wells and wellpoints were provided along the interior side of the cofferdam to improve its stability and control underseepage into the structure excavation.
- The Lomond cofferdam remained breached throughout the 2005 irrigation season and was reinstated in the fall of 2005. Following its reinstatement, the south portion of the reservoir was again lowered to 870 m to

facilitate abandonment of the original outlet structure, removal of the South Dam cofferdam and excavation of the intake channel upstream of the new outlet structure down to 865 m.

- Five additional 1.2 m diameter CSP conduits were installed in 2006 to increase the discharge capacity at the cofferdam between north and south portions of the reservoir.
- The Lomond cofferdam was again breached for operation of the reservoir in the 2006 season but was further reinstated for subsequent work to construct a new bridge at this location. The removal of the Lomond cofferdam, including the salvage of all of the gated conduits, was eventually performed by the bridge contractor.
- During periods of instream construction work, a turbidity curtain was installed to encapsulate the work area and turbidity monitoring was carried out on a regular basis.

4.3 West Abutment Slope

Construction of the new South Dam outlet structure was carried out at the base of the west abutment valley slope. In general, the base of the outlet structure excavation extended down to about 865 m at the inlet and 861 m at the outlet. Therefore the base of excavation was approximately 50 m below the crest of adjacent valley slope. Temporary excavation side slopes were 2H:1V and were initially intended to daylight on the lower section of valley slope.

The overburden deposits at the crest of valley slope are approximately 10 m thick and consist of glaciofluvial gravel interbedded with glacial clay till. The underlying bedrock is comprised of an upper zone of Horseshoe Canyon Formation and a lower zone of Bearpaw Formation both of upper Cretaceous age. The Horseshoe Canyon includes subhorizontal layers of sandstones, siltstones and silty claystones. The Bearpaw Formation is comprised of predominantly claystones and siltstones that are generally weaker than the Horseshoe Formation deposits. Although the contact between the formations is not precisely known, it is estimated to be locally at about elevation 870 m near the valley bottom.

Due to historical movements in this area, the stratigraphy near the base of west abutment valley slope is highly complex. Clay till and gravel zones were evident at the structure location and the depth to bedrock was variable. The lower slopes were also masked by colluvial soils and the location of the dam, near the base of a side coulee, further complicated the three dimensional geometry of the bedrock surface. The presence of relatively stiff foundation materials in the form of clay till and gravel soils along the base of west slope provided some advantages over the much softer alluvial and lacustrine deposits found elsewhere in the valley bottom.

The key component in the stratigraphic sequence of the valley slope was the presence of a thin bentonite layer at a depth approximately 10 m below the valley bottom. This layer was undetected in the original geotechnical investigations and became the primary trigger for subsequent stability problems that occurred during construction.

On upstream and downstream sides of the dam embankment, the structure excavation was connected to the intake channel and outlet canal excavations. During the course of excavation for the outlet structure and downstream connecting canal, significant slope movements were encountered on the overlying west valley slope adjacent to the excavations. As a result, additional geotechnical instruments were installed to monitor the slope movements, and further geological and geotechnical assessments were carried out to better identify the soil stratigraphy, the geological conditions, and the failure plane location and mechanism. Various remediation options were considered including additional excavation and flattening of the west valley slope, realigning and extending the outlet structure conduit so that the canal could be relocated further away from the valley, and concrete tangent piles. Ultimately, the adopted solution to mitigate ongoing slope movements consisted of additional slope flattening and the installation of a series of horizontal drains. Slope movements were first observed in September of 2004 during the initial excavations. This was followed by two stages of slope flattening in 2004 and 2005. After a period of monitoring, final mitigative measures for the horizontal drains were completed in January of 2007.

In total, approximately 30 slope indicators and 38 pneumatic and standpipe piezometers were installed in the dam embankment and foundation soils for monitoring the performance of the west abutment. In addition, ten horizontal drains extending up to 75 m into slope were installed to lower the water table in the slope. Slopes were flattened with several benches resulting in an average slope of about 5H:1V for the lower slope and about 2.5H:1V on the upper slope. After completion of all work, the rate of movement was reduced from an estimated rate as high as 800 mm/yr to a current rate less than 5 mm/yr. A typical section of the valley slope that shows the original and post construction condition is presented on Figure 5.

The steep portions of the upper valley slope consist of coarse gravel and layers of bedrock, including coal. These materials combined with the severe exposure conditions, particularly to high winds, made these areas difficult to reclaim this area. Several re-vegetation trial sections were carried out using various techniques including mulches, topsoil, hydroseeding, and drill seeding applications to assess the performance of each option.

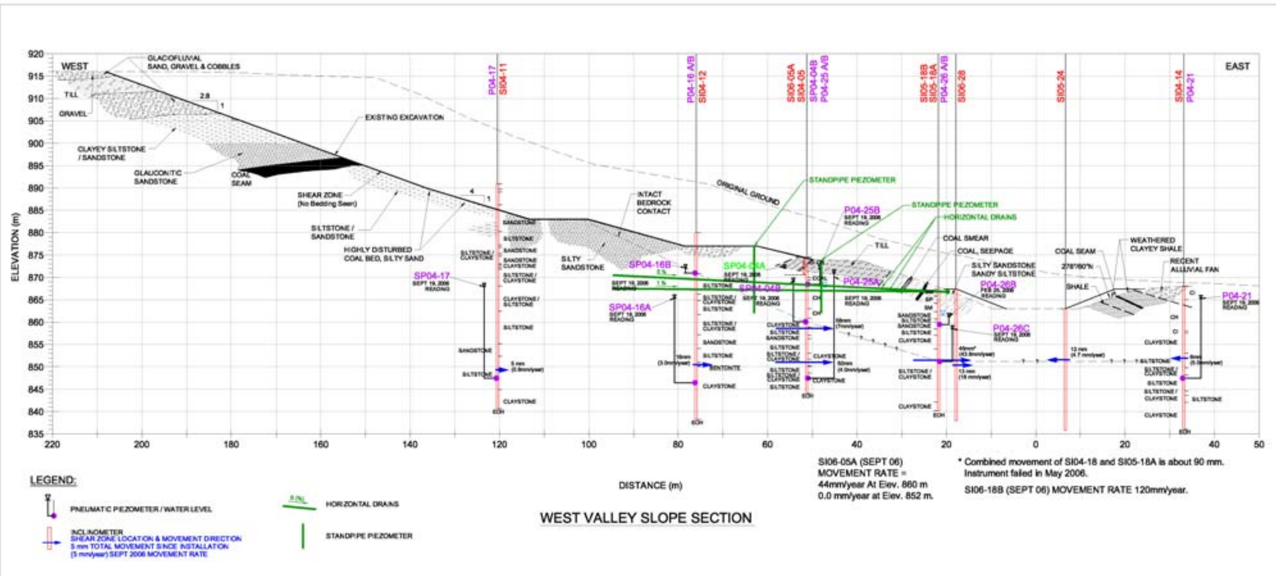

Figure 5: Typical section of valley.

The slope movements also raised concerns about the possibility for greater lateral soil pressures and translation movements to occur at the outlet structure stilling basin. As a result, a number of design modifications were incorporated at the stilling basin. These included incorporating closure sections to permit movements to occur for as long as possible during construction, placing a layer of compressible polystyrene adjacent to the basin walls to reduce the buildup of lateral pressures, designing the structure for passive lateral earth pressures, and modifying the structure expansion joints to accommodate some translation. Structure instruments consisting of survey points, joint movement plates and pressure cells were provided to monitor the performance of the structure. After completion of the work, no significant structure displacements or lateral pressure development have been observed.

4.4 Abandonment of the Original Outlet Structure

Abandonment of the original outlet structure was carried out after the new structure was completed and was functional. In-place abandonment of the original low level outlet structure at the McGregor South Dam was determined to be cost-effective and provided a low risk solution compared to other options involving demolition and removal of the entire structure. It provided significant advantages including eliminating the need to construct and remove an extensive cofferdam within the reservoir, and excavate and reconstruct a major section of the earthfill dam within a short winter construction season. It also avoided significant risks associated with potential cofferdam failure, particularly since the south dam is immediately upstream of Travers Dam, and both dams are in the Very High consequence category, and ensured that water supply from the reservoir would not be interrupted. The structure abandonment included demolishing the stilling basin, grouting the interior of the gatewell and conduits, and providing a granular drainage berm at the end of the downstream conduits. A more detailed description of the abandonment work is provided in a separate paper presented at the 2008 CDA conference (Mack et al., 2008).

5. TRAVERS INLET STRUCTURE

The Travers Inlet structure is situated in the SW ¼ Sec. 15-15-21-W4M at the downstream end of the 3 km long McGregor-Travers connecting canal that carries flow released from McGregor Reservoir into the north end of Travers Reservoir. The connecting canal is located along the east side of the valley at a level that is approximately midway between the FSL in McGregor Reservoir at 874.38 m and that of Travers Reservoir at 856.18 m. During canal operation, the head drop at the Travers Inlet structure varies from approximately 10 m with Travers Reservoir at its FSL to about 13 m when it is drawdown to normal low water level. The original Travers Inlet structure was constructed by the PFRA in the 1950's and had a design capacity of 51 m^3/s. The new structure is located approximately 180 m upstream of the original structure and its design capacity has been increased to 79.3 m^3/s. Although a variety of structure types were considered, a fixed weir chute structure with a horizontal hydraulic jump stilling basin was designed and constructed at the new site. The location of the Travers Inlet structure relative to the other project components is shown on Figure 1. The Travers Inlet site plan is shown on Figure 6.

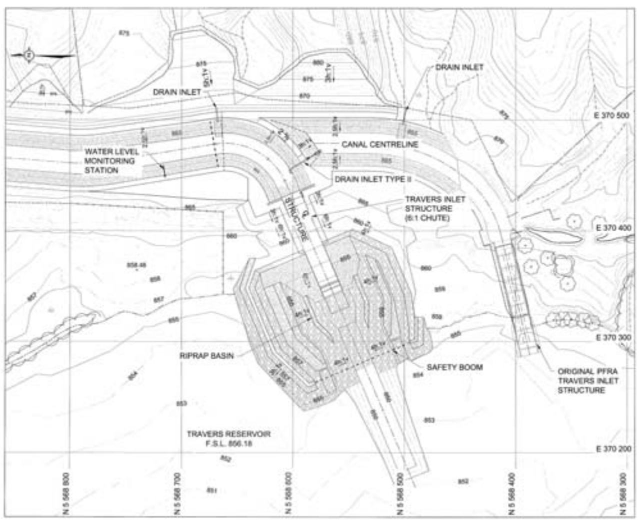

Figure 6: Travers Inlet site plan

The primary objective at the outset of the design was to identify a suitable location for the new structure using the following selection criteria. The ideal site for the new structure should have competent foundation conditions that would support the structure, provide stable slopes, facilitate groundwater seepage control, provide sufficient space to carry out construction excavations without impacting the operation of the connecting canal or reservoir, and allow ample contract time for construction of the new structure. Some of the challenges associated with each of these objectives are discussed as follows.

- Structure Location - An evaluation of potential structure locations was carried out and several sites were investigated. Several of the alternative sites considered were located near the north end of the Travers Reservoir. These sites provide ample work space between the canal and reservoir shoreline and also shortened the length of connecting canal that would have to be rehabilitated. However, the foundation soils at these locations were very soft with a high water table and therefore would likely pose significant construction and long term stability concerns. Generally, the stratigraphy along the upper valley slopes on the east side of the Travers Reservoir adjacent the connecting canal was comprised of clay till overlying Horseshoe Canyon bedrock. On the lower slopes, the competent till and bedrock strata dropped off in elevation and were overlain with a mixture of colluvial, alluvial and lacustrine deposits. These upper soils were generally very soft and increased in thickness towards the valley center. The site that was eventually selected, located near the original structure, is situated close to the base of valley slope and therefore had less of the softer soils overlying the competent clay till and bedrock. Since the contact between soft and stiff soils dipped towards the valley center, it was necessary to locate the structure as far as possible into the slope to ensure that the majority of the structure foundation was set on clay till.

- Operation of Existing Facilities – One of the major project requirements necessitated that the reservoir and existing Travers inlet and canal remain operational during the irrigation season throughout the construction period. After the most suitable structure location was selected, it was evident that the upper portion of the structure would have to overlap with the existing canal and therefore could interfere with its operation unless it was entirely constructed within the normal winter canal shut down period from October to April. The risk of not completing the work during a single winter period and thereby not being able to supply water to the BRID system was unacceptable. Therefore a plan was developed whereby work would be completed in two separate construction contracts.

- Contract Staging – The first contract was carried out during the winter of 2007 to 2008. Its main purpose was to realign the connecting canal and shift it further east into the slope by excavating and flattening the adjacent valley slope. In addition, Travers Reservoir was drawn down another 0.5 m below its normal winter level of 854.0 m, to provide access onto the shoreline where an earthfill cofferdam was constructed. Together the canal realignment and shoreline cofferdam would provide the necessary space and containment for construction of the new structure. A second contract was carried from 2008 to 2009 for construction of the new inlet structure. With the inclusion of the cofferdam, the reservoir was able to operate up to its FSL of 856.18 m during the irrigation season. At the same time, the realigned section of connecting canal was able to provide inflows to the original inlet structure. Using this contracting strategy, the construction schedule was significantly extended and instead of only having 6 winter months for construction, the contractor now had up to 11 months available of which more than half were during the spring/summer/fall period. In fact, the contractor made good use of this schedule and completed most of the work during the 7 months prior to December 2008 and was able to shut down entirely for two months during the coldest part of winter.

- Excavation Dewatering – Excavations in the stilling basin area were required to extend up to 9 m below the Travers Reservoir FSL. In addition, the excavation adjacent the shoreline was in the soft valley bottom sediments which included layers of water bearing silts and sands. Site investigations indicated that the greatest potential for seepage would occur on excavation slopes adjacent the reservoir shoreline. The position of the new structure involved excavation and exposure of a clay till foundation underlying the 6H:1V chute section. Since it was expected that seepage on the upslope side would be limited to sand pockets within the clay till, normal pumping was allowed for on this side. The remainder of excavation at the base of slope would however intercept seepage layers that could not be controlled by a normal sump pumping system. A horseshoe arrangement of wellpoints was designed and installed on three sides of the excavation adjacent the interior of the cofferdam. The wellpoints were spaced mostly at 1.5 m intervals with some end sections spaced at 3 m intervals where pre-drilling of the holes indicated less potential for seepage. Since a single row of wellpoints would not be adequate for the full depth of excavation, upper and lower rows were installed. All wellpoints were pre-drilled with an auger drill which necessitated access onto the soft soils with a relatively heavy drill. Special oil rig planks were used for access in very soft areas. Shallow trenching and localized pumping techniques were also used to drain surface layers for site access. A casing was washed into the pre-drilled hole, the wellpoint inserted and backfilled with sand and the casing retracted. A network of standpipe piezometers and slope indicators were installed surrounding the excavation and were monitored throughout the construction period.

After completion of the concrete work and placement of backfill and riprap in the basin area, the cofferdam was removed. The removal of the cofferdam and excavation of the intake channel extending up to 150 m into the reservoir were scheduled during the late fall and early spring periods when the reservoir was re-lowered to its winter level. The canal was tied into the new structure by constructing an embankment across the abandoned section of canal. In the unlikely event that the new Travers inlet structure experiences any difficulties in supplying water into Travers Reservoir it is feasible to restore service of the original structure by re-excavating the canal plug. The original Travers inlet structure will be abandoned after a suitable period of satisfactory performance of the new structure.

6. SUMMARY

The McGregor Reservoir Structures project was completed over a period of approximately 5 years from 2004 to 2009. Throughout the periods of construction, reservoir operations were able to continue with very little impact on water supply. This was due primarily to provision of temporary measures including several cofferdams combined with dewatering systems in deep excavations and close coordination with AENV for strategic operation of the reservoirs and canals.

7. ACKNOWLEDGEMENTS

The authors would like to thank Alberta Transportation for giving permission to publish and present this paper at the CDA 2009 Annual Conference, and also thank our respective companies for providing the required time and resources.

8. REFERENCES

Canadian Dam Association, 1999. “Dam Safety Guidelines”.

Klohn Crippen Berger Ltd. et al., “McGregor Reservoir Structures, Preliminary Design Report”, September 26, 2003.

Klohn Crippen Berger Ltd. et. al., 2005-2009. “McGregor Reservoir Project, Construction Report Nos. 1 to 14.”

D. Mack, T. Murray and B. Soutar, “Abandonment of the Low Level Outlet Structure at the McGregor South Dam”, Proceedings of the 2008 CDA Annual Conference.

Alberta Infrastructure and Transportation, Tender Contract No. 6814/04 South Dam Outlet System.

Alberta Infrastructure and Transportation, Tender Contract No. 6815/04 North Dam Inlet System.

Alberta Infrastructure and Transportation, Tender Contract No. 7559/07 McGregor-Travers Canal Realignment.

Alberta Infrastructure and Transportation, Tender Contract No. 7299/08 Travers Inlet Structure.

Photo 1: Completed McGregor North Dam

Photo 2: Completed Drop No. 4 Structure

Photo 3: Lomond Crossing Cofferdam Under Construction

Photo 4: Lomond Crossing Cofferdam and Control Structures in Operation

Photo 5: McGregor South Dam Stripping

Photo 6: McGregor South Dam Outlet Structure Excavation.

Photo 7: McGregor South Dam West Abutment Slide Scarp

Photo 8: South Dam Outlet Structure with Upstream Cofferdam

Photo 9: McGregor South Dam Outlet Structure

Photo 10: Completed South Dam and Outlet Structure

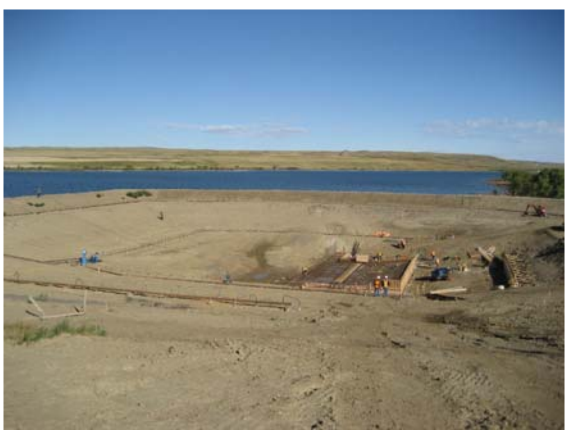

Photo 11: Wellpoint Dewatering System for Travers Inlet Excavation

Photo 12: Travers Inlet Structure Partially Completed

FITZSIMMONS RUN-OF-RIVER HYDROELECTRIC PROJECT – THREADING A NEEDLE THROUGH AN OLYMPIC VENUE

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ABSTRACT:

Fitzsimmons Creek Hydro Project is being constructed in the Resort Municipality of Whistler (RMOW) beside the Bobsleigh Track built for the 2010 Olympic Winter Games. The 7.5 MW run-of-river hydro project uses a 248-m head drop on the Fitzsimmons Creek from the intake, through a 3.4-km penstock, to the powerhouse equipped with a vertical Pelton turbine with 6 jets for a design flow of 4 m^3/s.

Space is very restricted within the valley of Fitzsimmons Creek, which has required the project structures to be shoe-horned into confined spaces within a narrow right-of-way. This has presented many geotechnical, hydraulic and construction challenges, such as:

- The intake/spillway structures are in a narrow section of the creek where there is no bedrock and where the high sediment load has to be passed over the structure, during flood conditions;
- A section of penstock is being installed in a narrow access road between the bobsleigh track (which is also known as the Whistler Sliding Centre) and the natural slope down to Fitzsimmons Creek. The slopes are very steep in this area – 35° to 44° and slope stabilization, using vertical micro-piles, has been required at the Men's Start and at Curves 4 and 7 of the bobsleigh track; and
- The powerhouse has to be tucked into the base of a steep slope away from potential outwash flooding from landslides that threaten to dam the creek.

RÉSUMÉ:

Le projet hydroélectrique de Fitzsimmons Creek est construit dans la municipalité touristique de Whistler juxtaposée à la piste de Bobsleigh nouvellement construite pour les Jeux Olympiques d'Hiver de 2010. Le projet au fil de l'eau de 7.5 MW achemine le débit de conception de 4 m3/s de la prise d'eau à travers une conduite forcée de 3.4 km jusqu'à la centrale, équipée d'une groupe Pelton vertical à 6 jets, sous une chute brute de 248 m. L'espace est restreint dans la vallée de la rivière Fitzsimmons, ce qui a forcé à construire les structures dans des endroits confinés dont des droits de passage sont également très étroits. Le projet a présenté plusieurs défis géotechniques, hydrauliques et de construction, tel que présentés ci-dessous :

- Les ouvrages amont, incluant la prise d'eau et l'évacuateur, sont construits dans une section étroite de la rivière. Étant donné que le roc n'est pas affleurant, les structures doivent être fondées sur des matériaux meubles. Une charge important de sédiments doit également passer au-dessus des structures en temps de crue.
- Une section de la conduite forcée est installée dans une route d'accès étroite entre la piste de Bobsleigh construite pour les Jeux Olympiques d'Hiver de 2010 et un talus naturel qui plonge jusqu'à la rivière Fitzsimmons. Les pentes sont très abruptes dans cette région – 35° – 44° – et des travaux de stabilisation des pentes utilisant des micros pieux ont été prévus au Départ des hommes, et aux courbes 4 et 7 de la piste ;
- La centrale est confinée à la base d'un talus abrupte la protégeant d'éboulement provenant de la rive gauche et qui menace de bloquer la rivière en conditions de crues.

1 INTRODUCTION

The Fitzsimmons Hydroelectric Project is located within the Resort Municipality of Whistler, British Columbia, approximately 100 km north of Vancouver, BC. The Fitzsimmons Creek flows in a steep valley between the Whistler and Blackcomb Mountains, where the world-famous Whistler-Blackcomb Ski Resort has established its premier ski runs and holiday resort.

Whistler is in the Coastal Mountains of BC. The intake of the Fitzsimmons Hydro Project is at an elevation of about 1,000 m. Winter minimum temperatures can drop below -20^{o} C and maximum summer temperatures can reach up to +38^{o} C.; the average annual temperature is 6.3^{o} C. The ski resort receives an average of about 10 m of snow at the summit, while at Whistler Village the normal snow fall is about 60 cm.

The intake of the hydro project is approximately 4.5 km upstream of the powerhouse on the Fitzsimmons Creek. The alignment of the penstock follows an old Forrest Service road for about the first two kilometres then is buried in a steep snow cat road between the intake and 2010 Winter Olympic Bobsleigh Track, before descending a steep slope to the powerhouse beside Fitzsimmons Creek, near the snow-making pond for the ski resort. The lower section of the penstock alignment is shown on Figure 1. The difference in elevation between the intake and the powerhouse is approximately 250 m.

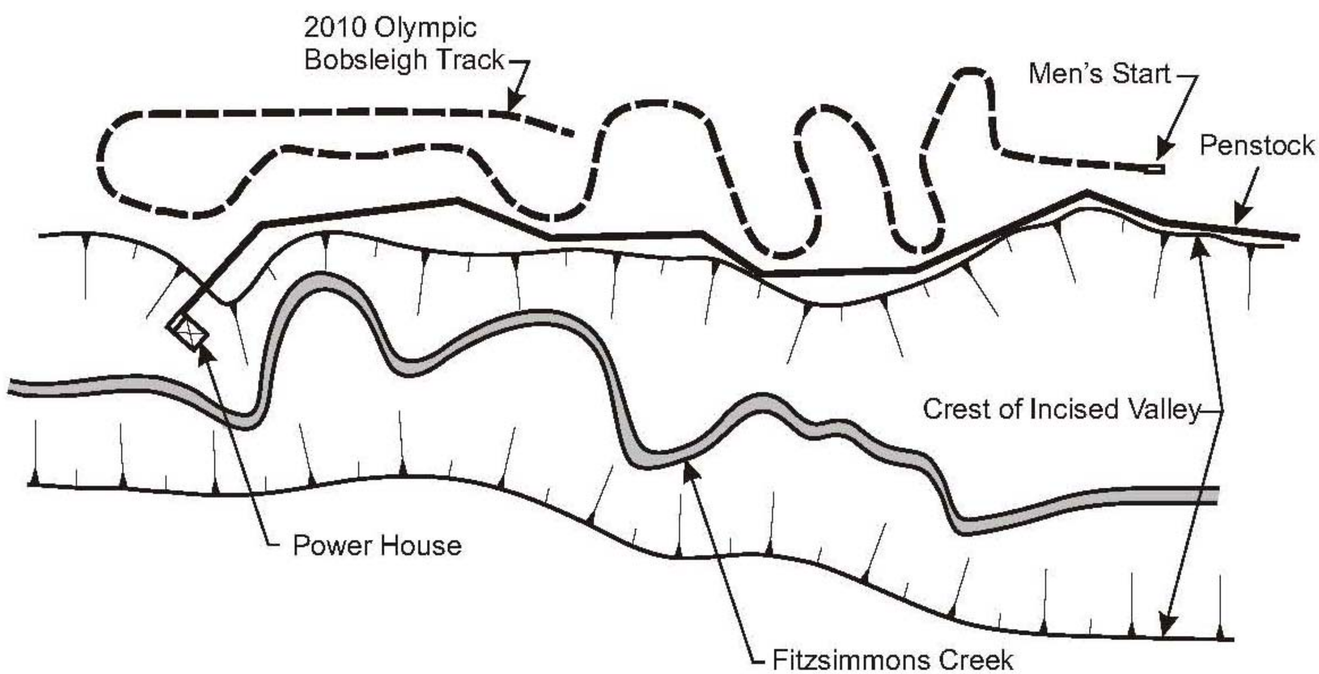

Figure 1- Plan of Downstream Half of Fitzsimmons Project

Construction of the 2010 Winter Olympic bobsleigh track began in 2005 and was not completed until approximately 4 months after the start of construction for the Fitzsimmons Hydroelectric Project. Initially, the developers of the hydro project had planned to install the penstock prior to, or in conjunction with, the bobsleigh track, however, due to permitting and regulatory issues, installation of the penstock was delayed until after the bobsleigh track was completed, so the penstock had to be “shoehorned” into the snow cat road, between the utilities for the bobsleigh track and the edge of a steep natural slope that drops down to Fitzsimmons Creek. The proximity of the nearly-completed Olympic venue poised several potential concerns, including undermining of retaining walls and mobilization of deep seated slope instability.

2 GEOLOGICAL SETTING

2.1 Site Geology

Fitzsimmons Creek is in a U-shaped, glaciated valley between Whistler and Blackcomb Mountains. The base of the valley at the project site is in-filled with glacial till, alluvial sediments, slope colluvium, and ice-contact deposits. These deposits form a gently sloping valley bottom which has been deeply incised to a steep-sided, V-shaped valley adjacent to the lower sections of the project. In this area, the penstock had to be installed very close to the edge of the steep slope. At the powerhouse site, this incised valley is approximately 60 m deep.

The bedrock at the site is foliated, metamorphic greenstones and phyllites. The intake and penstock are founded on till and glacio-fluvial sediments, except for the downstream end of the penstock and the powerhouse which are founded on bedrock at the toe of a steep slope.

2.2 Hazard Assessments

Several hazard assessments have been conducted in the Fitzsimmons Creek valley, primarily to assess slope failures and associated flood hazards to the Municipality of Whistler (Baumann, 2002a; Baumann 2002b; EBA 2005). Of particular concern are the potential slope failures in the incised, V-shaped, steep-sided river channel in the lower sections of Fitzsimmons Creek. Several ancient and recent slides have been identified, the most notable being the feature known as the Fitzsimmons Slump, which is on the opposite side of the valley from the new penstock and powerhouse. Other slides have also been identified that have a lesser or equal volume of slide material that could potentially Fitzsimmons Creek.

Various mitigation options have been studied to reduce the hazard from the Fitzsimmons Slump (EBA, 2005). Work began in the summer/fall of 2008 on the construction of a debris barrier on the Fitzsimmons Creek just downstream of the new powerhouse. The barrier is intended to provide protection to Whistler Village from outwash floods that could occur if a slide were to block the creek.

3 UPSTREAM WORKS

The intake and spillway (see Figure 2) for the Fitzsimmons Hydro Project are at an elevation of 995 m, approximately 4.5 km upstream of the powerhouse. These structures are located upstream of the incised, V-shaped section of the valley. The Coanda-type intake and spillway structures span the full width of the river channel and require only small embankment structures on each abutment. The intake raises the level of the creek by less than three metres and the headpond is fully contained within the normal floodway of the river channel. From the intake, the water flows through a HPDE conveyance pipe to a regulation chamber located approximately 40 m downstream. The regulation chamber is sized to reduce air entrainment before the flow enters the penstock.

A cross-section of the Coanda-type intake is shown in Figure 3. In a flood event, the spillway structure, as well as the Coanda intake structure, are used to spill the excess flow. The length of the Coanda overflow section is 20 m and the maximum design intake flow is 4.25m^3/sec in normal conditions. The combined Coanda and spillway structure is designed to pass the 1 in 200 year flow of 183 m^3/sec, 117 m^3/s of which is passed over the Coanda section.

The Coanda is designed to be a self-cleaning structure which prevents debris and large sediment from entering the penstock. This is accomplished by an acceleration plate at the head of the Coanda screen that produces a laminar flow over the screen thereby increasing the velocity of the water to the point where all debris on the screen gets passed over the weir (Douglass, 2008). The screen-size opening on the Coanda weir is 1 mm. The screen is also designed to resist impacts from debris and bed load material.

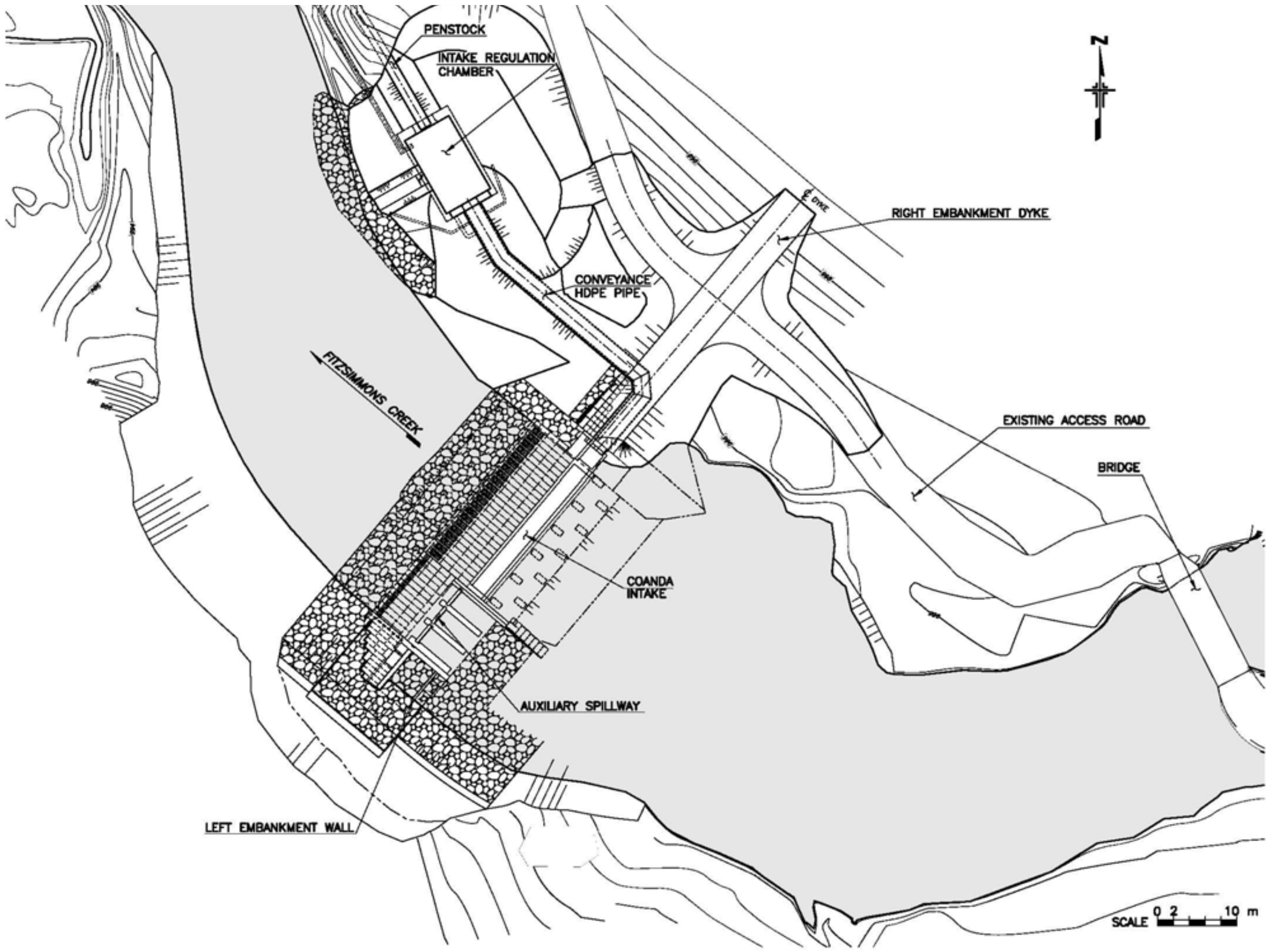

Figure 2- Plan view of Spillway and Intake

The 10.7 m wide spillway is on the left side of the creek channel (see Figure 2). The structure was designed to pass creek flows (88 m^3/s) during construction of the intake but it is also designed as an overflow spillway during floods. The spillway consists of three, 2.3 m wide sluiceways and an in-stream flow release gate. Two of the bays are equipped with concrete stoplogs while the last bay is equipped with a bottom gate which can be used for dewatering of the headpond and sediment flushing.

The intake and spillway structures are founded on glacial till and alluvial sediments. Detailed seepage analyses were carried out to determine whether a cut-off wall, an upstream impervious blanket or other measures are required to limit under-seepage and control uplift pressures. These analyses indicated that, although the foundation material has a permeability similar to a medium sand, the seepage flows under the intake and spillway structures will be about an order of magnitude less than the required in-stream flow release and seepage uplift pressures can be controlled by the installation of a drainage layer downstream of the crest. Therefore, no special measures were required to control seepage flows. This allowed for a large savings to the project as a cut-off structure or upstream blanket would have added a significant cost to the project.

To increase sliding stability, a key-way was added to the base slab. Also, to avoid undermining of the intake and the spillway structure, downstream protection had to be provided. The protection is designed for a velocity of 8 m/s and a pressure of 30 kPa. The geometry of the downstream protection is designed to project the water jet further downstream to minimize the potential for undermining the structures. The downstream protection consists of lock blocks covered by a thin reinforced concrete slab, which provides a monolithic structure and can withstand the vibration pressures.

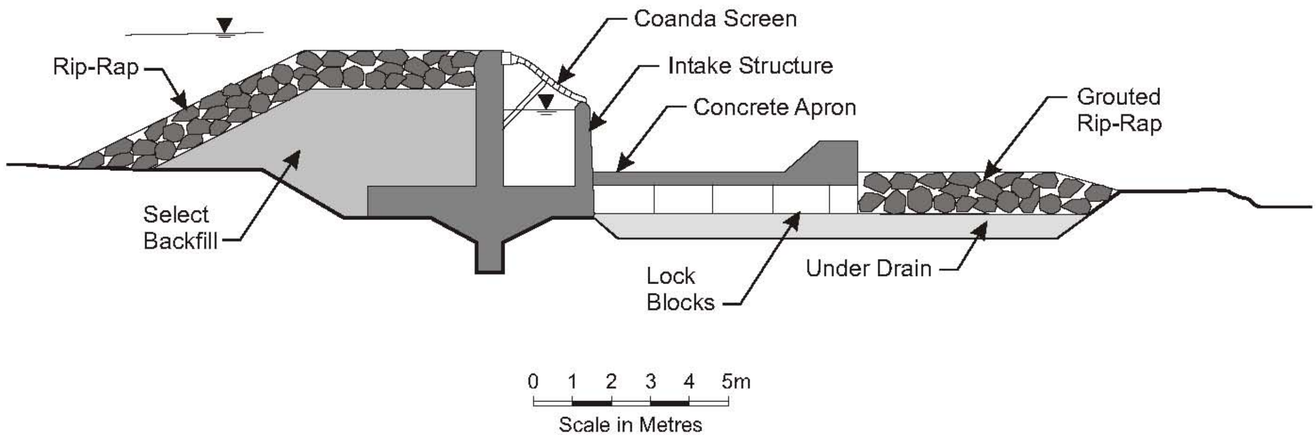

Figure 3 – Section Through Coanda-Type Weir

4 PENSTOCK

The penstock is 3.4 km in length and connects the intake to the powerhouse. It is installed just below original grade, mainly in existing mountain access roads, with some new right-of-way being at the upstream end of the pipe. For the first 2,000 m at the upstream end, where the static head is less than 195 kPa, the penstock is a 54 inch diameter HDPE pipe; it transitions to 48 inch diameter steel pipe downstream of this.

Upstream of the bobsleigh track, where the static head is less than 655 kPa, there is sufficient space for wide-radius bends in the pipe, so anchor blocks are not required. Through the area adjacent to the bobsleigh track, the pipe alignment had to be "threaded" between potentially unstable slopes, the bobsleigh track, the utilities for the track and the newly-constructed retaining walls. This necessitated a number of sharp bends where concrete anchor blocks are required to take the high lateral thrusts. The anchor blocks are typically 2.2 m wide by 2.2 m high and vary in length, depending on the weight required to resist the forces acting on the bends. The three anchor blocks at the downstream end of the penstock are on slopes greater than 20°. In these locations, post-tensioned, double corrosion protected (DCP) anchors are required to provide sufficient resistance to stabilize the anchor blocks and to minimize the volume of concrete even though the anchor blocks are founded on rock.

While selecting the alignment of the penstock to avoid the bobsleigh track and the utilities, it became apparent that the penstock would have to be constructed near or at the crest of some very steep slopes (35° to 44°) of marginal stability, particularly at the Men's Start area, and at Curves 4 and 7 of the track. This required detailed slope stability assessments, analyses, and review. Using the commercially available software SLOPE/W©, the designers analysed numerous cross sections through the penstock, which followed the steepest slope angles, perpendicular to the contours. In the most critical sections (Men's Start, Curve 4 and Curve 7), where the penstock is close to the edge of particularly steep slopes, the factor of safety was found to be less than 1.5. In these areas, additional slope stabilization measures were required, including micro piles, grade beams and lightweight backfill, as shown in Figure 4.

The micro piles are designed to act in shear to increase the sliding resistance along the potential failure surfaces. The micro piles, or pin piles, are 32 mm to 42 mm diameter reinforcing steel bars which were installed using an Odex drill rig that allowed for the advancement of a cased hole. The reinforcing bars were then fully grouted in the holes as the casing was removed. The depths and spacing, as well as number of rows, were determined through detailed stability analyses of closely spaced cross-sections.

To reduce the driving force and increase the factor of safety to at least 1.5, it was necessary to use lightweight cellular concrete as fill around the penstock in some of these areas. Light-weight cellular concrete (LWCC) consists of a cement slurry infused with a foaming agent to produce a hardened concrete product that weighs less than water. The bubbles that form in the cement slurry and the foaming agent are distributed such that there is no contact between the bubbles. This results in a concrete with a strength that can reach upwards of 1 MPa, with a density of approximately 500 kg/m^3. Due to the low durability of the LWCC, it had to be protected from freeze-thaw action and traffic. Where the LWCC could not be entirely backfilled with soil, cast-in-place cantilever walls were designed to retain the LWCC during construction and provide a barrier against freeze-thaw action.

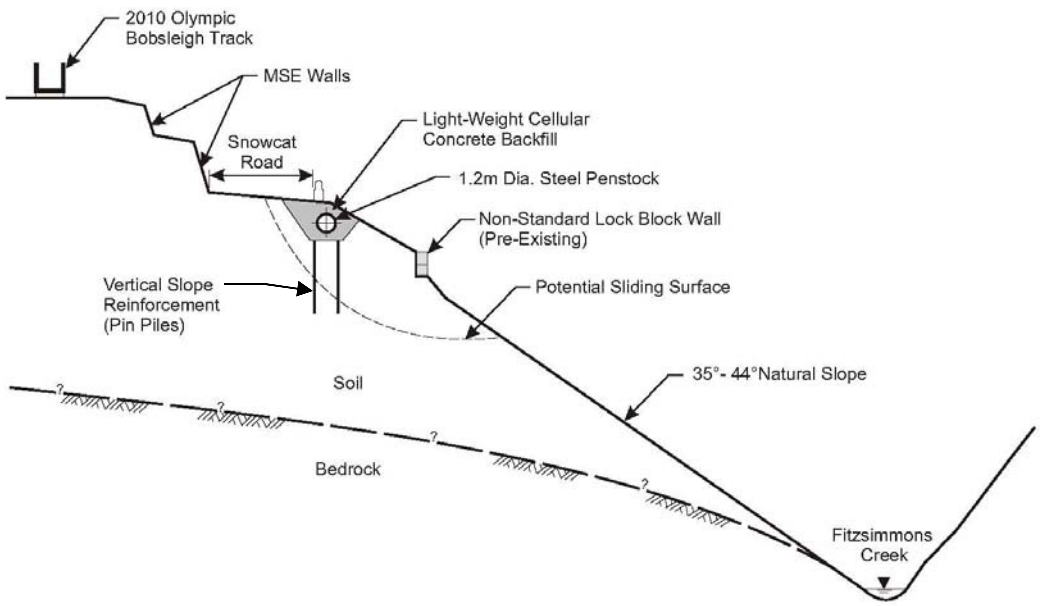

Figure 4 – Typical Cross Section of Penstock beside Bobsleigh Track

Pumice was also considered as an alternative to LWCC, to reduce material costs. As pumice is a locally available product, it was considered as a possible equivalent product to LWCC. However, pumice has several drawbacks that disqualified it as an acceptable replacement product. Pumice is about 2.5 times denser than LWCC when compacted in place in a saturated state. This would have required the installation of significantly more pin piles to stabilize the steep slopes. Pumice is a natural aggregate that will retain water and be susceptible to freeze-thaw degradation. This required the designers to specify a robust groundwater management system in the form of geomembrane and geo-composite drains. Furthermore, the pumice supply would not be available until late in the 2009 construction cycle making supply on the critical path. Consequently, the contractor had to disqualify the pumice due to its cost of placement, increase in pin piles, and its window of availability.

5 POWERHOUSE

The powerhouse contains a single, vertical 7.5 MW Pelton turbine equipped with 6 jets. The powerhouse structure is founded on rock and has a conventional concrete slab with a pre-fabricated steel frame superstructure. Discharge from the tailrace channel enters the existing Whistler-Blackcomb snow making pond near the pump house. The powerhouse is located on the east side of the pond and is “tucked in” at the toe of a steep natural slope, just downstream of a spine of rock so that it would be out of the line of debris flows in Fitzsimmons Creek. The area available for the powerhouse was relatively small and uneven. This necessitated a steep rock cut at the base of the natural slope to create a flat area large enough for the powerhouse and supporting infrastructure.

The slope behind the powerhouse has been excavated at 56 degrees in weathered and sheared bedrock. The cut slope is about 20 m in height and has shallow overburden at the crest of the cut face. The slope angle was selected to minimize rock support so only draped mesh is required to contain surface ravelling.

6 CONCLUDING REMARKS

The Fitzsimmons Creek Hydro Project is due to start producing power in 2010. As it is being constructed during the lead-up to the 2010 Olympic Games and is immediately adjacent to the bobsleigh track, close cooperation and coordination were required between the developer (Innergex/Ledcor Power), the contractor (Ledcor CMI), the Vancouver Olympic Committee (VANOC) and the design team (RSW/Golder). The restricted space that was available for the installation of the penstock and the powerhouse added greatly to the challenges of design and construction. There has been no interference with the operations of the bobsleigh track and no disturbance to the infrastructure from the construction of the project.

7 REFERENCES

Horizon Engineering Inc, Fitzsimmons Creek Run-of -River Hydroelectric Project Proposed Powerhouse Geotechnical Site Investigation Report, August 2002a

Horizon Engineering Inc, Fitzsimmons Creek Run-of -River Hydroelectric Project Proposed Intake Structure Geotechnical Site Investigation Report, August 2002b

Baumann Engineering, Initial Terrain Stability field Assessment of the Proposed Penstock route, Fitzsimmons Creek Area, Whistler, BC, December 2002a

Baumann Engineering, Overview Assessment of natural Hazards in the Fitzsimmons Creek Watershed, December 2002b

EBA Engineering, Fitzsimmons Creek Landslide Hazard Assessment, February 2005

Golder Associates Ltd, Review of Stability and Load restrictions snow Groomer Access Road at Curves 4 and 7, Whistler Sliding centre, Whistler, BC, December 2007

Douglass, S., “Coanda water Intake Basics,” Coanda Intakes website, 2008

Resort Municipality of Whistler, “Debris Barrier Construction Update,” RMOW website, 2008

OLDMAN DAM (ALBERTA):
MANAGING RISK OF SPILLWAY FAILURE

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ABSTRACT:
Alberta Environment, Water Management Operations operates and maintains the Oldman Dam, located on the Oldman River, 10 km northeast of Pincher Creek, Alberta. The dam was constructed between 1986 and 1991. It is 76 m high, 1200 m long, and impounds a reservoir with a storage capacity of 490,000 dam^3 of water. The dam is classified in the "Extreme" consequence category (CDA, 2007).

The dam has a concrete spillway that is 336 m long with a drop of 26.0 m. The spillway chute is 110 m wide at its crest, transitioning to 40 metres wide at the flip bucket. It is controlled by 7 radial gates, each 10 m wide by 9.8 m high, with a total discharge capacity of 2,970 m^3/s at Full Supply Level (FSL). The spillway is designed to handle the Probable Maximum Flood (PMF).

A 2007 dam safety review found that the dam generally was in good condition. One notable performance issue was the ongoing, slow deformation of the concrete spillway, which has shown no evidence of slowing down in the last 16 years of operation. While annual incremental movements are not large, the accumulated spillway movement and shearing of the foundation is significant.

This paper summarizes the performance of the spillway structure, discusses the possible causes of the deformation and their long-term impact, and presents the recommendations given in the dam safety review to manage the risk of spillway failure.

RÉSUMÉ:
La division de gestion de l'eau du ministère de l'Environnement de l'Alberta exploite et entretient le barrage Oldman qui est situé sur la rivière Oldman à 10 km au nord est de Pincher Creek en Alberta. Le barrage a été construit entre 1986 et 1991. Il mesure 76 m de hauteur, 1200 m de longueur et retient un réservoir d'une capacité de stockage de 490,000 dam3. Le barrage est classifié dans la catégorie à conséquence 'Extrême' (ACB, 2007).

Le barrage renferme un évacuateur de crues en béton ayant 336 m de longueur et une chute de 26 m. Il mesure 110 m de large à la crête, s'amincit jusqu'à 40 m au déversoir, et comporte 7 vannes radiales de 10 m de largeur et de 9.8 m de hauteur respectives qui permettent l'évacuation d'un débit de 2,790 m3/s au niveau maximal d'exploitation. L'évacuateur de crues a été conçu pour gérer un débit produit par la crue maximale probable.

Une évaluation de la sécurité du barrage complétée en 2007 démontre que le barrage est généralement en bonne condition. Un aspect notable de la performance du barrage est la déformation lente et progressive de l'évacuateur de crues en béton qui s'est perpétuée pendant les 16 dernières années d'opération. Même si les déformations annuelles ne sont pas élevées, la déformation cumulative de l'évacuateur de crues et le cisaillement de la fondation sont importants.

Cet article résume la performance de l'évacuateur de crues, discute des causes possibles des déformations et l'impact à long terme de ces déformations, et présente les recommandations déduites de l'évaluation de la sécurité du barrage pour la gestion du risque de rupture de l'évacuateur de crues.

1 INTRODUCTION

Alberta Environment (AENV), Water Management Operations (WMO) operates and maintains the Oldman Dam, located on the Oldman River, 10 km northeast of Pincher Creek, Alberta. The dam is located in Section 17, Township 7, Range 29, west of the 4th Meridian. The general location of the dam is shown in Figure 1 and Figure 2.

The dam was constructed between 1986 and 1991. It is 76 m high and 1200 m long (including dykes), has a crest elevation of 1125.5 m, and impounds a reservoir with a storage capacity of 490,000 dam3 of water at an FSL of 1118.6 m elevation (6.9 m freeboard). The average side slopes are 3.9H:1V on the upstream side and 4.2H:1V on the downstream side.

The dam has a concrete spillway that is 336 m long with a drop of 26.0 m. The spillway chute is 110 m wide at its crest, transitioning to 40 metres wide at the flip bucket. Seven radial gates control the flow, each 10 m wide by 9.8 m high, with a total discharge capacity of 2,970 m^3/s at Full Supply Level (FSL).

In accordance with the CDA Dam Safety Guidelines (2007), the dam is classified in the "Extreme" consequence category. The spillway is thus designed to handle the peak outflow associated with a Probable Maximum Flood (PMF) of 9320 m^3/s, at a maximum water level of 1124.5 m elevation at the dam (1 m freeboard).

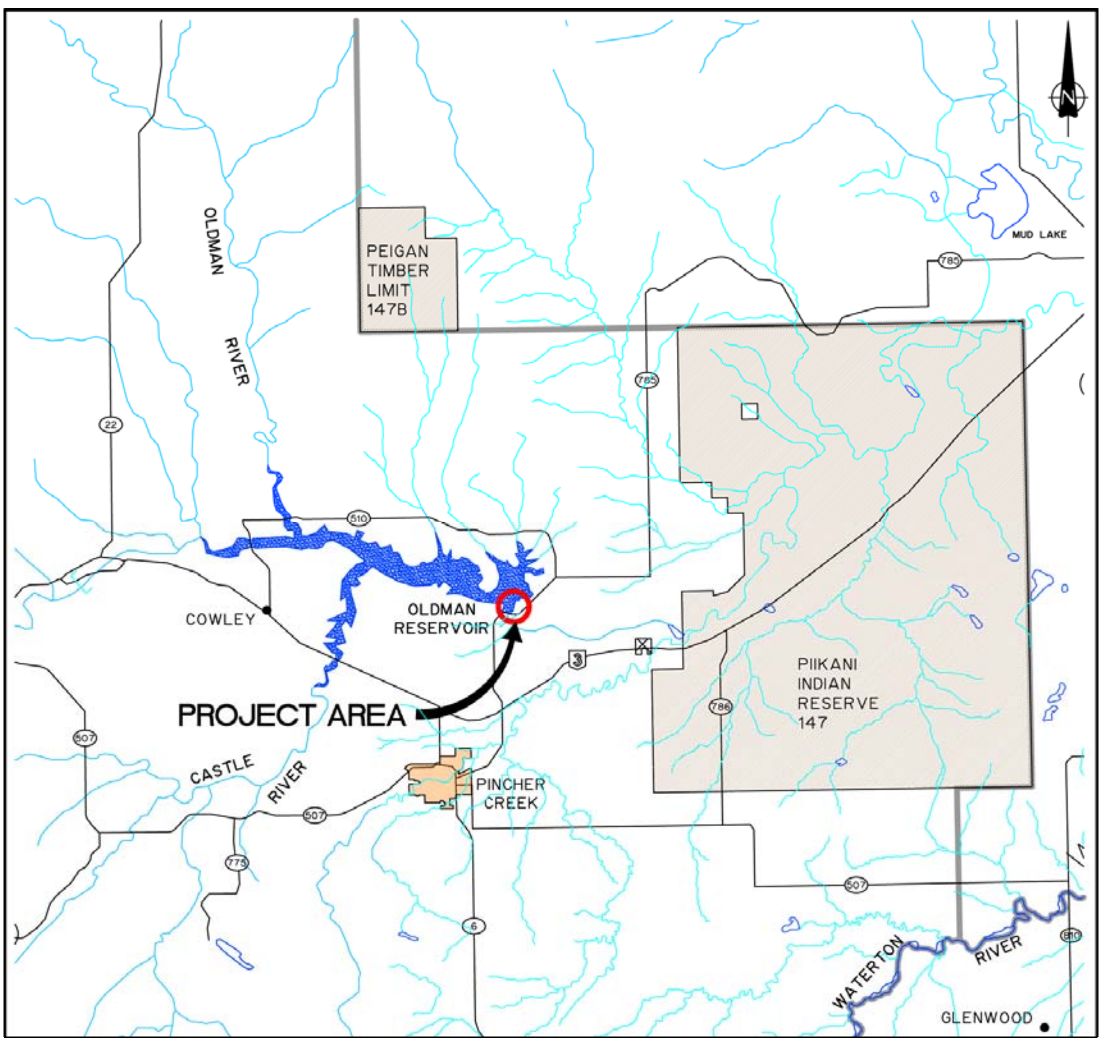

Figure 1: Site Location

Figure 2: Layout of Oldman Dam and Spillway (Photo courtesy of Tarin Resource Services Ltd.)

2 BACKGROUND

MPE Engineering Ltd. (MPE) and Thurber Engineering Ltd. (Thurber) jointly carried out a dam safety review for the Oldman Dam in 2007. The DSR found that the dam was generally in good condition with no significant performance issues except for an ongoing, slow deformation of the concrete spillway. The spillway has shown no evidence of slowing down in the last 16 years of operation. While annual incremental movements are small, a significant amount of movement and shearing of the foundation has accumulated in the 18 years since the end of construction.

This paper:

- Summarizes the performance of the spillway structure (as determined by slope inclinometers, extensometers, surface deformations, visual inspections and evidence of drain mal-performance).
- Discusses the possible causes of the deformation, and its long-term impact.
- Presents the recommendations given in the dam safety review to manage the risk of spillway failure.

3 SPILLWAY CONDITION

3.1 Spillway Head Works

The spillway head works were in good condition, showing only minor cracking in the ogee weirs and pier walls typical of a structure of this age (Photos 1 and 2). The drainage gallery within the head works also showed only minor cracking and seepage.

3.2 Chute Structure

The following observations were made of the chute structure:

- The expansion/closure joints at the transition between the spillway chute and the head works were performing well.
- Minor concrete spalling and cracking were seen at some of the joints of the chute walls, both on the inside and outside surfaces, although neither seems to have progressed much since a 2002 inspection.
- The concrete chute appears to be in good condition, with minor spalling at several transverse construction joints and transverse cracks across the width of the slab.
- A deformation survey completed during the previous DSR (2001) indicated that the lower portion of the chute slab has bulged approximately 28 mm, but this is not evident to the unaided eye.
- The concrete in the flip bucket shows some hairline cracking (likely shrinkage cracks) and all joints appear to be in good condition (Photo 3).
- The mud slab on the outside of the chute walls (both sides; not structural members) shows signs of distress at some transverse joints, with the uphill slab riding up on the downhill slab and the joints deteriorating (Photo 4). Some open cracks are evident around the drainage manholes as well.

3.3 Spillway Drainage System

There are two buried drain lines running parallel to the spillway, one on each side. Access is by manholes (MH) from the surface. Manholes near the bottom end of the spillway are showing signs of distress due to movement along a shear plane in the foundation. Cracking was observed:

- In MH7 on the left side of the spillway.
- In the barrels of MH8 on the left side of the spillway (Photos 5 and 6).
- In MH7 on the right side of the spillway, with some shearing of the precast section at the bottom of the manhole.

3.4 Summary of Spillway Condition

In summary, while it is known that movement is occurring along a shear plane in the foundation of the spillway, there is little evidence of this movement in the spillway structure itself. Minor distress is seen in the mud slab along the outside of the spillway and moderate distress is seen in the lower drainage manholes, which cross the elevation of the shear plane.

Photo 1: Spillway walls and floor.

Photo 2: Right side of spillway, showing mud slab, spillway walls and head works.

Photo 3: End of spillway and flip bucket.

Photo 4: Up thrust on spillway mud slab joint.

Photo 5: Cracking in drainage manhole barrel.

Photo 6: Left side of spillway, showing manholes for drain

4 SPILLWAY MOVEMENTS

4.1 General

Spillway movements have occurred due to shear displacement along a weak unit in the interbedded foundation mudstones (this unit varies in elevation from 1069 m to 1070 m on the right side of the spillway, to 1067 m on the left side). They have been monitored by visual inspections of the spillway and drainage system, precision survey of the surface of the chute, measurements of extensometers installed along the outside of the spillway walls, and measurement of slope inclinometers (SIs) installed on both sides of the spillway (outside the walls and in the cut slopes above the spillway walls). A summary of the visual observations is given in the previous section; this section summarizes the amount and rates of movement that have been recorded in the survey and geotechnical instruments to date.

4.2 Precision Survey

Through past measurements (2001 and before) the head works and upper section of the spillway chute have been shown to have experienced gradual but ongoing movement, mostly in the downstream direction, as well as "bulging" further down the chute section. There has been much concern expressed over the status and condition of the chute as the foundation seam R1 continues to experience gradual but ongoing movement. Unfortunately, no precision surveys have been run since 2001.

The results of the 2001 precision survey are shown on Figures 3 to 5.

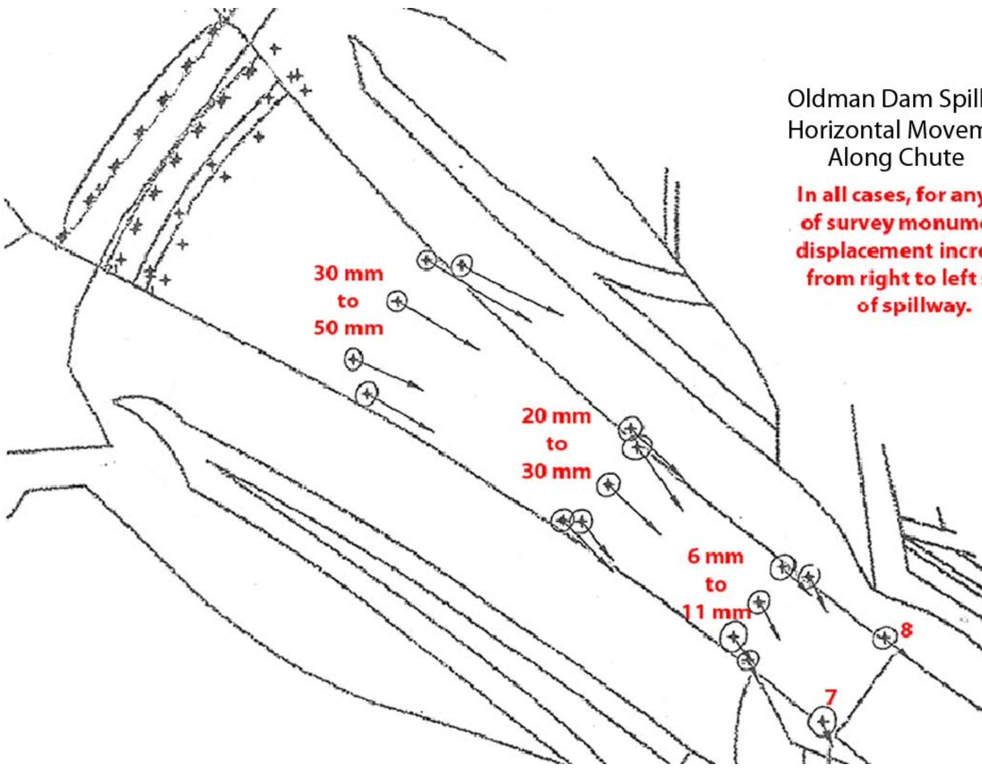

Figure 3: Oldman Dam Spillway – Horizontal Movement along Chute

At any location on the spillway, the rate of movement measured in the precision survey appeared to be relatively constant, with movement rates varying from 0.5 mm/year to 3 mm/year. However, the pattern of movements is spatially complex. The following patterns are noteworthy:

- Between 1997 and 2001, there were higher vertical and lateral movements in the centre of the chute than at the edges.
- Total movements showed a similar pattern. It is also clear (Figures 3 and 4) that lateral movements increased from the flip bucket (10 mm) to the head works (40 mm to 60 mm).
- The greatest heave of the spillway slab was noted mid-way between the head works and the flip bucket, with a total of 28 mm of heave, i.e., about 3 mm/year (Figure 5).

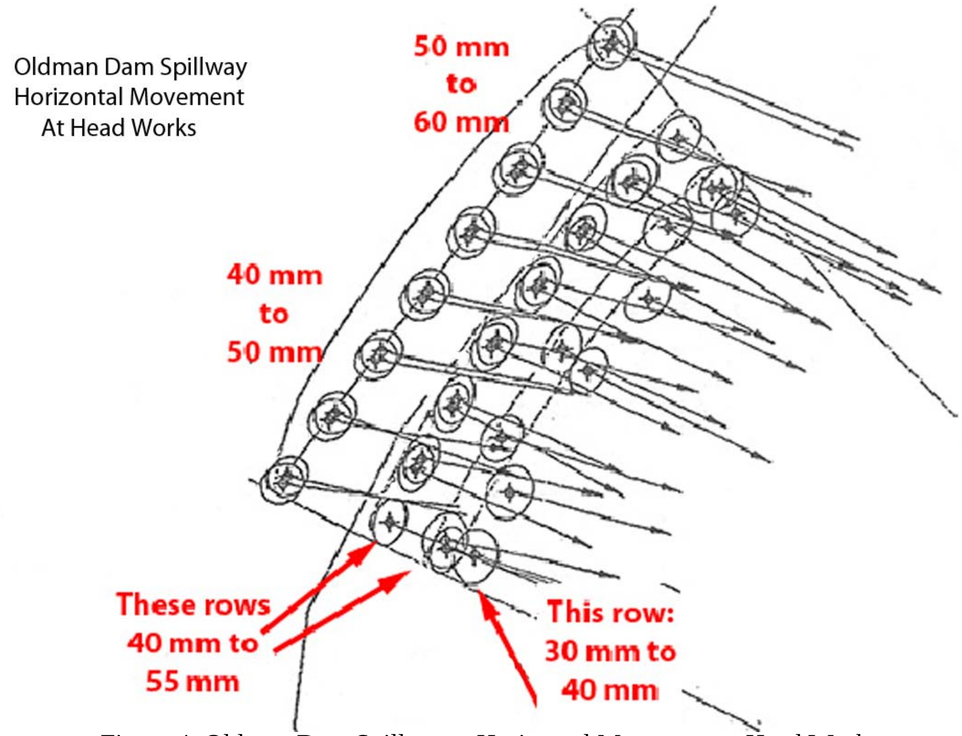

Figure 4: Oldman Dam Spillway – Horizontal Movement at Head Works

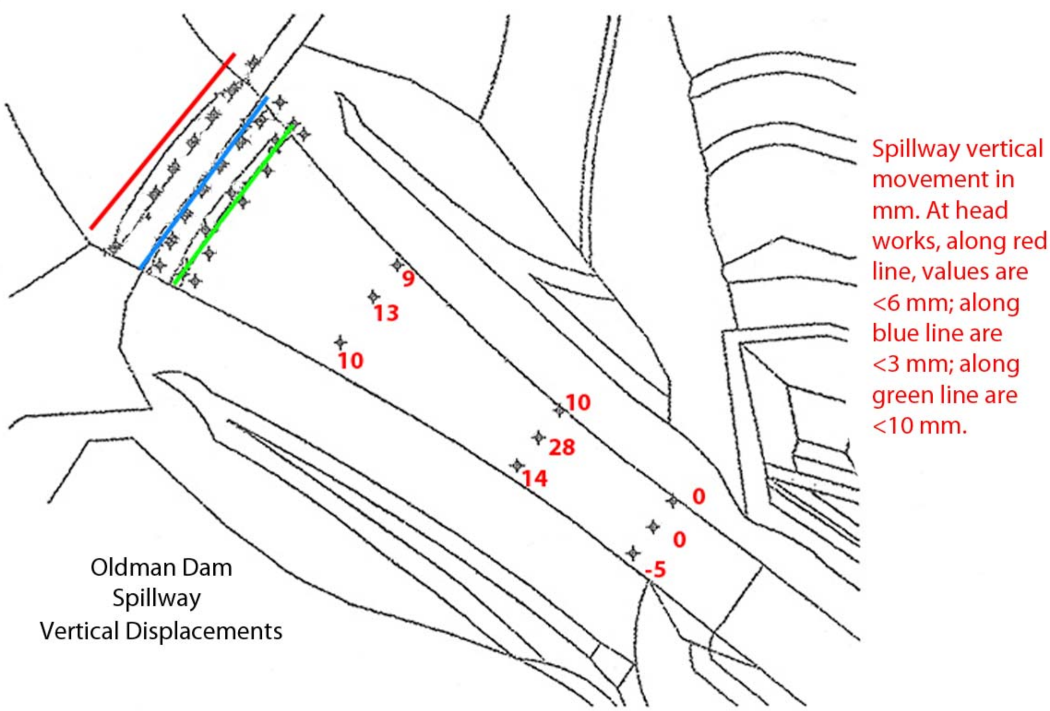

Figure 5: Oldman Dam Spillway – Vertical Movement at Head Works and on Chute

The 2001 DSR reported that down hill movement of the spillway slab could be a potential cause of high compressive stresses, since the flip bucket is a large massive structure that is not moving and the slab butts against the flip bucket without a joint to absorb the downward movement. At this time there is no visual evidence of spalling or other indicators of high stress occurring at the slab junction with the flip bucket.

There is also no obvious visual evidence that the chute structure, slab and walls, are experiencing any significant distress. As noted in Section 3.2, many transverse cracks across the slab were observed, but there was nothing to indicate that the slab surface was in state of high compressive stress.

The 2001 DSR proposed that the thick granular drainage layer under the slab could be absorbing and damping the movement in the foundation. This is a possible mechanism that would be effective at least up to a certain amount of displacement.

In summary, based on the precision survey of the spillway alone, there is no evidence to indicate that the structure is in any immediate peril. The fact remains that the spillway (but not the flip bucket) continues to move at a relatively constant (and slow) rate. From a structural perspective, the deflection measured in 2001 is very small relative to the width of the chute (i.e., 28 mm heave over a 40 m chute width) and even more insignificant relative to the length of the chute over which heave is occurring. A very simplistic structural check supports this idea and suggests that bending stresses in the slab from the 2001 level of measured deformation are insignificant relative to what would be necessary to cause a compression failure of the slab.

Considerable additional deformation would thus have to occur before the potential of a slab compression failure was significant. Quantifying the exact amount of deformation that would be necessary for a slab failure is difficult and complex, and not possible based on the information available. However, it is anticipated that major visible signs of distress (i.e., bulging and significant open cracks) would be apparent (quite possibly for some time) prior to a slab failure.

4.3 Movement of the Ground Adjacent to the Spillway from Slope Inclinometers

Slope inclinometers installed along each side of the spillway (Photo 7) show movement generally downstream and to the northeast, though there are some exceptions. Resultant movement vectors for the major shear zone in each inclinometer are shown on Figure 6.

Most of the movement is occurring in one foundation layer that dips to the northeast. South of the spillway, the movement zone is at Elevation 1069 m to 1070 m; north of the spillway, at Elevation 1067 m. A 2 m drop across the 100 m distance between inclinometers on opposite sides of the spillway suggests a dip angle on the shear zone of about 1 degree. The greatest ground displacements (up to 70 mm) have been noted in inclinometers adjacent to the lower (east) half of the chute.

While the accumulated movement on the SIs is substantial, movement rates are very low. The implications of the amount and rate of movement are discussed in Section 4.6 (below).

4.4 Movement of the Ground Adjacent to the Spillway from Extensometers

There are two rows of 7 extensometer installations, each row running along and about 10 m from each side of the spillway (Photo 7). Each installation consists of two extensometers, one vertical and one inclined upstream at 30 degrees from the vertical (Photo 8), and both extending to a depth of about 30 m (40 m at head works).

The extensometer data is more difficult to interpret than the SI data. Movements in the vertical and inclined extensometers are shown on Figure 7, which are interpreted as follows:

- The extensometers are shown in blue, and the movement of each anchor point with respect to the extensometer head is also shown in blue.

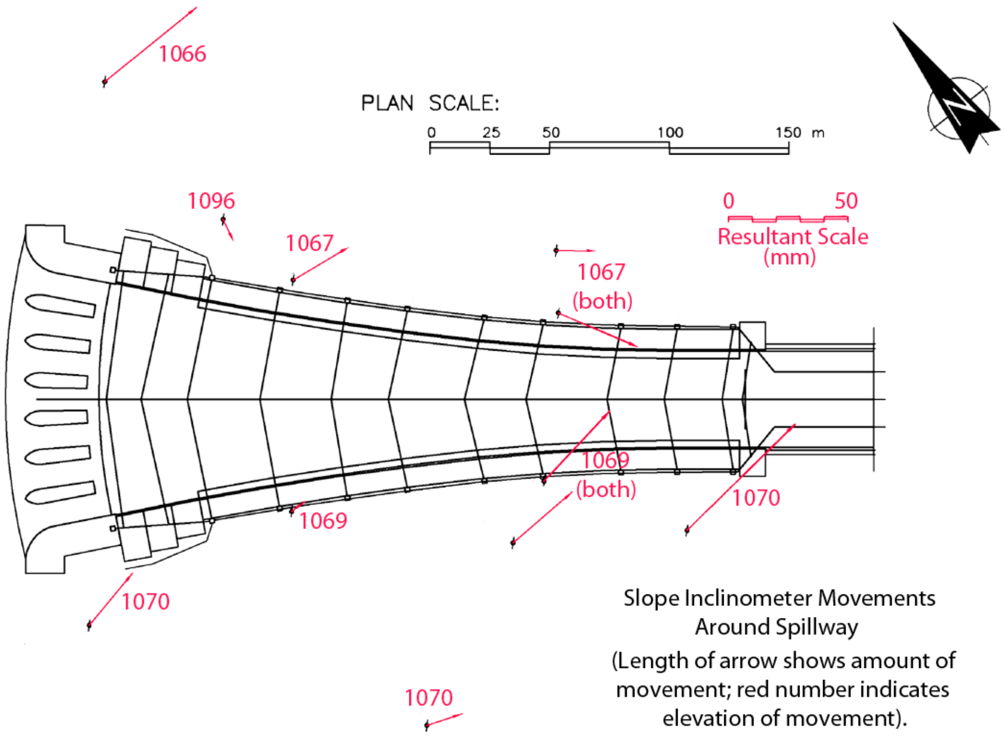

Figure 6: Slope Inclinometer Movements Around Spillway

- The movement of the extensometer head (with respect to the bottom anchor points) has been resolved into horizontal and vertical components, as shown in red (Figure 7).
- Also shown on this figure are the locations of adjacent inclinometers (in green), with the maximum incremental movements across their shear zones (in magenta).

The profiles indicate that the most significant movement is in the elevation 1065 m to 1070 m zone, which is at the same elevation as the cracking noted in manholes MH7L and MH7R. While accumulated movements are significant, over the last 10 years, movement rates have been low. Further discussion of movements adjacent to the spillway chute is included in Section 4.6.

Photo 7: Right side of spillway showing drain manholes, extensometers (yellow; middle left) and SI (blue).

Photo 8: Vertical extensometer (on left, showing head) and inclined extensometer (in middle).

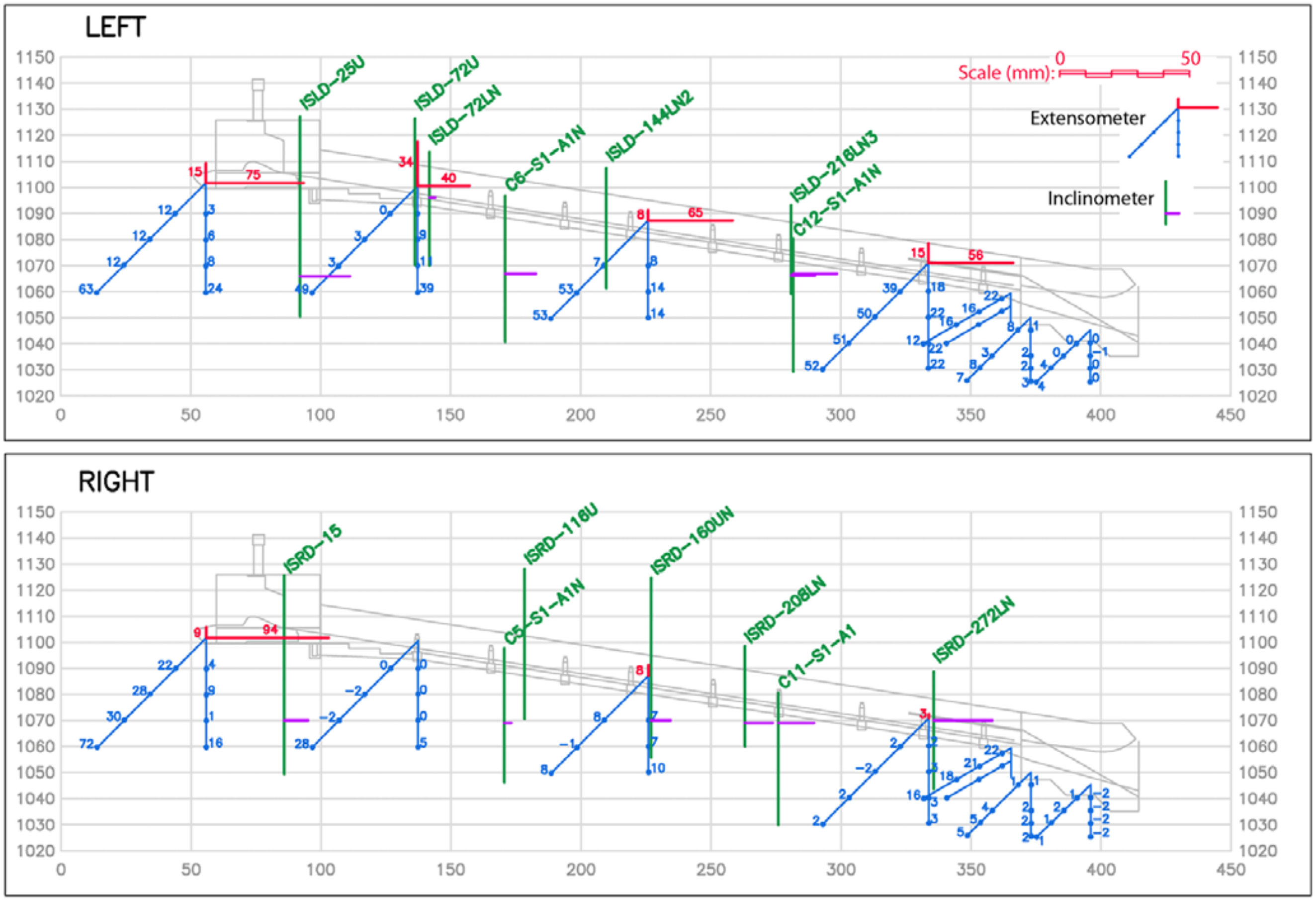

Figure 7: Plot of extensometer movements on left and right side of spillway. Values in blue are displacements of anchor points with respect to extensometer head (mm); values in red are interpreted displacements of head, resolved into X and Y components (mm). Magenta lines show depth of main displacement in SIs. [Axes of plots: vertical is elevation (m) and horizontal is distance along spillway (m)]

4.5 Indirect Evidence of Ground Movement

There is a series of vertical drains that run between a drainage gallery under the spillway head works and a deeper drainage tunnel (Tunnel A). In the past, these drains have been pulled from their boreholes for cleaning, but since 1995, it has not been possible to remove the drains. Probing of the drains indicates blockage at certain depths (much like an SI shearing).

The 1997 DSR speculated that movement along a weak zone in the foundation caused the obstruction of the drains, and that with continued movement, drain efficiency could be impacted. In addition, the grout curtain installed under the spillway head works, upstream of the drains, could be dislocated and rendered ineffective. A close examination of both drain flows and piezometers readings (for piezometers located on both sides of the drain curtain) indicates that there has been no increase in seepage through the area under the head works due to a faulty grout curtain or a loss of flow due to drain plugging.

However, the fact of the drain blockage still remains as indirect evidence that there is some shear in the foundation under the spillway head works consistent with the SI and extensometer measurements discussed earlier.

4.6 Stability of the Spillway and Implications of Spillway Movement

Stability analyses reported in the 2001 DSR indicate that the factor of safety against a rotational or translational failure through the spillway foundation is acceptable. The parameters used in the stability analyses were conservative. The 2001 DSR concluded that: "…It is therefore clear that the structure is not sliding on layer R1 (*the weak foundation unit*) due to hydrostatic loading…", that is, the slope (including the spillway) is inherently stable. The 2007 DSR team concurred with this conclusion.

The authors of the 2001 DSR go on to postulate that the cause of the observed movement is most likely swelling of the foundation mudstones, with movement being towards the unconstrained bedrock face, i.e., in a downstream or south-easterly direction, and with the rate of swelling being determined by the availability of seepage water from the reservoir. The 2007 DSR team agreed with that hypothesis.

Understanding the cause of the movements in this case is very important. Several points are noteworthy:

- The swelling mechanism is displacement driven rather than load driven (as in the case of slope instability).
- Because the weak layer has a low strength, near its residual value, the movement mechanism is considered to be ductile.

Taking both points together, this means that if one section of the spillway structure were to crack, there would be no expectation of a brittle ground response or of significant, quick additional movement.

The lateral displacements measured by each of the three methods discussed previously are shown in Table 1.

Table 1: Lateral Displacements Around Spillway (mm)

Instrument	**Top of Spillway**		**Middle of Spillway**		**Bottom of Spillway**	
	Right	**Left**	**Right**	**Left**	**Right**	**Left**
Survey of Spillway structure (slab and walls)	40	60	20	30	<10	<10
Extensometers (ground adjacent to spillway)	90	75	0	65	0	60
Inclinometers (ground near spillway)	40	20	55	50	70	n/a

Note: No inclinometer on left side of spillway flip bucket.

Note that while the magnitude of lateral movements observed in the survey results, inclinometers and extensometers is similar (Table 1), the pattern varies somewhat:

- Based on the inclinometer and most of the extensometer readings, it appears that the ground around the spillway has moved downstream in the order of 50 to 75 mm.
- The exceptions to the previous bullet are the extensometer readings on the right side, at the middle and lower parts of the spillway. For some reason, these extensometers are showing almost no lateral movement, even though movement is evident in nearby inclinometers.
- The precision survey indicates that the top part of the spillway has moved downstream in concert with the surrounding ground (values are a bit lower than the extensometers, but the last precision survey was done in 2001 and hence is somewhat out of date). The middle and lower portions of the spillway show less and less lateral movement, and at the flip bucket, the lateral movement is less than 10 mm. This suggests that the flip bucket is well anchored to the ground below the shear plane, which is not moving.

The patterns of movement described in Table 1 are consistent with the concept of a displacement driven mechanism, such as swelling of the foundation bedrock. The spillway structure follows the ground movement, except at the bottom end, where the flip bucket is firmly anchored into the lower ground (below the shear plane). This creates a zone of shear concentration in the structure upstream of the flip bucket, at about elevation 1068 m.

5 DISCUSSION OF SPILLWAY STATUS AND RECOMMENDATIONS

This section discusses the condition and status of the spillway, based on the visual observations and monitoring results discussed in the previous two sections, and also the likely outcome of continued spillway movement. It then recommends actions that could be taken to manage the risk associated with ongoing movement.

In the opinion of the authors, it is very unlikely that the spillway is already failing. As discussed in Section 4.2, basic structural checks indicate that considerable additional deformation would have to occur before the slab was close to failing in compression or bending. Total displacements, as indicated by various geotechnical instruments, are moderate (50 to 100 mm), but displacement rates are small. Movements have so far not impeded the operation of the spillway. The movement mechanism is considered to be ductile and displacement-driven rather than load-driven. All of these factors argue for time being available to develop an appropriate risk management plan for the spillway.

It is anticipated that significant visual signs of distress would be apparent (possibly for some time) prior to a compression slab failure. It is likely that more substantial damage to the spillway drain manholes would occur first, (requiring significant repair work and maintenance). Bulging and significant open cracks of the spillway slab or walls could also occur. In addition, movement rates would probably increase in the extensometers and slope inclinometers.

At present, there is insufficient information and knowledge of the complex mechanisms involved in the slab movement to completely understand and determine if “pre-emptive” action (to prevent excessive damage to the spillway structure) is desired or immediately required; and if so, what action would be the most effective. If the foundation shear zone continues to move, eventually there could be significant damage to the slab and structure, which could potentially lead to a serious dam safety event.

During the time between the appearance of damage to the spillway slab and completion of repair work, the spillway would be at risk from a flood event. While the probability of these two events coinciding is perceived to be relatively low, the consequences if this led to failure of the spillway would be very serious. Therefore, the 2007 DSR team recommended that a risk management plan be developed, that would include a strategy for the continued safe operation of the dam after a failure of the spillway chute, including details of flood handling.

The 2007 DSR team made several recommendations for further action to manage risk:

- Continue to inspect the spillway structure and drainage manholes semi-annually for increased signs of distress, and monitor existing instruments (on a prescribed schedule) for changing trends in performance (particularly movement rates).
- Complete a precision deformation survey of the spillway, and confirm how the movements have progressed relative to the previous surveys and the anticipated trend based on past measurement rates.
- Based on the results of the precision survey and other monitoring, develop a scope of work that identifies the next steps. This could include some or all of the following:
    - Additional instrumentation and monitoring.
    - More comprehensive site investigations and laboratory testing, which may involve additional instrumentation and possibly destructive and non-destructive testing.
    - Analysis and development of a plan to deal with continued spillway movement, which may include “pre-emptive” measures to prevent or lessen the impact of spillway failure.
    - A comprehensive Failure Modes and Effects Analysis (FMEA), to identify all high consequence failure modes, (particularly for high flood conditions).

- A Risk Management plan, describing the steps to be taken if the slab were to fail (due to ground movement), to control the risk of serious damage to and impairment of the spillway, caused by a near-simultaneous flood.
- Making any required changes to the OM&S Manual, the ERP and the EPP.

6 ACKNOWLEDGEMENTS

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7 REFERENCES

Breunig, M., Elson, B., Kitagawa, R., Sobkowicz, J.C., Cullum-Kenyon, S., 2008. “Oldman Dam – Dam Safety Review – Final Report”, MPE Engineering Ltd. – Thurber Engineering Ltd.

Canadian Dam Association, 2007. “Dam Safety Guidelines”.