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SYNOPSIS: The main dam at the Bighorn development consists of a zoned earthfill embankment with a concrete cut-off wall constructed through the river alluvium by the slurry trench technique. Upon first filling of the reservoir in 1972, erratic drops in piezometric heads in the alluvium upstream of the cut-off and significant downstream leakage prompted the construction of a weight berm at the downstream toe and implementation of a program of regular monitoring of all piezometers and seepage measuring facilities. Concern for the integrity of the structure was not allayed until completion of a dam safety evaluation in 1984. The paper summarizes the design and construction aspects of the main dam and concrete cut-off, documents the results of monitoring records of seepage and piezometric heads since reservoir filling and assesses the extent of reservoir siltation. Records and inspections demonstrate satisfactory performance of the structure.

INTRODUCTION

The Bighorn hydroelectric development, located on the North Saskatchewan River in the frontal ranges of the Rocky Mountains about 100 miles northwest of the town of Banff, Alberta, was completed in 1972. The project consists of a 300 ft high main dam, a 100 ft high closure dam, an emergency spillway, and a gated intake structure for the 22.5 ft dia. power tunnel in the left abutment leading to the 120 megawatt powerhouse.

The main dam, consisting of a zoned earthfill embankment with a total crest length of 1700 ft, is founded on a 210 ft deep deposit of sand, gravel and boulders. To minimize seepage through the pervious foundation alluvium, a concrete diaphragm cut-off wall aligned along the dam axis was constructed through the alluvium to key into bedrock using slurry trench techniques. The layout of the dam and appurtenant facilities are shown on Figure 1, and a typical section through the dam is illustrated on Figure 2. The emergency spillway and closure dam are located at the entrance of a natural gully about 1.6 mi north of the damsite.

Bighorn dam impounds Lake Abraham which, with a storage capacity of 1,165,000 acre-feet, is the largest man-made lake in Alberta. The project, owned and operated by TransAlta Utilities Corp. of Calgary, serves as a storage and flood control facility while providing power generation benefits.

The control of seepage about the dam was of particular concern in view of the disturbed condition of the sedimentary rocks forming the abutments and the locally high permeabilities of the overburden deposits, particularly within the deep riverbed sediments. The countermeasures which were adopted comprised blanketing of the riverbed and certain abutment slopes upstream of the dam, the installation of shallow drains at the downstream toe and the downstream slope of the north abutment, and the construction of a concrete diaphragm cut-off wall to control seepage through the deep bed deposits. The performance of the main earthfill dam and the diaphragm wall forms the principal topic of this paper. A description of the development, previously reported by Gordon and Rutledge (1972) and Forbes et al (1973), is given in the following sections.

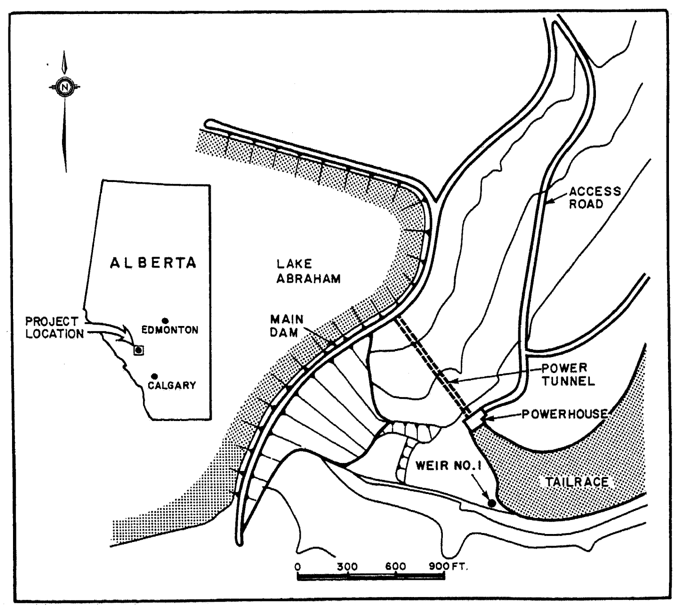
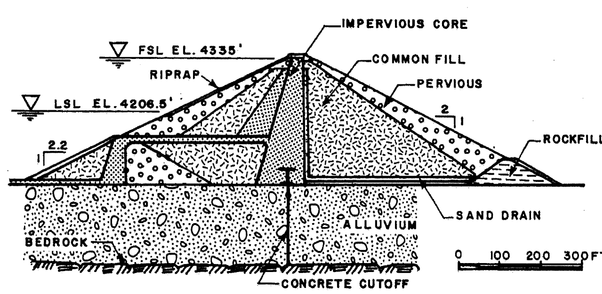


Fig. 1 Site layout



SITE CONDITIONS

The topography of the region is well suited for hydro-electric development, with the broad glacial valley of the North Saskatchewan River providing a large reservoir capacity upstream from the narrow canyon in which the dam is sited. The geological conditions are, however, less favourable, with the disturbance arising from the tectonic upheavals during mountain building masked by subsequent glaciation and post-glacial downcutting.

The dominant rock formations in the region comprise thick strata of interbedded siltstones and sandstones of the Cretaceous period which were extensively faulted, folded and tilted with the building of the Rocky Mountains. A major regional strike fault of this period traverses the project site about 1000 ft upstream of the dam axis and has been estimated to have moved more than 1.5 mi in the northeast direction along its strike. The thrusting has produced a series of closely-spaced parallel step faults in the adjoining sediments, in which the slippage ranges from a few inches to several feet, while numerous shear planes or bedding faults are evident in the weaker strata.

In the area of the canyon, the upstream limb of the regional fault is formed, in ascending order, by the Luscar and Mountain Park Formations of Lower Cretaceous origin, while the downstream limb is formed by the Blackstone, Bighorn and Wapiabi Formations of Upper Cretaceous origin. The downstream formations form the western arm of a broad syncline through which the canyon was downcut by valley glaciers, glacial meltwaters and the present river. Siting of the dam within the canyon locates it wholly downstream from the regional fault with only the impervious blanket extending onto the upstream formations.

The Blackstone Formation forms the upstream segment of the canyon and descends downstream below riverbed level beneath the Bighorn Formation which predominates within the abutment areas of the dam as shown on Figure 3. The contact between the two formations dips conformably with

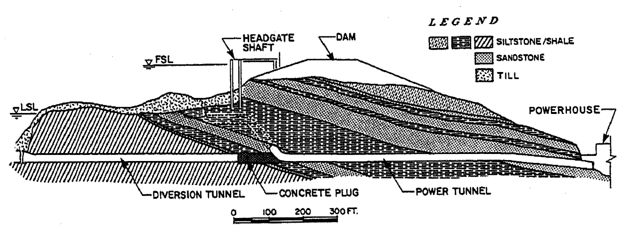


Fig. 3 Section through north abutment

the bedding at about 14 degrees downstream and is estimated to lie at a depth of between 40 and 70 ft below the riverbed surface at the location of the cut-off wall. The Blackstone beds of dark, fissile, often ferruginous shales and dark grey siltstones extend to heights of up to 165 ft above the valley floor at their upstream limit where they daylight within the reservoir. The overlying Bighorn Formation embraces three principal fine-grained grey sandstone strata separated by relatively massive beds of siltstone, while some thin partings of carbonaceous shale and conglomerate were encountered. The blocky sandstones of this formation constitute the hardest rocks in the project area. The Wapiabi Formation, which overlies the Bighorn on the downstream flanks of the canyon, consists of highly friable siltstones in a fractured and weathered condition.

The permeability of the rock formations was given particular attention in view of the fractures and disturbance evident in outcrops. However, while drill water losses were often experienced in the investigation program, the packer permeability tests indicated generally tight conditions, even within the brecciated zones along the fault planes and in the Wapiabi Formation. Bedrock permeabilities measured in both abutments generally increased from about 5×10^{-4} cm/sec near the surface to about 5×10^{-5} cm/sec at 260 ft depth, while tests at shallow depths in the valley floor gave about 1×10^{-4} cm/sec. The highest values, exceeding 1×10^{-3} cm/sec, were obtained in the higher strata near the abutment slopes.

The rock formations outside the canyon are capped by glacial drift comprising clean sands and gravels, silts, gravelly till and bouldery till. Though generally of shallow depth, the drift attains a depth of 165 ft on the north abutment within a linear depression in the bedrock surface. Seepage through this buried channel at higher reservoir levels was countered by blanketing the upstream exposure, constructing a series of shallow drains in the natural slopes downstream and installing three deep drainage wells collared at crest elevation.

The downcutting which formed the canyon extended through the cover of glacial drift, the Wapiabi and Bighorn Formations and into the Blackstone beds to a total depth of about 560 ft. Drilling in the valley floor indicated infill deposits extending to a maximum depth of 216 ft and consisting mainly of interbedded, river-sorted, silty to relatively clean sands and gravels in varying proportions, interspersed with bouldery talus from the canyon walls. It is noteworthy that the investigation of the bed was satisfactorily accomplished only with the advent of the Becker hammer drill, though it was difficult to appraise the consistency of the finer granular deposits and several lines of closely-spaced holes were needed to distinguish the massive boulders from the parent intact rock. The bedrock profile along the axis of the cut-off wall, and the boulders encountered, are shown on Figure 4.

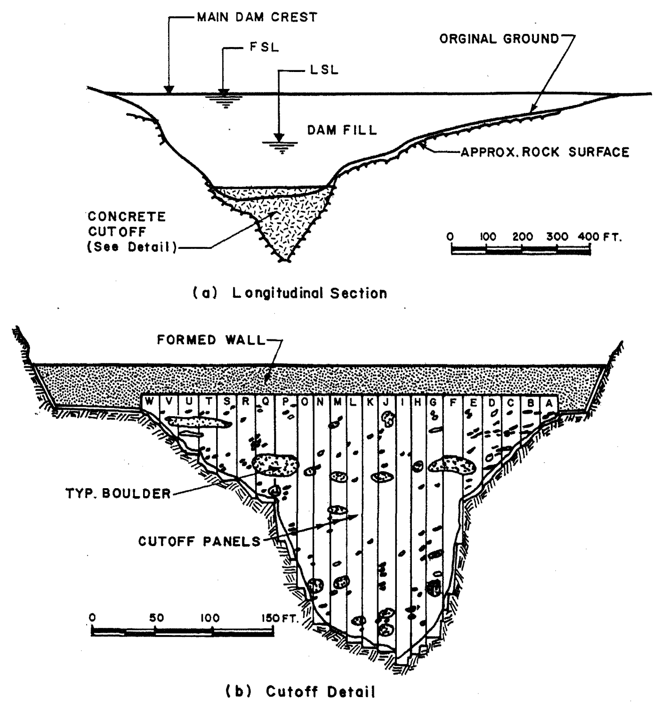


Fig. 4 Concrete cut-off construction

Extensive permeability testing in the riverbed deposits delineated three broad zones. In the uppermost zone, which comprises a 40 ft depth of silty sands and gravels, the horizontal permeability was found to be widely variable while the measured overall vertical permeability was 1×10^{-4} cm/sec. The underlying zone, which extends to 100 ft depth, consists mainly of uniform sands with some gravel and boulders. It was the most pervious zone within the depth tested, giving horizontal and vertical permeabilities of about 2×10^{-1} cm/sec and 5×10^{-2} cm/sec, respectively. The lowest zone extends to 140 ft depth and is composed of sand, coarse gravel and bouldery talus. The tests indicated overall permeabilities of roughly 1×10^{-1} cm/sec in the horizontal direction and 4×10^{-2} cm/sec in the vertical direction. The permeability of the bouldery deposits beyond 140 ft depth was not measured as a consequence of difficulties encountered in drilling through large boulders.

DAM DESIGN

The site was considered to be suitable for a zoned earthfill embankment structure founded on the riverbed deposits. The most critical factor in the design was the selection of the means of controlling seepage through the bed deposits, and detailed consideration was given to four alternatives, specifically, a grouted cut-off, an impervious upstream blanket, an excavated cut-off to bedrock and a concrete diaphragm wall keyed into bedrock. The use of a grouted cut-off was soon discarded due to uncertainties about its effectiveness and the high cost and time for implementation, while an upstream blanket was considered unsuitable in view of the limited space in the canyon for concurrent diversion of the river flows as well as the problems of ensuring an effective seal against the abutments. Although a core trench excavated to rock and backfilled with compacted impervious fill clearly constituted the most positive means to control seepage, its construction would have added at least six months to the construction schedule. Furthermore, its high direct cost would have been compounded by the added costs for related construction, such as a longer diversion tunnel, in comparison with a concrete diaphragm wall. The cost of dewatering alone was estimated to almost equal the cost of a diaphragm wall.

The relative advantages of a diaphragm wall were evident, particularly from a cost viewpoint. It could also be built during the winter, within a heated enclosure, and would not in consequence interfere with other construction activities or prolong the overall schedule. Nevertheless, the decision to adopt diaphragm wall construction was made only after extended deliberation of the possible adverse effects of anticipated settlements due to compression by the dam fill of loose zones in the foundation. There was concern that the settlements might be large enough to cause discontinuities in the wall sufficient to impair its effectiveness or within the impervious core as it settled downwards over the top of the wall. Another reason for hesitation was the limited precedent for the use of a concrete diaphragm cut-off wall under such high fill and hydraulic loadings.

The adoption of the diaphragm wall was based on the confidence expressed in its suitability by Dr. A. Casagrande who, together with Dr. R.M. Hardy, provided specialist geotechnical expertise to the project. To counter the effects of any disruption of the core arising from settlements relative to the wall, the wall was capped with a 30 ft wide galvanized steel plate surmounted by a 5 ft wide trench centred over the wall and filled with bentonite, as shown on Figure 5. As it was anticipated that differential heads across the wall exceeding 100 ft

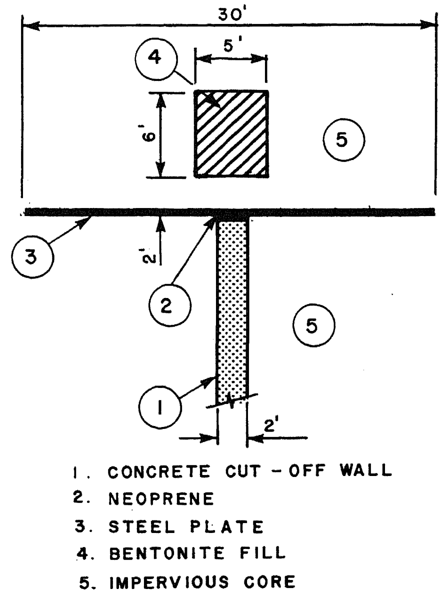


Fig. 5 Detail of cap at top of wall

would be sufficient to wash out the bentonite slurry bordering any imperfections or windows in the wall, the decision was taken to supplement the underseepage control by connecting the impervious core of the dam to the core of the integral upstream cofferdam, and to add a short upstream blanket.

Though the site offered an abundance of both impervious and pervious borrow, the zoning selected for the dam maximized use of the more readily available impervious glacial deposits within the core and inner shells, and limited the pervious materials, originating mostly from the riverbed, to the filters and thin outer shells. A filter of well-graded pit-run sand was provided between the core and the downstream common fill to protect against cracking the relatively brittle and low plasticity core material as a result of differential settlement. The dam was also arched upstream for the same reason.

DIAPHRAGM WALL CONSTRUCTION

The panel construction of the 2 ft thick cut-off wall is shown on Figure 4. Panel widths of 15 ft were used at the two ends of the wall where depth to bedrock did not exceed 120 ft and widths of 12.5 ft were used in the centre section where rock was deeper. The 15 ft wide panels were constructed in sequence, each being excavated through the bentonite slurry to bedrock by clamshell, and the excavation extended the requisite 2 ft into bedrock by percussion chisel. After clean-up a 2 ft diameter pipe was positioned vertically at the leading end of the excavation to prepare a concave interface for the next panel. The slurry was then displaced by tremie concrete with 3/4 in maximum size of aggregate and a mix designed for a 28 day strength of 4000 psi. The 12.5 ft wide panels, on the other hand, were constructed alternately. Guide holes were first drilled at the ends of an initial panel in order to maintain alignment, and the material between then excavated 2 ft into the bedrock as for the 15 ft wide panels. Concave interfaces were again left at the end of each panel.

Slurry losses within the more pervious deposits were reduced by the addition of sawdust and cellophane flakes. These additives proved insufficient in only one of the earlier guide holes where hole sealing was accomplished by backfilling with weak concrete. The guidehole was re-established by percussion chisel once the concrete had partially cured. Sloughing of the bed deposits at the top of panels N and M (Figure 4) led to the formation of a large bulb of concrete which was subsequently line drilled and severed from the wall.

Numerous boulders up to 18 ft in size were encountered and were either broken up by percussion chisel or drilled and blasted. To ensure that the wall penetrated bedrock and not boulders, soundings were made at frequent intervals and the rock cuttings were closely observed. Excavation was terminated at the specified 2 ft embedment depth only when bedrock had been positively identified. To enhance water tightness at the periphery, grouting was performed through vertical pipes embedded in the wall at 5 ft intervals.

Reinforcing steel cages were installed in the top 20 ft of the slurry displaced wall, and dowels were left projecting above its surface for juncture with the 25 ft high wall of conventional concrete that was later superimposed to extend into the impervious core of the dam. To further increase the seepage path a galvanized steel sheet 30 ft wide was installed on top of the wall after the latter had first been ground smooth and a thick neoprene pad added with rubber cement bonding to both concrete and steel. This is shown on Figure 5. Also to protect the impervious core against cracking near its base due to either arching or differential settlement a plug of soft bentonite paste with a dry density of 35 pcf was added in a trench 5 ft wide and 8 ft deep immediately above the wall during subsequent construction.

During the initial placement of core fill against the wall, a flow of water emerged on the riverbed downstream at the juncture of panels R and Q (Figure 4). The flow, originating as seepage through the upstream cofferdam, was attributed to an imperfect interface between the panels or possibly a window in the vicinity of the large boulder overlying bedrock in that area. The joint was thoroughly grouted from holes extending to the boulder. The grouting was apparently successful, because later it proved necessary to provide temporary drainage for water ponding at the same location on the upstream side of the wall.

TUNNEL DESIGN

The diversion tunnel had a length of 1700 ft and passed through the siltstone and sandstone strata of the Bighorn and Blackstone Formations in the north abutment (Figure 3). It was lined throughout with concrete 12 in thick. Later it was plugged with concrete at an intermediate point, and converted to a power tunnel with an inclined shaft sloping up to a surface intake from the downstream side of the plug. About 690 ft of the original tunnel downstream of the plug was utilized in the conversion. When completed the power tunnel had a total length of approximately 1350 ft and a finished diameter of 22.5 ft. The upstream 60% of its length has a concrete lining 24 in thick, and the downstream 40% (or 560 ft to be exact), where internal pressure exceeds the overlying weight of rock, has a steel liner.

To prevent the buildup of excessive hydraulic pressure behind the lining when the tunnel is unwatered, radial drain holes were drilled into the rock at several sections along the tunnel alignment during construction. The discharge from these holes, as well as any leakage from the tunnel, is collected in header pipes behind the lining and conducted to the tailrace. On the way the flow is measured by a venturi meter.

INSTRUMENTATION

To monitor the post-construction performance of the embankment and the seepage control components incorporated into the structure, piezometers, settlement gauges and movement hubs were installed during construction. A total of 46 pneumatic and standpipe piezometers were placed in the embankment, foundation alluvium and abutment to monitor piezometric head variation. Four settlement gauges were installed to measure the settlement of the foundation, embankment fill and cut-off wall respectively. Movement hubs consisting of 2 in diameter galvanized pipe 8 ft long were installed at 50 ft intervals primarily along the upstream edge of the crest to monitor dam displacement and settlement as a result of reservoir operation and embankment weight.

Upon first filling of the reservoir, when erratic drops in head were observed in the foundation piezometers and seepage was reported at the downstream toe, additional piezometers and seepage measuring weirs were installed downstream of the dam. In 1984 as part of the dam safety evaluation, an inclinometer was installed in the north abutment to monitor potential downdip displacement of the rock above the powerhouse.

DAM PERFORMANCE

Seepage - As indicated by flow measurements from Weir 1, located between the downstream toe of the dam and the tailrace (Figure 1), the variation of seepage flows with reservoir elevation since first filling of the reservoir is reasonably consistent indicating no major increase in seepage with time. However, since a significant portion of the total dam and foundation seepage is flowing through the river bed alluvium under the weir, it is possible that reasonably large variations in total seepage may occur before a change in Weir 1 flows is observed. Maximum seepage flows as determined from stream gauge measurements taken in the river channel downstream of the powerhouse when the tunnel headgate is closed have reached 28 cfs (Figure 6) whereas the peak flows recorded at Weir 1 have been about 3.5 cfs. The flow values for different reservoir levels given on Figure 6 represent stream gauge readings taken at least 8 hr after headgate closure to minimize the effect of riverbed and abutment storage outflow at lower river levels. However, outflow from riverbed and abutment storage is a significant component of the measured flows as indicated by the 1981 reading taken 21 days after gate closure.

Since the stream gauging station is approximately 0.5 mi downstream of the powerhouse, it was recognized that the measured seepage flows at this location included not only seepage through the dam and foundation but also headgate leakage as well as seepage through the abutments. To increase confidence in the measured values an estimate of seepage through the cut-off wall, i.e. through construction joints, cracks, etc., was made utilizing measured piezometric levels in the alluvium downstream of the wall, permeability values from pump tests and, since the bedrock channel below the riverbed narrows in the tailrace to about 1/2 the width of that at the cut-off, an average area of flow equal to 75% of the cut-off wall area below the river bed. The calculated seepage flow through the wall was thus estimated to be between 8 cfs and 13 cfs when the reservoir is at full supply level (FSL). To this was added leakage past the headgate, estimated at 7 cfs based on depth of water in the tunnel and the velocity of a floating wood chip observed during a tunnel inspection in 1984. An assessment of abutment leakage was more difficult to perform in view of the conflicting evidence available. The steep faces of the canyon walls downstream of the dam are visibly jointed and faulted but exhibit insignificant seepage above the

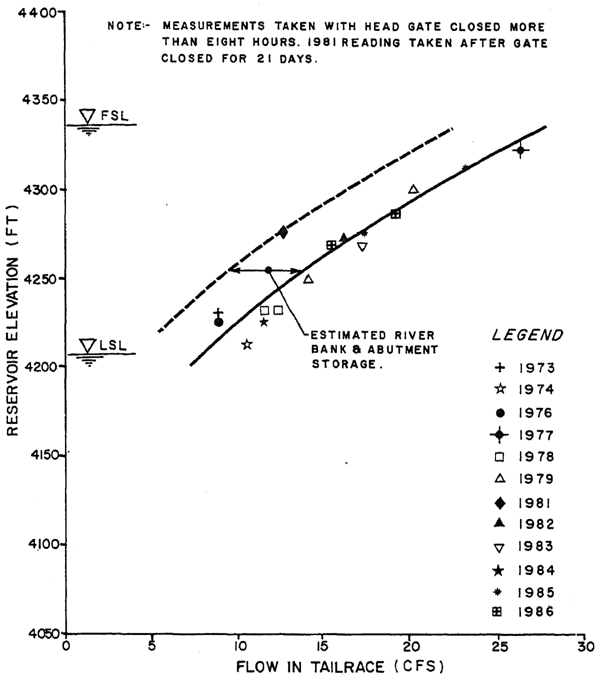


Fig. 6 Variation in seepage with reservoir level

river bed. Leakage from the diversion tunnel outlet in the north abutment rarely exceeds 0.5 cfs and piezometric heads within the left abutment rock remain well below FSL and are not significantly affected by changing reservoir levels. However, water losses were reported in a number of abutment drill holes and one hole required about 29 ft^3 of grout, bran, plaster of paris, calcium chloride and bentonite to seal the hole over a 100 ft interval. Allowing for random jointing, the total seepage through both abutments could be 2 to 3 times the quantity determined for intact strata and may be in the order of 6 cfs. Likewise an estimate of the leakage through bedrock below the cut-off contact was difficult to assess but, based on insitu pump tests, should be no greater than 1 cfs. It was concluded that the total calculated seepage from all sources when the generating units are not operating, the headgates are closed and the reservoir is at FSL was within the range of 18 to 27 cfs, which is comparable to measured flows in the tailrace after deducting the riverbed and abutment storage component.

Piezometric Records - Only five standpipe piezometers upstream of the cut-off have remained operational since 1972 to monitor piezometric heads in the river bed alluvium. Of these P25 is located 30 ft upstream of the wall and the tip is set approximately 60 ft below the original river bed (Figure 7). The other four, P45 through P48 inclusive, are collared on a berm on the upstream slope about 135 ft from the cut-off wall and are set just below the contact between the dam fill and the native gravels.

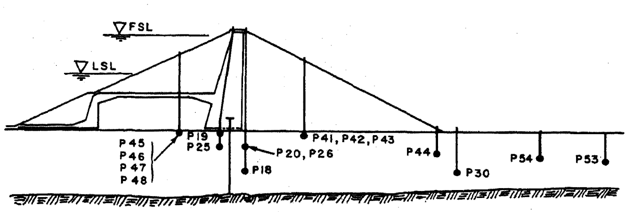


Fig. 7 Relative locations of foundation piezometers

Downstream of the cut-off eleven piezometers set in the alluvium are still functioning. Readings from P26, set closest to the wall and at about the same depth as P25, have been used, in conjunction with P25 readings, as an indication of the head drop across the cut-off.

As the reservoir was being filled in 1972, piezometric levels upstream of the concrete cut-off initially rose as expected in response to the rising reservoir level. When the reservoir reached a height about 150 ft above the river bed, however, P25 readings showed a sudden drop of 9 ft while P26 heads downstream of the cut-off rose about 3 ft. An additional drop of 15 ft in P25 and simultaneous increase of 4 ft in P26 occurred when the reservoir rose to 210 ft. At the same time, excessive seepage and sand boils were observed at the downstream toe of the embankment. The sudden drops in differential head were attributed at the time to "blow-outs" of the bentonite cake deposited in construction joints or other defects in the wall due to the high gradients across the wall. As a result of these observations and in accordance with the recommendations of the specialist consultants, a filter and buttressing berm 50 ft wide and 10 ft high was constructed along the downstream toe to enhance stability. Additional piezometers and a weir were installed adjacent to the downstream toe and a program of regular monitoring of all piezometers and seepage measuring facilities was instituted.

Over the years the piezometric levels in the alluvium immediately upstream of the cut-off gradually decreased until about 1982 when steady-state conditions were reached (Figure 8). Readings from piezometer P19,

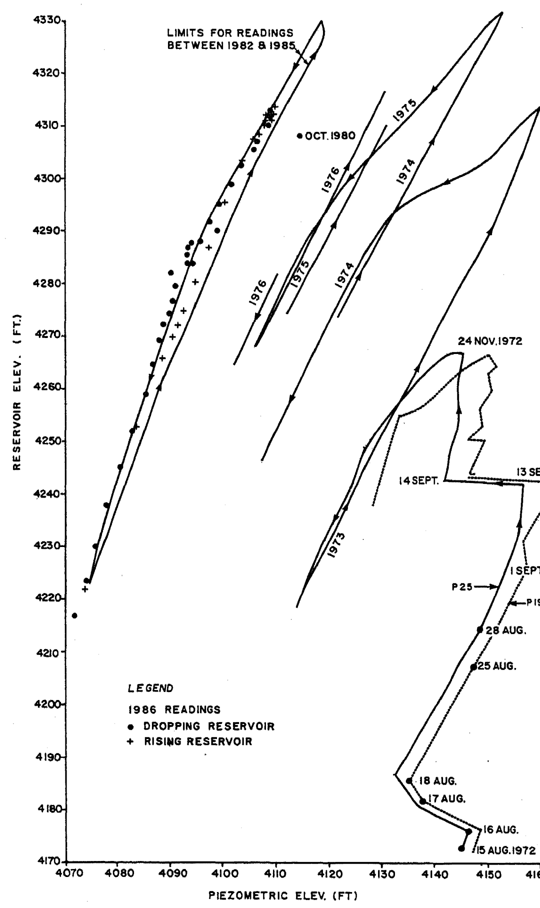


Fig. 8 Piezometric heads in P19 and P25

located about 30 ft upstream of the cut-off near the top of the river sediments about 100 ft south of P25, showed an almost identical variation in piezometric elevation as P25 until it became unserviceable in 1976. The records of these piezometers in the first few years of reservoir operation appear to confirm an abrupt and not insignificant reduction in the effectiveness of the concrete cut-off, resulting from either leaky construction joints in the panel wall, inadequate sealing of the wall at the bedrock contact or other imperfections in the concrete of unknown character.

Since about 1982, however, P25 readings have varied monotonously with reservoir level. This phenomenon is considered to be the result of either stabilized seepage conditions in the alluvium adjacent to wall apertures, or an increase in reservoir sedimentation or a combination of both effects. Records from piezometers P45, P46, P47 and P48, 100 ft farther upstream, are not particularly useful in explaining the current steady-state conditions across the wall. P46 readings indicate an hydraulic connection exists between this piezometer and P25, whereas piezometric levels in P45, P47 and P48 have remained relatively constant over the years (Figure 9). The linear relationship between P25 and P46 is not surprising as these piezometers appear from preconstruction river valley contours to lie in the main river channel. P45, P47 and P48, on the other hand, are located in the flood plain on either side of the main channel where less pervious sediments could be expected as a result of lensing and beaching effects.

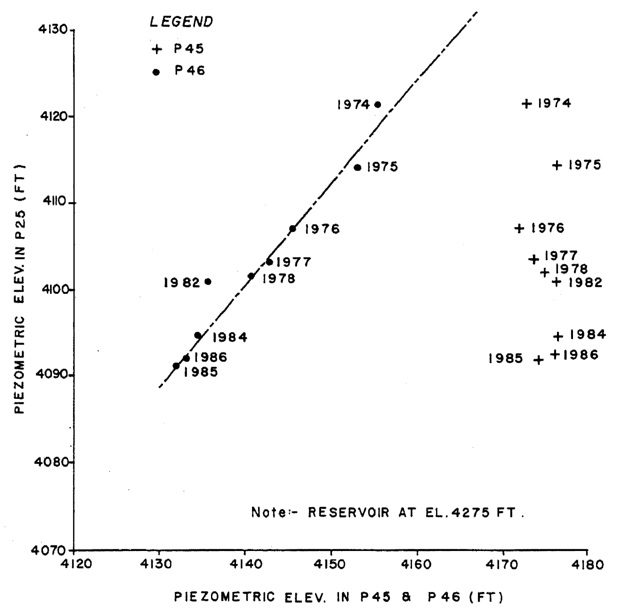


Fig. 9 Hydraulic connection between P46 and P25

An indication of the performance of the cut-off wall is illustrated on Figure 10. Except for erratic variations during the first few months following reservoir filling, the head drop across the wall has declined at a decreasing rate over the years until the early 1980's when conditions more or less stabilized (Figure 10a), varying only with reservoir elevation (Figure 10b). Downstream of the cut-off wall, the piezometric profile through the bottom of the chimney drain and underlying alluvium (Figure 11) indicates that hydrostatic conditions with heads 25 to 30 ft. higher than the tailrace level have prevailed in the alluvium since 1973. However, pneumatic piezometer G32, located in the chimney drain, denotes pressures in excess of hydrostatic suggesting the native sands and gravels are more pervious and acting as an underdrain. Piezometers downstream of P26 exhibit an average gradient over a distance of approximately 1300 feet between the cut-off and the tailrace channel of about 0.02, a value which has decreased only marginally since reservoir filling.

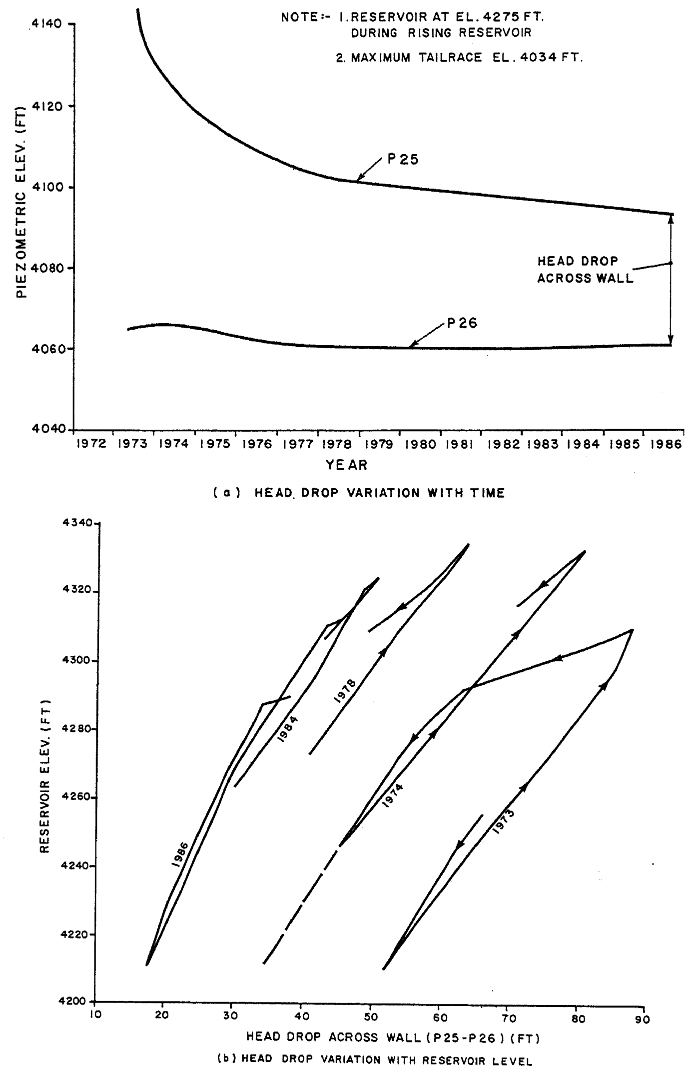


Fig. 10 Head drop across the cut-off

During initial reservoir filling, the impervious core of the main dam was not fully saturated and water from the pervious upstream shell and the foundation gradually seeped into the core as well as the common fill below the impervious blanket establishing a steady-state phreatic surface. Subsequent pressure reductions in the river gravel has allowed the foundation alluvium to act as an underdrain for the blanket and the impervious core (Figure 12). The hysteresis effects exhibited by pneumatic piezometer G37 also indicate the common fill under the upstream blanket is relatively impervious and the blanket is working satisfactorily (Figure 13).

Displacements - A survey carried out in 1984 on monuments installed along the dam crest indicated negligible downstream movement and a maximum settlement of 3.6 in. The crest profile remains from 6 to 12 in above design grade.

Settlement of the top of the cut-off wall, monitored with a settlement gauge installed during embankment construction, was reported to be 1.98 in at the end of construction (Gordon and Rutledge, 1972), with negligible settlement subsequently.

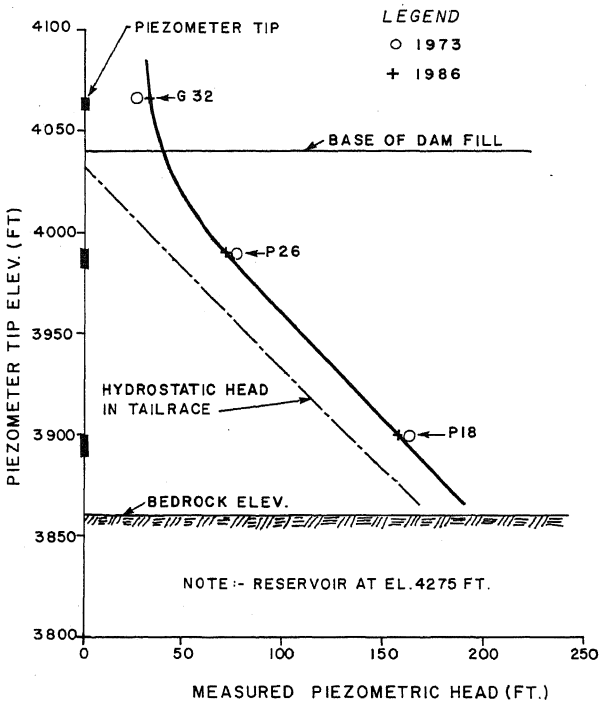


Fig. 11 Piezometric profile through alluvium downstream of cut-off

RESERVOIR SILTATION

To determine the extent of tightening of the lake bed upstream of the dam by siltation and thus ascertain whether the observed drop in piezometric heads in the alluvium upstream of the cut-off could result from reservoir siltation, an underwater survey was carried out by divers in the spring of 1986 at six locations in the reservoir below low supply level (LSL). The survey indicated that the accumulation of silt on the reservoir floor varied from 8 to 14 in with an average of about 12 in in the active reservoir area.

A rough estimate of the head loss through the silt blanket was made using D'Arcy's law as follows:

$$Q = kAi = kAh/t \quad (1)$$

where: Q = Quantity of seepage through the silt
k = Permeability coefficient for silt
A = Effective area of silt blanket contributing to seepage
i = Hydraulic gradient
h = Head drop across silt
t = Silt thickness

Assuming as a first approximation that the seepage through the alluvium and thus the silt layer varies as previously estimated from 8 to 13 cfs or about 10 cfs on average, the area of reservoir floor effectively contributing to seepage is, say, 1000 ft long by 400 ft wide, the permeability coefficient for the clayey silt is about $5x10^{-5}$ ft/min, and that the silt layer has a uniform thickness of 1 ft, the calculated head loss from Equation 1 is estimated to be about 30 ft when the reservoir is close to FSL. Although not a precise analysis, the calculated head loss demonstrates the potential of reservoir siltation for reducing piezometric heads in the pervious foundation. Whether, in fact, the actual head reductions observed are due primarily to siltation, to an imperfect cut-off, or to a combination of both effects, is not certain. Nevertheless, it is considered that the silt deposition to date has been instrumental in stabilizing seepage losses through the foundation alluvium and compensating for some leakage through the concrete cut-off wall.

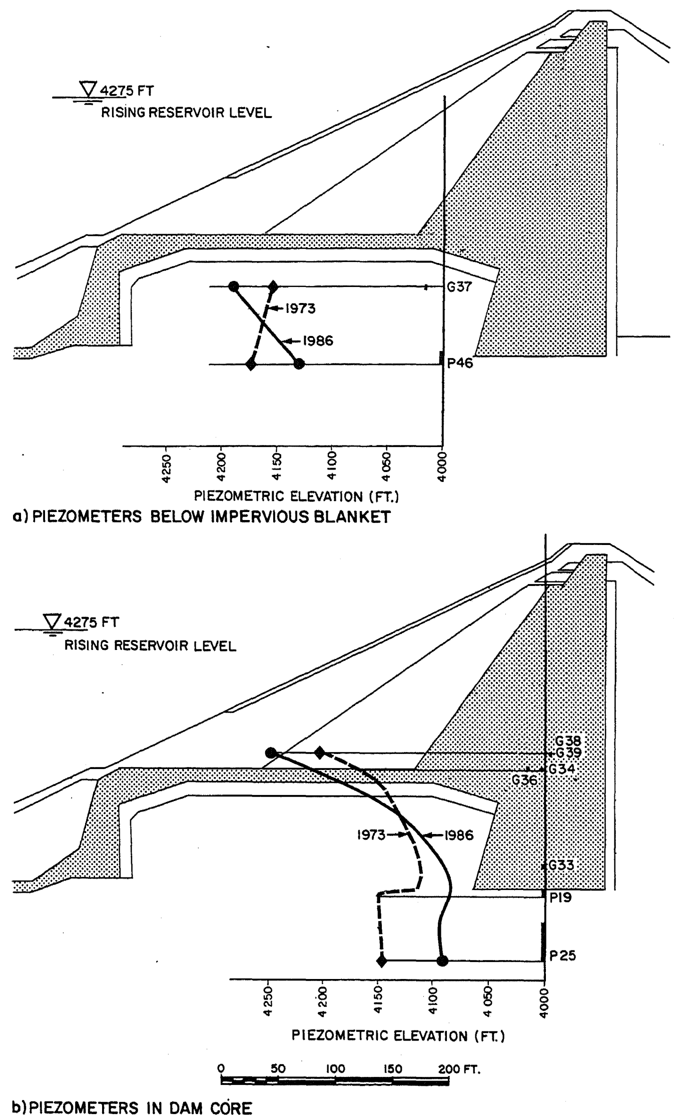


Fig. 12 Piezometric profile in embankment upstream of cut-off

TUNNEL PERFORMANCE

Periodic inspections of the tunnel when it is unwatered for routine plant maintenance indicate the steel lined section is in good condition. On the other hand some cracking of the concrete lining has occurred near the foot of the inclined section leading down from the headgate. Three cracks, located in the 10 o'clock, noon and 2 o'clock positions, have been observed extending downstream for a distance of 120 ft. The maximum crack width was about 3/16 in. Measurements with a lead scribe installed across one of these cracks in 1986 indicate that the annual variations in tunnel pressure (170 to 300 ft head change) and water temperature (range 32° to 55°F) are sufficient to cause the crack aperture to open and close by about 1/8 in from that observed when the tunnel is unwatered, giving a total movement of 1/4 in.

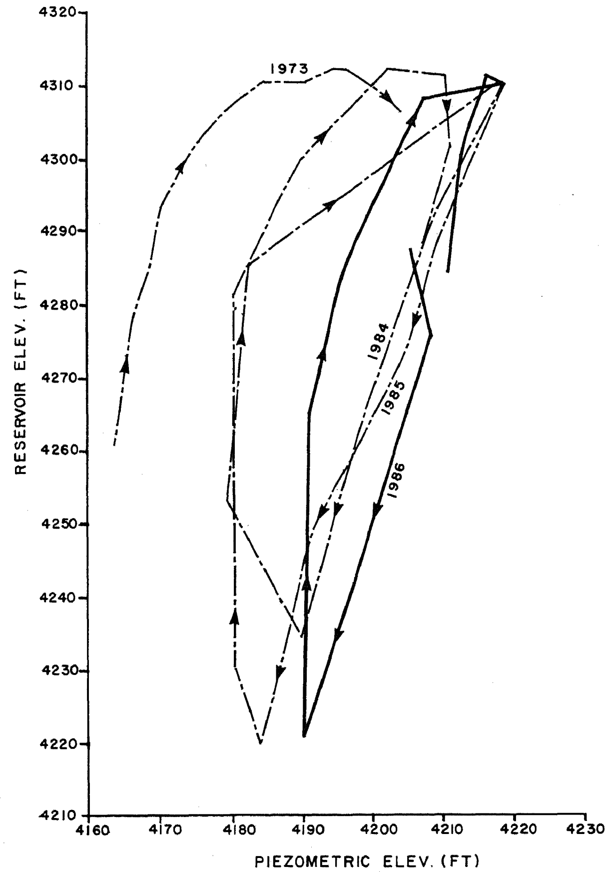


Fig. 13 Hysteresis effects in G37

Discharge from the tunnel drainage system when the tunnel is pressurized and the reservoir is at FSL is approximately 1800 Imperial gallons per minute. The discharge reduces the zero when the tunnel is drained signifying minor storage in the surrounding rock. In addition there appears to be no hydraulic connection between the tunnel drainage system and the river bed alluvium through faults and joints in the abutment rock. When the tunnel was unwatered in late 1972, the pressure head in the rock surrounding the tunnel dropped about 90 ft to Elev. 4095 ft as recorded in nearby piezometer P10 (Figure 14) whereas the piezometric heads in the river gravels remained within the range of Elev. 4148 (P19) to Elev. 4168 (P46).

DAM SAFETY EVALUATION

The safety evaluation of Bighorn dam was carried out in 1984 as part of a staged evaluation program for thirteen different projects comprising the hydroelectric generating system owned and operated by TransAlta Utilities Corp. (Wade et al, 1985). The evaluation was conducted in accordance with the Dam and Canal Safety Guidelines established by the Government of Alberta (1979). The guidelines designate, for new and existing dams, design floods ranging from full PMF for high hazard structures to 1 in 100 year floods for small storage ponds. These guidelines are similar to those adopted by other regulatory agencies such as the U.S. Corps of Engineers (1979) and the British Institution of Civil Engineers (1978). Present day river basin use and the more stringent safety requirements have resulted in a situation where many existing dams and spillways no longer exhibit acceptable factors of safety when considering the PMF design requirements.

The safety evaluation procedures used at Bighorn consisted of the following (Monenco, 1986):

Review Existing Data - The object of the file search was to obtain the original information on construction materials, geological assessment of foundations, design calculations and notes, drawings and reports that were developed during construction. These records were then reviewed together with a summary of the maintenance history.

Inspect Site - Site inspections were carried out systematically using prepared check lists with emphasis on problems or specific areas of concern as established through the file search and discussion with owner's staff. Surveys and measurements were undertaken to obtain structural dimensions and verify information on the construction drawings.

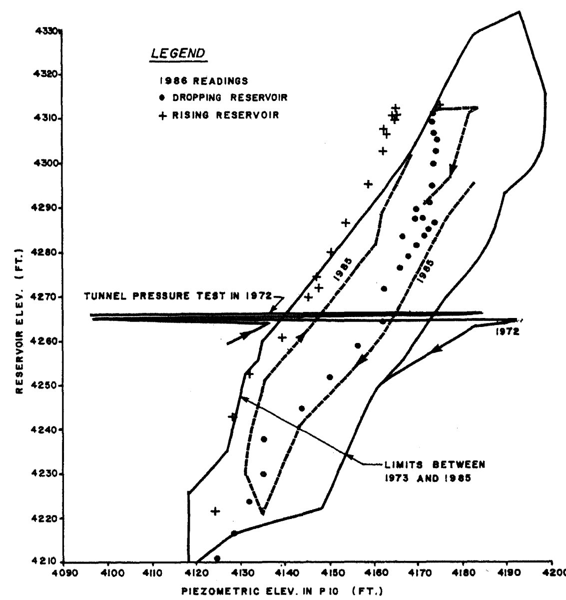


Fig. 14 Piezometric heads in north abutment

Exploratory Drilling and Monitoring Installations - The terms of reference established by the Safety Guidelines require that geotechnical properties and condition of embankment and foundation be verified by testing and instrumentation unless sufficient data exist to support the chosen parameters. Since considerable geotechnical information in the form of studies, reports, foundation investigations and construction records was available on file, drilling was limited to one hole in the north abutment for inclinometer installation and recovery of rock cores for strength testing. The objective in monitoring is to detect both gradual and rapid change of condition so that appropriate preventative action may be taken. Through the process of check lists, site inspections and evaluation of instrumentation records, long term performance was assessed. Leakage flows and piezometric levels, when plotted against corresponding reservoir levels, gave a good indication of the health of the dam.

Hydrology and Hydraulics Studies - Analyses of the river basin hydrology included a review of the reservoir operating criteria and available stream-flow records. Hazard potential for the project was assessed by carrying out routing studies for various floods up to the probable maximum flood (PMF) and by evaluating flood action plans.

Hydraulic studies were then made in conjunction with flood routing analyses to determine the capacity of the existing spillway structures. Using the spillway discharge rating curve, the head-discharge-load relationship for the turbines and the established operating procedures, it was found that the reservoir retaining structures would not be overtopped during the PMF.

Geotechnical and Structural Analyses - Stability analyses were carried out on each major structure considering records of seepage flows, piezometric and uplift pressures, geotechnical strength parameters and updated hydrologic criteria. Finally, an evaluation was made of the structural integrity of all concrete structures, including spillway gates, hoists and stoplogs to ensure each was sound and in satisfactory working condition.

Report - A dam safety evaluation report was prepared for the project that itemized the deficiencies found, recommended modifications or remedial work, revised operational practices, additional monitoring requirements and the frequency of future inspections. To ensure continued safety against unexpected failure and provide an early warning of deteriorating conditions in the future, critical monitoring procedures were outlined and "alarm" criteria established for piezometer levels and seepage flows.

CONCLUSIONS

As a result of the annual reviews of the piezometric and seepage data and the 1984 dam safety evaluation, it was concluded that the performance of the main dam and concrete cut-off-wall is satisfactory. Because of the magnitude of the seepage flows, however, it was recommended that readings of all monitoring installations be continued on a regular basis, that routine inspections of the facility be carried out by plant operating staff to ensure early detection of potential distress, and that a thorough inspection by professional engineers experienced in the design, construction and performance of earthfill and concrete dams be conducted every five years.

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