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SYNOPSIS: A case history of hydrostatic pressure development along the soil-structure interfaces of a water retaining structure is discussed in this paper to illustrate the use of instrumentation to verify expected performance during construction. For the project described, the development of hydrostatic pressure along the soil-structure interface during and after head pond watering was monitored using pneumatic piezometers. Monitoring of the piezometers detected a high hydrostatic pressure caused by a leaky contraction joint seal. Subsequent repair of the seal reduced water levels along the interface to expected levels, resulting in successful operation of the facility.

INTRODUCTION

The recently completed Pontook Hydroelectric Project is located north of Berlin, New Hampshire on the Androscoggin River. The project was was designed by E.C. Jordan Company and was constructed by C-E Hydro Power Systems, Inc. the project turnkey contractor. The project included construction of:

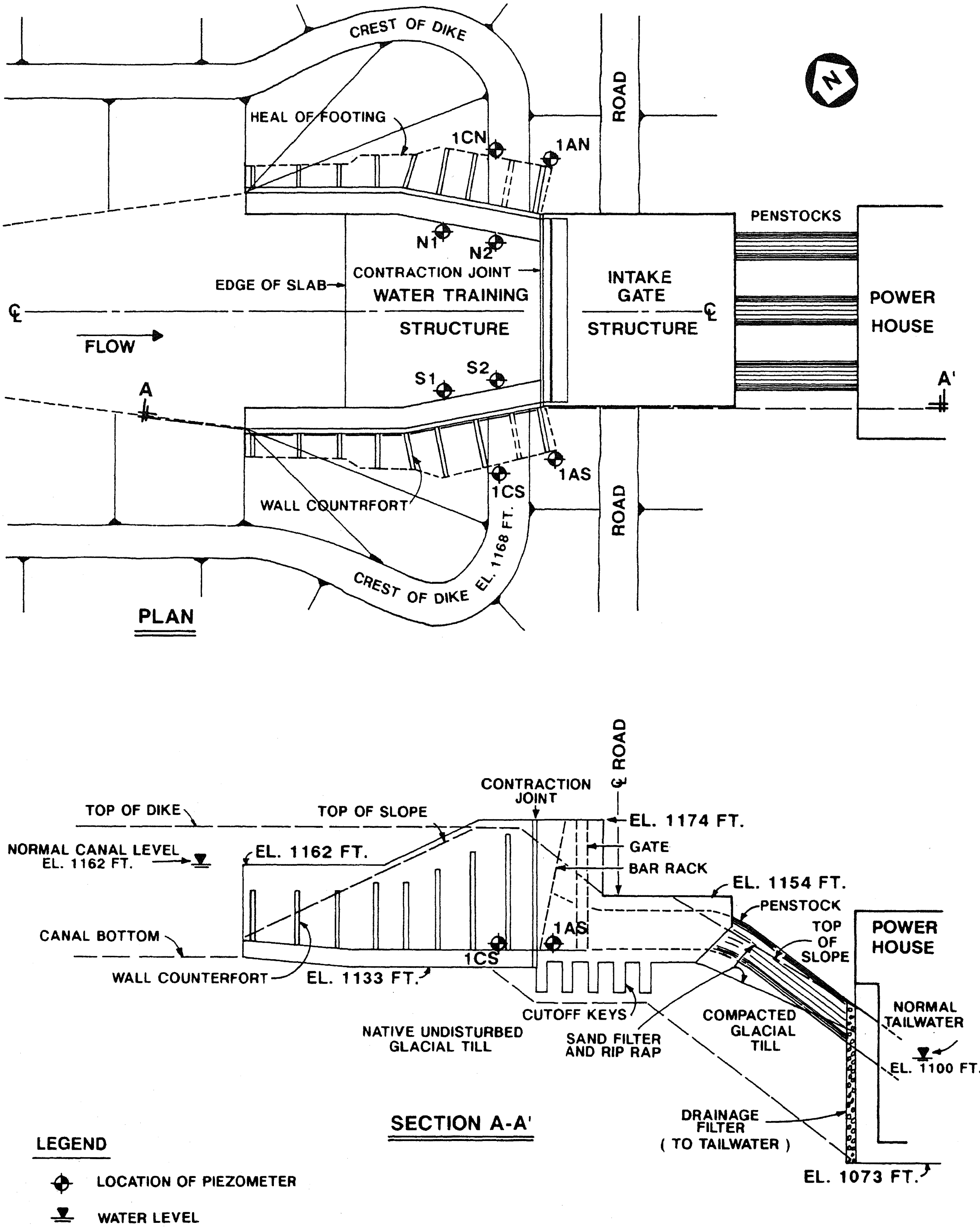
- an 800 foot (244 m) long timber crib and earthen embankment dam retaining a head of about 10 feet (3 m);
- a concrete canal headworks structure;
- a 6,000 foot (1,829 m) long unlined power canal consisting of an earth cut up to 70 feet (21 m) deep as well as side hill diked portions at the headworks and at the approach to the powerhouse penstock intake structure;
- a concrete penstock intake structure which trains water from the diked canal into three short steeply sloped 8-foot (2.5 m) diameter penstocks; and
- a concrete powerhouse containing 3 CE/ Neyrpic tubular turbines with a total installed capacity of approximately 10 MW.

This article focuses on the development of the hydrostatic pressure along the soil-structure interface between the canal dikes and the penstock intake structure. A schematic plan and sectional view depicting the configuration of the soil-structure interface along the penstock intake structure are presented as Figure 1.

SITE AND PROJECT DESCRIPTION

The Pontook Project is situated in the Androscoggin River Valley, located in northern New Hampshire. The area is a mountainous region characterized by northeast trending bands of metamorphic rocks lying in tight folds, produced by past mountain building events. The most significant of those events was the collision of the North American and European crustal plates, some 350 million years ago. More recently, during the Wisconsin glaciation, regional ice flow was sustained in the Pontook project area, with outwash accumulating further downstream. The ice eventually retreated in this valley and left sequences of stratified ice contact, outwash, and glacial till deposits above bedrock. Borings made for site development indicated the penstock intake area was underlain by a thick deposit of glacial till which was interbedded with braided gravel outwash deposits of up to a few feet in thickness. Bedrock was encountered over 150 feet (46 m) below the existing ground surface by exploratory borings.

The glacial till at the site was found to be a non-plastic, unsorted mixture of silt, sand, gravel, cobbles, and boulders which was widely graded with from 25 to 35 percent by weight passing the No. 200 sieve and had a coefficient of uniformity of more than 10. Although the native deposits of the glacial till contained some boulders greater than one cubic yard (1 cu. m) in size, no particles over eight-inches (20 cm) in diameter were permitted in the glacial till which was reused for construction of dikes and structural fills. Testing of the glacial till included a long term (two month) constant head permeability test in a fixed wall permeameter done on a remolded sample, the results of which indicated the glacial till had a hydraulic conductivity (permeability to water) of about 5 x 10^{-8} centimeters per second (1 x 10^{-7} ft/min.) at a dry density of about 125 pounds per cubic foot (1,204 Kg/m^3) or about 92 percent of maximum dry density. Other testing included pinhole dispersion testing, which indicated the till to be non-dispersive (not highly piping susceptible), however, observations on site indicated the till would generally ravel and erode when unconfined, until a self-



NOTE: DRAWING NOT TO SCALE

Fig. 1 Plan and Section of Penstock Intake Structure armoring of till stones prevented further erosion.

The penstock intake consists of two parts. These two parts are a water training structure and a gate structure. The training structure consists of two 25-foot (8m) high reinforced concrete retaining walls and a base slab which act as a transition from the earth-dike portion of the canal to the intake gate structure. The gate structure is also constructed of reinforced concrete, and in addition to providing gate control of the downstream terminus of the canal, it also acts to channel water from the training structure into three 8 foot (2.5 m) diameter penstocks. The training and gate portions of the penstock intake structure retain a 25 foot (8 m) of water. The two portions were designed structurally separate using a contraction joint to reduce adverse stresses which could develop in an otherwise monolithically constructed single structure. This contraction joint was designed to be water tight, using fused center bulb water stops and special caulking. By design, it is imperative that the joint not leak because of the relatively large retained head of water and the short distance, about 10 to 50 feet (3 to 15 m) from the contraction joint to the downstream side of the abutting dike slope. If significant leakage were to occur, the potential for piping along the soil structure interface would be significantly increased, with the possible result of a breach of the canal dike. As a result, it was determined that careful monitoring of the hydrostatic pressure development along the soil-structure interface during and after canal filling was necessary.

The penstock intake structure was constructed on undisturbed and recompacted glacial till as shown on Figure 1. The canal dikes were constructed of recompacted glacial till excavated from the cut areas of the 6,000 foot (1829 m) long canal which channels water to the penstock intake. These dikes directly abut the penstock intake forming a soil-structure interface. The penstock intake was designed to have piping resistance by counterforting the structure side walls and provision of cut off keys along the base of the structure (see Figure 1), thus providing a serrated interface with the dike and base soils and effectively increasing the seepage path. Because of concern for piping potential along the soil-structure interface and the large head drop (62 feet or 19 meters) between the canal and tailwater, four pneumatic piezometers (two on each side of the intake structure, as located on Figure 1, denoted as 1CN, 1AN, 1CS, and 1AS) were installed to monitor the development of the hydrostatic pressure across the soil-structure interface during and after initial canal watering. The piezometers were the double tube type, with a pressure range of 0 to 60 pounds per square inch (0 to 4.2 Kg/cm^2), actuated and monitored using dry nitrogen gas from a portable indicator box.

CONTRACTION JOINT SEAL

The penstock intake was constructed in numerous concrete pours during the late summer and early fall of 1986. After concrete placement for the training walls and intake structure had been completed and form work stripped, inspection of the contraction joint revealed that the fused center bulb water stop had apparently slipped out of position during concrete placement, having been cast parallel to the contraction joint rather than across the joint. This slippage of the water stop resulted in a non-positive seal against water passage through the joint. To rectify the situation, an exterior seal was designed and retrofitted on the water side of the contraction joint. The retrofit seal was composed of a Hyplon strip which covered the joint, held against the concrete and the joint by adhesive and plate metal strips which were through-bolted into the concrete. A protective steel shielding was secured over the seal to protect from abrasion by waterborne debris and the teeth of an adjacent trash rack cleaning rake.

INITIAL FILLING

After placement of the retrofit seal, the training walls and gate structure were partially filled with water for the first time. This initial filling was done without filling the entire canal by constructing an earthen cofferdam across the canal just upstream of the intake structure. The canal area between the cofferdam and intake structure was thus a relatively smaller, confined head pond of water for initial watering of the intake which could be drawn down rapidly if need be. This confined headpond area was filled with water to elevation 1156 feet (normal canal level is 1162 feet) over a period of about 36 hours. Elevation 1156 feet is equivalent to about 17.5 feet (5 m) of water in the headpond. As the headpond was filled, pneumatic piezometers outside of and adjacent to the contraction joint (piezometers 1AN and 1AS) registered pore pressure readings equivalent to the rising headpond level. Because of the potential of piping which could otherwise have occurred if the head condition were allowed to continue, the decision was made to immediately empty the confined headpond area and inspect the contraction joint rather than to continue filling the head pond to proposed normal levels. During drawdown of the headpond, which was dewatered at a rate of about 1 foot per hour (0.3 m/hr), the piezometers indicated a direct response with the decreasing water level. At all times during filling and dewatering, the other piezometers (piezometers 1CN and 1CS) registered pore pressures well below the headpond level, indicating seepage along the sides of the structure were not likely responsible for the high readings of piezometers 1AN and 1AS. After dewatering the headpond, careful inspections were made of the seal in an attempt to determine the cause of the apparent leakage. The inspections indicated that areas where the concrete surface was irregular, including the corners where the retaining walls meet the base of the structure, were not adequately sealed. Dye was injected on the soil side of the seal in an effort to see if the zones of leakage could be positively identified. The dye came through the seal in numerous locations indicating an inadequate seal. Prior to identifying the leaky seal, there was concern that underseepage below the structure may have occurred, contributing to the high measured pore pressures. Down stream drains and ditches located at the toes of the canal dikes were inspected to see if and observable piping had taken place. No evidence of such piping was noted. Further investigation of underseepage was done utilizing pump tests. These tests were conducted by digging sump pits at the upstream edge of the water training structure base slab, followed by continuous pumping of the pits. The piezometers were monitored before, during and after the pumping to see if a response of the piezometers could be observed, which would indicate that underseepage might have occurred. The tests indicated no response. Based on all the observations, it was concluded that the cause of the high measured pore pressure adjacent to the contraction joints was leakage of the contraction joint.

The retrofitted seal was redesigned and repaired to accommodate the difficult sealing conditions. The revised seal was essentially identical to the original retrofit seal except that a 1/4-inch (0.6 cm) thick hydrophylic membrane material was placed behind the steel plate strips. This material swelled to almost double its original size in the presence of water, forming a positive seal between the concrete, Hyplon and the metal strips. To monitor potential development of hydraulic pressure which could be an indication that underseepage below the structure may have developed, four additional piezometers (numbered N1, N2, S1, and S2) were installed through the base slab of the training structure, located as shown on Figure 1, to measure hydrostatic pressure below the structure.

HEADPOND REFILLING

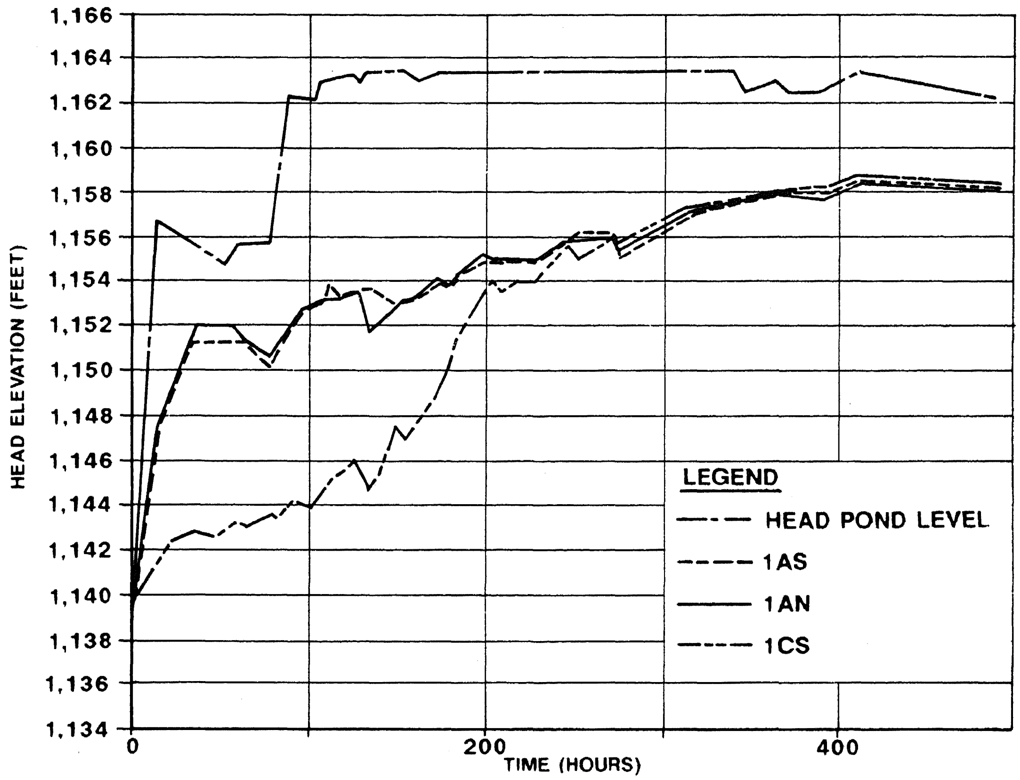
After the modifications to the retrofit seal were made, the canal was refilled at an average rate of about 1.5 feet per hour (0.5 m/hr) to an elevation of about 1156 feet. During this refilling, the piezometers readings were monitored about once per hour, however, piezometers 1CN and N1 were malfunctioning and did not provide usable data. Readings of the functioning piezometers during the first 500 hours are presented on Figure 2. During the refilling, hydrostatic pressures as indicated by the piezometers remained below headpond level. Piezometer 1CS registered a head unexpectedly less than 1AS and 1AN, probably due to its somewhat more isolated location within the glacial till beside the structure and away from the flow path of least resistance which is likely to be along the bottom of the water training structure and then along the sides of the gate structure. After filling the confined headpond to elevation 1156 feet, refilling was terminated and the elevation of the headpond was monitored with time over a 40 hour period. During this hold period, the piezometers initially indicated continued increase in the hydrostatic pressure at the soil-structure interface, generally followed by a leveling off or decrease in pressure. Also during the hold period, the headpond level decreased by 1.8 feet (0.5 m). The volume of water lost as indicated by the headpond level reduction was of the same magnitude as would be expected by the estimated normal gate leakage, so it appeared that leakage through the contraction joint had been substantially eliminated, reducing the previous concern that underseepage and piping may have developed during initial filling. After the hold period, head pond filling continued to about elevation 1162 feet, which was achieved about 100 hours after refilling of the headpond began. At this time, the cofferdam across the canal was removed.

The piezometers were closely monitored until it became apparent that steady state conditions had been achieved and that the contraction joint was not leaking. The hydrostatic pressure along the soil-structure interface, as indicated by the piezometers, appeared to come into equilibrium (indicating steady state flow) about 400 hours (17 days) after the headpond was filled, except for piezometer 1CS, which continued to rise for about 1100 hours, finally equilibriating about a foot or so in hydrostatic head above 1AN and 1AS. The final equilibrium levels of the hydrostatic pressure were at the approximate elevations predicted by steady-state seepage through an earthen embankment of similar profile to the soil-structure interface, however, this equilibrium occurred many magnitudes faster than would be indicated by the measured laboratory permeability (about 10^{-8} centimeters per second), indicating the permeability along the soil-structure interface is likely about 10^{-3} to 10^{-4} centimeters per second (2×10^{-3} to 2×10^{-4} ft/min). The piezometer readings were monitored frequently for a six month period to assure the steady state conditions remained constant, and that seal leakage did not restart. Currently monitoring is done on a seasonal basis by the station operators with no change from the expected steady state conditions as yet observed.

CONCLUSIONS

The following conclusions were made as a result of the experience of the Pontook penstock intake structure:

- water retaining structures with joints requiring mechanical seals and whose stability is sensitive to piping should be equipped with a means of determining if such seals are functioning as designed;
- such seals should be initially tested under controlled circumstances (a situation where drawdown can be implemented quickly, if necessary, is desirable);



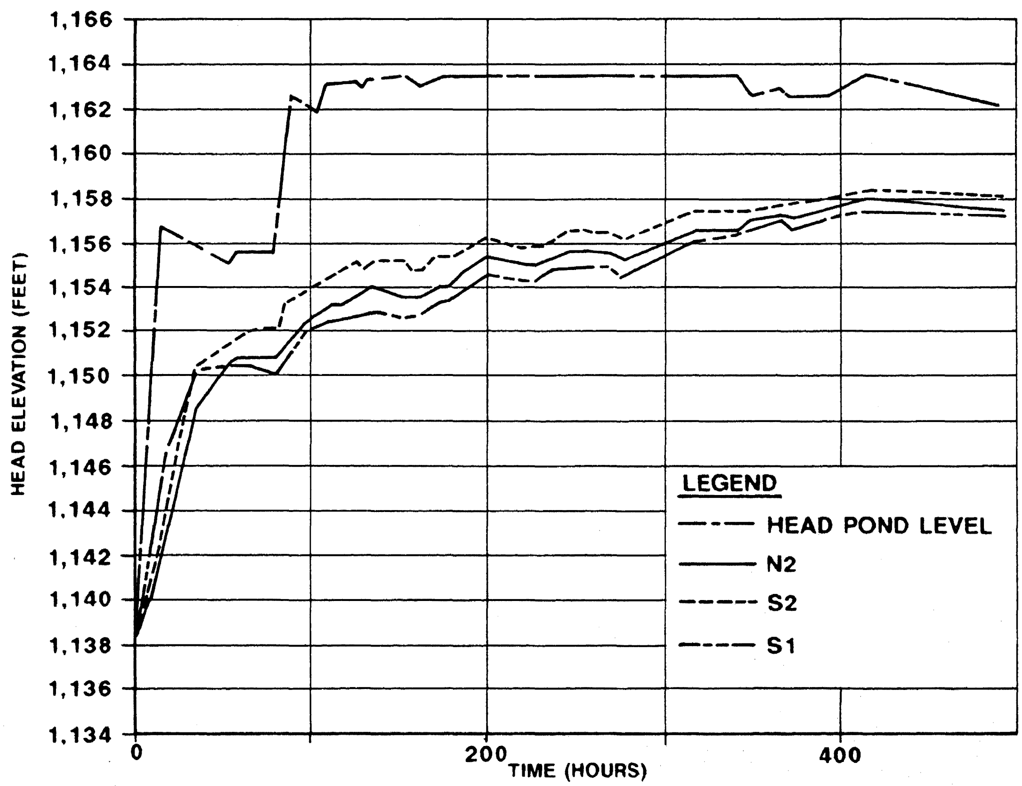


Fig. 2 Piezometer Response vs Head Pond Level After Final Seal Repair

- adequate redundancy in instrumentation (Piezometers, in this case) should be provided as a safe guard against inevitable malfunctions; and

- under normal circumstances (no leakage at the contraction joint) the development of the hydrostatic pressure at the penstock intake along a soil-structure interface appears to develop significantly faster than would be indicated by the permeability of the soil placed at the interface.