Construction and performance monitoring of a new remedial dam in northwestern Ontario

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ABSTRACT
The remedial dam, located in northwestern Ontario, is one of a series of block dams constructed for retention of the headwater. It was originally constructed in the 1950’s as an earthen retention dam in a rock saddle partly filled with 0 m to 9 m overburden of firm fissured Lake Agassiz clay overlying alluvial and fluvial silty sand, gravel, cobbles, and boulders over a granitic bedrock. In 2014 the dam experienced a slump movement on the upstream slope of the eastern section of the dam. A previous paper presented at GeoNiagara 2021 discussed the original construction, site investigations, and detailed design of the remedial dam. This present GeoCalgary 2022 companion paper discusses the instrumentation, construction using the Observational Approach, select construction challenges and the solutions that were developed and implemented. Construction was completed and the new dam was commissioned in Fall 2019. Construction and post-construction instrumentation monitoring during operation is presented for nearly three years of data.

RÉSUMÉ
Le barrage correctif, situé dans le nord-ouest de l'Ontario, fait partie d'une série de barrages-blocs construits pour retenir le cours supérieur. Il a été construit à l'origine dans les années 1950 comme un barrage de rétention en terre dans une selle rocheuse partiellement remplie de morts-terrains de 0 m à 9 m d'argile ferme fissurée du lac Agassiz recouvrant du sable limoneux alluvial et fluvial, du gravier, des galets et des rochers sur un substrat rocheux granitique. En 2014, le barrage a connu un mouvement d'affaissement sur le versant amont de la partie est du barrage. Un article précédent présenté à GeoNiagara 2021 a discuté de la construction originale, des enquêtes sur le site et de la conception détaillée du barrage de réparation. Le présent document d'accompagnement de GeoCalgary 2022 traite de l'instrumentation, de la construction à l'aide de l'approche observationnelle, de certains défis de construction et des solutions qui ont été développées et mises en œuvre. La construction a été achevée et le nouveau barrage a été mis en service à l'automne 2019. Le suivi de l'instrumentation de construction et post-construction pendant l'exploitation est présenté pour près de trois ans de données.

1 INTRODUCTION
This paper is a continuation of the companion paper written for GeoNiagara 2021 (Kurz et al. 2021) that discussed the original 1950’s construction, site investigations and design of the remedial dam. This paper discusses select challenges encountered during construction and solutions of the remedial dam. Monitoring during construction and post-construction is presented.

The dam site is in Northwestern Ontario situated within the Superior Province of the Canadian Shield. Regional bedrock was identified as massive granodiorite to granite. Topography throughout the Canadian Shield is dominated by bedrock highs, often near or exposed at the surface, and overburden filled lows. The overburden soils are relatively thin except in low bedrock areas or beside bedrock ridges and generally consist of glaciolacustrine, glaciofluvial, and fluvial deposits along with organics.

The original dam was built in the 1950's as a zoned embankment dam with an inclined clay core. The dam was approximately 360 m long with a maximum height of 8.5 m and a crest width of approximately 4.6 m. The average crest elevation was approximately El. 320.56 m, and the average foundation and toe elevation was between El. 312.0 m to 313.5 m, climbing at the abutments.

Based on previous Dam Safety Periodic Reviews (DSPR), the downstream stability of the original dam did not meet Ministry of Natural Resources and Forestry (MNRF) 2011 dam safety criteria. In August 2014, a slump in the east section upstream slope of the dam unexpectedly occurred, making the investigation, analysis, and remediation of the dam a priority. Immediate dam safety actions implemented by the owner included increased surveillance and inspections including installation of remote camera monitoring, review of instrumentation data, and seepage monitoring.

The owner investigated several options, as discussed in Kurz et al. (2021), before initiating the design for a new downstream remedial embankment and a new impervious core of the dam in March 2017. The remedial design involved construction of a zoned embankment dam with a 1 m wide cement-bentonite wall (CB Wall) vertical seepage cut-off designed to make positive contact with the bedrock foundation. The CB Wall was designed to act as the new primary seepage cut-off through the embankment and the foundation overburden soils. The upstream slope of the original dam was stabilized by offloading it to below the reservoir level, and re-grading the slope as part of the remedial work. Overall, the remedial dam was designed to meet MNRF (2011) dam safety criteria. Figure 1 illustrates a plan view of the remedial embankment dam constructed at the toe of the original dam. Figure 2 illustrates a general cross-section of the remedial dam and the offloaded...
original dam. Figure 3 illustrates a general profile along the centreline of the CB Wall core alignment.

In May 2019, construction activities commenced, and substantial completion was achieved in late November 2019. Impoundment to the full supply level (FSL) of the reservoir was achieved in early December 2019 allowing the owner to resume normal operations since the reservoir was lowered in 2014 to minimize load on the dam. The site was actively monitored during construction and remains monitored for post-construction dam safety performance.

2 ESTABLISHING SAFETY AND MONITORING PROTOCOLS DURING CONSTRUCTION

Construction of the remedial dam involved excavation and preparation of materials at the toe of an in-service dam that
had experienced a slump movement. Considering the presence of the slump on the existing structure, dam safety during construction was a major concern. As a result, several protocols were implemented to monitor the performance of the existing embankment for signs of movement that would initiate emergency protocols.

2.1 Stockpile Material

In early 2019, the contractor established a 1 ha laydown area a few hundred meters from the dam site to stockpile the material being produced at the sand pit approximately 2 km south of the dam site and the quarry approximately 14 km south of the dam site. The contractor was required to haul a minimum of 50% of the estimated total rock fill material contract quantities prior to conducting any excavation or foundation preparation work at the site as stipulated in the specifications.

This requirement was to ensure material was on-site to enable the owner’s engineer and the contractor to react and mitigate any emergency stability or seepage problems during the initial excavation and foundation preparation phases for the remedial dam construction.

2.2 Telltale Strips and Survey

Prior to commencement of any construction activity at the site, the contractor installed five (5) concrete telltale strips on the downstream slope of the original dam as a dam safety monitoring element. The strips were constructed by pouring a low-slump concrete on the slope using an excavator and bucket.

The purpose of the telltale strips was to provide a means to visually detect potential adverse movement of the original rockfill dam. That is, if the original embankment exhibited movements induced by construction activities in the downstream toe area, the movement would manifest as cracks or deformations within the telltale strip. Figure 4 illustrates an example telltale strip. These strips were visually inspected daily throughout construction, until construction of the remedial embankment buried the strips.

In addition, the contractor, regularly surveyed the pre-existing monitoring pins in the original dam to monitor potential deformations of the original dam during construction. No movements were observed in the telltale strips or monitoring pins during construction.

2.3 Vibrating Wire Piezometers in the Foundation

Twenty-eight (28) vibrating wire (VW) piezometers were installed in the foundation soils to monitor groundwater pressures and gradients during and following embankment construction. Five (5) of seven (7) VW piezometers installed during the investigation phase were able to be salvaged where their positions did not conflict with the shear key or CB Wall alignments. The piezometers were installed after construction of the permanent sand drainage blanket and were monitored throughout construction and following construction. The pore water pressure response to embankment loading (B-bar response) within the foundation silty clay deposit was monitored during fill placement and was the basis for establishing phased embankment elevation hold points in the specifications to respect construction slope stability criteria.

The locations of the piezometers were strategically spread throughout the site and included nests of VW piezometers with tip elevations ranging from within the bedrock to within the clay or silt overburden, to the sand blanket. VW leads from the various locations were strung to six (6) common hub locations along the alignment of the toe of the dam for ease of data collection. A few additional VW piezometers were installed later during construction with collection / monitoring points on the crest of the remedial dam and are discussed in Section 4. Only the piezometric data for the piezometers in clay are shown herein to highlight the monitoring program.

2.4 Excess Pore Water Pressure Monitoring (B-Bar)

At the design stage (Kurz et al. 2021), the low hydraulic conductivity of the foundation clay was anticipated to result in embankment fill induced excess porewater pressures (B-bar) that would temporarily reduce the slope stability until those pressures dissipated. Given the low hydraulic conductivity, the rate of excess porewater pressure dissipation could theoretically take years to occur. As such, the structure, at final height, was designed to meet long-term MNRF 2011 safety factors without the dissipation of construction induced excess pore water pressure. That is, a B-bar = 0.7 was assumed within the design.

During construction, the observational approach was taken to continuously monitor the porewater pressure response during the construction of the embankment. A plot was established for resident engineering staff to follow during construction such that piezometer readings could be converted to an estimated B-bar response based on the elevation of the fill at any time. Should the B-bar response yield a safety factor FOS < 1.30 at any given stage, embankment fill operations would be suspended until the pressures dissipated sufficiently to an acceptable value.

Figure 5 shows the plot including the estimated B-bar response in the clay based on piezometer readings during construction and is discussed further in Section 4. The piezometric increases generally coincided with initial fill loading and significant precipitation events during construction rather than the surcharge loading induced by embankment fill placement. The clay at this site was fissured and has been thought to provide drainage relief within the clay mass, and therefore a possible cause of the lower than estimated B-bar response.
3 CONSTRUCTION

Figures 1, 2, and 3 show the general plan, profile, and section of the design remedial dam. The design consists of a zoned embankment rockfill dam with a central 1 m wide CB Wall seepage cut-off. The new structure was positioned immediately downstream of the original embankment dam slope. The alignment of the new CB Wall core was offset 5 m from the downstream toe of the original dam which governed the position of the structure. The CB Wall was designed to make positive contact with the bedrock foundation for its full length and constructed as slurry trench panels cut from the crest of the new embankment at predetermined interim lift elevations.

The following sections discuss challenges experienced during construction.

3.1 Overburden Foundation

Preparation of the overburden foundation involved stripping vegetation and excavating the overburden soils or previous fill material down to the design elevation. This elevation promoted drainage away from the core and towards one of three locations that eventually would be open channel weir locations. Any clay material determined, through the excavation of the shear key, to be thicker than 2 m, was trimmed down to 2 m to meet the design slope stability factors of safety. Excess clay was stockpiled for future use as impervious fill.

During stripping of the fill material between approximately STA. 1+300 to 1+330 (east end of the site), an unanticipated deep organics deposit was encountered between approximately 20 m to 35 m from the toe of the original dam. This deposit was largely composed of logs suspected to have been previously used as a corduroy road, perhaps during original construction of the dam, that was disposed of in this area, and subsequently covered by clay and granular materials. The organics deposit was completely sub-excavated down to a competent clay subgrade at approximate El. 310.7 m and totaling approximately 370 m$^3$ of unanticipated extra excavation. Given the lower base elevation, this excavation would trap groundwater in the foundation, as such, a drainage outlet was developed to relieve seepage toward the nearest toe drain outlet to provide foundation drainage relief.

As discussed in the companion paper, Kurz et al. (2021), there were some challenges with the foundation clay material. This clay had an upper weathered mottled brown-grey layer and a thicker layer of fissured grey clay. The clay was described to be fissured and had a nuggety and friable structure. This resulted in challenges achieving compaction of the foundation clay and more ‘working’ of the material was necessary. Similarly, challenges were encountered during the excavation of the shear key where some of the sidewalls were observed to collapse. The problem was mitigated by minimizing the time the trench was open, limiting traffic and work in the vicinity, shortening the open trench lengths, and benching or widening the top width of the trench where required.

3.2 Bedrock Foundation

The bedrock foundation at the west abutment was relatively smooth with minor undulations with a gentle slope rising westward. The bedrock joints at the west abutment were generally tight and no major open vertical or sub-vertical joints, or shear zones were observed. This allowed for relatively easy construction at this abutment.

At the east abutment, however, the bedrock rose steeply (vertical in places) toward the east and was observed to be highly irregular with steep blocky, vertical faces and a cliff that plunged below the design sub-grade elevation approximately 3 m south of the dam axis. The elevation of the toe of the cliff was not determined. A deep sub-vertical shear zone was observed at STA. 1+394 approximately 100 to 200 mm wide with a strike angle that aligned it generally upstream to downstream and sub-perpendicular to the dam axis. The east abutment bedrock also had other dominant sub-vertical and sub-horizontal joint sets. The shear zone was observed to be tightly infilled with sediment. This shear zone was cleaned to the maximum extent possible and then filled with dental concrete to be flush with the adjacent bedrock surface.

The observed shear zone along with other dominant sub-vertical and sub-horizontal joint sets warranted construction of a grout curtain at this abutment to improve the water tightness of the rock. Figure 6 provides an aerial photo of the bedrock condition at the east abutment after dental concrete placement and prior to the grout curtain construction. The bedrock was mapped, and a split-spaced, up-stage grouting program was developed and

Figure 5. Global Stability - B-Bar vs. Fill Lift Elevation.

Figure 6. Aerial view of bedrock (east abutment).
implemented. In addition, the dam core alignment was optimized to ensure the core was founded on the most ideal, level bedrock at the abutment.

### 3.3 Embankment Construction

The working area onsite was confined and restricted between the laydown area and manoeuvring excavators, dozers, loaders, and rock trucks on site. The buttressing of the remedial dam to the existing dam and the location of the CB Wall made effective work planning a challenge. The contractor had to make efficient use of space and rotation of equipment to construct the dam and minimize delays. The primary challenge was the placement of the filter materials at a 10V:1H batter with the adjacent rockfill material while ensuring effective placement and a narrowing work area at the top of the fill as the embankment height increased (See Figure 2).

To minimize the risk of long-term internal migration of the filter and transition granular materials into the void-rich rockfill, finer rockfill material was to be selectively placed adjacent to the sand and gravel zone, with coarser rockfill placed in the outer zones of the shell. This became challenging as the downstream rockfill area became narrower as the embankment height increased. The contractor worked with both a dozer and excavator to achieve this, and where necessary used excess transition gravel material to fill specific voids.

To place the filter material effectively in the narrow work area, the contractor made efficient use of a spreader box typically used on roadworks projects. The rock trucks were able to end-dump into the loader-mounted spreader box and place a relatively uniform lift of filter sand material at the correct width.

To accommodate unforeseen bedrock features at both abutments, the design abutment details were modified to best suit the alignment of the CB Wall on the bedrock. This resulted in including a clay core detail locally at each abutment that the CB Wall keyed into and terminated at a specified embedment depth. To construct the clay core abutment, the contractor placed the clay fill at appropriate lift thicknesses for compaction, overbuilt the edges and carefully trimmed the sides to the neat line geometry. Tarps were used to minimize drying and desiccation prior to the placement of the adjacent lifts of filter material, which were built much quicker than the clay core. Figure 7 is a photo of the clay core construction at the east abutment.

Challenges with clay conditioning and placement were encountered due to excessive wetting and drying from weather conditions, excessive time to place lifts, lift thickness, compaction challenges, among other factors.

### 3.4 Cement-Bentonite Wall

The primary water retaining element of the remedial embankment and underlying overburden foundation was the cement-bentonite diaphragm cut-off wall (CB Wall). The CB Wall was constructed from a top elevation of El. 320 m, through the embankment and overburden foundation down to the bedrock surface contact to create a positive seepage cut-off using a long-reach excavator and the slurry trenching technique.

The CB Wall was constructed in two major stages (or panels) due to excavator reach limitations to construct the CB Wall as a slurry trench where the bedrock was deep near the east abutment area. Figure 4 shows the profile of the CB Wall core along the length of the dam with the approximate panel locations with the measured depth of the CB Wall and bedrock contact versus the interpreted bedrock depth based on investigations. The general sub-surface bedrock profile was found to be consistent with the investigations, except at the east abutment where the bedrock was unexpectedly found to be steep and blocky.

Immediately at the east abutment, the CB Wall was initially constructed from west to east with the excavator positioned on the abutment, so that the excavator bucket could pull upward to ensure that the overburden at the bedrock contact was adequately scraped away and replaced with cement-bentonite slurry. The excavator was then repositioned to the embankment and proceeded to continue and complete the Stage 1 panel from east to west (with excavator advancing backward and excavating the trench towards the west abutment). Given reach limitations of the excavator, the Stage 1 panel of the CB Wall was constructed from the embankment crest at the interim elevation of El. 317 m.

The Stage 2 panel was constructed continuously and progressed starting from the west abutment and advanced toward the east abutment from the embankment crest once it was built to an interim elevation of El. 320 m. A minimum cold joint depth was specified to be a 1.0 m overlap between separately constructed CB Wall panels. Therefore, a horizontal cold joint exists between Stage 1 and Stage 2 at an average elevation of El. 315.6 m and a sub-vertical cold joint exists between Stage 1 and Stage 2 between STA. 1+287 to 1+295.

Challenges with the construction of the CB Wall primarily existed when constructing the Stage 1 panel at the deepest section of foundation overburden, where the bedrock was deepest near the east abutment. The soils at this location were dense and difficult to remove with the long-reach equipment. In addition, boulders that were encountered needed careful removal from the narrow trench. The contractor used several excavator-mounted implements, including an aggressive multi-ripper bucket, a hydraulic hammer (hoe-ram), and a rotary drum cutter with carbide teeth to loosen rock / boulders and the dense silty sand. A 1 m wide bucket with paddle teeth was used to ensure appropriate width of the CB Wall was achieved. The contractor employed the use of a second long-reach
excavator for redundancy to minimize down-time as it was necessary for the slurry trenching to be a continuous operation to ensure operations remained well ahead of trench sections where slurry that was setting up and curing.

4 PERFORMANCE MONITORING

4.1 Construction Monitoring

4.1.1 Vibrating Wire Piezometers

A total of forty-one (41) VW piezometers were monitored throughout construction using the Observational Approach (Peck 1969). Some instruments were decommissioned throughout construction and thirty-three (33) are currently monitored. The instruments were aligned to common locations and the multi-channel dataloggers were installed. An existing barometer on-site was used for barometric correction of the data. Since construction, all digital instrumentation including VW piezometers, the barometer, and a weather station are remotely monitored through a data acquisition system with cellular telemetry.

During construction of the main embankment, the piezometers were used to monitor the B-bar response (Figure 3) after construction of the 0.5 m thick sand blanket across the footprint of the new embankment. The monitoring was primarily focused on the piezometers in the foundation clay within the limits of the embankment. The response of the piezometers was measured and plotted daily on a predetermined relationship between fill elevation versus B-bar versus slope stability (in terms of safety factor). During construction, it was found that the maximum B-bar response was experienced near the start of initial fill placement but then the response became attenuated with increasing fill height. This was thought to be due to pore water pressure dissipation through the clay fissures, and drainage relief to the underlying silty sand layer, the overlying sand filter blanket, and other sand-filled trenches cut in the foundation clay to satisfy other design intents. In general, a B-bar response of 0.0 to 0.5 was ultimately realized due to embankment construction.

Figure 8 shows the measured piezometric response of select piezometers in the clay foundation during various stages of embankment construction from approximately El. 314 m (July 3, 2019) to El. 320 m (Sept 15, 2019) and up to the start of impoundment (Oct 22, 2019). Key events are noted on the plot. In general, the piezometric level in the clay increased by 1.0 to 1.5 m over the course of construction. For reference, prior to impoundment, the headpond was maintained at around El. 314.5 m.

This data shows increases in piezometric readings due to noted precipitation events as well as the general construction of the embankment. During excavation of the CB Wall, a rise in piezometric elevation is noted during both Stage 1 and Stage 2. Select piezometers (e.g., PZ3-2019) showed immediate reaction to the excavation of a CB spoil pit as well as both filling and dewatering of the pit, implying a hydraulic connection. The piezometric rise in the clay piezometers during Stage 1 CB Wall construction was a result of raising the embankment to El. 320 m or the nearby CB spoil pit and not associated with the CB Wall itself given the locations of the piezometers. However, during Stage 2, the stark piezometric increases are attributed to the liquid column of slurry as the slurry trench approached various transducers. The piezometers all show dissipation of the excess pore pressures as the CB Wall cured.

The monitoring program also afforded the opportunity to monitor the temperatures from each piezometer to complement the piezometric data. The temperature monitoring, as may be expected, became interesting during the excavation of the CB Wall. Figure 9 illustrates the temperature of three nested piezometers located near the CB Wall alignment. The data begins after initial installation and shows temperatures increase during the CB Wall excavation and then dissipate. The lowest piezometer, PZ9-2019, installed in bedrock, shows the least impact; however, the middle piezometer (PZ10-2019 in silty sand overburden) and the upper piezometer (PZ11-2019 in the filter sand) show more impact, reaching a temperature of 20°C. The dissipation of temperatures (or heat of hydration) from the CB Wall took place over nearly 5 months since CB slurry passed this location. Temperatures have since normalized within the ground showing minimal seasonal variation except for the upper piezometer showing seasonal variations over 4°C.

4.1.2 Crest Survey

In addition to the daily monitoring of the concrete telltale strips (Section 2.2), the settlement markers (or survey pins) in the original dam were monitored to determine if there...
was any movement as a result of the construction activities or otherwise. Thresholds were established as +/- 4 cm of movement and were surveyed using RTK GPS methods (accuracy +/- 2 cm) every few days to complement daily visual inspections.

A few anomalous surveys showed survey points at or near the threshold value; however, no observation on site indicated movement, and the overall trend in data indicated no measurable movement of the dam. The survey of the original pins continued until the offloading of the original dam commenced in early November 2019.

4.2 Impoundment and Post-Construction

After the final Stage 2 panel of the CB Wall achieved appropriate strengths, as confirmed from testing sample cylinders, impoundment of the new remedial dam took place on October 26, 2019. Up to this point, the original dam served as a cofferdam; however, offloading of the original dam and crest finalization of the new remedial dam was to be completed following successful impoundment of the newly constructed dam.

Impoundment occurred in three phases. Phase 1 involved pumping water between the original and new embankments at a controlled water level rise of 1 m per day for approximately 7 days until the water level between the two dams was balanced with the headpond. Phase 2 involved a controlled breach of the original dam upon reaching the specified water elevation between the dams, at which time the owner began raising the entire headpond using the plant flow control equipment. During both of the first two phases, the headpond elevation was increased from approximately El. 314.5 m to approximately El. 317.75 m. Phase 3 increased the headpond elevation up to FSL at El. 318.2 m.

4.2.1 Vibrating Wire Piezometers

During impoundment, the piezometers were carefully observed for performance of the CB Wall and any adverse piezometric response. Figure 10 is a continuation of the data presented in Figure 8 through impoundment and performance monitoring to early 2022. At the onset of impoundment, nearly all piezometers showed an increase ranging from approximately 0.1 to 0.7 m over approximately 7 days. After full impoundment, the piezometric response stabilized and only displayed minor seasonal fluctuations in the months following impoundment as the piezometric conditions reached steady-state.

Since impoundment, some piezometers have shown minor dissipation trends (e.g., PZ7-2017 and PZ14-2019), while most of the piezometers have remained relatively consistent. One piezometer, PZ5-2019, showed a drop of nearly 0.5 m in April 2020 but has since increased slightly and remained relatively stable since then. Overall, the piezometric data, including data for the sand / silt and bedrock piezometers have been demonstrating good performance of the remedial dam with post-construction readings that are within the ranges anticipated at the design stage.

4.2.2 Crest Survey

At the time of impoundment, temporary survey pins were installed for monitoring to complement visual inspections. This data is not provided herein; however, the data showed no measurable indication of movement including settlement. After construction, permanent survey monitoring pins (SMPs) were installed and were surveyed using a total station. Figure 11 shows the elevation survey collected for the first year after construction. The measured data is within the post-construction thresholds of +/- 2.5 cm, and the noted variability in data is attributed to the tolerances of the survey convention and repeatability.
Most foundation consolidation settlement may have occurred on a relatively immediate basis during embankment fill construction, and before it could be measured with the survey pins which were put in place after the embankment was built to full height. The general absence of measured post-construction settlement is further corroborated with the negligible B-bar response observed in the clay foundation.

4.2.3 Monitoring Weirs

Three (3) monitoring weirs (shown in Figure 1) were installed to collect and facilitate monitoring of seepage before being discharged into the marsh immediately downstream of the dam. The dam foundation was strategically graded to provide positive drainage to respective weirs. Steel pipe piles were installed and socketed into bedrock and the 90° V-notch weir plates, embedded in the clay overburden, were welded to the piles to restrict frost heave movement of the weir plate. The open-channel weirs were outfitted with high-precision piezometers located approximately 1.5 m upstream from the weir plates to facilitate remote monitoring of seepage flow rates. Each weir is also monitored manually to perform visual checks on seepage quality (suspended sediment), and verification of the piezometer readings using a staff gauge and a clamp-on trough to allow for flow measurements using a control volume and stopwatch.

Figure 11 illustrates an example of the monitoring from the west weir (Weir 1). Data shows good correlation between the manual readings and the piezometric readings, as expected. Occasionally, debris such as leaves, or algae were observed to obstruct flow through the weir which yielded anomalous “increases” in flow. Similarly, snow melt, and precipitation events cause increases in the weir discharge. Ice has also been observed to cause anomalous data in the winter. These data anomalies, that don’t necessarily represent seepage flow, warrant regular maintenance and visual inspection of the weirs.

Based on seepage modeling at the design stage, an estimated range of steady-state seepage was between 15 and 220 L/min. Currently, the approximate base seepage rate is 40 L/min excluding snow melt and precipitation events. Careful monitoring of the weirs is needed to complement the overall performance monitoring of this CB Wall core dam.

5 CLOSING REMARKS

This paper presents the construction and monitoring of a remedial earthen dam in northwestern Ontario that replaced a dam that had experienced unexpected slump movements. The paper presented challenges that were encountered and addressed during construction. Minor modifications to the design were completed to suit unforeseen site conditions. The dam safety monitoring of both the original dam, the new remedial dam during construction, and monitoring of the new dam during first impoundment and for over two years following construction. Based on regular visual inspections and instrumentation monitoring, the dam has been performing as designed since the end of 2019.

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7 REFERENCES

