Construction of the A21 Dike at the Diavik Diamond Mine

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ABSTRACT
The Diavik Diamond Mine is located on East Island in Lac de Gras, Northwest Territories (Canada). The A21 Dike is the third water-retaining dike constructed within the lake to enable dewatering of the area enclosed by the dike and allow open pit mining of the A21 Kimberlite pipe. The A21 Dike is a zoned rockfill embankment with a seepage cut-off wall. The cut-off wall comprises a single-row grout curtain in bedrock, cutter soil mixing panels through the dike embankment and foundation soil, and jet grout columns through the foundation soil and soil/rock interface. Changes in design and construction methods from the previous A154 and A418 Dikes were introduced with the intent of reducing costs, optimizing the project and accommodating the site conditions and schedule constraints. Although these changes led to a project that was recognized to be less robust than the previous dikes, the A21 Dike was completed on time and under budget and is currently performing as intended. This paper describes the main components and construction activities of the A21 Dike.

RÉSUMÉ
La mine de diamants Diavik est située sur l'île Est au Lac de Gras, Territoires du Nord-Ouest (Canada). La Digue A21 est la troisième digue de rétention d'eau construite au lac pour permettre l'assèchement de la zone délimitée par la digue ainsi que l'exploitation à ciel ouvert de la cheminée de kimberlite A21. La Digue A21 est un remblai construit en enrochement avec un mur parafouille. Le mur parafouille comprend un rideau d'injection dans le substrat rocheux, des panneaux de 'cutter soil mixing' à travers le remblai et les sols de fondation, et des colonnes 'jet grouting' à travers les sols de fondation et l'interface entre les sols de fondation et roc. Des modifications de conception et méthodes de construction par rapport aux digues précédentes, A154 et A418, ont été introduites dans le but de réduire les coûts, d'optimiser le projet et de prendre en compte les conditions de chantier et les contraintes d'horaire. Bien que ces changements ont mené à un projet reconnu comme étant moins robuste que les digues précédentes, la Digue A21 a été réalisée dans les délais et en deçà des sommes budgétés, et performe comme prévu. Ce document décrit les composantes majeures de la Digue A21 et les activités principales de la construction.

1 INTRODUCTION

The Diavik Diamond Mine site is operated by Diavik Diamond Mines (2012) Inc. (DDMI) and is located on the East Island in Lac de Gras, approximately 300 km north-east of Yellowknife in the Northwest Territories (Canada). The mine site includes three open pits, A154, A418 and A21, and corresponding diamondiferous Kimberlite pipes, which are all located at bathymetric depressions just offshore in Lac de Gras. Each open pit is partially surrounded by a water-retaining dike constructed within Lac de Gras to enable dewatering of the area enclosed by the dike for open pit mining. The construction of the A154 and A418 water-retention dikes was completed in 2003 and 2006, respectively. Open pit operations at the Diavik Diamond Mine site started in 2003 and since then has produced more than 100 million carats of diamonds. The current mine plan has production continuing to 2025.

The A21 Dike is a ring dike, approximately 2200 m long and up to 25 m high. Major elements of the design and construction included on-land and underwater foundation preparation, embankment fill placement, vibro-densification of the embankment core, installation of the seepage cut-off wall (COW), and installation of instrumentation, thermosyphons, relief wells and pumping wells. Instrumentation installations included vibrating wire piezometers (VWP), slope inclinometers (SI), thermistors, magnetic extensometers, geophones and survey markers. After dewatering of the area enclosed by the A21 Dike, a downstream toe berm and a seepage collection system were constructed.

Construction of the A21 Dike was carried out from November 2014 to September 2018 with dewatering between November 2017 and March 2018. Quality control for construction included but was not limited to bathymetric surveys, verification coring through the COW, field and laboratory testing of construction materials, water pressure testing within the grouted bedrock and in-situ hydraulic conductivity testing of the COW. Open pit mining of the A21 Kimberlite pipe is currently ongoing.

This paper describes the main components of the A21 Dike and the main construction activities. A summary of the main challenges encountered during construction is also provided.

2 SITE CONDITIONS

The mine site is located on an island where personnel are transported via airplane. Primary resupply of materials, equipment and fuel are transported from Yellowknife via a temporary ice road built and operated during the winter. Figure 1 shows the approximate location of key mine site infrastructure. The A21 Dike and A21 Kimberlite pipe are located at the south end of the mine site.

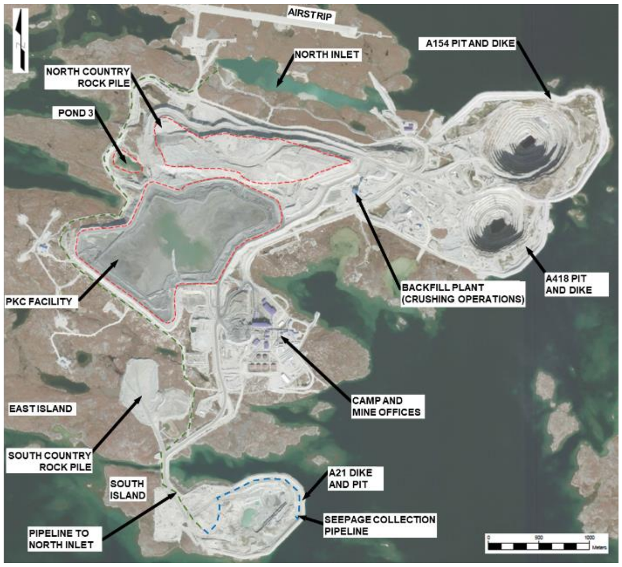

Figure 1. 2018 satellite image showing the Diavik Diamond Mine Site, and approximate locations of site infrastructure.

Located 220 km south of the Arctic Circle, the mine site is within the arctic climate region and in a zone of continuous permafrost. Daylight reaches a minimum of four hours per day in winter, and a maximum of 20 hours per day in summer. Climate at the site typically comprises long, cold winters and very short, cool summers. The range of mean monthly air temperature is summarized in Figure 2.

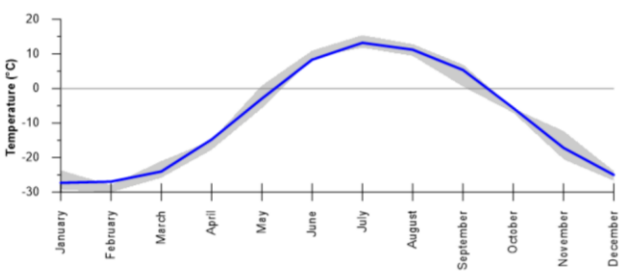

Figure 2. Historical mean monthly temperature (blue) and range of mean monthly temperatures for the A21 Dike construction period 2015 – 2018 (grey).

The Lac de Gras area is in the Canadian Shield. The rock formations at the mine site are primarily Archean, Proterozoic, or Eocene in age. Kimberlite pipes occur as volcanic cores injected into the much older country rock and are generally aligned with southwest-northeast trending axis that represent zones of structural weakness exploited by the intrusions. At the A21 Dike, the typical stratigraphic sequence consists of soft, primarily silty lakebed sediments overlying stiff lakebed sediments and ablation till, in turn overlying an undulating to hummocky bedrock surface. The bedrock is tonalite and quartz diorite with pegmatite intrusions. Observed periglacial features are typical of continuous permafrost areas. Physical weathering (frost wedging and frost shattering) typically occurs on exposed bedrock and in boulder fields near the lake shore.

3 A21 DIKE COMPONENTS

3.1 Dike Embankment

A plan view and a typical cross-section of the A21 Dike are shown in Figures 3 and 4, respectively. The dike is a zoned rockfill embankment with upstream and downstream shells built out of 900 mm minus run-of-mine (Zone 3) and 200 mm minus crushed rockfill (Zone 2), respectively, and with a central core of crushed 50 mm minus sand and gravel (Zone 1). A 1 m thick filter blanket of 50 mm minus crushed granular material (Zone 1C) is beneath the downstream shell of the dike, to protect against internal erosion due to seepage through the dike foundation. The filter blanket allows foundation seepage water to drain, while providing a filter element for the foundation material. Larger rock blocks of the run-of-mine material were selectively placed on the upstream slope near lake level to help protect the upstream slope against wind and wave erosion.

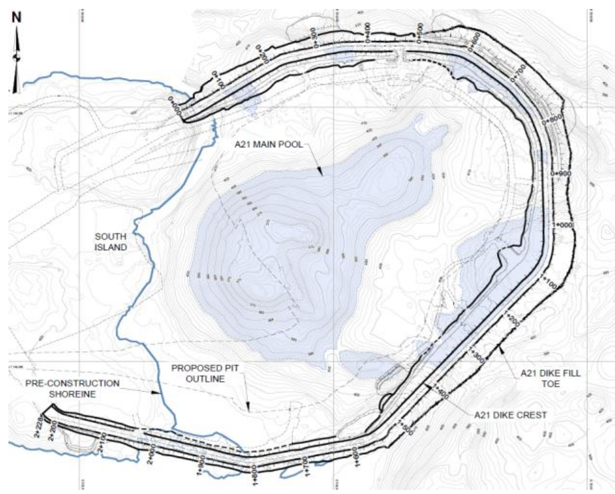

Figure 3. A21 Dike plan view showing localized pools formed after dewatering.

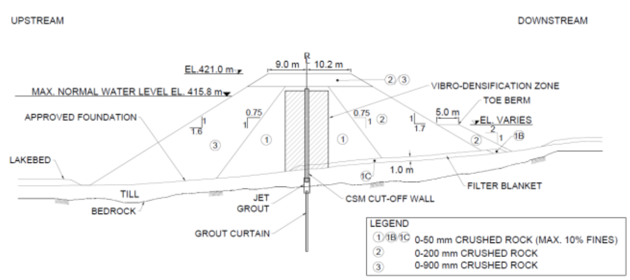

Figure 4. Typical A21 Dike design cross-section.

As a cost savings measure, the design crest width was reduced from 25 to 19.2 m, resulting in a reduced overall dike width compared to the previous dikes at Diavik. The A21 Dike has a maximum height of 25.5 m (crest at elevation 421m) and the upstream and downstream embankment design slopes are approximately 1.6H:1V and 1.7H:1V, respectively. Stabilization buttresses were required at select locations to improve the stability of the dike during construction and in the long-term.

3.2 Seepage Cut-Off

Near the lake shore the dike is founded on permafrost, whereas in deeper waters the dike foundation is unfrozen (talik) foundation. Two different seepage cut-off systems were adopted in permafrost and in talik.

The cut-off element on unfrozen foundation consists of a plastic concrete diaphragm wall through the embankment core material and into the upper till foundation, a bedrock grout curtain extending from the upper bedrock surface to bedrock depths of 15 and 25 m and a jet grout wall which overlaps both the diaphragm wall and the bedrock grout curtain, tying the three COW components together. The top of the COW is generally at elevation 417.5 m.

Within the dike sections founded on permafrost, the frozen soil and bedrock act as the seepage barrier below the dike embankment and plastic concrete diaphragm wall. Thermosyphon groups were installed at the talik-permafrost boundary to ensure the maintenance of the boundary location and to keep the COW keyed to the frozen ground. The south talik-permafrost boundary had shallow permafrost over talik which, along with the three-dimensional features of the area of the south abutment, posed particular challenges (Arenson et al., 2019).

3.3 Relief Wells

The purpose of the relief wells was primarily to reduce potential excess pore pressures in the foundation at select deep dike areas downstream of the COW during dewatering. The typical relief well had two well screens, one located in bedrock and the second screen located in the dike fill just above the foundation. The relief wells are located near the downstream edge of the dike crest.

3.4 Toe Berm and Seepage Collection System

The permanent seepage collection and drainage system controls the water level inside the downstream section of the dike by collecting water at select locations and pumping it to the North Inlet (see Figure 1). The system includes two Dike Pumping Stations (DPS), three pump wells, a main collector pipeline and its pumping system. The pump wells are located at select locations on the downstream shell of the dike to manage ponding water within the downstream shell of the dike. The two DPS are installed at the downstream dike toe to manage seepage through the dike and ponding water at localized depressions downstream of the dike toe. The insulated and heat traced collection pipeline is situated on the crest of the toe berm constructed along the dike toe after dewatering. The toe berm crest is approximately 5 m wide and its height ranges from 0.5 to 9.8 m above the prepared foundation.

3.5 Instrumentation

Geotechnical instruments were installed within the foundation, embankment and toe berm to monitor the performance of the dike. The installed instrumentation comprises VWPs, thermistors, SIs, survey markers, magnetic extensometers and geophones. An Automatic Data Acquisition System (ADAS) was installed to collect and transmit VWP data by radio frequency from four ADAS shacks installed on the crest of the A21 Dike.

3.6 Comments

To make the A21 project feasible and improve project economics, changes were made to reduce some of the redundancy that was incorporated into the design and construction methods of the previous dikes. These changes and the unfavourable site conditions identified during construction of the A21 Dike led to a project that was recognized to be less robust than the previous A154 and A418 Dikes, however the project benefitted from calibration based on the performance of the previous dikes.

The till cover in the A21 Dike area of the lake was found to be thinner (typically less than 3 m) and coarser (less silt, more cobbles and boulders) than in the foundation of the other dikes, resulting in less protection against seepage and less robustness in resisting the potential for piping under the hydraulic gradients in the dike foundation. The hazard associated with these issues is increased by the narrower dike cross section and especially by the significantly smaller minimum set back between the toe of the dike and the crest of the pit wall.

The objective of the minimum set-back is to position the dike outside the zone which may be excessively influenced by the open pit development and the zone which may be affected by pit wall rock stability. A key reduction of the redundancy that was incorporated into the design of the previous dikes was the decrease of the set-back from the downstream dike toe to the pit crest, from 100 m for the A154 and A418 Dikes to 50 m for the A21 Dike.

During construction, design changes, procedure optimizations and remedial measures were employed where required to mitigate risks. The remaining uncertainties led to more focus on the QA/QC program during construction.

4 A21 DIKE CONSTRUCTION

Construction of the A21 Dike began in November 2014 and was completed by September 2018. The following sub-sections provide a summary of the A21 Dike construction materials and methods, including the optimized construction methods that differed from those used for the previous dikes and the main modifications from the original design to accommodate the encountered site conditions, the construction deficiencies and the schedule constraints.

4.1 Construction Materials

Non-reactive mine rock used for producing construction materials was selected by DDMI from a local stockpile of waste rock, and either stockpiled near the A21 dike abutments (Zone 3) or crushed and screened to prepare Zone 1/1B/1C, Zone 2 and plastic concrete aggregate. A total of approximately 3.8 million tonnes of material

(Zone 1/1B/1C, Zone 2 and plastic concrete aggregate) was prepared for construction.

Quality Control (QC) and Quality Assurance (QA) gradation testing of granular materials during material preparation, stockpiling and construction was undertaken. When production gradation test results were outside the gradation specifications, corrective actions were taken to bring the material production back into compliance. Corrective actions were also undertaken with the out-of-specification materials, resulting on more than 15,000 tonnes of material removed from the stockpiles and not used for A21 Dike construction.

4.2 Construction Components

4.2.1 Foundation Preparation

Foundation preparation was carried out to provide a competent foundation capable of supporting the dike structure. The A21 Dike is founded on either stiff lakebed sediments, glacial till or bedrock. Unacceptable materials for the dike foundation, such as organic materials, soft lakebed sediments, boulder lags (underlying Zone 1 and Zone 2 fills) and ice-rich soils were removed from the dike and toe berm footprints by excavators (used on-land, in shallow water and in the toe berm area after dewatering) or by dredging with a cutter-suction dredge and/or barge mounted crawler cranes equipped with heavy duty clam shells (in deep water).

Foundation preparation in deep water was carried out to refusal of the clam shell or cutter-suction dredge. In some locations, a clam shell pass was used after dredging refusal. A Real-Time Kinematic (RTK) Global Positioning System (GPS) unit mounted on the top of the crane mast or cutter-suction dredge ladder was used to track the position of the clam shell or cutter head during dredging. In shallow water or on land, foundation preparation was carried out to excavator bucket refusal or as directed by field personnel based on the observed excavated material and foundation surface (if visible).

Foundation preparation of QC and QA included the observation of dredged material, collection of samples for gradation testing, pre- and post-foundation preparation surveys. RTK GPS surveys and the execution of test-pits were done in shallow water and on-land. In deep water, bathymetry surveys were conducted using multi-beam bathymetric sonar equipment along with pre- and post-dredging dive inspections. Dive inspections consisted of visual observations, probing the thickness of residual softer material and sampling the foundation materials. Diver observations were logged with audio and video recording.

4.2.2 Fill Placement

The construction materials were sourced from the local stockpiles and transported to the fill placement sites by split-hull hopper barges (marine works) or haul trucks.

Marine construction was used for placement of the filter blanket (Zone 1C) and for the construction of stabilization berms. The 1 m thick filter blanket material in deep water areas was placed using barge mounted crawler cranes equipped with smooth lip clam shells. The loaded clam shell buckets were lowered through the water to the target location on the prepared foundation. The Zone 1C was released as close to the lakebed as possible to reduce the material drop height and minimize material segregation. Upon approval of the foundation preparation, stabilization buttresses were constructed where required by placing select Zone 2 or 3 material from lake surface using split hull hopper barges. Bulldozer and/or excavator placement onto approved foundation was used for filter blanket construction in shallow water areas, where the clam shell could not be used due to barge access restrictions. The thickness of the placed filter blanket was verified using the same GPS and sonar techniques used to verify the foundation preparation surface.

Upon approval of the filter blanket construction, Zone 1, 2 and 3 materials were dumped by haul trucks near the embankment advancing face just above lake level, and then pushed frontward and downward with a bulldozer to create slip displacement of the fill. This method was selected to minimize the rolling segregation of the materials. Material Zones 1, 2 and 3 were sequenced to maintain the design cross-section zone contacts. Additional material was then placed (maintaining the design zonation) to approximately 2 m above lake level (nominal elevation 418 m) to build a temporary construction platform to allow the installation of the COW and dike instrumentation.

The embankment was raised to the final dike crest elevation of 421 m after construction of the COW (see below). The dike raise was completed by placing and compacting material in layers using excavators, dozers and an 18-ton smooth drum vibratory compactor. The downstream toe berm fill, including the extension of the filter blanket under the toe berm, was placed in layers on approved foundation after dewatering.

Embankment construction QC and QA included observation of material placement, sample collection for gradation testing, and pre- and post-fill placement surveys. Bathymetric surveys were conducted regularly with the multi-beam sonar equipment to obtain surfaces of the stabilization buttress, filter blanket and dike upstream, downstream and advancing slope faces. As shown in Figure 5, the resolution of the bathymetric surveys was such that the material zone contacts below lake level could be identified on the advancing dike face.

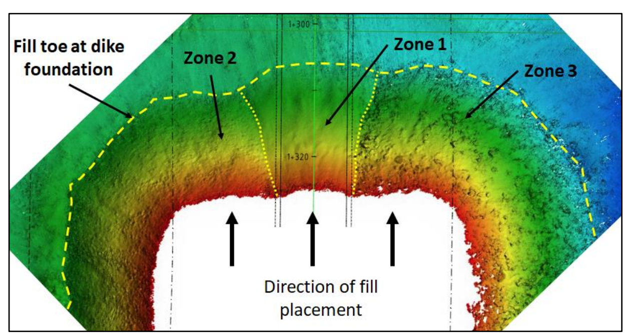

Figure 5. Multi-beam bathymetric sonar survey of the advancing dike face below lake level, with material types discernable by imagery texture.

Above water or in shallow water, hand held digital RTK GPS was used to conduct as-built surveys of the placed fill. RTK GPS units mounted on the top of the crawler crane mast (filter blanket placement) and on the split hull hopper barges (stabilization buttresses) tracked the position of each dump load.

4.2.3 Vibro-Densification

Vibro-densification of the Zone 1 dike core was required to stiffen the core material within a 10 m wide area around the diaphragm wall to reduce deformations in response to pool dewatering.

Vibro-densification work was completed from the temporary construction platform at elevation 418 m using 27 m long electric vibro-probes mounted on crawler cranes. The vibro-probes penetrated and densified the Zone 1 fill by the application of high energy vibration coupled with water and air jets supplied under pressure via probe tip and side nozzles. At a typical vibro-densification location, the probe penetrated to a depth of approximately 0.5 m above the approved foundation. Once reaching the target depth, the densification was performed by incrementally raising the probe and holding at the new depth or letting the probe settle back to the original depth. Several operational parameters were adjusted to optimize the vibro-densification effectiveness, including the withdrawal increment and hold times, tip and side air/water pressures, and probe point spacing. While the densification was being performed, additional Zone 1 material was added at the ground surface using a loader. A total of 2283 initial densification columns were executed.

Dynamic probing heavy (DPH) testing (European Standard ISO/DIS 22476-2) and the volume of added material were used as the main control method. Production log data recorded during densification, including penetration depth, bottom hold times, side pressure consistency, withdrawal interval and peak amperage consistency and pattern over the entire column were also considered. DPH tests were conducted pre- and post-densification and after re-densification, where needed. After initial vibro-densification, re-densification was required at locations that did not reach target depths and areas in which densification did not meet the DPH based acceptance criteria. Becker Penetration Tests (BPT) were conducted in critical areas for additional verification of the fill density. In a portion of this area, where the densification did not meet the design criteria, the diaphragm wall was widened as a remedial measure.

4.2.4 Curtain Grouting

Curtain grouting was executed from the temporary construction platform at elevation 418 m to treat fractures and joints in the bedrock to reduce the hydraulic conductivity of the bedrock below the dike. Single row curtain grouting was adopted along most of the length of the dike, except in the area of a major fault, as described in Section 4.4.3 below. Curtain grouting work involved drilling, hole washing, water pressure testing, and pressure grouting. A total of 886 curtain grout holes were completed following a Primary - Secondary hole sequence, with higher order holes (Tertiary, Quaternary and Quinary) to achieve the closure criteria if not achieved with the Primary and Secondary holes.

Holes were drilled along the dike reference line to the design depth through the dike embankment, till foundation, and into bedrock using down-the-hole (DTH) hammer drilling with air flush in the rock. Temporary steel casing supported the holes through the dike embankment and foundation soils. Drill parameters, including crowd speed (or rate of penetration), rod rotation speed, crowd pressure (or weight on bit) and torque were recorded digitally. Review and processing of drill data along with observations at the drill enabled determination, among other parameters, of top of bedrock elevation, casing depth, and final curtain grout hole depth.

All grout holes were washed with high pressure water flushing upwards and radially along the entire grout hole length in rock. Then, the holes were water-pressure tested in 5-metre intervals (stages) isolated by packers to determine the hydraulic conductivity of the rock mass over that interval prior to grouting and to assess the nature of potential fracture infilling. Hydraulic conductivity was expressed in Lugeon units. Select holes were surveyed with an optical televiewer. Data were processed to evaluate orientation of the bedrock structures and the images were used to evaluate rock type, fracture aperture and infilling. Downhole ShapeAccelArray (SAA) survey was completed on select drill holes to determine the 3D position of the hole.

The top stage of each hole, and all stages with a measured Lugeon (Lu) value greater than 3 were pressure grouted. Each hole was then completely backfilled through the foundation soils and embankment with grout using the tremie method. Curtain grouting verification holes were drilled using the DTH method, washed, water pressure tested, surveyed with SAA and optical televiewer (OPTV), and grouted.

During drilling, the air pressure was reduced as much as possible and constant observation of air return by the driller was implemented to minimize the risk of uncontrolled release of air pressure into the dike foundation. VWPs were installed in the areas of highest risk for slope instability to monitor pressure changes within the dike foundation during curtain grouting operations.

4.2.5 Cutter Soil Mixing and Shallow Cut-off Wall

The diaphragm COW was constructed by two methods: Cutter Soil Mixing (CSM) in the higher sections of the dike (where Zone 1 had been vibro-densified), and conventional slurry trench (“Shallow Cut-Off Wall” or SCOW) in shorter dike sections.

The construction of the CSM COW along the dike reference line involved pre-drilling of overlapping 1.18 m diameter cased holes to replace Zone 1 dike fill and at least 3 m of foundation soils with concrete aggregate (12.5 mm minus), followed by slurry injection and mixing with the CSM equipment. The pre-drilling allowed for removal of boulders in the foundation till, which required a number of different tools. The slurry injection and mixing process was completed in overlapping 2.8 m long and 0.8 m wide panels built following a primary-secondary sequence, with the secondary panels being typically mixed shortly after the mixing of adjacent primary panels. A total of 2139 pre-drill holes and 668 CSM panels were completed.

The operating parameters of the CSM rig were recorded using an onboard data recording system. The CSM cutter head position was logged continuously using a depth sensor and an inclinometer within the cutter head assembly. QA and QC personnel conducted sampling and testing of the concrete aggregate, CSM cement-bentonite (C-B) slurry and mixed plastic concrete. In addition, QA and QC reviewed CSM production logs to confirm consistent production parameters, such as slurry consumption, percent slurry injected during downstroke, penetration rate, withdrawal rate, cutter wheel rotation speed, and geometric requirements, such as depth alignment and minimum overlaps. The pre-drilling cased hole depth and verticality survey data and the positioning information from the CSM cutter head instrumentation were used to plot pre-drill cased holes and CSM panels and assess CSM panel overlaps and depths.

The construction of the Shallow Cut-Off Wall (SCOW) consisted of excavating a continuous open trench to refusal on bedrock or frozen till backfilled with C-B slurry. A total of 416 m of SCOW were constructed. Trench depth measurements were generally collected at 2 m longitudinal intervals during and immediately after SCOW construction to assess trench wall stability. At the CSM to SCOW connections, detailed measurements were conducted to determine the connection profile and confirm minimum overlap requirements. C-B slurry samples were collected from the mixing plant and the SCOW trench for QC and QA testing.

4.2.6 Jet Grouting

The jet grout columns were executed along the dike reference line from the temporary construction platform at elevation 418 m using split spacing and “hybrid wet-in-wet” sequences. The “hybrid wet-in-wet” sequence consisted of groups of up to seven adjacent columns (referred to as a “panel”) that were jet grouted in a wet-in-wet sequence (i.e., without significant passage of time for grout to set between jetting adjacent columns). Another panel would be jetted, skipping a length of up to 5 adjacent columns. The skipped length of panels would be jetted later after setting of the concrete on the initial “primary” panels.

Jet grouting trials were conducted to establish and optimize the production jet grouting procedures and parameters. During the trials, constructed column diameters and associated column overlap for a given set of jetting parameters were not robustly established due to the variability of the till in the dike foundation. An assumed column diameter was used for different lift rates and specific jetting parameters to estimate the average jet grouting COW width and to determine whether remediation columns were required. A double fluid jet grouting method was used.

Before jetting, pre-drill holes were advanced at a 0.75 m spacing using rigs equipped with water powered DTH hammers. The holes were drilled through the constructed CSM panels, the till foundation and into the upper part of the grout curtain in bedrock. Then, a second rig lowered the jetting assembly into the pre-drilled hole and the column was jetted during withdrawal. The jetting assembly included a jet grout monitor with an outer air nozzle surrounding an inner grout nozzle.

During pre-drilling, the drilling parameters were digitally recorded, and the 3D position of each pre-drill hole was determined using a total station and a downhole SAA. The as-built data of pre-drilled holes was evaluated against the adjacent completed jet grout column geometry and used to prepare jet grouting instruction sheets providing the production jetting parameters. The parameters varied based on the material being treated.

Jetting parameters, including withdrawal rate, rod rotation speed, grout slurry flow and pressure, and air pressure, were also digitally recorded. Samples of grout slurry were collected from the grout plant and the spoil return. A downhole sampling tool was used to collect in-situ wet samples for field and laboratory testing.

The pre-drill and jetting records were used to plot completed jetted columns and assess column overlaps and depths. Area with insufficient inferred overlap between columns or between jet grouting and the CSM wall or the curtain grout were remediated by installing additional columns. A total of 2128 jet grout columns were completed, including 215 remedial columns (10%).

4.2.7 Verification Coring

Verification coring was performed to evaluate the quality of the completed COW, including the CSM, jet grouting, and upper curtain grout portions of the COW (curtain grout verification holes were previously completed as part of the curtain grouting works). A total of 151 verification core holes were completed. The core was visually inspected, and SAA, optical televiewer (OPTV) and hydraulic conductivity testing were completed in the core holes prior to tremie backfilling with grout. Select core samples were tested in the laboratory.

4.2.8 Instrumentation Installation

Drilling for the installation of VWPs, thermistors, SIs, survey markers, magnetic extensometers and geophones outside the COW was completed with track mounted drill rigs with retrievable Odex air percussion DTH hammer. To minimize the risk of damage to the COW, drilling for the installation of instruments along the dike reference line was completed with either drill rigs equipped with a tri-cone mud rotary system or PQ3 coring equipment, or using a rig equipped with water powered DTH hammers (as used for jet grout pre-drilling). As part of the QA/QC program, deviation survey of the holes drilled through the COW was completed prior to the installation of instruments.

A total of 20 SIs, 179 VWPs, 63 thermistors, 212 survey markers, 41 magnetic extensometers and 28 geophone piers were installed at the A21 Dike to monitor dike performance during and after pond dewatering. Dike instrumentation was installed either from the final crest at elevation 421 m or from the temporary construction platform at elevation 418 m and then extended to elevation 421 m during the subsequent raise of the dike. Multi-conductor cables were installed to connect the VWPs to ADAS shacks to allow near real-time remote monitoring. All other instrumentation data (SIs, thermistors, survey markers, extensometers and geophones) are collected manually by DDMI.

4.2.9 Toe Berm and Seepage Collection System

Construction of the toe berm included foundation preparation and placement of a Zone 1 filter blanket layer (minimum 0.5 m thick), followed by the placement of Zone 2 or 3 material over and downstream of the filter blanket. Zone 2 material for toe berm construction was sourced from stockpiles or from the surplus Zone 2 material recovered from the regrading of the dike downstream slope completed after dewatering.

Drilling for pump wells was completed with track mounted rigs equipped with Odex air percussion DTH hammer. Drilling and installation of the well casing was completed from the final dike crest or from the temporary construction platform, requiring the extension of the pump well during the dike raise to the final dike crest elevation. Installation of the pumping infrastructure at each pump well location was completed after the dike raise. Pipeline segments installed on the dike downstream slope were used to connect the pump wells to the main collection pipeline situated on the toe berm crest.

The construction of the DPS was performed when the main pond was drawn down with only localized ponds left at the DPS locations. The local ponds were dewatered as much as practical with temporary pumps, and temporary construction platforms were built to allow for material stockpiling and to accommodate machinery movement for construction. The DPS consist of buried vertical corrugated and perforated steel culverts installed on a granular bed, which in turn was placed on prepared foundation. The granular bed is an extension of the toe berm filter blanket. Excavation for foundation preparation, installation of the culvert, and backfilling around the culvert was performed segmentally, with temporary sumps and pumps used to manage water. The pumping infrastructure at each DPS and the connection to the main collection pipeline were completed after locally raising the toe berm to its final elevation.

4.3 Quality Control and Quality Assurance

QC was undertaken by the contractor executing each component of work. The design engineer was responsible for QA. In addition to verification coring described above, QC and QA consisted of field inspections, field and laboratory testing, and review of production logs and survey data. Approval of each component of work was based on the review of the available QA/QC records and was required prior to the start of subsequent work activities.

4.4 Construction Challenges

During construction, the original design was adjusted to accommodate the encountered site conditions, construction deficiencies and schedule constraints to reduce risk where possible. Mitigative measures were implemented to remediate the deficiencies encountered. The following is a summary of select challenges encountered during construction.

4.4.1 Weather

Most of the construction work was completed in the short summer months, except for some of the curtain grouting, instrument installation, dike raise, toe berm and seepage collection system work that was completed in winter. Although weather conditions throughout most of the A21 Dike construction were generally favorable for construction, various adverse weather conditions caused construction delays. Strong winds caused interruptions during activities involving large cranes/rigs (marine works, vibro-densification, jet grouting and CSM). Marine works were also delayed by wave storms. All construction activity was interrupted during lightning storms and white out conditions.

In cold weather, the top of the recently completed CSM and SCOW was insulated with tarps and warmed with frost fighters to allow the grout to set. Drilling for curtain grouting and instrumentation in cold weather required the installation of heated tents to protect equipment and personnel. Frozen equipment, hoses or pipelines caused delays during curtain grouting and jet grouting. The ADAS and pumping stations are protected with heated shacks.

4.4.2 Dike Foundation Conditions

The foundation conditions at the A21 Dike were found to be less forgiving than at the A154 or the A418 Dikes. At the A21 Dike, the till foundation overlying bedrock is thinner and more coarse-grained than under the previous dikes, meaning there is less capacity to control seepage gradients and erosion under the embankment, hence more reliance is placed on engineering solutions such as filters, and the cut-off wall components. The till at the A21 Dike foundation appears to have a larger quantity of cobbles and boulders than at the foundation of the previous two dikes, which resulted in more issues with deviation of curtain grout and jet grouting drill holes. The presence of boulders adds challenges to jet grouting due to the risk of leaving untreated ‘shadow zones’. Managing this risk required extended jet grout field trials, drilling of a larger than planned number of verification core holes to check areas of potential poor treatment, and remedial jet grout columns.

4.4.3 Station 0+600 m Fault Treatment

A steeply dipping (approximately 80 degrees) fault zone was encountered on the northeast area of the dike (near dike Station 0+600 m) that required additional treatment not described above. The fault strike is approximately NE/SW, nearly perpendicular to the dike reference line. Closure was not achieved in the upper 12 m of the fault zone during curtain grouting. After a large number of curtain grouting holes along three rows of grouting, high water flows and wash out in water pressure tests, sand in the return water and low grout takes were still being observed. The interpretation of these observations was that the fault zone had sand infilled fractures of high permeability which were not amenable to cement grouting.

The presence of potentially erodible materials in the fault zone, increased the potential for seepage and piping. Final treatment of the fault zone was completed with deep jet grouting (up to 40 m deep below the dike working platform). Drilling and testing of a larger than planned number of verification core holes was carried out to determine the quality of the treatment.

4.4.4 Increased CSM COW Thickness

Vibro-densification of the Zone 1 fill along the highest portion of the dike did not meet the acceptance criteria even after re-densification works were undertaken. At the end of the vibro-densification work, DDMI made available a Becker Penetration Test (BPT) rig to check the densification of the Zone 1 in these areas. The BPT results confirmed the inadequacy of the densification on a significant portion of the tested length. In the areas considered most critical, the width of the CSM COW was increased from 0.8 to 1.2 m to compensate for the lower than specified fill density, which increases the likelihood of excessive deformation of the fill transferring more load to the diaphragm wall (drag forces and bending moments), potentially leading to increased fracturing at the bottom of the diaphragm wall due to excessive compressive stresses.

4.4.5 CSM Compressive Strength and Stiffness

The CSM method introduces more variability to the final diaphragm wall product than the conventional slurry trench method used on the previous dikes since it does not use plastic concrete produced in a plant under better controlled conditions. The results of tests done in the laboratory on cast samples and samples cored from the wall indicate that a significant portion of the wall was built with plastic concrete of lower strength and higher stiffness than specified. The higher stiffness leads to concentration of stresses which compounds the problem of the low strength. Once the initial results were obtained, the project team worked with the contractor to modify the plastic concrete mix to reduce its stiffness while maintaining or improving the strength, however there was limited success. Additional stress-deformation analyses were conducted to refine the acceptance criteria by reducing the load factor (by removing the uncertainty on the wall density since actual values could be used) and by developing specific criteria for different ranges of wall height and estimated vibro-densification conditions.

4.4.6 Jet Grouting to Treat Local CSM Defects

CSM inclination records indicated that some panels did not meet the panel overlap requirements, especially at depth. Additionally, some unmixed aggregate pockets were observed in the CSM wet bulk samples, verification core, and OPTV images. Unmixed pockets in the CSM concrete are expected to have limited impact because the pockets were found to be randomly distributed and to be sufficiently small to be encapsulated by CSM plastic concrete. Moreover, the CSM plastic concrete is non-erodible, and unmixed aggregates are filter compatible with Zone 1 directly downstream of the cut-off wall. Nevertheless, remedial work was done in the areas where unmixed aggregate pockets were found. Remedial work was completed by re-mixing relatively fresh CSM panels, locally jet grouting CSM defects, and constructing jet grout columns upstream of a CSM panel.

5 DEWATERING AND DIKE PERFORMANCE

Figure 6 shows the A21 Dike configuration during the summer of 2017, before dewatering of the internal pond enclosed by the dike. The internal pond was dewatered using two pumping systems. The Phase A system consisted of large capacity submersible pumps discharging directly into Lac de Gras, as the water quality allowed. Phase B system included submersible pumps discharging into the North Inlet, where the water could be treated before discharge into Lac de Gras.

Figure 6. Aerial view of the A21 Dike (foreground) looking north during construction of the COW in 2017. On the right (background), A418 and A154 Dikes and open pits.

Pond dewatering started after the completion of a fish salvage program to catch live fish within the A21 pond and release them to Lac de Gras, the completion of key dike elements (COW, relief wells, pump wells and dike instrumentation), and after confirming the functionality of the thermosyphon blocks and frozen foundation conditions.

The initial maximum allowed pond drawdown rate was 0.4 m/day. The drawdown rate was permitted to gradually increase during dewatering (up to 0.7 m/day) based mainly on review of instrumentation readings, which indicated responses within the expected range, and visual observations, which did not indicate any signs of poor dike performance. This drawdown rate increase was due to maintaining pumping rates as the pond area was shrinking.

Monitoring of dike performance continued by the design and construction team after dewatering until the substantial completion of the A21 Dike on September 2018, when the A21 Dike surveillance program was transferred to DDMI Operations. Continual monitoring of the dike is ongoing as the A21 Pit is being developed (including blasting and pumping activities) and is required for the lifespan of the dike. At the time of writing this paper and based on the available instrumentation information, the A21 Dike was performing as expected and as intended in the design.

6 CONCLUSIONS

This paper summarizes the construction of the A21 Dike, which began on November 2014 and was substantially completed on September 2018. Based on field observations and QC/QA data collected during the A21 Dike construction, the construction activities were carried out in general conformance with the design and its approved modifications. Despite the challenges encountered during construction, the A21 Dike was completed on time and under budget. At the time of writing this paper the A21 Dike was performing as expected and as intended in the design.

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8 REFERENCES

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