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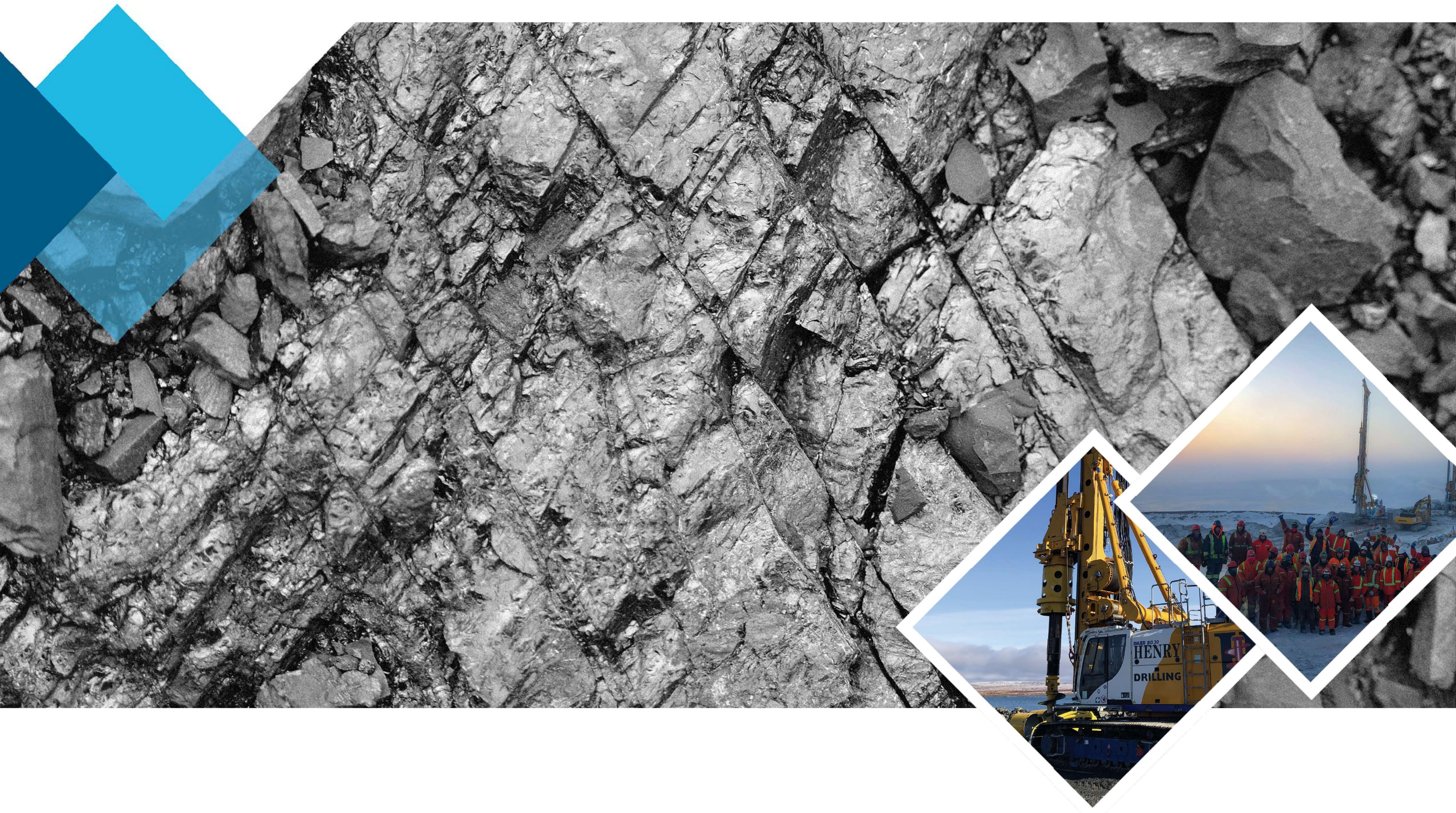
Assistance during construction of the water management and geotechnical infrastructures at the Amaruq project

As-Built Report of Whale Tail Dike – Report Only

Agnico Eagle Mines Limited



Mining & Metallurgy

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Quebec, June 5th 2020

Mr. Yan Côté
Engineering Superintendent
Agnico Eagle Mines Limited
Meadowbank Division
Baker Lake, Nunavut, Canada
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Subject: Detailed Engineering of Water Management and Geotechnical Infrastructures at Amaruq Mine
As-Built Report of Whale Tail Dike
Our file: 658309-0000-56ER-0001-00

Dear Mr. Côté,

We are pleased to submit the final version of the as-built report for the above referenced project.

Do not hesitate to communicate with the undersigned should you have any questions regarding the content of this report.

Truly yours,

SNC LAVALIN INC.

Yohan Jalbert, P.Eng.
Project Manager
Mining and Metallurgy

YJ/bsp

List of Revisions

Revision				Revised pages	Remarks
#	Prep.	Rev.	Date		
PA	MDJ/JL/SM/AA	AA	2019-03-08	All	Issued for internal review
PB	MDJ/JL/SM/AA	AA	2019-03-22	All	Issued for client review
PC	MDJ/JL/SM/AA	YJ/TX	2020-04-09	All	Issued for client review
00	MDJ/JL/SM/AA	YJ/TX	2020-06-05	All	Final Report

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Executive Summary

The construction of the Whale Tail Dike was initiated in July 2018 and completed in June 2019. The construction process of the dike is discussed in the current as-built report.

The construction of the dike includes earthworks such as foundation preparation, fill placement and dynamic compaction, construction of a secant pile cut-off wall made of a cement-bentonite mix, curtain grouting of the bedrock foundation of the dike as well as instrumentation.

The quality control and quality assurance program included foundation preparation and fill placement approvals, casing installation monitoring, cement-bentonite slurry testing and secant piles installation approvals during the construction of the cut-off wall, as well as grout mixes testing, bedrock drilling monitoring and water pressure tests oversight during curtain grouting. During the course of the work, 42 field adjustments and/or design changes were applied to optimize construction activities.

1.0 INTRODUCTION

1.1 Context

Agnico Eagle Mines Limited, Meadowbank Division (''AEM'') is operating Whale Tail Pit, a satellite deposit on the Amaruq property, as a continuation of mine operations and milling at the Meadowbank Mine. The Amaruq Exploration property is a 408 km² site located on Inuit Owned Land, approximately 150 km north of the Hamlet of Baker Lake and approximately 50 km northwest of the Meadowbank Mine in the Kivalliq region of Nunavut. The property was acquired by Agnico Eagle in April 2013 and is subject to a mineral exploration agreement with Nunavut Tunngavik Incorporated. The Meadowbank Mine is an approved mining operation and AEM is extending the life of the mine by constructing and operating the Whale Tail Pit.

As part of this infrastructure, Whale Tail Dike is an important dewatering dike that is required to enable mining of the open pit located in the north part of Whale Tail Lake. The construction of Whale Tail Dike occurred between June 16th 2018 and February 19th 2019 for the construction of WTD up to elevation 157.0 m. The construction of the thermal cap to elevation 159.0 m occurred from July 7th to August 5th, 2019.

1.2 Objective and Scope

This as-built report describes and summarizes all the construction works carried out during Whale Tail Dike construction, including the following elements:

- An overview of the project, the organisational chart during construction, design elements that were emitted as technical memorandums and design variations deemed necessary;
- A summary of the construction activities and a global review of the entire project based on challenges faced by the various stakeholders over the advancement of the project;
- Quality control and quality assurance (QC/QA) tasks, compilation of data obtained from field surveillance and observations as well as laboratory and field testing;
- As-built drawings;
- A summary of the instrumentation installed on the dike.

1.3 List of acronyms

ACCRONYMS OR ABBREVIATION	DESCRIPTION
°C	Degrees Celsius
AEM	Agnico Eagle Mines Limited, Meadowbank Division
CB	Cement-bentonite
CF	Coarse Filter material
cm	centimetre
d/s	Downstream
DTH	Down-The-Hole hammer
El.	Elevation
FF	Fine filter material
GHD	GHD consultant
Henry Drilling or HD or HFDI	Henry Foundation Drilling Inc.
IFC	Issued for Construction
KCG	Kivalliq Contractors Group Ltd
m	Metre
Max	maximum
MDRB	Meadowbank Dike Review Board
MET	Meadowbank Engineering Team
Min	Minimum
mm	millimetre
NAG	Non Acid Generator
PAT	Portable Agitator Tank
QA	Quality Assurance
QC	Quality Control
STA	Station
u/s	Upstream
UCS	Unconfined Compressive Strength
WPT	Water Pressure Test
WTD	Whale Tail Dike

2.0 PROJECT DESCRIPTION

2.1 Description of Whale Tail Dike

The Whale Tail Dike (WTD) is located on a shallow plateau of the Whale Tail Lake floor with approximately 2 m depth of water. This plateau is located between deeper sections of the lake with water depths of approximately 12 m. Once in operation, the downstream side of the dike will be dewatered, and the upstream side of the dike will allow a 3.5 m raise of water prior to being discharged towards Mammoth Lake by gravity. The discharge will be through South Whale Tail Channel, a new blasted channel located west of the WTD. The channel will be built within the existing A20, A45 and Mammoth Lake to reroute 2,400 ha of watershed. The raising of the water elevation will change the thermal regime of the flooded lands that could degrade the permafrost, especially in the Whale Tail Dike area.

Whale Tail Dike was designed as an about 755 m long zoned embankment comprising of rockfill, coarse and fine filter materials. From the original ground, its height is about 8 m, the crest has a minimal width of 13.4 m and built to the elevation of 159 m. The cement-bentonite (CB) cut-off wall was constructed to the elevation of 157.0 m through the central zone which consisted of fine filter materials that were densified by dynamic compaction.

2.2 Extent of Work

Whale Tail Dike construction included the following main components:

- Installation of turbidity barrier;
- Construction of the initial rockfill platforms;
- Foundation preparation;
- Key trench excavation, sediments removal and bedrock cleaning;
- Fill (rockfill, coarse filter and fine filter) placement, compaction and slope profiling;
- Dynamic compaction of fine filter materials;
- Secant pile installation for cut-off wall, including the western extremity called “slurry wall”;
- Curtain grouting of bedrock;
- Instrumentation installation;
- Final rockfill lift to El. 159.0 m.

2.3 Design, Technical Specifications, and Construction Drawings

The detailed design for Whale Tail Dike was prepared by SNC-Lavalin (2018). The Technical Specifications and Drawings for Construction package were issued on July 5th, 2018 (Table 2-1). Table 2-2 presents the list of drawings “Issued For Construction” (IFC).

It should be noted that the drawings mentioned in Table 2-2 are for the curtain grouting completed in 2019. Grouting activities that occurred in 2020 is not covered in this report.

Table 2-1: WTD Design Report and Technical Specifications

No.	AEM #	Title
651298-2400-40EF-0001-00	6118-E-132-002-SPT-001	Technical Specifications for the Construction of Whale Tail Dike
651298-2400-40EF-0001-01	6118-E-132-002-SPT-001	Technical specifications update – Section 5.0 – Drilling and Grouting that has been modified according to site conditions
651298-2700-4GER-0001	6118-E-132-002-TCR-007	Detailed Engineering of Water Management and Geotechnical Infrastructures at Amaruq

Table 2-2: WTD Construction Drawing List

No.	Title	Revision
651298-2500-4GDD-0000	Location map and drawing index	02
651298-2500-4GDD-0001	General arrangement plan	02
651298-2500-4GDD-0002	General plan of field investigation locations	02
651298-2500-4GDD-0003	Whale Tail Dike plan with field investigation locations	02
651298-2500-4GDD-0004	Whale Tail Dike plan view	01
651298-2500-4GDD-0005	Whale Tail Dike excavation, stripping and blasting	01
651298-2500-4GDD-0006	Plan and stratigraphic profile of Whale Tail Dike	01
651298-2500-4GDD-0007	Whale Tail Dike typical design sections (1/2)	01
651298-2500-4GDD-0008	Whale Tail Dike typical design sections (2/2)	01
651298-2500-4GDD-0009	Whale Tail Dike typical sections at 20 m intervals	01
651298-2500-4GDD-0010	Whale Tail Dike construction stages	01
651298-2500-4GDD-0011	Whale Tail Dike instrumentation plan	00

2.4 Roles and Responsibilities

The Drawings and Technical Specifications for Whale Tail Dike were developed by SNC-Lavalin and reviewed by the AEM's Meadowbank Engineering Team (MET) and Meadowbank Dike Review Board (MDRB). Kivalliq Contractors Group Ltd (KCG) was contracted by AEM for the construction of the Whale Tail Dike. Menard Canada Inc. and Henry Foundation Drilling Inc. (Henry Drilling), both specialized contractors, were contracted by KCG as sub-contractors for dynamic compaction and secant pile cut-off wall installation, respectively. The AEM Representative was the Geotechnical Engineer or Specialist on-site and was responsible for managing and planning the construction. AEM's Representative was only present on site during the day shift.

The quality control (QC) program was carried out by GHD consultant (GHD), under the direction of AEM. GHD role was the supervision of the construction work and materials according to the Technical Specifications. GHD oversaw all monitoring tasks, including quality of cut-off wall and grout curtain as well as all the laboratory and field tests.

SNC-Lavalin was responsible for the design of WTD and the quality assurance (QA) program on-site with the presence of QA inspectors and acted as a technical support for the grout curtain. The QA supervisor oversaw the inspector's reports from distance without attending the site. SNC-Lavalin also provided technical input and overseeing compliances with Drawings and Technical Specifications.

The surveyor (under KCG) was contracted by AEM for site surveys including lines, grades, piles locations, casing tolerances and grout hole locations as per design requirements.

Except for the key personnel from the Contractor (and sub-contractor), all QA/QC personnel that worked on-site on a regular basis were on a 2-week in and 2-week out rotation. Due to logistic conditions, this involved a gap or the absence of personal of about 10 hours during the transition for every 2-week rotation. In addition, only QA/QC personnel and the Contractor worked during day and night shift (12 hour shift).

The parties and key personnel that were involved during the construction of Whale Tail Dike are listed in Table 2-3.

Table 2-3: Key personnel involved during the construction of Whale Tail Dike

Name	Title
Agnico Eagle (AEM)	
Julie Bélanger	Project superintendent | Amaruq
Frédérick L. Bolduc	Geotechnical coordinator
Alexandre Lavallée	Geotechnical coordinator
Pier-Éric McDonald	AEM Representative (Geotechnical engineer)
Patrice Gagnon	AEM Representative (Geotechnical specialist)
Olivier Jacques	Dike supervisor
Jean-François Béland	Dike supervisor
SNC-Lavalin – design and quality assurance (QA)	
Yohan Jalbert	Project manager and WTD designer
Ali Ameli	QA supervisor, off site support – Vancouver office
Angie Arbaiza	Quality assurance inspector
Tezera Azmatch	Quality assurance inspector
Mathieu Durand-Jézéquel	Quality assurance inspector
Jonathan Leblanc	Quality assurance inspector
Muhammad Umar	Quality assurance inspector
Tom Xue	Grout curtain designer
Mazin Toma	Grout specialist for grouting
Saleem Muhammad	Grout specialist for grouting

Name	Title
GHD consultant – Quality control (QC)	
Daniel Pedneault	QC supervisor
Marc-Olivier Bernard	Quality control representative
Mathieu Côté	Quality control representative
Maxime Côté	Quality control representative
Denis-Jacques Coulombe	Quality control representative
Cédrick Fillon-Tremblay	Quality control representative
Dominic Fournelle	Quality control representative
Mélina Guay	Quality control representative
Amadou Oury-Diallo	Quality control representative
Daniel Roy	Quality control representative
Kivalliq Contractors Group Ltd – Contractor	
Serge Lalancette	Superintendent
Dany Pageau	Superintendent
Jonathan Gilbert	Nunavut Operations Manager
Bernard Vachon	TCG manager (responsible for contact with HDFI)
Damien Gagnon	Project manager
Christopher Gilbert	Project manager
Jeannot Gagnon	Assistant project manager
Jonathan Audet-Croft	Field technician

Name	Title
Audrey Bilodeau	Field technician
Frédérick Tremblay	Grouting specialist (Secant Piles)
Ken Lachance	Grouting specialist (Pressure grouting)
Marc-André Blackburn	Lead surveyor
Mikaël Lévesque	Lead surveyor
Menard Canada Inc. – Dynamic compaction (sub-contractor of KCG)	
Adrien Viateau	Project manager
Rachid Bennis	Operation and project manager
Hubert Guimont	Technical manager
Henry Foundation Drilling Inc. – Secant piles cut-off wall specialized contractor (sub-contractor of KCG)	
Don Henry	President
Owen Langton	Operations manager
Cole Allester	Superintendent
Alex Zapantis	Project coordinator
Dewet	Project coordinator
External laboratories	
SNC-Lavalin inc.	

The line of communication defined by AEM and applied during the construction process is presented in Figure 2-1.

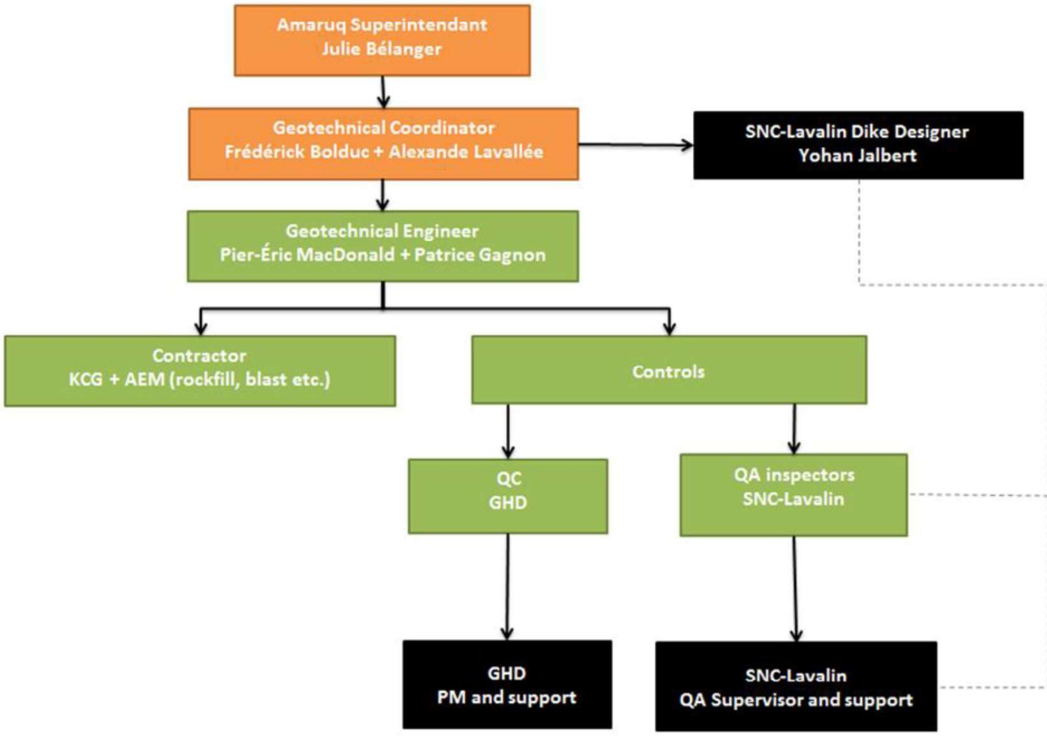

Figure 2-1: Communication chart

2.5 Construction Documentation

AEM was responsible for collection, distribution and storage of related construction documentation which includes the following:

- SNC-Lavalin IFC Drawings and Technical Specifications;
- KCG Daily Construction Report;
- Menard Dynamic Compaction Daily Report;
- Henry Drilling Daily Construction Report – Secant Piles Cut-off Wall;
- AEM Daily and Weekly Construction Meeting Minutes;
- AEM and QA Project Manager Weekly Minutes of Meeting;
- Email communications between all stakeholders;
- QC Shift Reports;
- QC Test Results and Registers;
- Survey Data and Documents;

- Approval Forms;
- Environmental Inspection Reports;
- AEM Daily Supervisor Report;
- KCG Drilling and Grouting Reports during Injection;
- QA Shift Reports;
- Request For Information (RFI);
- Site Memorandums and Technical Memorandums.

2.6 Schedule and Construction activities

The main construction activities of Whale Tail Dike were carried out from July 15th 2018 to August 5th, 2019. Table 2-4 presents the construction schedule for the main work items for the construction of Whale Tail Dike. A detailed construction schedule is shown in Appendix D.

Table 2-4: Timeline of the main construction activities of Whale Tail Dike

Activity	Beginning	End
Turbidity barriers installation	July 15^{th,} 2018	July 27^{th,} 2018
Initial rockfill platforms	July 25th, 2018	August 03rd, 2018
Key trench excavation, sediments removal and bedrock cleaning	August 03rd, 2018	August 13th, 2018
Fill (rockfill, coarse filter and fine filter) placement, compaction and slope profiling	August 06th, 2018	February 18th, 2019
Dynamic compaction of fine filter material	August 28th, 2018	September 17th, 2018
Secant pile installation for cut-off wall	September 1st, 2018	December 4th, 2018
Grouting of bedrock	November 26th, 2018	February 12th, 2019
Instrumentation	January 03rd, 2019	February 19th, 2019
Final rockfill lift to El. 159.0 m	July 7th, 2019	August 5th, 2019

3.0 DESCRIPTION OF THE CONSTRUCTION ACTIVITIES

The work procedures followed during the construction of the Whale Tail Dike are discussed in the following subsections. Selected photographs of the work progression taken throughout the construction are presented in Appendix A. Contractor construction work methods are presented in Appendix J.

3.1 Turbidity Barrier Installation

Five rows of turbidity barriers were deployed prior to material placement in Whale Tail Lake to control TSS during construction work. The design of the turbidity barriers was done by SNC-Lavalin and the installation was done by the contractor under the supervision of AEM. No as-built report is available from this installation.

3.2 Foundation Preparation

The following subsections describe the foundation preparation for each zone. The objective of the foundation preparation was to reach bedrock by removing all the overlying material along the centerline or reach an acceptable foundation (ice rich poor material). The excavation was completed underwater into the key trench and in the abutments from stations ±0+060 to 0+810.

3.2.1 Key Trench Excavation at the Lake portion

The foundation preparation in the lake portion consisted on removing the lakebed sediments until exposing 5 m in width of bedrock. This activity was crucial since the cutoff wall was going to be anchored 1 m into bedrock. In order to allow excavation of the key trench in this sector, the initial rockfill platforms were constructed (refer to section 3.3.1). The excavation of key trench started at the west lakeshore, progressing east. The key trench started from about station 0+130 to 0+730. As expected, the highly turbid water limited the observations during the cleaning of the bedrock. In general, two excavators were used for the cleaning of the key trench. An excavator, equipped with a bucket with teeth (e.g., CAT 395D and Komatsu 490), removed the bulk of materials. Then, a long-reach excavator equipped with a bucket with a lip (e.g., Komatsu PC 490 LC) carried out the final clean-up. The shovel removed all lakebed material present in the trench until bedrock was reached and a clean bucket was observed.

During the trench excavation, a zone of heavily fractured bedrock was encountered from Station 0+360 to Station 0+370. In this sector, the theoretical bedrock was supposed to be at elevation of ±147.0 m. Nevertheless, the bedrock was encountered at a lower elevation and the quality of the rock was poor. Cleaning of the key trench had to be stopped at an elevation of 144.0 m (maximum reach of the long-reach excavator) without reaching a satisfactory (or sound) bedrock surface. The key trench was approved without going deeper than 144.0 m with the intent of additional grouting in this zone. The rockfill platforms were then re-profiled accordingly.

As specified in the construction drawings, a frozen foundation was expected between stations 0+150 to 0+200 and between 0+500 to 0+700. Therefore, more stringent controls were implemented in this zone in order to validate if the bedrock had been reached (more details in section 4.2). Presence of frozen material was identified between 0+555 and 0+575 and additional cleaning was done. In the west abutment, the esker seemed to be frozen between stations 0+140 to 0+195 but thawed fast without apparent ice lenses.

Procedures were adapted to let the foundation melt for a certain time before continuing excavation until reaching a stable foundation.

3.2.2 West Abutment Excavation

The key trench at the west abutment was excavated above water level from about Station 0+060 to Station 0+140. The excavation and cleaning were completed using a Komatsu 450, a CAT 345D and a dozer D8T with a ripper.

No ice lenses were observed during the excavation of the esker material at the west abutment (up to station 0+130), or into the bedrock. From Station 0+155 to 0+180, bedrock surface was significantly above inferred bedrock surface. Excavation activities were performed with caution in this zone to make sure that it was indeed the bedrock surface and not remaining frozen esker material. The excavation from Station 0+060 to Station 0+140 ended in bedrock. Bedrock in this area was found to be highly fractured and irregular with outcrops that required additional excavation of the foundation with a hydraulic rock hammer mounted on an excavator to obtain a reasonably even surface. The smooth surface was required to ease a good compaction of fine filter and to reduce risks of problems during the installation of the piles.

Following the excavation of the abutment, it was assessed that the slurry trench would be built between Stations 0+069 and 0+092 and the first secant pile from station 0+089 going eastward, based on actual bedrock profile and the minimal 1.5 m depth operational requirement for secant pile drilling.

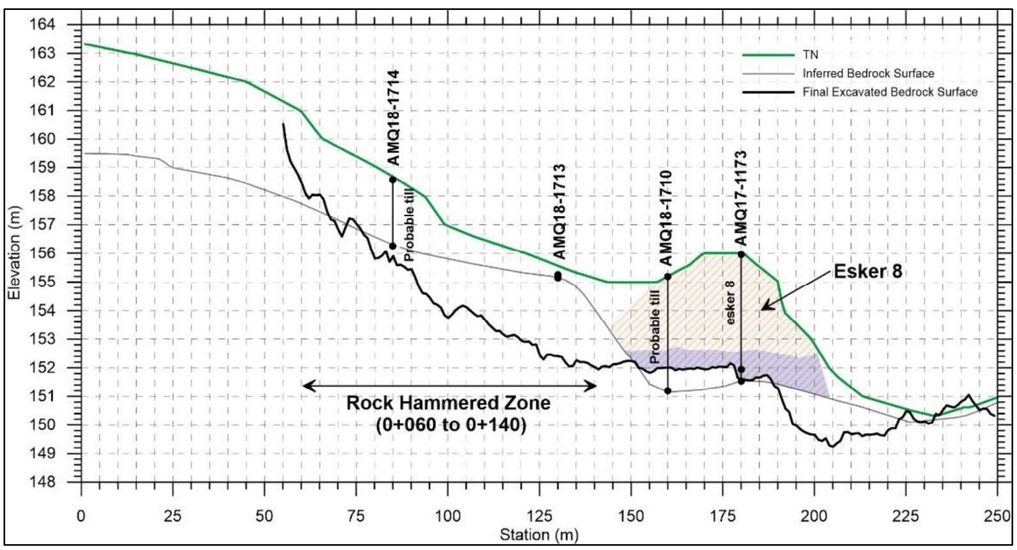

Figure 3-1: Excavation of the West abutment

3.2.3 East Abutment Excavation

After leaving about 30 m plug from shoreline to work dry, the east abutment is identified from Station 0+727 to 0+835. Blasting was originally intended for this sector, but its excavation was not on the critical path of the schedule and material was easily excavated by letting it thaw on several days. During the excavation of the east abutment, ice lenses were observed especially from Station 0+773 to 0+810. It was decided to remove all ice-rich material from Station 0+750 to 0+820. The ice was removed about 2.5 m deeper than initially expected. Highly fractured bedrock was reached in the footprint between Stations 0+739 and 0+772. From 0+772 to 0+808, the foundation was on frozen soil visually free of excess ice.

The excavation of the east abutment stopped at Station 0+835, where the natural ground reached the elevation 157.0 m.

3.3 Fill Placement and Compaction

The construction of Whale Tail Dike implies the use of three (3) types of materials that were produced on site: Rockfill, Fine Filter (FF) and Coarse Filter (CF) material. As mentioned previously, rockfill material comes from the quarry and consists of suitable, NAG and non-leachable material. The produced Fine Filter material (FF) had a nominal gradation between 0-20 mm, the Coarse Filter materials (CF) from 0-150 mm and the rockfill from 0-1000 mm.

All controls and gradations are discussed in section 4.0.

The placement of fill materials can be summarized by the placement of (1) the initial rockfill platforms, (2) the materials into the key trench up to the elevation of the rockfill platforms and (3) all materials above these platforms.

3.3.1 Initial Rockfill Platforms

According to the construction drawings, two (2) working platforms at elevation 154.0 m were specified to keep the working platforms above the expected WTL water level of 153.5 m. However, since the lake elevation after freshet was lower than expected (152.7 m), it was agreed between AEM, the QA and the Designer to lower the rockfill platforms to elevation 153.5 m to reduce the amount of rockfill material required for construction.

Both upstream and downstream initial platforms were constructed simultaneously between stations 0+175 and 0+725 using 0-1000 m NAG and non-leachable rockfill. Rockfill placement started at the east abutment of WTD and progressing towards the west abutment. Rockfill material was transported from the stockpile area to the platform location with hauling trucks (100 and 150-tons) and the material was dumped on the platform and spread using two (2) D8 bulldozers, one of which was equipped with GPS on its blade. External safety berms, which were part of the rockfill platforms, were maintained during the whole construction and were kept for safety purposes since both platforms were used as access roads to circulate from east to west abutments. Inner safety berms along the key trench were gradually removed during construction to be able to work in the trench.

Since the platform elevation was modified from 154.0 m to 153.5 m, the rockfill containing particles with more than 0.5 m dimensions was no longer suitable at the west abutment. Hence, it was decided to place coarse filter material instead of rockfill between Stations 0+140 and 0+195.

3.3.2 Fill Placement in Key Trench to El. 153.5 m

The key trench located in the sector of the lake extends from 0+205 to 0+727. The fill placement to El. 153.5 m is comprised of fine filter and coarse filter materials along the initial rockfill platform slopes.

The fine filter (FF) and coarse filter (CF) materials were transported with hauling trucks and were placed with the bucket of the Komatsu PC 450 LC excavator, equipped with an onboard GPS, from the bottom of the key trench up to el. 153.5 m (upward placement) to prevent particle segregation. The underwater control points along the toe of the placed CF were taken with the GPS-equipped excavator. Prior to any placement, a cleaning of the slopes was completed. The surveyor also checked the control points at the crest to verify if the required slope profile was met as per design drawings.

Due to the significant lakebed thickness in some areas, the excavators that were working in front of the fill placement front had to do several passes to ensure a proper cleaning of the key trench. The sediments would accumulate in the cleaned key trench if not backfilled right away. In some occasions, the bathymetric survey was conducted after a first pass of cleanup but QA/QC representatives requested a second pass of cleanup to be carried out. GPS spot checks after final cleanup revealed that the actual lakebed elevation was lower in elevation compared to the initial bathymetric survey. As an impact, the actual thickness of coarse filter material was lower compared to the Drawings since the projected lines and grades were done using the original bathymetric survey. The rockfill and coarse filter slopes had to be reprofiled to meet the required specifications.

To mitigate the risk associated to potential construction delays, AEM implemented a new work method consisting of projecting the slopes as if the bedrock was one metre below the surveyed lakebed elevation. This buffer ensured that the minimum thickness of the coarse filter met the Drawings in circumstances where second cleanups lowered the lakebed elevation initially approved.

The placement of the FF underwater was done by placing the bucket of the excavator at the bottom of the trench and filling the trench from the bottom towards the top to avoid segregation. The CF was placed first and the front of the advancement was kept about 5 m in front of the FF placement to avoid cross-contamination of the filter materials.

As mentioned in section 3.2.1, a deeper than expected excavation of the key trench was performed between Stations 0+360 and 0+370. Therefore, the placement of the filter material had to be adjusted for this portion of the trench. The coarse filter material was placed first on the downstream slope of the key trench to provide better access for the clean-up of the center portion. To avoid CF of the upstream side reaching the centerline, a blanket of FF material was placed first up to elevation 148.0 m. This was followed by placement of coarse filter material on the upstream slope of the key trench. From 148.0 to 153.5 m, FF placement occurred as originally planned by pushing the material downward in the trench using the bucket of the excavator.

3.3.3 Fill Placement from Elevation 153.5 m to 157.0 m

The fine filter placement was carried out with the bucket of an excavator in one lift to elevation 157.0 m. Compaction of fine filter was initially carried out by the weight of the excavator, by tamping with the excavator's bucket, and finally dynamically (refer to section 3.3.5). The compaction process was monitored visually.

Strips of coarse filter material were placed on the rockfill platform at elevation 153.5 m to avoid fine filter to be sitting directly on rockfill material. This field adjustment was necessary because the base of the filters was wider due to the lower elevation of the rockfill platforms.

The section of the slurry trench was built from 0+065 to 0+090 by placement of fine filter material in 500 mm lifts and its densification was achieved with a P10000 vibratory plate compactor. Between stations 0+330 and 0+385, placement of the filters was completed in two (2) lifts to ensure an appropriate compaction in the deepest section where the core was densified in two (2) stages as discussed in section 3.3.5. From stations ±0+740 to 0+800, rockfill was placed as thermal cap on the u/s side of the excavated foundation and a layer of CF was placed over it. The purpose of this modification was also to avoid water ponding on the excavation and preventing FF migration into poor and unsupported natural ground.

The overall surface was compacted with the dozer (D8 dozer) and the passage of trucks.

The rockfill placement was carried on both side of the dike's centerline in two (2) lifts of 1.5 m and 2.0 m from elevation 153.5 m to 155.0 m and from elevation 155.0 m to 157.0 m, respectively, by a dozer (D8 bulldozer). Compaction requirement was met for each lift with passes of the dozer and the traffic of heavy machineries/equipment, according to Technical Specifications.

3.3.4 Fill Placement from Elevation 157.0 m to 159.0 m

Rockfill material from elevation 157.0 to 159.0 m was placed several months upon completion of the dike up to elevation 157.0 m. A bulldozer was used to place the final lift of rockfill material in July and August of 2019.

3.3.5 Dynamic Compaction of FF

Dynamic compaction of the core of WTD was performed by a specialised contractor, Menard Canada (Menard), under KCG supervision. The compaction started at station 0+092 to keep a certain distance from the slurry wall (that ends at 0+089) and finished at station 0+840. Menard's work program is presented in Appendix J-1 and was developed based on Meadowbank Bay Goose Dike's design requirements.

The compacted area was subdivided in six (6) sectors or zones based on the thickness of the material to be densified. The compaction energy was generated from a 15-ton tamper lifted by a crane and dropped from 18 m. The purpose of the work was to densify the FF core of the dike.

The work was performed from the working platform on fine filter surface at elevation ~ 157.0 m, except from Station 0+330 to 0+385, where the compaction sequences were performed in two stages: at elevation 154.2 m and then at elevation at 157.0 m. This was due to the thickness of fine filter material within these stations associated with deeper bedrock foundation than expected. At the end of each sequence, the produced craters were backfilled with fine filter material up to elevation 157.0 m.

KCG conducted a final dynamic roller compaction (12 tons Caterpillar CS76) at the surface after having backfilled all craters. Two (2) passes were performed at the center of the fine filter zone while three (3) passes were performed on the coarse filter (downstream) and at the transition between fine filter and rockfill (upstream). An overlap of 1.0 m (half the length of the roller) was maintained during compaction while covering the entire width of the fine filter zone.

3.3.6 Quantities

The quantities of the fill material placed during the construction of WTD[2] are presented in the table below.

Table 3-1: Quantities of NAG material

TYPE OF MATERIAL	QUANTITIES
Fine filter	*55,279m³*
Coarse Filter	*29,974m³*
Rockfill	*115,764m³*
Excavation	*53,398m³*

3.4 Secant Pile Cut-off Wall

In general, the secant piles were constructed by using three (3) drill rigs (i.e. two BG30 and one BG28). Each drill rig had different setup depending on its required equipment, whether the drill rig was used for casing installation or rock socket drilling. The secant piles were drilled with a spacing that varies between 0.75 and 0.80 m (center to center). A summary of this activity is presented in Table 3-2 and as-built documentation is available in Appendix J. The construction of the secant pile wall started from the West towards the East. All piles started with the installation of a 1.0 m diameter casing pushed to the bedrock by removing the material inside the casing using an auger. Once the bottom of the casing was anchored into bedrock, another drill rig was mobilized to drill a minimum of 1.0 m deep into the bedrock using a 0.9 m diameter hammer drill bit. Once the drilling process completed and the secant pile cleaned, the piles were filled with a cement-bentonite mix using the tremie method. Details of the secant piles installation are presented in this section.

[2] From WT_PROGRESS_QUANTITIES FINAL.xlsx, in table "Total of each type of material "

Table 3-2: Summary of Secant Piles at WTD

Station	0+089 to 0+832
Secant pile quantity (unit)	978
Spacing (Center-to-center)	0.75 – 0.80 m
Average height (m)	8.7
Maximum height (m)	18.3
Total length of piles (m)	8 540

3.4.1 Casing and Rock Socket Drilling

The casing used for the secant piles had an external diameter of 1.0 m and a wall thickness of 5 cm. Each casing was drilled and pushed within the densified fine filter material until bedrock surface was reached. Then, the casing was drilled further into the surface bedrock (approximately 300 mm) until it seemingly seated on an even bedrock surface. The fine filter material within the casing was then removed by an auger while pushing the casing deeper until bedrock was reached.

A drill rig mounted with a Down-The-Hole (DTH) hammer, consisting of seven (7) 6 inches drill bits mounted below a calix basket to retrieve cuttings, was used to drill 1.0 m depth of rock socket at a diameter of 0.9 m below the casing shoe. Compressed air was used during drilling and at the end of the process to clean the hole.

Henry Drilling employed an alternative method for socket drilling in poor quality rock starting October 6th, 2018 for primary and secondary piles. This was done to avoid a significant slurry loss at the casing shoe elevation near the bedrock surface. This alternative method consisted in using a core barrel to penetrate approximately 1.0 m into the bedrock and pushing the casing deeper into bedrock (e.g. in Pile 362). The DTH hammer was then used to clean the rock socket without drilling any deeper into bedrock. Drilling of rock socket for tertiary piles was carried out using a DTH hammer.

Some problems occurred during casing drilling operations to seat the casing of Pile 374 (anchoring) on the bedrock surface. At a depth of approximately 8.0 m during the drilling process, the casing was stuck and could not be removed. HFDI drilled an additional pile (374b) next to the casing of Pile 374 to release pressure probably caused by fine filter friction along the casing and extract the casing. Pile 374b is intersecting Piles 375 and 376 and was drilled at the same target depth as Pile 375. Henry Drilling also encountered problems after trying to remove the casing of Pile 680, which was stuck after being filled with CB slurry. An additional Pile was added (678b) to move the casing of Pile 680. Pile 680 had to be re-drilled and filled with CB mix again with Pile 678b. Another pile (#355) encountered the same problem with a casing stuck into the embankment. To extract the casing, an excavation of 3 m deep was done downstream of the wall. The cavity was backfilled with fine filter and compacted with the bucket of an excavator.

3.4.2 Cement-Bentonite Slurry Preparation

The batch plant for CB slurry production was provided and operated by Henry Drilling. A layout is shown in Figure 3-2. The procedure for CB slurry preparation at the batch plant, located at approximately 100 m south of the east abutment, is as follows:

- The bentonite slurry was prepared by mixing bentonite with lake water (heated in boiler tanks) in a separate mixer for approximately 5 minutes (bentonite was not pre-hydrated but a high shearing method was used and believed to be equivalent to pre-hydration for 24 hours);
- Dry cement was mixed with water separately from the bentonite slurry in another mixer;
- The bentonite slurry was then pumped with the cement mix in one of the two Portable Agitator Tanks (PATs), having a 5.5 m^3 capacity each, and mixed for 10 minutes;
- The Lignosulfonate powder admixture (ARBO S01P) was mixed with water using a manual mixer, before being added to the PATs.

Production of cement-bentonite slurry started on September 10th 2018 but many problems occurred at the batch plant and the first pile was poured on September 16th, 2018.

A contingency plan for manual mixing was put in place after some issues between October 20th and 27th 2018. HD and KCG decided to mobilize the injection unit (required during the grouting operations) at the batch plant to produce the cement-bentonite mixes in order to be able to produce CB slurry in case of major shutdown of the cement plant. This contingency option was only used during night shift on October 26th and 27th 2018 and was then retained as a stand-by option until the end of the cut-off wall construction.

The manual procedure adapted for CB slurry preparation using the injection unit went as follows:

- Buckets were manually filled (18 kg of dry cement in each bucket) and mixed with water in the injection unit mixer. The volume of water in the mixer was controlled manually. The cement mix was then pumped in the PATs. A total of six (6) cement mixes were needed to fill both PATs;
- Bentonite slurry was prepared at the batch plant, as in the original procedure, and pumped in the PATs;
- Admixture was then added into the PATs as in the original procedure.

The mix proportions were nevertheless subject to slight fluctuations due to inaccuracies in the weighing system and other operation issues. Controls were adapted according to this procedure and discussed in section 4.5.

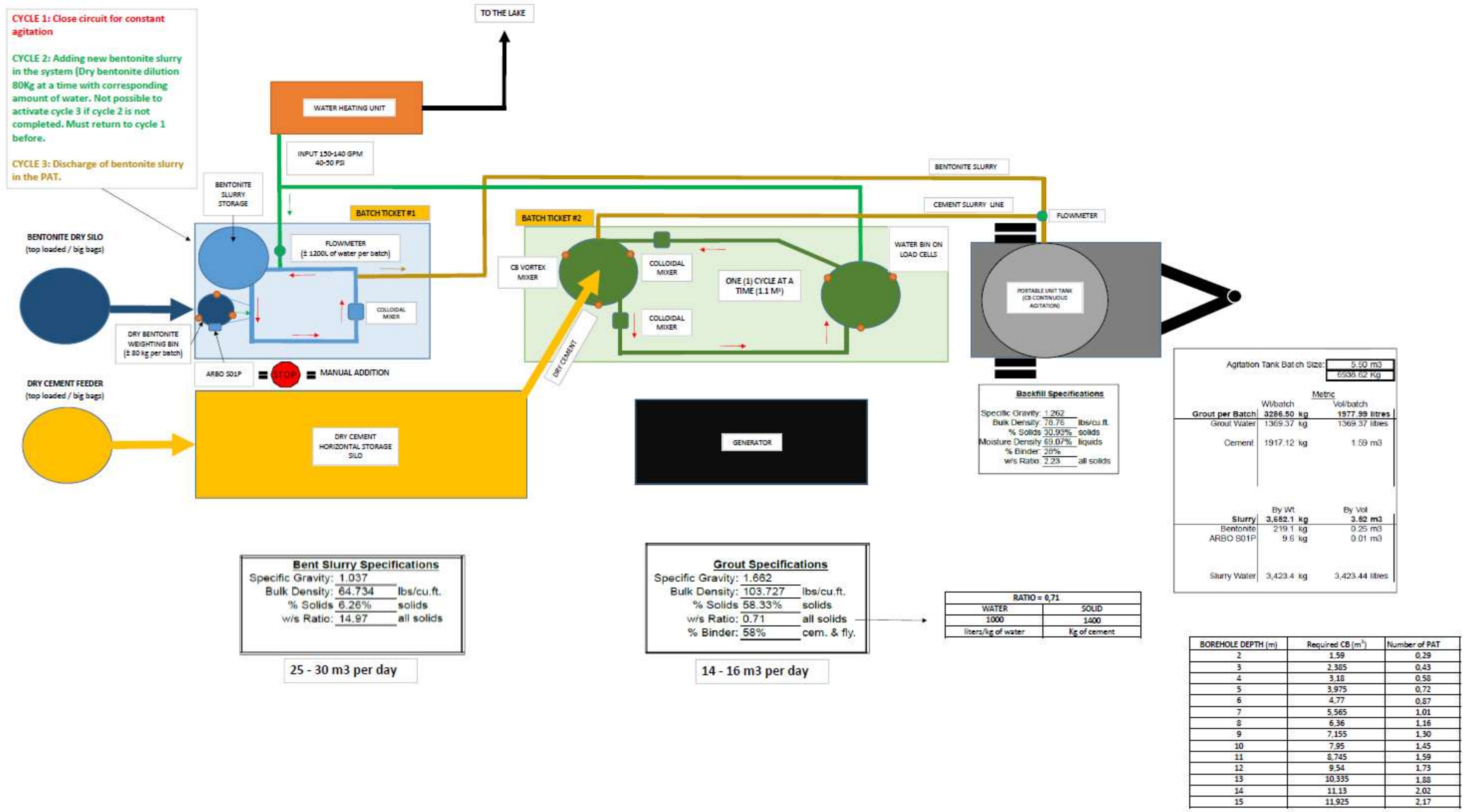

Figure 3-2: Layout of Batch Plant (From KCG)

3.4.3 CB Slurry Mixes

CB slurry mix proportions, initially from proportions used in the laboratory during design phase, were adjusted to site conditions during the few initial trial mixes conducted by KCG. The initial cement-bentonite slurry trials produced at the beginning of the production were placed in the slurry trench located in the west abutment.

The cement content in the slurry mix was further decreased over the course of the CB slurry production for several reasons:

1. Controls on the mixes gave results above criterion (compressive strength).
2. The quantity of CB slurry to be produced compared to theoretical was higher due to slurry loss[3] ;
3. AEM's concerns on limited amount of GU cement in the site inventory;

From November 30th to December 4th 2018, HE cement available on-site, was used as the inventory of GU cement ran out. To do so, trial mixes using HE cement were performed by varying the cement/water ratio to establish the required proportions in order to respect the Technical Specifications. These trial mixes were proposed and established in advance as it was expected that GU cement would run out prior to the end of the construction.

Table 3-3 shows the nominal mix proportions that were approved by the Designer and/or QA throughout the CB slurry mix activity.

[3] around 50% more based on AEM Representative

Table 3-3: Cement-Bentonite Slurry Mix Proportions

ID	Mix A or B (1)	Mix C	Mix E (2)
Description	**Initial batch plant setup**	**Approved modification (cement decrease)**	**Mix using HE cement**
Weight of Cement (kg)	1917	1770 (1725-1770)	1920-1943
Weight of Bentonite (kg)	219.1	220.0	220.0
Bentonite Moisture Content (%)	11	11	11
Mix Water (kg)	4755	4904	4795-4873
Admixture (kg)	0-9.6	9.6	9.6/4.8
Density (g/cm^3)	1.26	1.24	1.26
Cement Content (kg/m^3)	351	307-324	348
Cement: Water ratio (C/W)	0.40	0.36	0.40
Bentonite / Water ratio (%)	4.1	4.0	4.0 - 4.1
Admixture / Cement (%)	0-0.50	0.54	0.25-0.50
Dates Used (2018)	September 01 to October 08	October 09 to November 30	November 30 to December 04

Notes

(1) Mix A has an admixture and Mix B has no admixture.
(2) Mix D was only tested but not used for production.

3.4.4 Cement-Bentonite Transportation and Pouring

The transportation method of CB slurry on WTD was subject to changes and modifications over the course of the construction, especially with regards to weather conditions, as follow:

- CB slurry was initially transported in Portable Agitator Tanks (PATs) mounted on a trailer and pulled by a loader which allowed transportation while stirring the slurry mix. The PATs were later used as fixed agitator tanks to mix hydrated cement, hydrated bentonite and admixture together;
- KCG considered the option of slurry transportation by pumping the slurry through a pipeline from the batch plant to pile locations. This option was not successful during the trial stage due to pumping issues and was not used during the construction;
- KCG finally decided to transport CB slurry from the batch plant using a vacuum truck. The tank on the truck was insulated to protect CB slurry against freezing. The truck was equipped with a pump system to transfer the slurry into the tremie pipe. During freezing conditions, the pump system was protected in a heated trailer moved by an excavator. The truck capacity was of 11.0 m^3 which corresponded to the volume of two (2) full PATs to fill the vacuum truck. During the work, KCG were requested to change the work method to avoid traffic on the cured piles with the equipment.

Placement of CB slurry was done inside the casing and through a tremie pipe placed at the bottom of the rock socket. Interruption in the tremie placement was not allowed, and the piles were backfilled with CB slurry from bottom to top in one go though. However, interruptions occurred in several instances when the tank would get emptied during the backfilling of the piles. Placement of CB slurry was stopped when there was no more clean water coming out at the top of a casing but CB slurry.

3.4.5 Casing Extraction and Top-Ups

Once the piles were filled with CB slurry, the casings needed to be removed before the slurry gel time was reached and thus avoiding damages on the periphery of the pile. The gelation time was calculated from the moment of contact between water and cement. After pouring the CB slurry, it was observed that the slurry level dropped over time due to permeation into the fractured rock socket, slurry leakages at the casing shoe-bedrock interface, or penetration into the fine filter around the periphery of the pile after the casing was extracted. A portion of the drop was the result of the compensation for the volume of steel casing with a wall thickness of 0.05 m.

To compensate these drops, top-ups were required in most of the piles to ensure that the top of the cut-off wall reached elevation 157.0 m as per design drawings and Specifications. To show the importance of horizontal cold joints, QA also provided a technical memorandum (Appendix B[4]) on cold joint issues including mitigation options. The advice was to top the piles within the gelation time of the CB slurry to avoid cold joints formation. The gelation time was typically of 5 hours, depending on the mix proportions and cement type.

3.5 2019 Curtain Grouting

The grout curtain is composed of a single row located 0.7 m offset at the upstream side from the centreline of the secant pile cut-off wall. According to the revised specifications, primary holes were designed 12 m center to center with secondary holes in mid-space between primary holes using a split spacing method. Both primary, secondary and tertiary holes were mandatory holes and quaternary holes were added if a high grout take was observed in the adjacent holes or the adjacent hole could not be effectively grouted due to caving in the hole. Similarly, quaternary holes were recommended if a high grout take was observed in one of the two immediate adjacent holes.

The original design required a grout curtain be installed between Station ±0+180 and ±0+730. Based on the bedrock conditions, observed permafrost close to the east abutment and time constrain, rock grouting was only carried out between Station ±0+180 and ±0+516.

Three (3) stages per borehole were carried out for grouting of a typical ±10 m borehole within the bedrock:

- Stage I represents the bottom stage of grouting (6 m to 10 m);
- Stage II was the middle stage between 2 m and 6 m;
- Stage III represents the top stage close to the bedrock surface (0 – 2 m).

[4] 658309-0000-64ER-0001-00 | Gel/Set Time – Slurry Placement – Casing Extraction – Cold Joint

Target pressure was calculated at the center of each stage prior to start of grouting. The target pressure was calculated by adding 0.2 bar for each meter length of hole in the fine filter material and 0.5 bar for each meter length in the bedrock. Adjustments were made for groundwater and grout column in the grout lines above the mid-stage point for calculating the required gauge pressures during the injection process. Due to the low grout takes observed in the top stage in majority of the holes, effective on January 14th, 2019, the pressure was increased by 0.5 bar for Stage III.

On-site grouting instructions were discussed in the daily construction meeting among KCG's Grouting specialist, AEM Representative and SNC-Lavalin's grouting specialist.

The casing drilling for the first series of grout holes started on November 28th , 2018, while the cut-off wall was being completed at the eastern end of Whale Tail Dike. The grout holes were then drilled into the bedrock and the first primary hole (P-504) was grouted on December 15th , 2018. After completing the mandatory primary holes and as advised later all the secondary holes, the tertiary holes were drilled and grouted if high grout take (>200 L/m) was observed in the preceding series of holes. Drilling in bedrock was first carried out in the primary holes. However, due to the poor bedrock condition and time lagging between drilling holes and grouting, it was observed that some holes collapsed before grouting was completed. Instructions were provided to perform downstage grouting to mitigate the hole collapse issue. But due to the logistic and time constrains, the contractor made some changes on the drilling operation to minimize the time lagging between drilling crew and grouting crew.

Eight (8) quaternary holes with a spacing of 1.5 m were drilled and grouted due to high grout takes observed in adjacent holes.

Casings backfill started after most of the grout holes were completed. The grout for casing backfilling was cement grout with water/cement ratio of 2/1 and 7.5% bentonite. The use of Mix B (presented in section 3.5.1) was agreed to be used to backfill the casings from top.

3.5.1 Grout Mixes

Field trail tests were performed prior to start grouting to ensure that the grout mixes met the viscosity, stability and strength requirements. Five (5) grout mixes; Mix A, Mix A+, Mix B, Mix C and Mix D with different water-cement ratios and admixtures were tested on December 12th and13th, 2018. The QC representative conducted tests on the grout mixes to assess the physical properties of each mix. Three (3) additional mixes; Mix D+, Mix E and Mix F were developed later on with higher viscosity values, considering the observation of very high grout takes in the fractured bedrock. Mix D*, Mix E, and Mix F were developed during production without trial mixes prior to their use.

The composition and properties of various mixes used for grouting at WTD are shown in Table 3-4.

Table 3-4: Grout Mixes

Ingredient	Unit	Mix A	Mix A^{+}	Mix B	Mix C	Mix D	Mix D^{+}	Mix E	Mix F
Water	Litre	56	56	56	53	50	50	50	50
Cement Type III	kg	80	80	80	80	80	80	120	100
Glenium 3030NS	ml	320	320	-	-	-	-	-	-
Rheomac 450	ml	-	-	-	300	432	432	432	
Masterset FP20	ml	-	600	-	-	-	-	-	-
Celbex 653	ml	-	-	-	-	-	250	375	
W/C Ratio		0.7	0.7	0.7	0.7	0.6	0.6	0.4	0.5
Expected Density Results									
Mud-Balance	g/cm^3	1.66	1.67	1.67	1.70	1.72	1.72	1.93	1.83
Mix Test Results									
Marsh Funnel time	s	39	36	41	119	150	>180 (1)	>180 (1)	n/a
Mud-Balance	g/cm^3	1.67	1.64	1.65	1.70	1.69	1.64	1.80	n/a
Initial Set Time	min.	375		458-305	331-429	330			
Lombardi Plate	Pa	3	n/a	5.6	21.9	22.5	22.5	n/a	n/a
Bleeding after 2 hrs	%	3.3	4.5	2.6	1.0	1.0	n/a	n/a	n/a
(1) Mixes are too thick to flow through Marsh Funnel.									

3.5.2 Grout Injection

Initially high mobility grout (Mix A) was used and if pressure did not build up, thicker grout mixes B, C, D, D*, E and F were injected. Grout injection continued until refusal criterion was reached under maximum pressure (P_{max}) or the maximum grout volume (Q_{max}) was injected. In most cases, when P_{max} for a stage was reached and grout flow rate came below 1 liter to 3 liters per minute, the P_{max} was maintained for 10 minutes before the grouting of the stage was terminated.

According to the specification, after a stage is completed, the supply valve should be closed, and the packer kept inflated until the pressure in the hole is dissipated. Due to freezing conditions observed in most of the holes in the upper part of the pipes exposed to the air temperature, the packer was deflated shortly after the completion of a stage and the grout pipes were removed without waiting for the pressure inside the hole to naturally dissipate to zero. The two (2) other stages were injected in a similar manner.

All holes were grouted with ascending or upstage grouting method (from bottom to top) except P-378 and S-402 that were grouted with combined descending and ascending methods. In general, high grout takes were observed in holes between Stations 0+324 and 0+468 and some other scattered locations as well. The top stage in the majority of the holes was observed as low grout take.

P-180 and P-192 were each grouted with a single stage due to ice observed in the hole during drilling. Mix A* with cold weather admixture (MasterSet FP20) was used due to permafrost zone. Grouting of S-186 was cancelled due to hole in the permafrost zone and the casing was backfilled prior to bedrock drilling.

Holes P-348, S-354, S-384, P-396, P-468, S-486, S-498 and S-510 collapsed prior to grouting and could not be cleaned initially to install packers. These holes were re-drilled and re-grouted.

In some holes, grout bypass was observed, and the situation was fixed by raising the packer about 1 m. In a few other grout holes, grout bypass was observed in more than one instance or the packer could not be placed and the two (2) stages were combined and the hole grouted in two (2) stages. The as-built data such as stage lengths, grout takes, and P_{max} were presented in an excel file (WTD_INJECTION CASING_REV4.xls, received February 21st , 2019 and WTD drilling and grouting.xls received February 23rd, 2019) which are presented in Appendix K-5.

3.5.2.1 Water Pressure Tests

Water pressure tests were performed in some of the holes proposed by the Designer to evaluate the hydraulic conductivities and groutability of the foundation rock. Due to the limitation of the contractor's equipment and time constrains, only a limited number of water pressure tests were carried out for comparison purposes.

Water pressure tests were performed by inflating the packer at the top of the bottom stage and then water was pumped in the hole by applying 1/3 of the predetermined maximum target pressure (P). When pressure stabilized, the water flow rate and injection pressure were measured every minute for five (5) minutes. A stage was completed in five (5) steps by applying 1/3P, 2/3P, P, 2/3P and 1/3P. A water test at the middle and top stage was completed by using a double packer and following steps similar to the bottom stage.

Water pressure tests were completed in holes P-516, S-474, T-357, S-306, T-297 and T-267.

3.6 Instrumentation during Construction

In order to monitor the ground temperature near the secant piles, a thermistor string was installed on September 21st, 2018 at station 0+336, 2.25 m upstream of the dike centerline. Originally planned to be a temporary installation, the thermistor was not removed and is still functioning.

Two (2) additional thermocouples were installed in secant piles by QC personnel during the cut-off wall construction. Thermocouples were installed in piles 142 (centre of the pile) and 254 (downstream border of the pile) to follow temperature evolution of CB slurry. Temperature was measured from October 08th, 2018 to October 12th, 2018 in pile 142 and from October 20th, 2018 to November 23rd, 2018 in pile 254.

Readings of all the instrumentation installed during the construction works are presented in Appendix C-3.

Both upstream and downstream water level was measured between August 03rd and November 02nd 2018 by KCG surveyor. During this period, measurements showed that upstream and downstream water level was constant at an elevation of 152.7 m.

3.7 Permanent Instrumentation

Instrumentation installation report and drawings are presented in Appendix C-2.

3.8 As-Built Drawings

Following construction and using the survey data provided by AEM and KCG, As-Built drawings were prepared by KCG and reviewed by AEM. Table 3-5 presents a summary of the most current as-built revision of the drawings. A complete set of as-built drawings is included in Appendix K-3.

Table 3-5: List of As-Built Drawings

No.	Title	Date
CON-FD-164	Plan View As-built	2019-06-05
CON-FD-078	Slurry Wall As-built	2018-10-03
CON-FD-169	Instrumentation Plan View As-built	2019-06-05
CON-FD-173	Profile Secant Wall and Instrumentation As-Built	2019-06-05
CON-FD-173	Profile Secant Wall As-Built	2019-06-05
CON-QT-119	Final quantities – Rockfill Capping El. 159.0	2019-08-06

4.0 QUALITY CONTROL AND QUALITY ASSURANCE (QC/QA)

4.1 General

As previously mentioned, GHD, working under the supervision of AEM, carried out the QC program as per the Drawings and Technical Specifications for all the construction activities. The daily reports prepared by the QC Representatives are presented in Appendix I-7. Daily surveys were conducted by KCG to verify quantities under AEM's supervision. The supervision of drilling operation was carried out by QC representatives and quantities/tolerances measured by the Surveyor. The QC and the Surveyor provided the results to QA and QA approved if control processes were in place and if the specifications criteria were met. QA representatives conducted visual inspection of the work and reported the facts to the AEM Representative. Deviations from drawings and specifications were also collected by the QA representative and reported to AEM Representative and the Designer for their best decisions.

QA did not instruct, supervise and/or measure the quantities of the Contractor or Surveyor work on-site. QA did routine inspection of Work, carried out some spot checks, and compared the data presented in various spreadsheets. The Daily and weekly reports from the QA are presented in Appendix I-1 and I-2 respectively.

The AEM Representative conducted inspections of the work during the construction of WTD on a routine daily basis and was supported by the Dike Supervisor in the daily supervision of the Work. AEM did not have a representative during night shifts. Instructions were exchanged before and after each shift.

The Daily Construction Meetings for Whale Tail Dike activities were held every morning with all parties present on-site, i.e. AEM, GHD (QC), SNC-Lavalin (QA), KCG (Contractor) and their sub-contractor, Menard and Henry Drilling. The surveyor was not always present in the meetings but was represented by KCG. The meetings covered a review of the construction progress of the previous 24 hours, planning for the following 24 hours, discussions on Health and Safety, control and quality issues, and site progress and problems. The Minutes from these meetings, prepared by AEM, are presented in Appendix J-3.

One QA Representative followed the construction activity from July 27th, 2018 to February 19th, 2019. Night shift QA Representative ended on December 5th since grouting activities occurred only during day shifts. The QA and QC worked on a 2-week rotation basis.

The number of QC personnel in day and night shifts varied based on construction activities as follows:

- One day-shift and one night-shift QC during earthwork activities (a second day-shift QC arrived later during this construction activity to work in the laboratory);
- Two day-shifts and two night-shifts QC during the first month of the cut-off wall construction. A third day-shift QC was present after the first month of construction;
- One day-shift QC during the grouting related activities and in few occasions that grouting continued in the night shifts

Assistance from QA Representative was necessary in a number of occasions to compensate the absence of QC representatives during the QC turnover or when not enough QC representatives were on-site to cover multiple tasks.

Some site pictures were taken by QA representatives from various components of the work. A limited number of photos was attached in the QA shift and weekly reports. A selection of photographs throughout the construction period is presented in Appendix A.

The QC/QA responsibilities, including test frequencies on grouting (Appendix 2 of Specifications) was modified during the initial stage of the site operations to be adapted to the on-site capability. The modified program and test frequencies approved by Designer are shown in Appendix B.

The main QC/QA activities on various project components, listed in Section 1.4, are detailed below.

4.2 Foundation Approval

As part of the QC/QA program, the foundations were approved before placing any material over the natural ground. The criteria for foundation approval are itemized in the approval forms and generally include:

- The cleaning and stripping were adequate;
- The excavation was enough to reach the required grades and limits;
- A competent bedrock was observed;
- For excavations below water, elevation of bottom of excavation is at or below the expected bedrock obtained from site investigation data except close the east abutment where the Contractor, the QA and QC Inspectors were able to confirm visually the presence of the bedrock.

In the underwater key trench, the bathymetric survey was affected by the turbidity issues caused from considerable sediments in the water caused by the excavation in the key trench. Starting from Station 0+328, AEM and the Contractor agreed to use a GPS-equipped excavator instead of standard bathymetric survey tools. The excavator equipped with GPS recorded survey points in an approximate 1-m grid in order to provide the actual bathymetry of the key trench. The surveyor accompanied by the QC representative then used a boat to obtain manual readings at 5-m intervals along the centerline at the bottom of the key trench. The data obtained from the GPS-equipped excavator and manual readings were further compared with the expected bedrock profile from construction drawings and boreholes from field investigation where available. Typically, 20-m long segments were approved by QC/QA upon the review of these three sources and indications from the field observation. Otherwise, QC/QA representatives asked for additional spot checks where major discrepancies had been observed between manual readings and GPS survey from KCG. If lakebed sediments were still found inside the key trench, the cleaning of the section started over as well as its survey. The final bedrock surface in the key trench segments were recorded in the Approval forms. These forms are supported by survey plans and sections of each approved segment.

Table 4-1 presents a summary of the foundation approvals during the construction. The corresponding approval forms are presented in Appendix H-1. It should be noticed that the shovel survey and the boat equipped GPS didn't allow for slopes bathymetry, therefore some differences are displayed in the survey plan layout and sections.

Provisions were made by doing rough excavation between 0+500 and 0+700 at least 5 days ahead to clean the foundation and allow for permafrost thawing if any. Everyday, excavation of the foundation was done at selected locations between these stations. The elevation at the bottom of the excavation was then measured and compared against date from previous days to see if there was a change of elevation, which would indicate that the foundation might be frozen and that regular scraping was necessary to attain a stable elevation over a 2-day period.

Table 4-1: WTD Foundation Approval List

Station	Completion	Approval date	Doc. #
0+060	X	2018-08-14	20180814-NS-01
0+130	X		
0+130	X	2018-08-13	20180813-DS-02
0+150	X		
0+150	X	2018-08-12	20180812-DS-01
0+205	X		
0+205	X	2018-08-05	20180805-DS-01
0+255	X		
0+255	X	2018-08-06	20180806-DS-01
0+295	X		
0+295	X	2018-08-07	20180807-DS-01
0+328	X		
0+328	X	2018-08-10	20180810-DS-01
0+355	X		
0+355	X	2018-08-13	20180813-NS-01
0+375	X		
0+375	X	2018-08-15	20180815-DS-01
0+395	X		
0+395	X	2018-08-15	20180815-DS-02
0+415	X		
0+415	X	2018-08-16	20180816-DS-02
0+435	X		
0+435	X	2018-08-17	20180817-DS-01
0+475	X		
0+475	X	2018-08-18	20180818-DS-01
0+495	X		
0+495	X	2018-08-18	20180818-DS-02
0+515	X		
0+515	X	2018-08-19	20180819-DS-02
0+555	X		

Station	Completion	Approval date	Doc. #
0+555	X	2018-08-20	20180820-DS-01
0+575	X		
0+575	X	2018-08-20	20180820-DS-04
0+595	X		
0+595	X	2018-08-22	20180822-DS-01
0+615	X		
0+615	X	2018-08-22	20180822-DS-08
0+635	X		
0+635	X	2018-08-23	20180823-DS-01
0+655	X		
0+655	X	2018-08-23	20180823-DS-02
0+675	X		
0+675	X	2018-08-24	20180824-DS-01
0+705	X		
0+705	X	2018-08-25	20180825-DS-04
0+727	X		
0+727	X	2018-08-22	20180822-NS-01
0+739	X		
0+739	X	2018-08-16	20180816-DS-03
0+772	X		
0+772	X	2018-08-19	20180819-NS-01
0+790	X		
0+790	X	2018-08-20	20180820-NS-01
0+808	X		

Additional attention was given when encountered bedrock was higher than expected bedrock elevation.

4.3 Fill Materials and Placement

During fill placement, it was ensured that the materials gradation was controlled to be within the allowable ranges, the materials quality was visually acceptable, the placement technique limited any segregation and the maximum allowable lift thickness was not exceeded.

4.3.1 Rockfill

The technical specifications required a well graded crushed rockfill material. For Quality Control purposes, the gradation was assessed visually by QC and QA representatives inspired from procedures in ASTM D5519. Since the rockfill included particles with nominal maximum size of 1000 mm, no conventional sieve analysis test could be carried out. Some inconsistencies in the rockfill materials were noted during the construction of the platforms, i.e. considerable amount of fine fractions and presence of boulders exceeding 1.5 m in diameter. Overall, the rockfill material was within the specified ranges and larger boulders were placed in the exterior of the platforms.

4.3.2 Filter Gradation

Samples of fine filter and coarse filter materials were taken by QC Representatives during site operation at the required frequencies specified in the Technical Specifications or as agreed by QA. The sampling took place from the stockpiles to adjust crushing screens in a way to respect gradations limits and also *in situ* at the time of placement of materials. At the beginning of the project, it was noticed that the fine filter near the production plant was being stockpiled in a manner creating segregation. QC Representatives notified the contractor several times. The issue was fixed after a few weeks. The sampling process and segregation issues were overseen and follow ups made by QA.

The fine and coarse filter samples were tested for particle size distribution to ensure that the gradation limits of the Technical Specifications were met. Modifications on the crusher configuration were done as required by the Contractor when required. The samples were also tested for water content (results are included with the gradation curves). QA monitored the laboratory procedures and gradation test data processing and provided feedback for modifications.

The results of the QC laboratory tests are presented in Appendix E for fine filter and coarse filter. Figure 4-1 and Figure 4-2 compile the gradation curves. The gradation ranges recommended in the Technical Specifications are also shown.

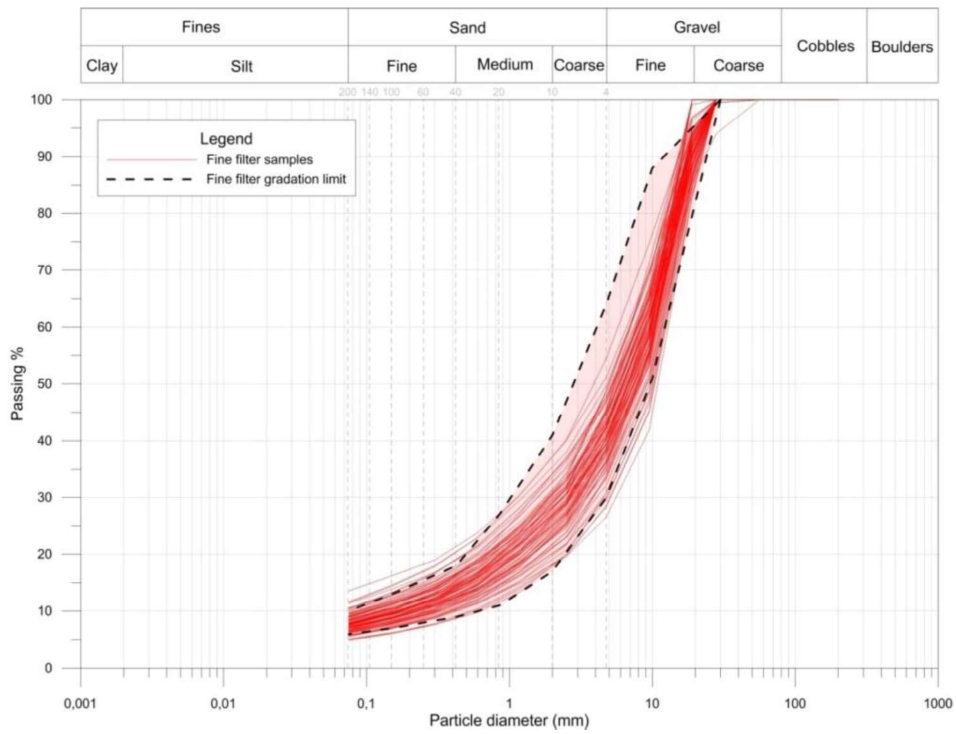

Figure 4-1: Fine Filter Gradation Curves and Recommended Ranges in Technical Specifications

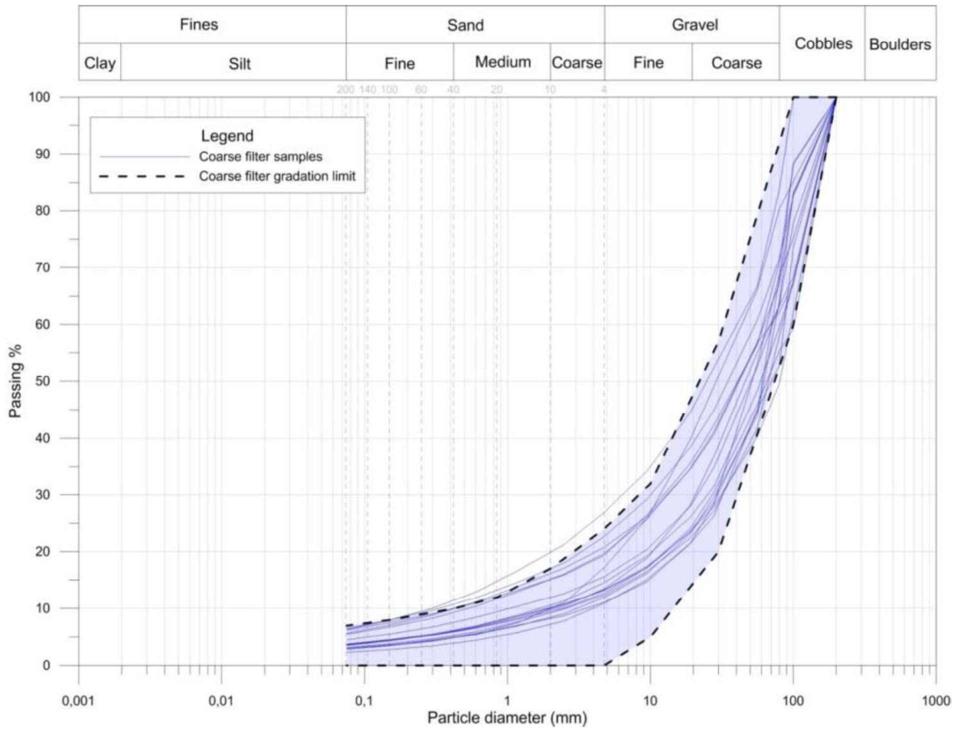

Figure 4-2: Coarse Filter Gradation Curves and Recommended Ranges in Technical Specifications

The results of gradation tests on fine filters (Figure 4-1) indicate that the materials were predominantly gravel-sand mixtures to gravel-sand-silt mixtures (GP-GM) and met the Technical Specifications limits. The gradation curves for some samples were slightly outside the permissible ranges for particle sizes ranging between 2.0 and 10.0 mm. Nonetheless, the material was still accepted by QC/QA Representatives based on visual evaluations as the difference was not believed to affect the performance of the filter negatively.

The results of gradation testing on coarse filters (Figure 4-2) indicate that the materials were predominantly well to poorly graded gravel with little to no fines (GW, GP), both in the stockpile and *in situ,* and met the Technical Specifications requirements. The placed materials were also visually acceptable.

4.3.3 Approvals

As part of the QC/QA program, approvals were issued for fill materials and placement in compliance with Specifications. The items for approval included:

- Lines and graded were respected as per drawings;
- Fill was free of ice, snow or deleterious materials;
- Gradation was within the allowable ranges;
- Lift maximum thickness was respected;

- Placement method minimized segregation.

After the surveying of the foundation and providing the limit of the area to be approved, the form was prepared and signed by representatives from the Contractor, QC, AEM and QA.

The approval forms are presented in Appendices H-2, H-3 and H-4 for rockfill, fine filter and coarse filter materials, respectively. Table 4-2 to Table 4-4 summarize the approval processes for the placement of rockfill, fine filter and coarse filter materials respectively.

In Table 4-2, rockfill was placed in three (3) stages, i.e. upstream initial platform to elevation 153.5 m, downstream initial platform to elevation 153.5 m and in both upstream and downstream from elevation 153.5 m to 157.0 m. The approval forms are placed in Appendix H-2.

In Table 4-3 and Table 4-4, fine and coarse filter materials were placed in two (2) stages in the key trench, i.e. to elevation 153.5 m and from elevation 153.5 m to elevation 157.0 m. The approved area in these tables corresponds to the approval forms in Appendix H-3 for fine filter material and Appendix H-4 for coarse filter material.

In some sectors, no approval forms were completed. However, these activities (rockfill, fine filter and coarse filter placement) were monitored by QA and QC during the Construction. The lack of approval doesn’t imply a lack of controls or quality of the work. The sectors where no approval form was completed are also shown in Table 4-3 and Table 4-4.

Table 4-2: Approval of the rockfill placement

Station	Initial platform - U / S (South) @ 153.5 m			Initial platform - D / S (North) @ 153.5 m			El. 157 m		
	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #
0+140	West abutment			West abutment			X	2018-08-19 2018-08-21 2018-08-29	20180819-NS-03 20180821-DS-11 20180821-DS-12 20180829-NS-10
0+195							X		
0+195	X	2018-08-03	20180803-NS-01	WTL shoreline at 0+200 D/S			X		
0+200	X			X	2018-08-03	20180803-NS-01	X		
0+228	X			X			X		
0+228	X	2018-08-02	20180802-DS-01	X	2018-08-02	20180802-DS-01	X		
0+235	X			X			X		
0+235	X			X			X	2018-08-19 2018-08-21	20180819-NS-03 20180821-DS-10 20180821-DS-11 20180821-DS-12
0+240	X			X			X		
0+240	X			X			X	2018-08-21	20180821-DS-09 20180821-DS-10 20180821-DS-11 20180821-DS-12
0+257	X			X			X		
0+257	X	2018-08-02	20180802-NS-01	*Approval's missing D/S from 0+269 to 0+257*			X		
0+269	X			X	2018-08-02	20180802-NS-01	X		
0+270	X			X			X		
0+270	X			X			X	2018-08-21	20180821-NS-04 20180821-DS-09 20180821-DS-10 20180821-DS-11
0+285	X			X			X		
0+285	X			X			*U/S @157 m D/S @155 m*	2018-08-21	20180821-NS-04 20180821-DS-09 20180821-DS-11
0+295	X			X			*U/S @157 m D/S @155 m*		
0+295	X	2018-08-02	20180802-NS-01	X	2018-08-02	20180802-NS-01	*U/S @157 m D/S @155 m*		
0+300	X			X			*U/S @157 m D/S @155 m*		

Station	Initial platform - U / S (South) @ 153.5 m			Initial platform - D / S (North) @ 153.5 m			El. 157 m		
	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #
0+300	X			X			*El. 155 m*	2018-08-21 2018-08-29	20180821-DS-11 20180829-NS-04
0+320	X			X			*El. 155 m*		
0+320	X	2018-08-01	20180801-NS-02	X	2018-08-01	20180801-NS-02	X	2018-08-27 2018-08-29 2018-09-02	20180827-DS-06 20180829-NS-04 20180902-NS-01
0+340	X			X			X		
0+340	X	2018-08-01	20180801-NS-01	X	2018-08-01	20180801-NS-01	X	2018-08-27 2018-09-02	20180827-DS-05 20180827-DS-06 20180902-NS-01
0+355	X			X			X		
0+355	X			X			X	2018-08-27 2018-08-28 2018-09-02	20180827-DS-05 20180828-NS-01 20180902-NS-01
0+360	X			X			X		
0+360	X			X			X	2018-08-28 2018-08-29 2018-09-02	20180828-NS-01 20180829-NS-05 20180902-NS-01
0+370	X			X			X		
0+370	X	2018-08-01	20180801-NS-02	X	2018-08-01	20180801-NS-02	X		
0+384	X			X			X		
0+384	X	2018-07-31	20180731-DS-01	X	2018-07-31	20180731-DS-01	X	2018-08-22 2018-08-29	20180822-NS-05 20180829-NS-05
0+395	X			X			X		
0+395	X			X			X	2018-08-22 2018-08-23 2018-08-29	20180822-NS-05 20180823-NS-04 20180829-NS-05 20180829-NS-11
0+405	X			X			X		
0+405	X	2018-07-31	20180731-NS-01	X	2018-07-31	20180731-NS-01	X		
0+415	X			X			X		
0+415	X			X			X	2018-08-22 2018-08-29	20180822-NS-05 20180822-DS-06 20180829-NS-05 20180829-NS-11
0+420	X			X			X		
0+420	X			X			X		

Station	Initial platform - U / S (South) @ 153.5 m			Initial platform - D / S (North) @ 153.5 m			El. 157 m		
	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #
0+445	X			X			X	2018-08-22 2018-08-23	20180822-NS-05 20180822-DS-06 20180829-NS-05 20180829-NS-11
0+445	X	2018-07-30	20180730-DS-01	X	2018-07-30	20180730-DS-01	X		
0+450	X			X			X		
0+450	X			X			X	2018-08-22 2018-08-29	20180822-NS-05 20180822-DS-04 20180829-NS-08 20180829-NS-11
0+483	X			X			X		
0+483	X			X			X	2018-08-22 2018-08-29	20180822-NS-05 20180822-DS-04 20180829-NS-01 20180829-NS-08 20180829-NS-11
0+495	X			X			X	2018-08-22 2018-08-29	20180822-DS-04 20180829-NS-01 20180829-NS-08 20180829-NS-11
0+495	X	2018-07-30	20180730-NS-01	X	2018-07-30	20180730-NS-01	X		
0+510	X			X			X		
0+510	X			X			X	2018-08-26 2018-08-29	20180826-NS-05 20180829-NS-01 20180829-NS-06 20180829-NS-08
0+555	X			X			X		
0+555	X			X			X	2018-08-26 2018-08-29	20180826-NS-04 20180826-NS-05 20180829-NS-01 20180829-NS-06
0+560	X			X			X		
0+560	X	2018-07-29	20180729-DS-01	X	2018-07-29	20180729-DS-01	X		
0+583	X			X			X		
0+583	X			X			X	2018-08-26 2018-08-29	20180826-NS-04 20180826-DS-06 20180829-NS-01 20180829-NS-06
0+600	X			X			X		
0+600	X	2018-07-29	20180729-NS-01	X	2018-07-29	20180729-NS-01	X		

<table>
<tr><th rowspan="2">Station</th><th colspan="3">Initial platform - U / S (South) @ 153.5 m</th><th colspan="3">Initial platform - D / S (North) @ 153.5 m</th><th colspan="3">El. 157 m</th></tr>
<tr><th>Completion</th><th>Approval date</th><th>Doc. #</th><th>Completion</th><th>Approval date</th><th>Doc. #</th><th>Completion</th><th>Approval date</th><th>Doc. #</th></tr>
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2018-08-29</td><td rowspan="4">20180827-NS-03
20180827-DS-04
20180829-NS-06
20180829-NS-12</td></tr>
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<tr><td>0+650</td><td>X</td><td rowspan="4">2018-07-28</td><td rowspan="4">20180728-DS-01</td><td>X</td><td rowspan="4">2018-07-28</td><td rowspan="4">20180728-DS-01</td><td>X</td></tr>
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2018-08-29</td><td rowspan="2">20180827-NS-03
20180829-NS-02
20180829-NS-06
20180829-NS-12</td></tr>
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2018-08-29</td><td rowspan="3">20180827-DS-03
20180829-NS-02
20180829-NS-09
20180829-NS-12</td></tr>
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<tr><td>0+727</td><td>X</td><td rowspan="4">2018-08-22</td><td rowspan="4">20180822-NS-02</td><td>X</td><td rowspan="4">2018-08-22</td><td rowspan="4">20180822-NS-02</td><td>X</td></tr>
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20180829-NS-02
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20180829-NS-12</td></tr>
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2018-08-27
2018-08-29</td><td rowspan="2">20180817-NS-03
20180827-DS-03
20180829-NS-09
20180829-NS-12</td></tr>
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Station	Initial platform - U / S (South) @ 153.5 m			Initial platform - D / S (North) @ 153.5 m			El. 157 m		
	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #	Completion	Approval date	Doc. #
0+739	East abutment			East abutment			X	2018-08-17 2018-08-25 2018-08-27 2018-08-29	20180817-NS-03 20180825-NS-02 20180827-DS-03 20180829-NS-09 20180829-NS-12
0+756							X	2018-08-17 2018-08-25 2018-08-27 2018-08-29	20180817-NS-03 20180825-NS-02 20180827-NS-05 20180829-NS-07
0+756							X		
0+768							X		
0+768							X	2018-08-19 2018-08-25 2018-08-27 2018-08-29	20180819-NS-04 20180825-NS-02 20180827-NS-05 20180829-NS-07
0+785							X		
0+785							X	2018-08-25 2018-08-27 2018-08-29	20180825-NS-02 20180827-NS-05 20180829-NS-03 20180829-NS-07
0+797							X		
0+797							X	2018-08-25 2018-08-29	20180825-DS-01 20180828-NS-03 20180829-NS-07 20180829-NS-13
0+815							X		
0+815							X	2018-08-25 208-08-29	20180825-DS-01 20180829-NS-13
0+835							X		

Table 4-3: Approval of the fine filter placement, including its dynamic compaction

Station	KEY TRENCH			TO EL. 157 m		
	Fine Filter Completion	Approval date	Doc. #	Fine Filter Completion	Approval date	Doc. #
0+065	-			X	2018-08-29	20180829-NS-18
0+092	-			X		
0+092	-			X		
0+120	-			X		
0+120	X	2018-08-21	20180821-DS-02	X	2018-08-21	20180821-DS-02
0+135	X			X		
0+135	X	2018-08-18	20180818-NS-03	X		
0+209	X			X		
0+209	X	2018-08-21	20180821-DS-02	X		
0+230	X			X		
0+230	X	2018-08-17	20180817-NS-04	X	2018-08-17	20180817-NS-04
0+260	X			X		
0+260	*No approval between 0+260 to 0+317*			X	2018-08-21	20180821-DS-03
0+300				X		
0+300				X		
0+317				X		
0+317	X	2018-08-27	20180827-NS-01	X		
0+330	X			X		
0+330	X			X	2018-08-31	20180831-NS-01
0+335	X			X		
0+370	X			X		
0+370	X	2018-08-17 2018-08-27	20180817-NS-02 20180827-NS-01	X		
0+380	X			X		
0+380	X			X	2018-08-21	20180821-DS-01
0+383	X			X		
0+383	X	2018-08-17	20180817-NS-02	X		
0+395	X			X		
0+395	X	2018-08-18	20180818-NS-02	X		
0+435	X			X		
0+435	*No approval between 0+435 to 0+471*			X		
0+447				X		

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<tr><td>0+480</td><td>X</td><td>X</td></tr>
<tr><td>0+480</td><td colspan="3" rowspan="4">No approval between 0+480 to 0+495</td><td>X</td></tr>
<tr><td>0+490</td><td>X</td></tr>
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<tr><td>0+695</td><td>X</td><td rowspan="2">2018-08-26</td><td rowspan="2" colspan="2">20180826-DS-05</td><td>X</td></tr>
<tr><td>0+727</td><td>X</td><td>X</td></tr>
<tr><td>0+727</td><td>X</td><td rowspan="4">2018-08-23</td><td rowspan="4" colspan="2">20180823-NS-02</td><td>X</td></tr>
<tr><td>0+738</td><td>X</td><td>X</td></tr>
<tr><td>0+738</td><td>X</td><td>X</td><td rowspan="8">2018-08-26</td><td rowspan="8">20180826-DS-03</td></tr>
<tr><td>0+739</td><td>X</td><td>X</td></tr>
<tr><td>0+739</td><td>X</td><td rowspan="2">2018-08-26</td><td rowspan="2" colspan="2">20180826-DS-03</td><td>X</td></tr>
<tr><td>0+748</td><td>X</td><td>X</td></tr>
<tr><td>0+748</td><td>X</td><td rowspan="2">2018-08-26</td><td rowspan="2" colspan="2">20180826-DS-03
20180826-NS-01</td><td>X</td></tr>
<tr><td>0+772</td><td>X</td><td>X</td></tr>
<tr><td>0+772</td><td>X</td><td rowspan="2">2018-08-20
2018-08-26</td><td rowspan="2" colspan="2">20180820-NS-03
20180826-DS-03
20180826-NS-01</td><td>X</td></tr>
<tr><td>0+805</td><td>X</td><td>X</td></tr>
<tr><td>0+805</td><td>X</td><td rowspan="2">2018-08-20</td><td rowspan="2" colspan="2">20180820-NS-03</td><td>X</td><td rowspan="4">2018-08-29</td><td rowspan="4">20180829-NS-19</td></tr>
<tr><td>0+808</td><td>X</td><td>X</td></tr>
<tr><td>0+808</td><td>X</td><td rowspan="2">2018-08-26</td><td rowspan="2" colspan="2">20180826-NS-01</td><td>X</td></tr>
<tr><td>0+815</td><td>X</td><td>X</td></tr>
<tr><td>0+815</td><td colspan="4">-</td><td>X</td><td rowspan="4">2018-08-25</td><td rowspan="4">20180825-DS-03</td></tr>
<tr><td>0+830</td><td colspan="4">-</td><td>X</td></tr>
<tr><td>0+830</td><td colspan="4">-</td><td>X</td></tr>
<tr><td>0+835</td><td colspan="4">-</td><td>X</td></tr>
</table>

Table 4-4: Approval of the coarse filter placement

Station	KEY TRENCH			TO EL. 157 m		
	Coarse Filter Completion	Approval date	Doc. #	Coarse Filter Completion	Approval date	Doc. #
0+065	-			X	2018-08-29	20180829-NS-16
0+085	X	2018-08-14	20180814-DS-01	X		
0+095	X			X	2018-08-29	20180829-NS-14
0+130	X			X		
0+130	X			X	2018-08-21	20180821-DS-04 20180821-DS-05
0+150	X			X		
0+150	X	2018-08-13	2018013-DS-01	X		
0+205	X			X		
0+205	*No approval between 0+205 to 0+215*			X		
0+215				X		
0+215	X	2018-08-06	20180806-NS-01	X		
0+220	X			X		
0+220	X			X	2018-08-21	20180821-NS-02 20180821-DS-04
0+240	X			X		
0+240	X			X	2018-08-21	20180821-DS-04 20180821-DS-06
0+245	X			X		
0+245	X	2018-08-07	20180807-NS-01	X		
0+280	X			X		
0+280	X	2018-08-08	20180808-NS-01	X		
0+300	X			X		
0+300	X			*155 m*	2018-08-21	20180821-DS-04
0+320	X			*155 m*		
0+320	X	2018-08-10 2018-08-11	20180810-DS-02 20180811-NS-01	X	2018-08-27 2018-08-31	20180827-NS-02 20180831-NS-02 20180831-DS-01
0+355	X			X		
0+355	X	2018-08-14	20180814-NS-02	X		
0+375	X			X		
0+375	X	2018-08-16	20180816-DS-01	X		
0+385	X			X		
0+385	X			*155 m*	2018-08-22	20180822-NS-04
0+390	X			*155 m*		
0+390	X			X	2018-08-22	20180822-NS-04 20180822-DS-03
0+395	X			X		
0+395	X	2018-08-17	20180817-NS-01	X		
0+410	X			X		

Station	KEY TRENCH			TO EL. 157 m		
	Coarse Filter Completion	Approval date	Doc. #	Coarse Filter Completion	Approval date	Doc. #
0+410	X			X	2018-08-22 2018-08-24	20180822-NS-04 20180824-DS-06
0+415	X			X		
0+415	X	2018-08-18	20180818-NS-01	X		
0+455	X			X		
0+455	X	2018-08-18	20180818-NS-01 20180818-DS-03	X		
0+475	X			X		
0+475	X	2018-08-18	20180818-DS-03	X		
0+483	X			X		
0+483	X			X	2018-08-22 2018-08-24	20180822-DS-02 20180824-DS-06
0+495	X			X		
0+495	X	2018-08-19	20180819-NS-02	X		
0+515	X			X		
0+515	X	2018-08-20	20180820-NS-04 20180820-DS-02	X		
0+520	X			X		
0+520	X			X	2018-08-22 2018-08-26	20180822-DS-02 20180826-DS-07
0+540	X			X		
0+540	X			X	2018-08-24 2018-08-26	20180824-NS-04 20180826-DS-07
0+555	X			X		
0+555	X	2018-08-21	20180821-NS-01	X		
0+570	X			X		
0+570	X			X	2018-08-24 2018-08-26	20180824-DS-05 20180826-DS-07
0+575	X			X		
0+575	X	2018-08-21	20180821-DS-08	X		
0+595	X			X		
0+595	X	2018-08-22 2018-08-23	20180822-DS-07 20180822-NS-01	X		
0+600	X			X		
0+600	X			X	2018-08-26	20180826-NS-02 20180826-DS-07
0+615	X			X		
0+615	X	2018-08-23	20180823-NS-01	X		
0+633	X			X		
0+633	X	2018-08-23	20180823-DS-03	X		
0+645	X			X		
0+645	X			X	2018-08-26 2018-08-28	20180826-NS-02 20180828-NS-02
0+652	X			X		
0+652	X			X	2018-08-27 2018-08-28	20180827-NS-06 20180828-NS-02
0+655	X			X		

Station	KEY TRENCH			TO EL. 157 m		
	Coarse Filter Completion	Approval date	Doc. #	Coarse Filter Completion	Approval date	Doc. #
0+655	X	2018-08-24	20180824-NS-01	X		
0+674	X			X		
0+674	X	2018-08-24	20180824-DS-02	X		
0+689	X			X		
0+689	X			X	2018-08-27	20180827-NS-06 20180827-DS-02
0+695	X			X		
0+695	X			X	2018-08-27	20180827-DS-01 20180827-DS-02
0+702	X			X		
0+702	X	2018-08-26	20180826-NS-03 20180826-DS-01	X		
0+727	X			X		
0+727	X	2018-08-22	20180822-NS-03	X		
0+740	X			X		
0+740	X	2018-08-19 2018-08-25	20180819-NS-06 20180825-DS-02	X		
0+745	X			X		
0+745	X			X	2018-08-26 2018-08-29	20180826-DS-04 20180829-NS-15
0+755	X			X		
0+755	X			X	2018-08-26 2018-08-29	20180826-DS-04 20180829-NS-15
0+770	X			X		
0+770	X	2018-08-25	20180825-DS-02	X		
0+790	X			X		
0+790	X	2018-08-20 2018-08-25	20180820-NS-02 20180825-DS-02	X		
0+805	X			X		
0+805	X			X	2018-08-29	20180829-NS-15 20180829-NS-17
0+808	X			X		
0+808	X	2018-08-25	20180825-DS-02	X		
0+815	X			X		
0+835	X			X	2018-08-29	20180829-NS-17

4.4 Dynamic Compaction

Dynamic compaction was performed according to Menard's work program presented in Appendix K-1. Controls were done by the Contractor (Menard) and reported to AEM representatives. There was no additional control performed by QC/QA for this activity.

Daily reports were produced by the specialized contractor and are presented in Appendix I-4. Furthermore, the dynamic compaction was summarized in a final technical report produced by Menard and presented in Appendix K-1. Each plan section of dynamic compaction was surveyed and approved by AEM, QC/QA Representatives to verify that the proposed work layout was followed.

Table 4-5 summarizes the dynamic compaction approval process referring to the approval forms presented in Appendix H-5.

Table 4-5: Dynamic Compaction Approval Forms

Station	Dynamic compaction		
	Completion	Approval date	Doc. #
0+065	No dynamic compaction below 0+092		
0+092			
0+092	X	2018-09-03	20180903-DS-01
0+120	X		
0+120	X		
0+135	X		
0+135	X		
0+209	X		
0+209	X		
0+230	X		
0+230	X		
0+260	X		
0+260	X		
0+300	X		
0+300	X	2018-09-06	20180906-DS-01
0+317	X		
0+317	X		
0+330	X		
0+330	X		
0+335	X	2018-08-31 2018-09-06	20180831-DS-02 20180906-DS-01
0+370	X		
0+370	X		
0+380	X		
0+380	X	2018-09-06	20180906-DS-01
0+383	X		
0+383	X		
0+395	X		
0+395	X		

Station	Dynamic compaction		
	Completion	Approval date	Doc. #
0+435	X		
0+435	X		
0+447	X		
0+447	X		
0+471	X		
0+471	X		
0+480	X		
0+480	X		
0+490	X		
0+490	X		
0+495	X		
0+495	X		
0+505	X		
0+505	X		
0+530	X		
0+530	X		
0+540	X		
0+540	X		
0+545	X		
0+545	X		
0+575	X		
0+575	X		
0+590	X		
0+590	X		
0+595	X		
0+595	X	2018-09-07	20180907-DS-01
0+600	X		
0+600	X		
0+625	X		
0+625	X		
0+647	X		
0+647	X		
0+655	X		
0+655	X		
0+673	X		
0+673	X		
0+695	X		
0+695	X		
0+727	X		
0+727	X		
0+738	X		
0+738	X		
0+739	X		

Station	Dynamic compaction		
	Completion	Approval date	Doc. #
0+739	X		
0+748	X		
0+748	X		
0+772	X		
0+772	X		
0+805	X		
0+805	X		
0+808	X		
0+808	X		
0+815	X		
0+815	X		
0+830	X		
0+830	No dynamic compaction beyond 0+830		
0+835			

4.5 Cut-off Wall Construction

A total of 975 secant piles were drilled and filled with CB slurry for the construction of the cut-off wall from September 16th to December 04th, 2018. Piles 374b and 678b had to be added for the reasons explained in Section 3.0. A third additional Pile 668b was drilled as a quaternary to compensate the excessive deviations in Piles 668 and 669.

After drilling and subsequent cleaning operations, the depth of the bottom of rock socket was measured by QC and reported to QA. The Contractor filled an excel register (named “secant pile register”, placed in Appendix F) to collect their measurements which were often compared with QC results. The final elevations, casing verticality and offset deviation were also measured by the surveyor. QA relied on measurements carried out by QC representatives and by the surveyor for approval of rock sockets, prior to pouring the CB slurry in the piles. This protocol was used for primary and secondary piles. The target depths of bottom of rock socket of tertiary piles were based on adjacent primary and secondary piles, i.e. the deepest of the adjacent primary or secondary piles was used. This new protocol was established by AEM and approved on October 14 because HFDI drilling operators could hardly feel the bedrock surface on tertiary piles drilled mostly through the CB material unlike primary and secondary piles.

The QC/QA activities on secant piles mainly included controls on the following items:

- Casing Drilling Tolerances;
- Depth of Rock Socket;
- Cement-Bentonite Slurry Preparation;
- On-Site Tests on CB Slurry Mix;
- Cement-Bentonite Slurry Placement;

- Drilling of Tertiary Piles;
- Field Test on Cement-Bentonite Backfill;
- Laboratory Tests on Cement-Bentonite Backfill.

4.5.1 Casing Drilling Tolerances

When the bedrock surface was reached, Henry Drilling noted the total length of the casings installed. The bedrock surface was frequently encountered below the previously surveyed elevations, especially in the zone with poor rock conditions between Stations 0+340 and 0+375. In these circumstances, the actual depth of the bedrock encountered, provided by Henry Drilling, was compared with the surveyed rock elevations obtained from initial excavation of the key trench. Vertical deviations and location offsets were systematically measured by the surveyor prior to pouring cement-bentonite slurry into the secant casings. The data were compiled by the surveyor and the as-built pile tolerances included in a spreadsheet submitted by AEM on February 6th, 2019 as presented in Appendix K-4.

In the latest data, the casing location offsets are included in the as-built pile spacing, which varies between 0.59 and 0.90 m. Although the verticality in 80 single piles exceeded the allowable maximum value of 0.75 %, the compilation of the surveyor data does not indicate any gaps between the secant piles. This can be explained due to the favourable conditions developed by the actual spacing and a smaller deviation in the adjacent pile, since it is the combination of the as-built spacing and the verticality of each of the adjacent piles that govern the magnitude of the overlap and hence wall thickness at the interface.

A minimum wall thickness of 0.247 m above bedrock surface (i.e. at the bedrock interface) can be deducted from the as-built drawing. At the rock sockets (i.e. below the bedrock surface where the diameter of each pile is 0.9 m instead of 1.0 m) the as-built data revealed the presence of 13 gaps. Overall, the Surveyor records show that the actual overlap and wall thickness at the bedrock interface meet or exceed the initial target values.

There was only one exception that the Surveyor reported a gap at the interface of Piles 668 and 669, which was remedied by installation of quaternary Pile 668b on December 1st, 2018.

Further information can be obtained from the Technical Memorandum on pile spacing in Appendix B.

4.5.2 Depth of Rock Socket

Depths of the bottom of the rock socket were approved by QC and QA Representatives prior to pouring CB slurry in each secant pile to ensure that the minimal rock socket depth requirement was respected from the bedrock surface per the Technical Specifications. Additional control was put in place to determine required rock socket depth by defining the lowest elevation that a given pile would hit the bedrock surface. This was particularly important when steep bedrock slopes were encountered in order to obtain the minimal rock socket depth requirement. The approval included the verification that the bottom of rock socket was clean of ice, snow, rock debris, filter or any external materials. The presence of water did not allow the visual inspection of the bottom of the rock socket. The socket cleanliness was assessed by feeling the bottom of the rock socket with measuring tools (tape with plumb bob) to identify if unwanted materials accumulated at the bottom of the secant piles. The approval forms for rock sockets are presented in Appendix H-6.

The method adopted to measure the rock socket depth was modified, based on the bedrock surface elevations encountered by Henry Drilling during casing installation (as discussed in section 4.5.1). All the changes were discussed with AEM. For the tertiary piles, the minimum depth of the rock socket was established as the lowest elevation of the adjacent pile as described in section 3.3.1.

The sections of the cut-off wall subject to variations from original design were:

- Between Sections 0+743 and 0+773, secant piles required a minimum rock socket of 2.0 m following MDRB recommendations for the east abutment. The measurement for the depth of rock socket was made based on HD's encountered bedrock surface because the bedrock in this sector had not previously been exposed;
- Between Sections 0+775 and 0+825, the minimal rock socket depth was approved based on HD encountered bedrock surface, since it was often observed that the encountered bedrock surface was lower than expected;
- Between Sections 0+826 and 0+832 (at east abutment), no rock socket was required, due to the presence of frozen ice-poor till, observed during stripping of the foundation.

Rock socket depth were measured and compiled by QC Representatives in the latest spreadsheet available in Appendix F. The data received show that the average rock socket depth was 1.37 m with depths ranging from 0.47 m to 3.6 m. Based on specific attention by QA representatives on rock socket depths, QA believes that all rock sockets are within the tolerance stated in Technical Specifications (1.0 $\pm$ 0.1 m), except for 6 piles, i.e. Piles 65, 317, 321, 329, 417 and 914, having socket lengths of 0.89 m, 0.47 m, 0.52 m, 0.89 m, 0.84 m and 0.80 m, respectively.

AEM sent the final secant piles table from the Contractor on February 06th 2019, compiling rock socket depth in the cut-off wall. This secant piles table is presented in Appendix K-4.

4.5.3 Cement-Bentonite Slurry Preparation and Testing

4.5.3.1 Mix Preparation in Batch Plant

The Quality of CB slurry was controlled by checking the properties of materials (water, cement, bentonite, admixture) and the mixing procedures were employed per the Technical Specifications requirements.

The controls in the mixing procedures were carried out during initial trial mixes on-site. No laboratory test results were available from the Contractor before the start of the Work. The CB slurry materials produced in early stage were poured in the slurry trench, where there was a lower expectation for slurry quality.

Between November 07th and November 21st 2018, there were discrepancies in the density of slurry mix prepared at the batch plant in comparison with what was expected for the target mix. It was observed during this period that an overall lower cement percentage was used resulting in lower density values. The discrepancies were noted in daily reports and discussed in daily construction meetings.

The Contractor was asked to calibrate batch plant weighing system and provide the weighing system accuracy to QC/QA after discrepancies in the density of slurry mix was observed (as low as 1.20 when the density of the approved mix was expected to be 1.24 g/cm^3). QC Representative tested the cement weighing system with the contractor by comparing theoretical Specific Gravity of cement with mud balance readings. The results showed the weighing system to be calibrated properly. The batch plant was on occasion operated manually. When operated manually, it was difficult for the operator to control the exact mix proportions of CB slurry, hence explaining mud balance reading variations.

Prior to mixing, pH tests were conducted on water with support from the Environment Team at Amaruq. The results were generally between 7.2 and 8.5 which were not expected to adversely affect the mix. When test results met the specifications, the mix could be transported to the pile locations.

4.5.3.2 On-Site Tests on CB Slurry Mix

The properties of CB slurry were controlled by QC Representatives at the batch plant for each full portable agitator tank (PAT) prior to transportation to the pile locations by a truck. The performed tests were:

- *Viscosity measurement using Marsh Funnel*
- *Density measurement using Mud Balance.*
  Table 4-6 was developed by QA in spreadsheet format to predict and verify the required mud-balance readings performed by QC for various cement, bentonite, water and admixture compositions (white boxes) used on-site. One of mixes is shown in Table 4-6.

Figure 4-3 shows the evolution of density and viscosity readings of the cement-bentonite slurry over the course of the cut-off wall construction.

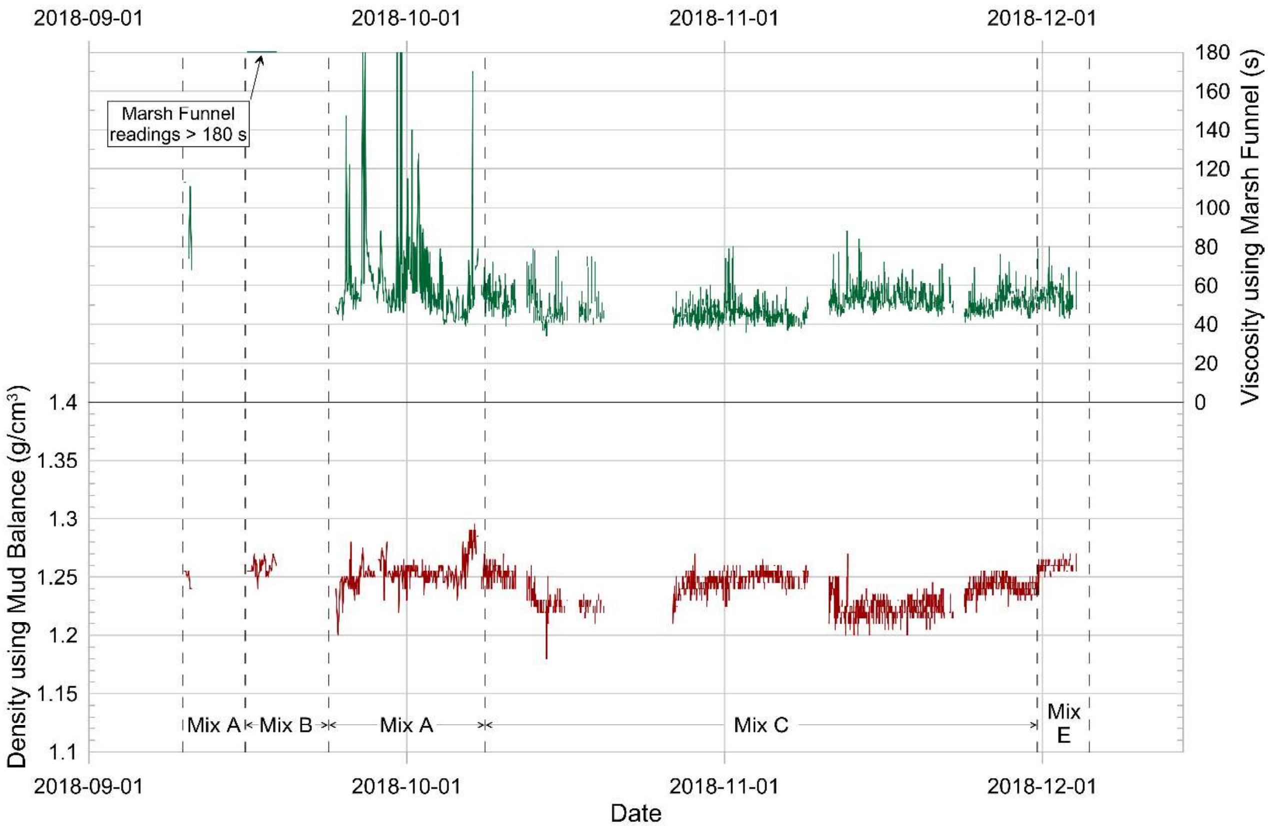


Figure 4-3: Density and viscosity readings of the cement-bentonite slurry during the construction of the cut-off wall

- *Bleeding Test*
  The test was carried out in on the CB slurry mixes produced in the batching plant. Bleeding test results are compiled in Appendix G-1. The QC test records show that bleeding values were less than 2.2% after two hours in the approximate 60 bleeding tests carried out during the construction of the cut-off wall. Although, the specification required measuring the bleeding at 2 hours, the laboratory test continued up to 24 hours. This would not change the conclusions made, because the bleeding values even after longer testing periods are well below the specification requirements of 5%. Several bleeding test results were discarded at the initial stages as the test cylinder was not capped in accordance with standard test procedure. One bleeding test was required per day. Deviations regarding test frequency as per the ''Modified Test Frequency Program" were noted in the shift reports or communicated with AEM in the Daily Construction Meetings.
- *Temperature of CB slurry*
  This provided an indication of the cement hydration and closeness to the gelation time of the slurry.

Table 4-6: Mix Proportion Table for Mix Density Prediction

Mix Used for Production (Dec 2, 2018)			
Mix E (HE Cement)			
Materials	**Mass, kg**		**Density, kg/m^3**
Weight of Cement	1943		3150
Weight of Bentonite	217		2780
Moisture in Bentonite, %	11		-
Mix Water	4873		1000
Admixtures (ARBO S01P)	4.8	0.25%	435
Density, g/cm3	1.26		
Cement Content, kg/m3	347		
Water:Cement ratio (W/C)	2.5		
Cement:Water ratio (C/W)	0.40		
Cement:Bentonite ratio (C/B)	10.1		
Bentonite:Water, %	4.0		
Solid %	30.4		

Test results are compiled in Appendix G-1. When the CB slurry mix had major issues or was not pumpable, (e.g. Batch #150), the slurry batch was rejected by QC Representative prior to be transported to pile locations.

4.5.4 Cement-Bentonite Slurry Placement and Top-Ups

The slurry was placed through a tremie pipe at the bottom of rock socket. When there was water within the casings, QC and QA representatives paid attention to ensure that the slurry was placed smoothly to allow overflow of clean water at the top of casing. Following this visual observation, QC Representatives would give the instruction to stop CB slurry placement. In addition, QC/QA monitored that no cold joint was allowed during tremie placement and the pile backfilled in one go. There were few occurrences that this advice was not followed (e.g. Piles 578, 393, 214 and 11).

The placement of CB slurry into the secant piles and the subsequent extraction of casing were required to be within the gelation time of slurry. As previously mentioned, the gelation time was calculated from the moment of contact between water and cement. After issues with late extractions of casings in some of the secant piles and per QA notifications, contractor arranged to extract the casings shortly after CB slurry placement to avoid disturbance of the slurry that was in the process of jellification. Despite the above, some casings were still extracted after the anticipated gel time and in few occasions (e.g. Pile 118) the pile had to be re-drilled. In the majority of piles, the placement was within the gel time or marginally met the criteria. QA representatives notified the QC/Contractor when this practice was not followed (e.g. in Piles 77, 377, 359, 362, 578, 662, 711, 717, 12, 16).

CB slurry drops were noticed in most of the piles after slurry placement and continued at a decreasing rate after casing removal until the backfill materials set. The CB slurry elevation required to be maintained to the required height (i.e. at elevation 157.0 m) with top-ups. Top ups were occasionally carried out with delays after the CB slurry in the piles had started to jellify or set. QA discussed the potential for development of cold joints between the old slurry and the top-up fresh slurry in the piles with AEM and site crew. AEM communicated a procedure on November 5th, 2018 to facilitate placement of top-ups in a timely manner – within the slurry gelation time. The procedure stated that every second truck was being designated to top-ups before resuming CB mix placement in any new casings. Following this communication, top-ups were performed rapidly (often right after casing removal) and the casings extracted with less delays. In few occasions, the top-ups with more than 1.0 m drop were done without inserting the slurry delivery hose at the bottom of the previously poured slurry. QA advised to use a flexible hose of sufficient length in order to reach the CB slurry surface below any bled or accumulated water, avoiding the slurry placement in a free fall manner. The work method was modified accordingly.

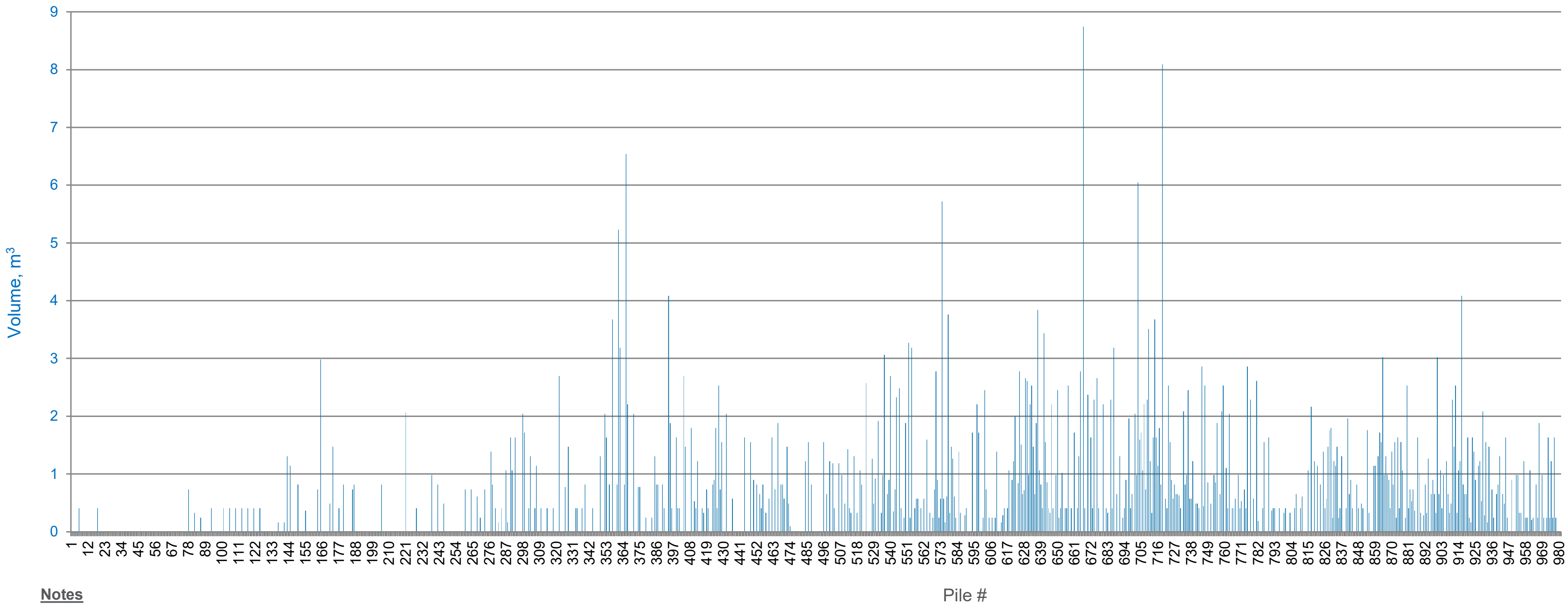
There were also few occasions that top ups were placed on top of the ice that was formed on top of the old slurry (e.g. Pile 630 and 839). QA emphasised that any ice or external materials on the surface of old slurry be removed before pouring fresh CB slurry in the piles during the top ups. After the slurry drops in piles, to avoid cold joints at top of the set slurry, some piles were drilled using an auger on a depth of approx. 0.5 m prior to placement of fresh slurry. In addition, an UCS was done on a mold prepared to look at the effect of cold joint after 24 hours (Refer to the Weekly Report 20181031-WR in Appendix I-2). The UCS result was 230.6 kPa after 8 days of curing which satisfied the criterion. However, the impact on the permeability has not been tested.

The issues on top-ups and generation of cold joints were recorded in the QA daily and weekly reports. Top up measurements were compiled by QC Representatives in the Secant Piles Register presented in Appendix F. A similar follow up was made by QA and recorded in the shift reports compiled in Appendix I-1. Figure 4-4 summarizes the acquired information from QA regarding volume of slurry drop in piles.

When present, QA Representative recorded the slurry drops in the piles after CB slurry placement and casings removal.

In addition, QA advised covering the pile tops while the slurry was still unset to prevent workers fall into the freshly backfilled pipes.

Figure 4-4: Estimation of Volume of Slurry Drop in Piles



Notes

- Data Source: QA Shift Reports (estimations), measured after slurry placement from working platform
- No Column Shown when No Slurry Drop Reported
- 0.5 m Drop (0.4 m^3) was assumed, when Slurry Drop reported but Drop Height Not Provided (Drop Volume = πr^2 x Drop Height (r = 51 cm, allowed 1 cm enlargement near top)
- Drops due to slurry loss can be contributed to followings:
  - Slurry penetration in the bottom and periphery of rock socket
  - Slurry loss at casing shoe location near bedrock-fine filter interface
  - Slurry penetration into Fine Filter (Coarse Fill) materials after casing extraction
- Diagram may show trends only. Some piles were topped-up in multiple times and could not be reported due to difficulties during operation.
- Actual Volume of Slurry Loss could be up to several order higher as loss of slurry starts from the moment that slurry is placed and the rate usually decreases with time

4.5.5 Installation of Tertiary Piles

The procedure for tertiary piles was to establish the target depth based of the deepest depth of the two adjacent piles (corresponding primary and secondary piles) instead of determining the depth of bottom of rock socket with encountered bedrock surface. The reason of this change was because the Contractor's drilling operators could hardly feel bedrock in these tertiary piles during the drilling process.

As per Technical Specifications, a minimum UCS of 50 kPa was required in order to drill tertiary piles in primary and secondary piles. The approval to drill tertiary piles was given verbally by QA for a certain number of piles when the strength criterion was met and approval forms emitted for larger sections of the cut-off wall. All the forms are presented in Appendix H-7. Based on UCS and vane tests, the minimum strength of the piles was obtained after 2 to 3 days.

The placement and on-site testing of the CB slurry mix used for tertiary piles followed the same process as in the primary and secondary piles discussed above.

4.5.6 Field Test on Cement-Bentonite Backfill

Field Vane Shear Tests were performed in the slurry trench backfill in west abutment and in the secant piles after a curing time of approximately 13.5 to 58 hours. Test results are presented in Appendix G-5. Measured shear strength was greater than 160 kPa (limit of test apparatus) for a curing time of ~ 20 hours.

It was not possible to manually push the rod of the apparatus into the backfill materials during Testing and the help of an excavator bucket was required by QC Representative to carry out most of the Field Vane Shear Tests. This could, however, cause inaccuracies in the test results. There was, however, little added value for the shear test results, when in a later stage, adequate UCS test results became available to control early strength gain of CB backfill. Nevertheless, effective November 10th, 2018, it was decided to end the vane tests due to their destructive nature when the deviation of the instrument was such that it reached the edge of the secant pile.

4.5.7 Laboratory Tests on Cement-Bentonite Backfill

QC was responsible for all aspects of testing in the on-site and external laboratories. The QA tasks mainly included the observation of sampling, sampling frequency and number of specimens, testing schedule, overseeing the execution of tests in the on-site laboratory, an overview of the test results received from QC and sporadic control of the test results recorded in the QC "secant pile register".

4.5.7.1 Pinhole tests

The pinhole test results are compiled in Appendix G-2. They were carried out as per ''Method A'' of ASTM D4647/4647M-13 to determine the erosion potential of the set CB slurry in the adjacent primary and secondary piles prior to drilling the casings for tertiary piles.

QA Representatives observed that pinhole test set-up in the laboratory did not allowed application of constant head pressure as per standard procedures. The laboratory set-up was never modified but QC Representatives paid attention to manage a larger or equal head pressure than required in the standard.

A total of 12 tests were conducted on samples cured for 1 to 5 days. All test results were indicative that the CB backfill used in production work was non-dispersive, classified as ND1, thus minimising the chance for erosion of the freshly placed backfill during casing installation for tertiaries.

Pinhole tests were not systematically performed by QC according to the “Modified Test Frequency Program” (presented in Appendix B) and this was notified by QA in the shift reports and daily construction meetings. Nevertheless, the results being constantly favorable, did not suggest a high priority for frequent pinhole tests.

4.5.7.2 Unconfined Compressive Strength (UCS)

The UCS tests were performed for short term and long-term conditions:

- The short term (1 to 3 days) or early strength tests were carried out to determine if the minimum strength requirement of 50 kPa was achieved in order to approve the drilling of the tertiary piles into the adjacent secant piles. All the samples tested reached an early strength of 50 kPa after 2 days of curing, except for samples CB-WTD-052 and CB-WTD-081, which reached the early strength requirement after 4 and 3 days, respectively.
- The long-term (28 and 56 day) tests were carried out to confirm the compliance with specification of a minimum 200 kPa for backfill strength. Most of the samples tested reached this requirement after 5 days of curing.

The results are presented in Appendix G-3 and compiled in Figure 4-5. The tests were performed according to the “Modified Frequent Program” once issued on October 10, 2018. The modified frequency program is presented in Appendix B.

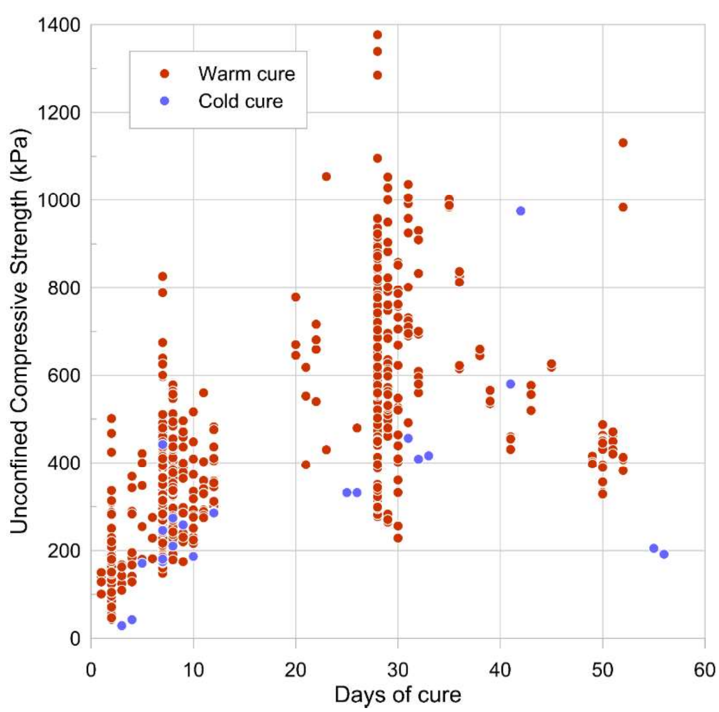


Figure 4-5: Unconfined Compressive Strength (UCS) Chart

The tests were initially performed on samples cured for 1, 2, 3, 28 and 56 days in cold and warm conditions. After November 3rd, 2018 all the UCS tests were performed in warm curing conditions due to the warm conditions monitored within the piles on site using thermocouple readings within the secant piles.

Most of the UCS tests were carried out with a platen slightly smaller than the samples, making deviations from the ASTM requirements. At occasions, the smaller platen resulted in punching of the loading plate into the CB samples. Right size platen was received between October 24th and October 31st, 2018. Results varied up to 10 kPa if the Standard platen was not used. This did not have a significant impact on the strength requirements because the strength values were well over the specification requirements.

4.5.7.3 Permeability Tests

Permeability tests were conducted in an external laboratory, (i.e. SNC-Lavalin (Qualitas), sub-contracted by GHD). A total of five (5) shipments were prepared and sent by QC.

Permeability tests were performed on 26 samples after 18 to 46 days of curing. Samples were selected by QC and QA Representatives in order to have an overall representativeness of the wall with minimal information gaps. Permeability test results are summarized in Table 4-7 and detailed provided in Appendix G-4.

The test values were in compliance with specification requirements for samples of CB backfills after 28 days of curing, except for tests SK-03, SK-08, SK-18 and SK-22. The permeability of these 4 samples were slightly greater than 10^{-6} cm/s but still in the same order of magnitude.

For CB slurry using HE cement, permeability results were approximately one order of magnitude lower than the results for CB slurry using GU cement, exceeding specification requirements.

No further permeability testing of samples were required by Designer/QA, because the existing relatively consistent permeability test results were believed to represent the backfill quality in the entire secant pile cut-off wall. All the changes regarding CB mix proportions during production (i.e. reduction in GU cement content and usage of HE cement) were also well represented and characterized in terms of permeability.

Table 4-7: Permeability Tests Results on Cement-Bentonite Backfill

Test #	Date of test	Sampling date	Days of cure	Permeability (cm/s)	Pile #	Mix	Type of cement	C/W ratio
SK-01	22-10-2018	2018-09-10	42	8.75E-07	Slurry wall	Mix A	GU	0.40
SK-02	26-10-2018	2018-09-10	46	5.27E-07	Slurry wall	Mix A	GU	0.40
SK-03	24-10-2018	2018-09-16	38	1.87E-06	41	Mix B	GU	0.40
SK-04	24-10-2018	2018-09-16	38	1.38E-06	41	Mix B	GU	0.40
SK-05	25-10-2018	2018-09-18	37	9.07E-07	93	Mix B	GU	0.40
SK-06	25-10-2018	2018-09-18	37	1.05E-06	93	Mix B	GU	0.40
SK-07	31-10-2018	2018-09-24	37	5.37E-07	101	Mix A	GU	0.40
SK-08	31-10-2018	2018-09-27	34	3.15E-06	99-103	Mix A	GU	0.40
SK-09	01-11-2018	2018-09-30	32	5.80E-07	229	Mix A	GU	0.40
SK-10	01-11-2018	2018-10-03	29	5.90E-07	38-50-54	Mix A	GU	0.40
SK-11	02-11-2018	2018-10-05	28	7.08E-07	273-277	Mix A	GU	0.40
SK-12	05-11-2018	2018-10-07	29	1.13E-06	118-114	Mix A	GU	0.40
SK-13	09-11-2018	2018-10-10	30	4.67E-07	198-329-333	Mix C	GU	0.36
SK-14	22-11-2018	2018-10-30	23	9.10E-07	Tests HE mix	Mix E	HE	0.40
SK-15	22-11-2018	2018-10-30	23	7.88E-07	Tests HE mix	Mix E	HE	0.40
SK-16	26-11-2018	2018-10-27	30	1.24E-06	367-379	Mix C	GU	0.36
SK-17	18-11-2018	2018-10-31	18	7.60E-07	449-427	Mix C	GU	0.36
SK-18	28-11-2018	2018-10-31	28	2.13E-06	Test GU mix	Mix D	GU	0.32
SK-19	03-12-2018	2018-11-04	29	1.00E-06	424	Mix C	GU	0.36
SK-20	03-12-2018	2018-11-05	28	7.40E-07	585	Mix C	GU	0.36
SK-21	06-12-2018	2018-11-08	28	1.72E-06	288	Mix C	GU	0.36
SK-22	13-12-2018	2018-11-15	28	3.27E-06	695-691-679	Mix C	GU	0.36
SK-23	19-12-2018	2018-11-21	28	1.88E-06	726-722	Mix C	GU	0.36
SK-24	07-01-2019	2018-11-25	43	1.28E-06	616-648	Mix C	GU	0.36
SK-25	08-01-2019	2018-11-29	40	8.52E-07	907-931	Mix C	GU	0.36
SK-26	09-01-2019	2018-12-01	39	9.21E-07	816-820	Mix E	HE	0.40

Notes

- **Mix A contained 0.5% admixture (ARBO) while Mix B contained none.**
- **Mix D was only tested but not used for production.**

4.6 Grout Curtain

The original design required a grout curtain at the centerline of the cut-off wall. As such the specifications required that grout holes be drilled from the centerline of the cut-off wall by rotary drilling and continued 10 m below the bottom of the rock socket section of the cut-off wall. Prior to the start of grouting and to preserve the integrity of the cut-off, the location of the grout curtain was modified so that the grout holes be drilled with an upstream offset of 0.7 m from the centerline of the cut-off wall and depth limited to 10 m below the bedrock surface. The offset was determined to avoid grout holes to intersect cut-off wall and considering piles actual deviation. The drawings and design specifications were revised accordingly to reflect the above changes or variations. The modified grout curtain plan, longitudinal section and details are shown in a set of drawings in Appendix B.

4.6.1 Grout Mixes

Prior to starting the grouting program, trial grout mixes were tested to ensure that they met the viscosity and stability requirements. Five (5) mixes; Mix A, Mix A*, Mix B, Mix C and Mix D with various water-cement ratios and admixtures were tested on December 12-13, 2018. The QC representative conducted tests on the grout mixes to assess the physical properties of each mix. The trial mix tests were conducted using a 40 kg cement bag per mix. Three (3) other mixes Mix D*, Mix E and Mix F were developed later on with higher viscosity values, considering the observation of very high grout takes in the fractured bedrock. Mix D*, Mix E, and Mix F were developed during production without trial mixes prior to their use. Mix F was used for the first time at Hole S-390 on December 30th, 2018. After the contractor ran out of Celbex 653 on January 9th, 2019, only Mixes A, B, C and D were used until the end of grouting operations. As notified in the Weekly Reports, the simultaneous use of Celbex and Rheomac 450 admixtures, from different suppliers, was not according to the specifications. It was noticed in one occasion that Mix F was too thick to penetrate, showing early refusals.

After 40 kg cement bags ran out on December 24th, 2018, it was decided to use 1000 kg cement bags. 20 kg buckets were filled and used for each batch as required. Some issues were noted with the cement from 1000 kg bags. Pieces of plastics, wood and cement lumps were observed in the cement. A screen was installed at the inlet of the agitating tank, but it was not enough to remove all impurities. QA had advised to install another screen at the inlet to prevent impurities get to the mixer in the first stage, but this was not practiced. The impurities caused the grout return valve to be clogged, causing an uncontrolled flow of grout towards the holes. This may be one of the reasons for excessive spikes in the pressure-time graph as seen in grout holes P-492, P-504, P-468, P-480, S-402 and S-198.

Starting on January 1st, 2019, higher than specified values for bleeding were reported for Mix A. From January 7th, 2019 Mix A was no longer used and was replaced by Mix B which also met the criterion of high mobility grout.

Figure 4-6 shows the evolution of density and viscosity readings of the grout mix over the course of the curtain grouting.

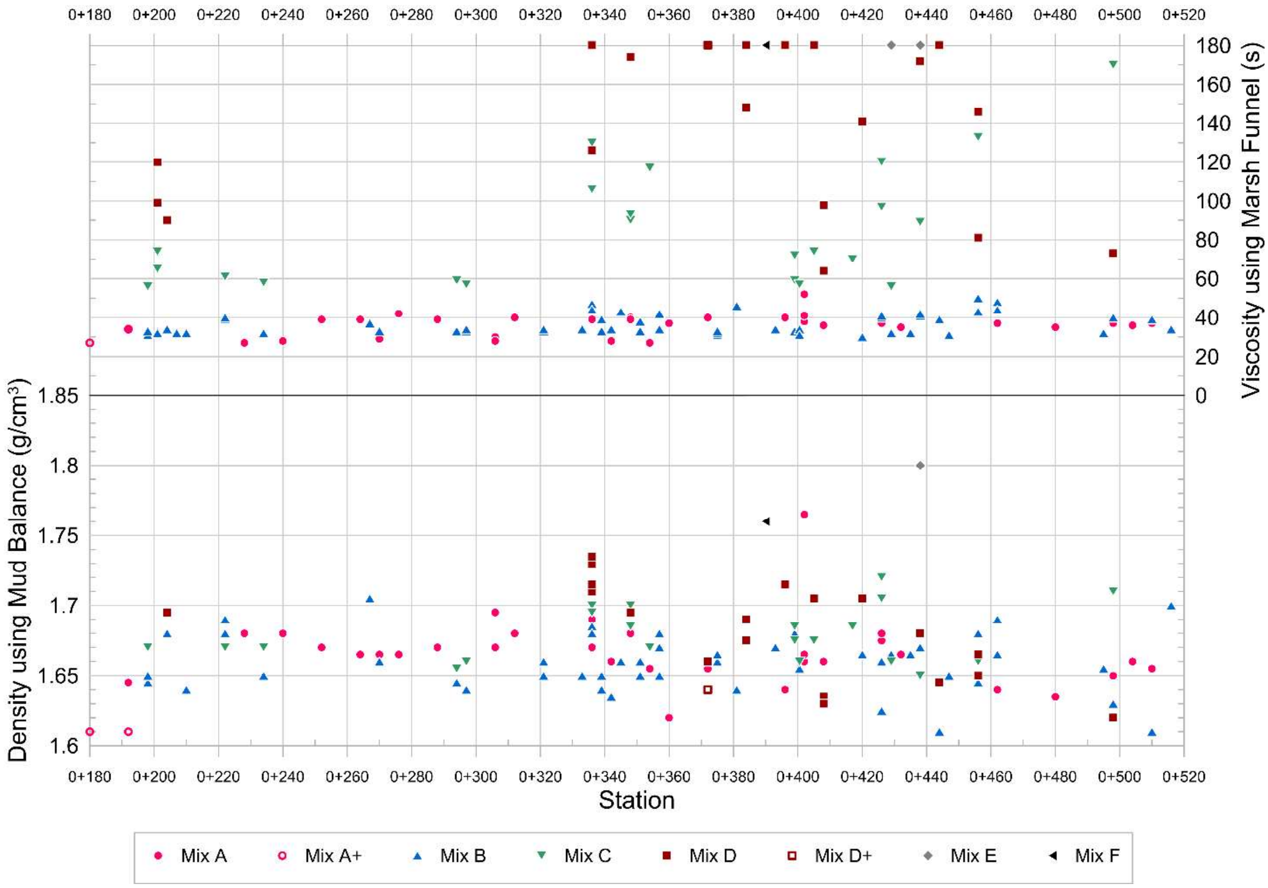


Figure 4-6: Density and viscosity readings of the grout mix during curtain grouting

4.6.2 Casing installation and Bedrock Drilling

The maximum vertical deviation of a drill hole casing was specified to be 2% of its drilled length. The reported data show that the majority of the casings met this requirement. Only 15% of the casings had vertical deviations between 2% and 5%. However, considering the azimuth values for these holes, the effect on the overall grout curtain performance is expected to be negligible.

Casings were to be anchored approximately ±0.3 m in the bedrock until it seats entirely on the rock. Cuts given to the driller were indicative only. So, the casing shoe elevation was defined by the driller when the bedrock is encountered at a given place. The drilling records[5], presented in Appendix K-5, show an average embedment of 0.3 m. In general, in 13% of grout holes the bedrock was encountered more than 0.5 m below the bedrock elevation in the adjacent secant pile. In some locations, longer embedment lengths (up to 1.3 m)

[5] WTD_INJECTION_CASING (REV 4), Received February 21, 2019.

were recorded. E.g. in T-201, P-216, S-222, P-228, S-234 and S-378, where the casing shoe elevations appear to be below the bottom of the secant pile bedrock socket.

The drilling record also shows that in 10% of the holes, the bedrock was encountered more than 0.5 m above the reported rock elevation in the adjacent secant pile. In an extreme case at Hole T-369, the bedrock was encountered 3.4 m higher and the driller was confident that the bedrock was encountered at higher elevation. No information was available from the QC representatives during the drilling operation. The reasons for the above inconsistencies were not clear. Nevertheless, QA advised the site crew on January 4th, 2019 to avoid excessive drilling of the casing into the bedrock.

As per specification, after completion of drilling, a hole had to be flushed with clean water for a minimum of five (5) minutes or until clean water starts coming out of the hole, but in practice, holes were only washed out for a period of about 2 minutes. According to the driller, during flushing, clean water starts coming out of the hole in less than 2 minutes.

Quaternary holes were predominantly grouted in the absence of QC/QA representatives as AEM had priority for QC/QA representatives to be present at other dikes.

Non-compliances observed during grouting were discussed in the Daily Meetings or recorded in the QA Shift Reports and Weekly Reports. These were not recorded into the RFI log. A brief is mentioned below:

- At T-417, bottom two stages were completed in the day shift and night shift crew mistakenly backfilled the hole without grouting the third stage. No additional hole was drilled to cover third stage grouting in this hole.
- At some occasions, when grouting for the hole was not completed on the same shift due to Q_{max} reached or some other reasons, grout backflow was observed in the part or on an entire succeeding stage and the crew was not able to properly grout that stage, e.g. at P-324, S-426, T-201 where the stage length was not measured prior to stage grouting.
- The first working pressure gauge on the collar was installed on January 5th, 2019. The pressure gauges installed previously at the collar did not function correctly.
- The pressure transducer for data acquisition system stopped working on January 4 th, 2019 and from January 4 th, 2019 to January 10 th, 2019 no computer data was available for pressure measurement. The operator noted pressures manually from the pressure gauge, which was not very accurate due to a minimum division on the gauge of 2 bar. Therefore, it was not possible to accurately measure the pressure in fraction of 2 bars. A new pressure transducer was installed on January 10 th, 2019. The pressure sensor was down again on January 30 th, 2019 but fixed on February 1st, 2019.
- As per specification, rock temperature in the hole should be monitored using a thermistor but no thermistor was installed, and rock temperature was not measured prior to grout injection in the holes.
- Holes were not washed with clean water immediately prior to the start of the grouting and the crew was depending on the hole being washed by the drilling crew, even though there were delays of a few days to more than a week between drilling and grouting the hole.
- Higher than the specified grout temperature of 10 °C to °20 C was observed at a number of occasions. This mostly happened to the first morning batch probably due to an overheating of cement and water during the night (refer to QC grout curtain register attached in Appendix M).
- The Vicat apparatus was not available on-site at the start of the grouting operation. The first Vicat test was performed on January 1st, 2019 after the apparatus became available on-site. A total of seven (7) Vicat tests were performed during the grouting program - two (2) tests on Mixes A, B and C

each and one (1) test on Mix D. The set time values obtained are included in Table 2-3. The results are nevertheless approximate and influenced by the grout mix temperature during the test.

- Marsh Funnel was calibrated using water and found an error of 10 seconds on January 1st, 2019. A new Marsh funnel was tested and used afterwards. Some of Marsh values reported prior to January 1st , 2019 may not be accurate.
- Flowmeter and pressure gauge calibrations were not done as frequently as required by specification. Calibration was performed only once on Jan 10 th, 2019 on the flowmeter and pressure gauge during the grouting operation, but the results were not submitted to QA.

4.6.3 Water Pressure Test (WPT)

The target pressure for different pressure steps for the WPT could not be achieved for many of the tests performed such as in T-357, T-297 and T-267 due to limited pumping capacity of the equipment. Water pressure tests at S-306 and S-474 were not performed as per standard procedure for Lugeon test and the results are not reliable. QA worked with the Contractor to calculate the correct effective applied pressures in the subsequent tests. It was noticed that water pressure test results for 2nd and 3rd stage of P-516 were also not reliable because pressure readings were noted manually from pressure gauge. Also due to excessive vibration of the gauge needle, the pressure measurements were not accurate.

The water pressure tests planned for Q-406.5 and Q-466.5 were cancelled. Results are presented in Appendix I-6. Due to the limitations mentioned in Section 3.5.2.1, the water pressure test results were not reliable and were used in decision making for grouting closure.

4.6.4 Forms and QC/QA Activities

The grouting forms were prepared and/or approved by QA, prior to the start of the field work. This included Casing Drilling Report, (Rock) Drilling Daily Report, WPT forms and Grout Application Record. The forms were filled in by the Contractor during site operation and then, together with injection data files, reviewed by QA promptly, when they were received from the Contractor on daily basis. The casing drilling, casing deviation measurement (rock) drilling, Water Pressure Test (WPT), grouting application record and injection data files are presented in Appendices I-6 and K-5.

QC was mainly involved in on-site and laboratory testing but in some occasions QC representatives were on other tasks and were not available for grouting work. Some of the QC tasks could not be done or covered by QA representative when they were not present. QC compiled the grouting data in an excel spreadsheet called “grout curtain register” presented in Appendix M.

QA initially reviewed the grouting submittals including participation in technical discussions with AEM, Contractor and QC during the course of grouting. The QA strategy on-site (mainly day-shifts) included field observation tasks, providing support to grouting specialist and preparing the as-built report. The QA representative was accompanied by SNC-Lavalin grout specialist from December 10th to December 19th, 2018 and from December 31st to January 31st, 2019.

QA was not present during bedrock drilling and washing as well as some of the grout injection operations, especially, towards the end of grouting work. QA tasks included review of records on casing drilling, casing deviations, bedrock drilling, hole cleaning, mix preparation, grouting equipment, system layout and applied injection pressures. This included an overview of the grouting materials used, on-site testing and real-time data on the computer screen for pressure, grout flow rate and Contractor's field notes.

When deficiencies were observed, QA informed AEM and the Contractor to take corrective actions verbally in the Morning Construction Meeting and/or through emails, Daily and Weekly reports.

5.0 DESIGN VARIATIONS

Design variations/changes and field adjustments occurred during the construction of Whale Tail Dike. This was to adapt the initial design to the conditions encountered during the field work and to accommodate the project's requirements in terms of cost and schedule. The decisional chart of the design changes is presented in Figure 5-1 and were documented in the minutes of meetings, emails and technical memoranda.

The list of the technical memoranda that describes field adaptation and modification from the initial Technical Specifications and Drawings is presented in Table 5-1. The technical memoranda are also presented in Appendix B. The complete list of variations is presented in a so-called RFI List (Request For Information) in Appendix L. The RFI serial numbers were changed in this last version from those submitted earlier in the weekly reports.

Other local field adjustments were made during construction when the encountered conditions differed from the expected conditions. These local field adjustments were discussed with the QA on-site and implemented by AEM's Representative. Field adjustments were documented in the QA Representatives weekly and daily reports (Appendices I-1 and I-2).

Table 5-1: List of Technical Memoranda Emitted During Construction

No.	Title	Revision
658309-0000-64ER-0001	Spacing of secant piles	00
658309-0000-64ER-0002	Modified test frequency program for CB slurry	00
658309-0000-64ER-0003	Early strength development in CB slurry	00
658309-0000-64ER-0004	Gel/Set time – Slurry placement – Casing extraction – Cold joint;	00
658309-0000-64ER-0006	Conditions for early dewatering (superseded by 658309-0000-64ER-0009)	00
658309-0000-64ER-0007	Figure on the modified instrumentation location	00
658309-0000-64ER-0008	Stress-Deformation analysis of deeper cement-bentonite secant pile walls	00
658309-0000-64ER-0009	Whale Tail Dike Commissioning criteria	01

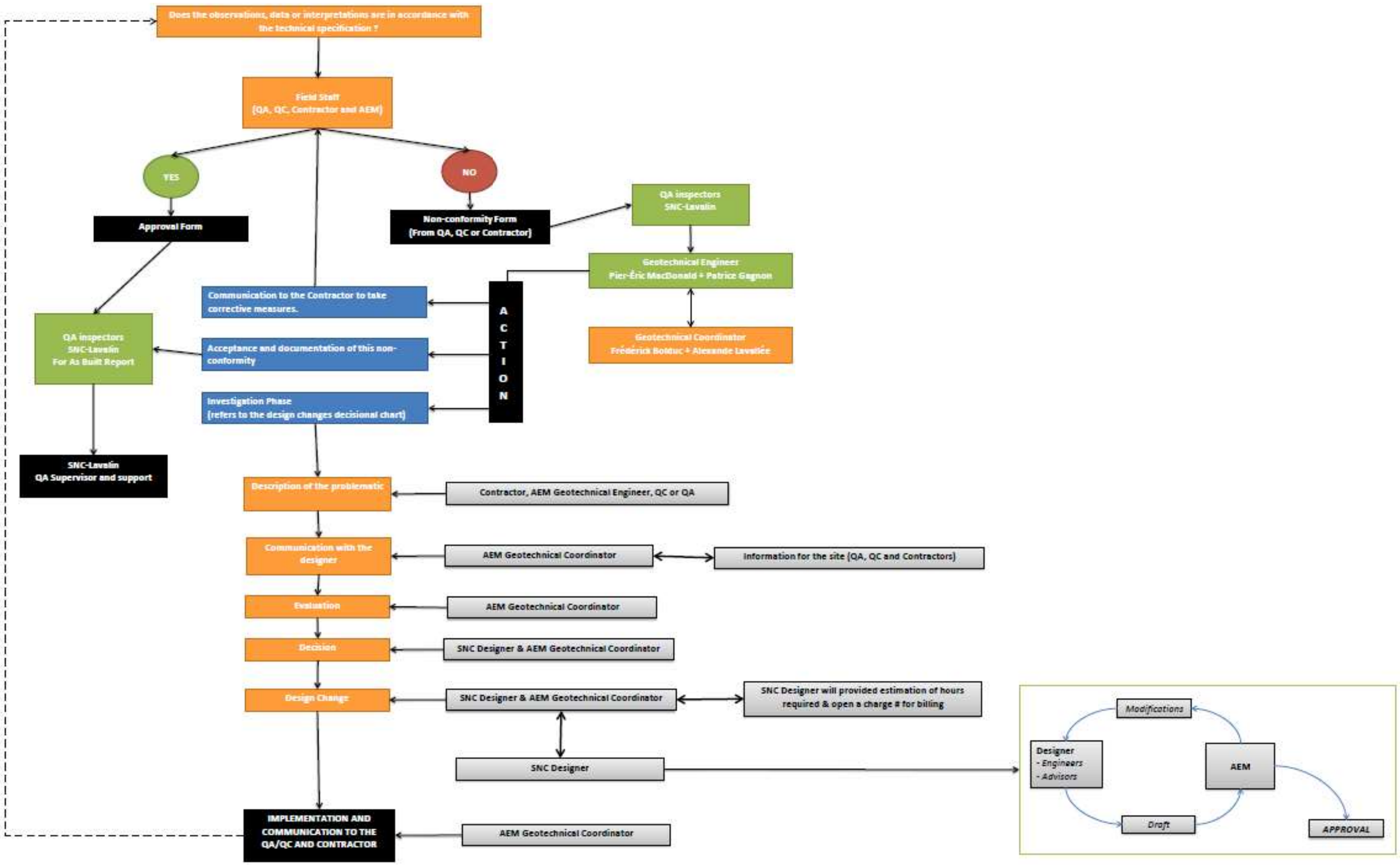


Figure 5-1: Decisional chart during design changes

6.0 PERSONNEL

This report has been prepared by Jonathan Leblanc, with support of Angie Arbaiza, Mathieu Durand-Jézéquel, Muhammad Saleem and Muhammad Umar and revised by Yohan Jalbert and Tom Xue. This report has been read and approved by Yohan Jalbert.

We trust that this report is to your satisfaction. Should you have any question, please do not hesitate on contacting us.

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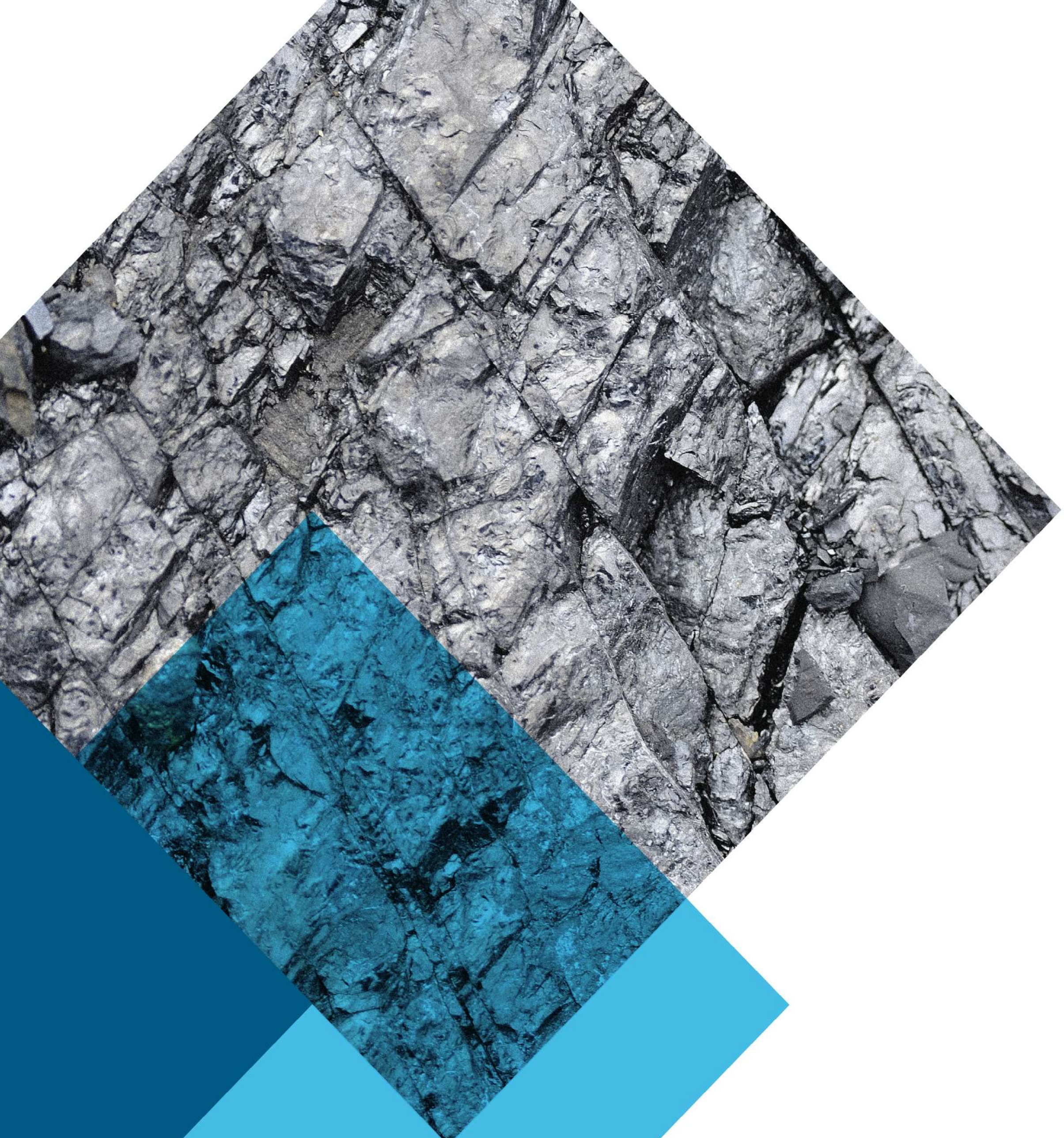

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