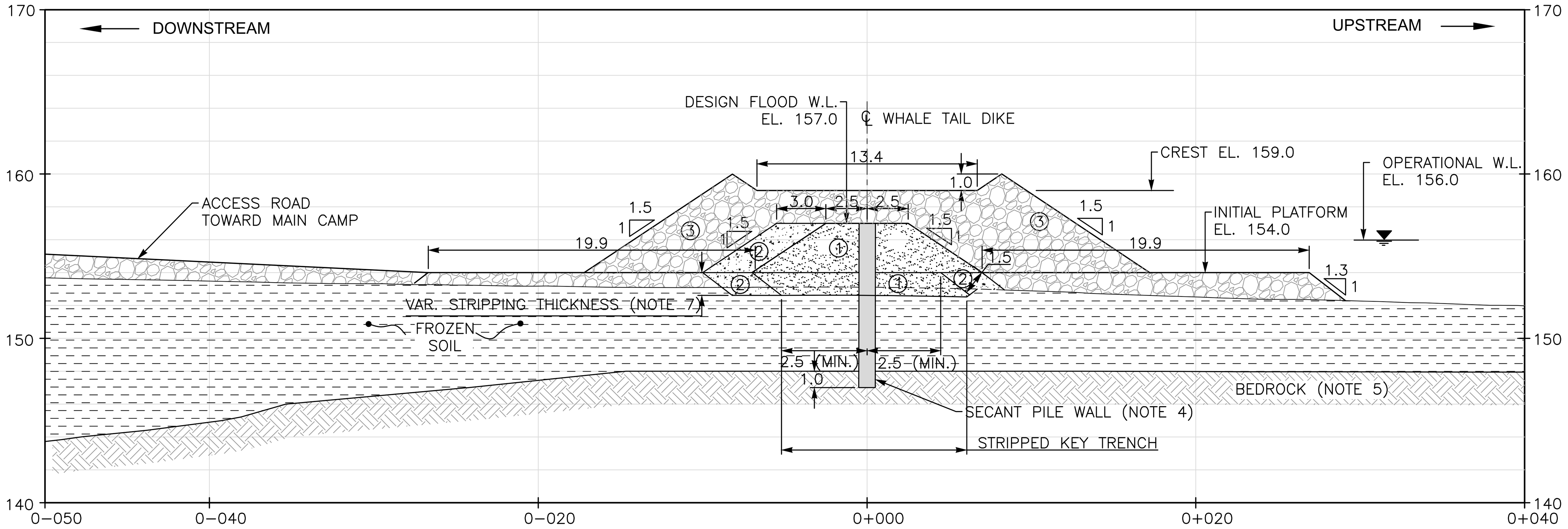
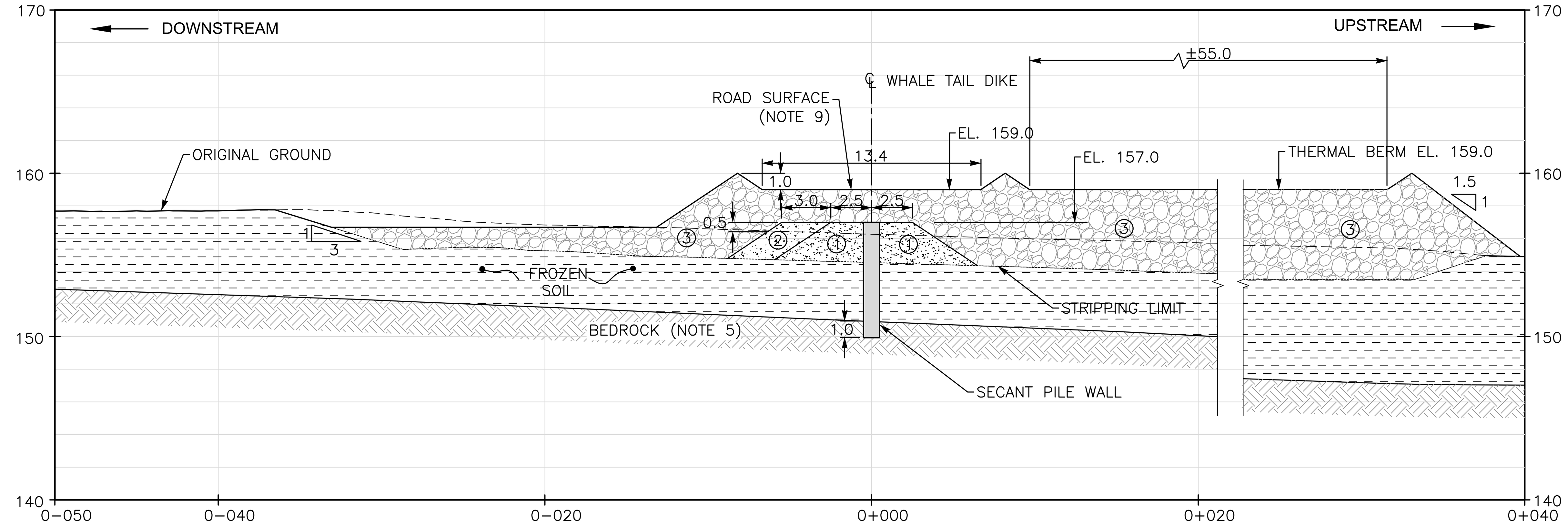


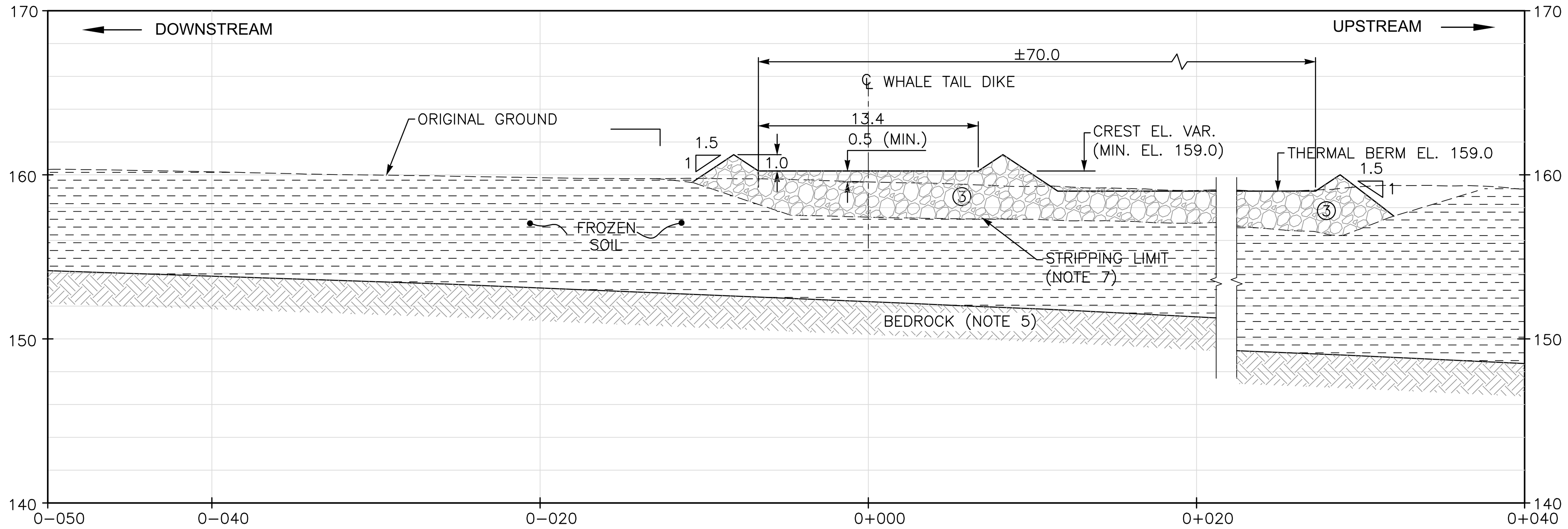
SECTION G-G ACCESS TO THE N/S INITIAL PLATFORMS AT STA. 0+705

SCALE 1: 250



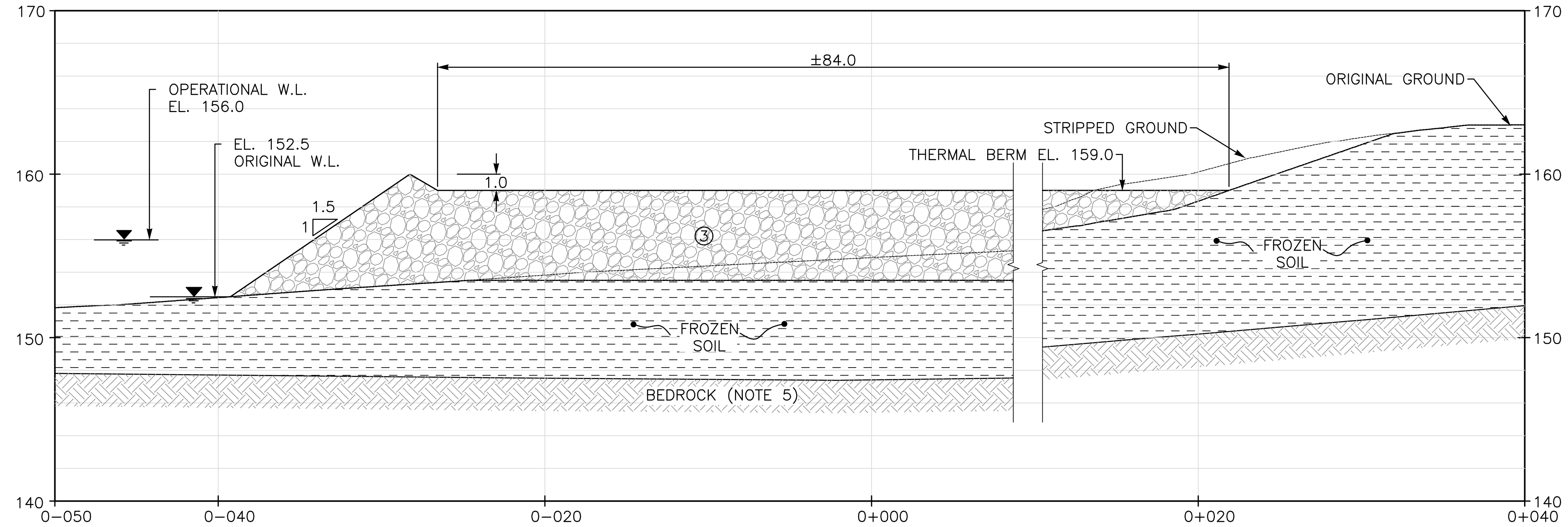
SECTION H-H EAST ABUTMENT AT STA. 0+775

SCALE 1: 250



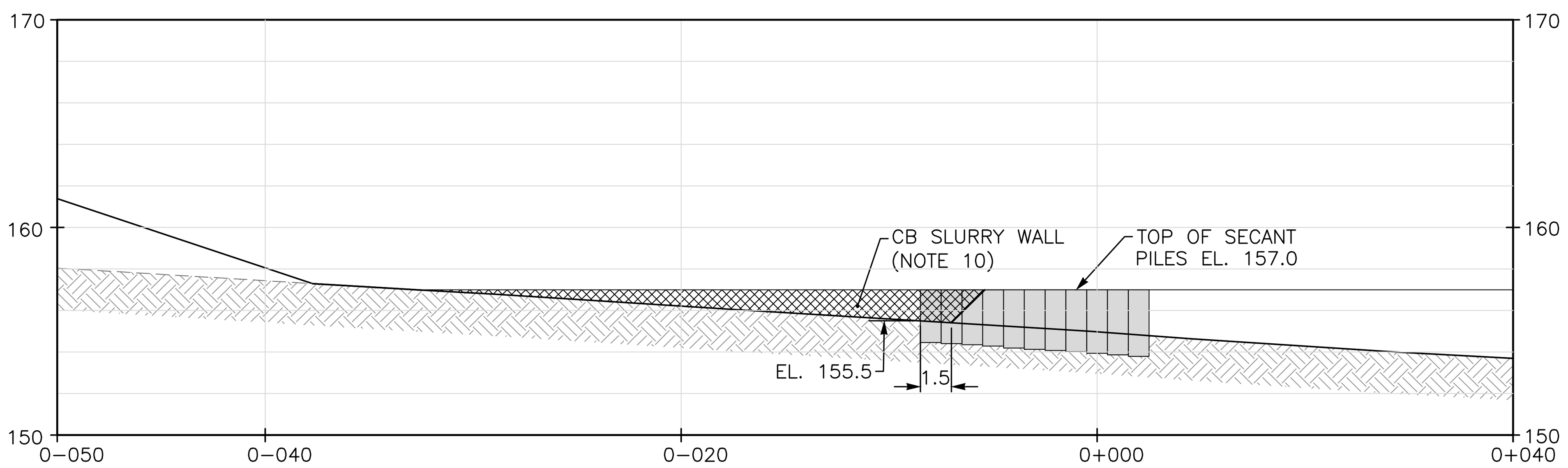
SECTION I-I EAST ABUTMENT AT STA. 0+820

SCALE 1: 250



SECTION J-J THERMAL BERM

SCALE 1: 250



DETAIL 3 - JUNCTION BETWEEN SLURRY AND SECANT PILES WALL

SCALE 1: 250

LEGEND FOR FILL MATERIAL

- (1) FINE FILTER
- (2) COARSE FILTER
- (3) ROCKFILL (NOTE 8)

NOTES:

1. WORKS SHOWN ON THIS DRAWING SHALL BE EXECUTED IN ACCORDANCE WITH THE APPLICABLE TECHNICAL SPECIFICATIONS.
2. ALL ELEVATIONS ARE IN METERS.
3. SEE DRAWING 0003 FOR THE LOCATION OF SECTIONS G-G TO J-J. THE LOCATION OF DETAIL 3 IS PRESENTED ON SECTION C-C, DRAWING 0007.
4. LOWER LIMITS OF PILES MAY BE ADJUSTED ON SITE BASED ON FIELD CONDITIONS. SECANT PILES SHOULD BE EXTENDED INTO BEDROCK (1m MIN.).
5. THE BEDROCK SURFACE AND ELEVATION WAS ESTIMATED. PRELIMINARY INVESTIGATION WILL HAVE TO BE DONE PRIOR TO THE CONSTRUCTION TO CONFIRM BEDROCK CONFIGURATION IN THE AREAS OF THE END OF THE SECANT PILE WALL.
6. WIDTH OF INITIAL PLATFORMS (19.9m) IS DEFINED BY AEM REQUIREMENTS FOR THE PASSAGE OF 100 TON TRUCKS (DUAL LANE). WIDTH OF WHALE TAIL DIKE (13.4m) IS DEFINED FOR PICKUP TRUCKS, DUAL LANE.
7. LIMITS SHOWN FOR ALL SECTIONS ARE BASED ON INTERPRETATIONS AND WILL VARY BASED ON NEW FIELD INFORMATION (INVESTIGATION, STRIPPED BEDROCK LEVELS ETC.).
8. THE MAXIMUM SIZE OF ZONE 3 ROCKFILL IS 1000mm. HOWEVER OVERSIZE ROCKFILL UP TO 1.5m IN DIAMETER IS ALLOWED IN THE EXTERIOR 15m OF THE ROCKFILL PLATFORMS WHERE THE PLATFORM THICKNESS IS GREATER THAN 2m.
9. ROAD SURFACING TO CONSIST OF 0.3m OF MINUS 50mm CRUSHED ROCK PLACED ON THE FULL WIDTH OF THE DIKE AT FINAL ELEVATION.
10. IN THE WEST ABUTMENT, CB SLURRY WALL WILL OVERLAP SECANT PILES BY 1.5m. SECANT PILES WILL EXTEND TO LOCATION WHERE BEDROCK IS AT EL. 155.5m.

ISSUE REGISTER

ISSUE No	REV.	DATE (Y/M/D)	PURPOSE OF ISSUE
04	00	2018-05-07	ISSUED FOR CONSTRUCTION
03	PC	2018-01-15	ISSUED FOR MDRB COMMENTS
02	PB	2017-12-22	ISSUED FOR CLIENT COMMENTS
01	PA	2017-12-09	ISSUED FOR INTERNAL COMMENTS.

ISSUE REGISTER

ISSUE No	REV.	DATE (Y/M/D)	PURPOSE OF ISSUE

REVISION REGISTER

No	REVISION DESCRIPTION	DATE (Y/M/D)	*	**

INITIALS: * DESIGNED ** APPROVED

REFERENCE DRAWINGS

No	
	Amaruq Baseline Studies, Potentially Environmentally, Significant Features. Project: DA14-053-04. Dougan & Associates.

PROFESSIONAL SEAL

REGISTERED PROFESSIONAL ENGINEER — A.A. ARBAIZA GUZMAN — LICENSEE — 2018-05-07 — NT/NU

REGISTERED PROFESSIONAL ENGINEER — D-Y JALBERT — LICENSEE — 2018/05/07 — NT/NU

PERMIT TO PRACTICE
SNC-LAVALIN INC.
Signature ______ P.Eng.
Date 2018-05-07
PERMIT NUMBER: P 260
The Association of Professional Engineers, Geologists and Geophysicists of NWT/NU

SNC·LAVALIN
Mining & Metallurgy
5500, des Galeries Blvd., bur. 200, Quebec (Quebec), Canada G2K 2E2
Telephone: (418) 621-5500, Fax: (418) 621-8887

PREPARATION	APPROVAL
DESIGNED: A. ARBAIZA, P. ENG.	PROJECT DISCIPLINE ENGINEER: G. HAILE, ENG.
DRAWN: D. LAN	PROJECT ENGINEERING MANAGER: Y. JALBERT, P. ENG.
CHECKED: Y. JALBERT/L. MACPHIE	CLIENT
DATE: 2018-05-07	PERMIT TO PRACTICE: D. LEAHY, P. ENG.
SCALE: AS SHOWN	

CLIENT: AGNICO EAGLE

PROJECT: AGNICO EAGLE – MEADOWBANK DIVISION
AMARUQ PROJECT

TITLE: WHALE TAIL DIKE
TYPICAL DESIGN SECTIONS
SHEET 2/2

PROJECT No	SUBDIVISION	SUBJECT	SERIAL	REV.
651298	2500	4G DD	0008	00

A1-HOR-FRAME-EN (SI)

ISSUE No	REV.	DATE (Y/M/D)	PURPOSE OF ISSUE
04	00	2018-05-07	ISSUED FOR CONSTRUCTION
03	PC	2018-01-15	ISSUED FOR MDRB COMMENTS
02	PB	2017-12-22	ISSUED FOR CLIENT COMMENTS
01	PA	2017-12-09	ISSUED FOR INTERNAL COMMENTS

ISSUE REGISTER

ISSUE No	REV.	DATE (Y/M/D)	PURPOSE OF ISSUE

ISSUE REGISTER

No	REVISION DESCRIPTION	DATE (Y/M/D)	*	**

INITIALS: * DESIGNED ** APPROVED

REVISION REGISTER

No	
	Amaruq Baseline Studies, Potentially Environmentally, Significant Features. Project: DA14-053-04. Dougan & Associates.

REFERENCE DRAWINGS

PROFESSIONAL SEAL

PERMIT TO PRACTICE
SNC-LAVALIN INC.
Signature P.Eng.
Date 2018-05-07
PERMIT NUMBER: P 260
The Association of Professional Engineers, Geologists and Geophysicists of NWT/NU

SNC·LAVALIN
Mining & Metallurgy
5500, des Galeries Blvd., bur. 200, Quebec (Quebec), Canada G2K 2E2
Telephone: (418) 621-5500, Fax: (418) 621-8887

PREPARATION	APPROVAL
DESIGNED A. ARBAIZA, P. ENG.	PROJECT DISCIPLINE ENGINEER G. HAILE, ENG.
DRAWN D. LAN	PROJECT ENGINEERING MANAGER Y. JALBERT, P. ENG.
CHECKED Y. JALBERT/A. ARBAIZA	CLIENT
DATE 2018-05-07	PERMIT TO PRACTICE D. LEAHY, P. ENG.
SCALE 1 : 1 250	10 0 10 20 30 40 50m

CLIENT
AGNICO EAGLE

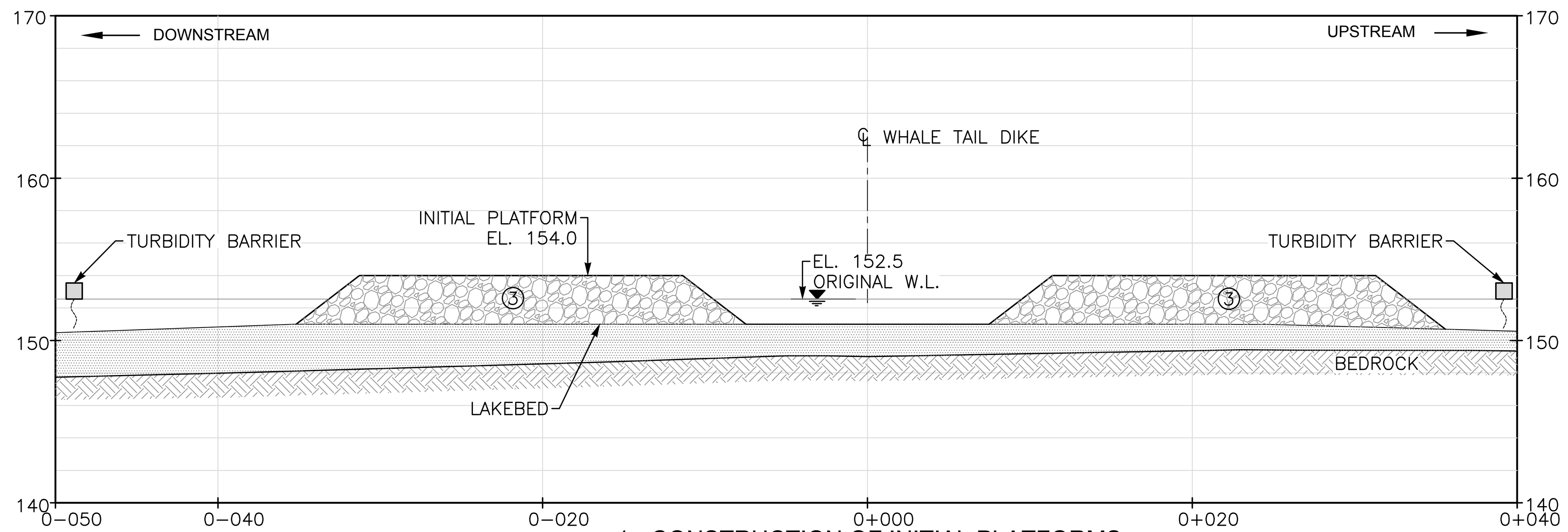
PROJECT
AGNICO EAGLE – MEADOWBANK DIVISION
AMARUQ PROJECT

TITLE
WHALE TAIL DIKE
TYPICAL SECTIONS AT 20m INTERVALS

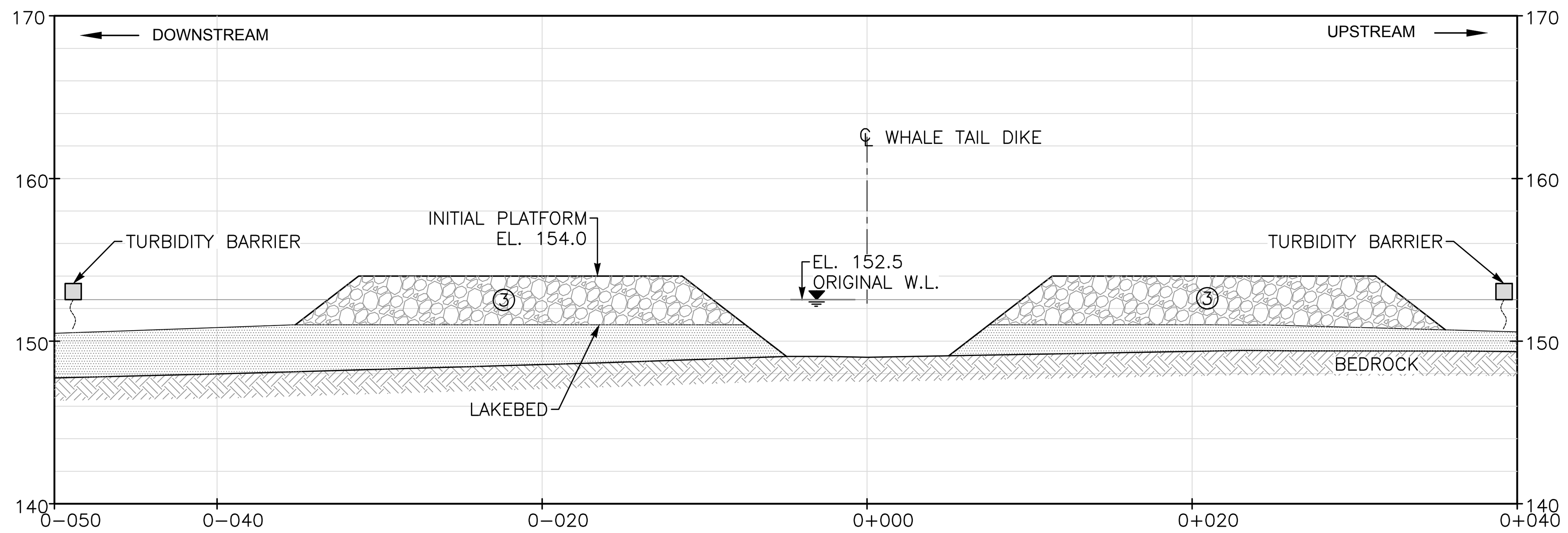
PROJECT No	SUBDIVISION	SUBJECT	SERIAL	REV.
651298	2500	4G DD	0009	00

A1-HOR-FRAME-EN (SI)

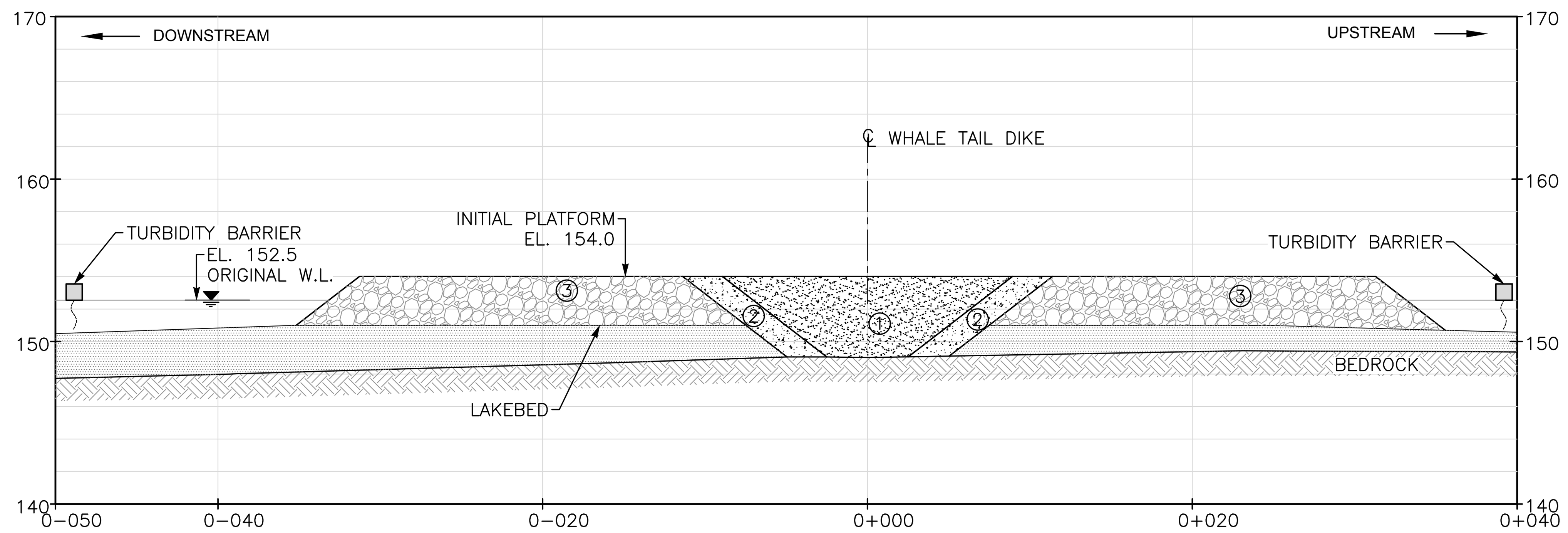
LAST SAVE: 2018/05/07 - 3:22pm WM-Geotech Amaruq\40_DD_DESSINS\00\651298-2500-4GDD-0009-00.dwg
PATH: T:\proj\651298 Detailed Eng



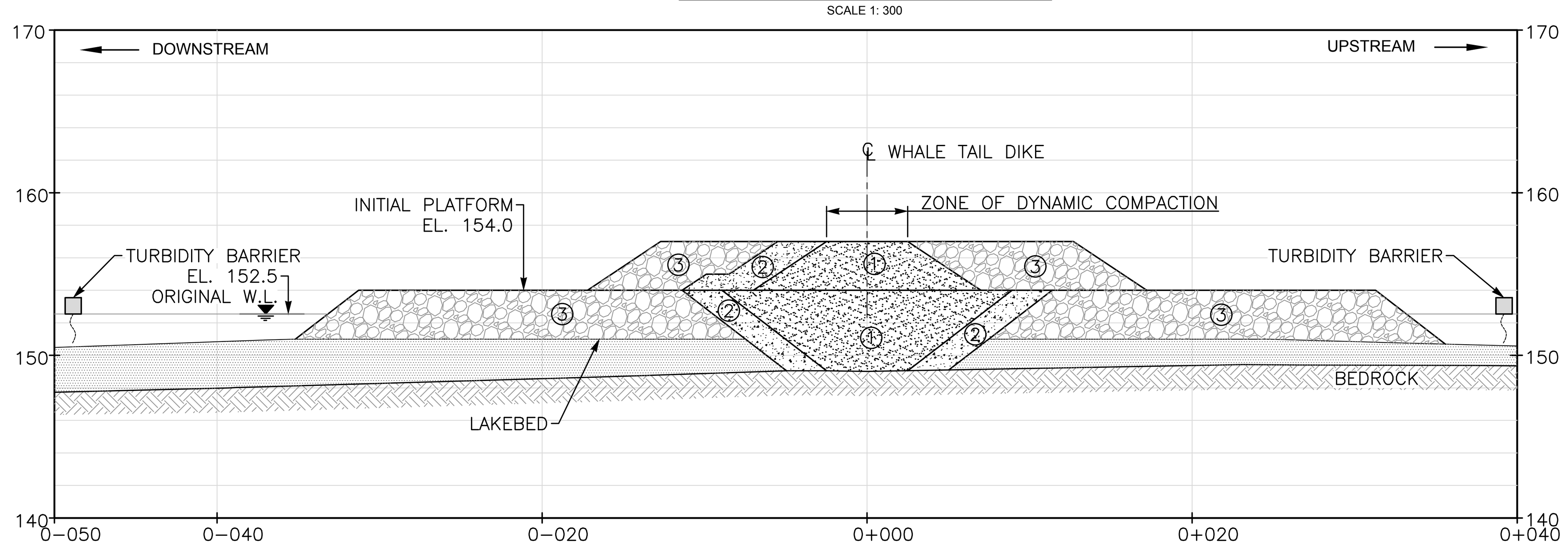
1 - CONSTRUCTION OF INITIAL PLATFORMS
SCALE 1: 300



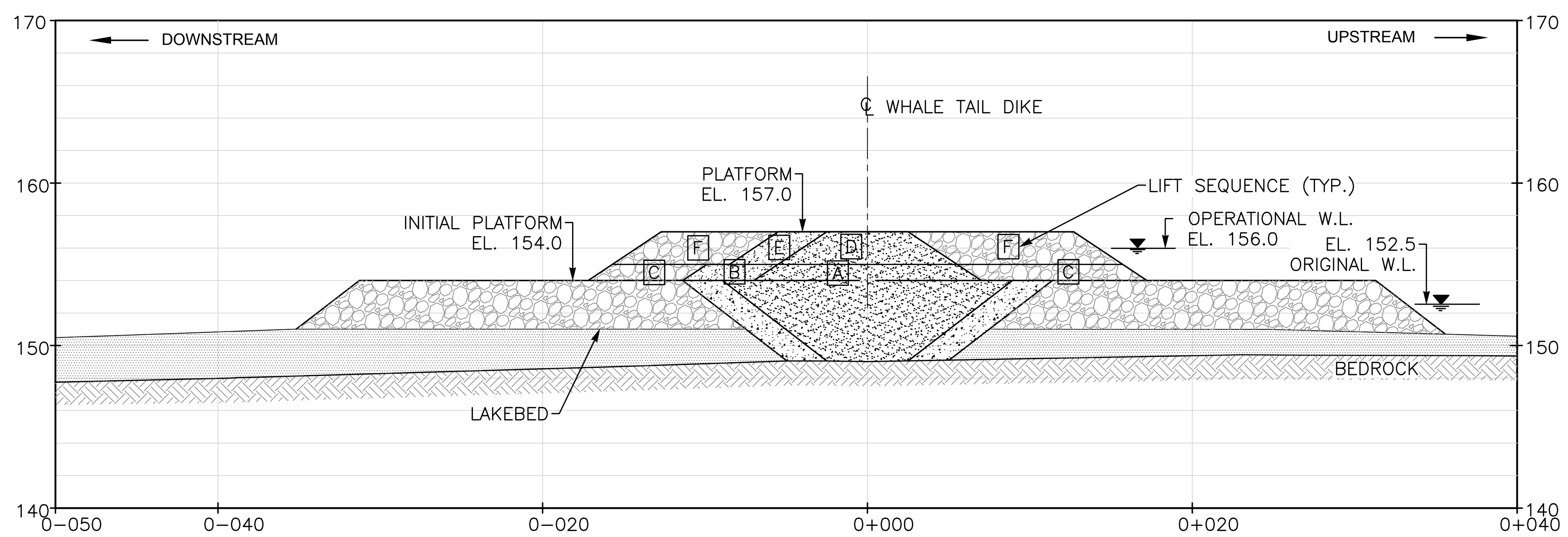
2- EXCAVATION IN KEY TRENCH
SCALE 1: 300



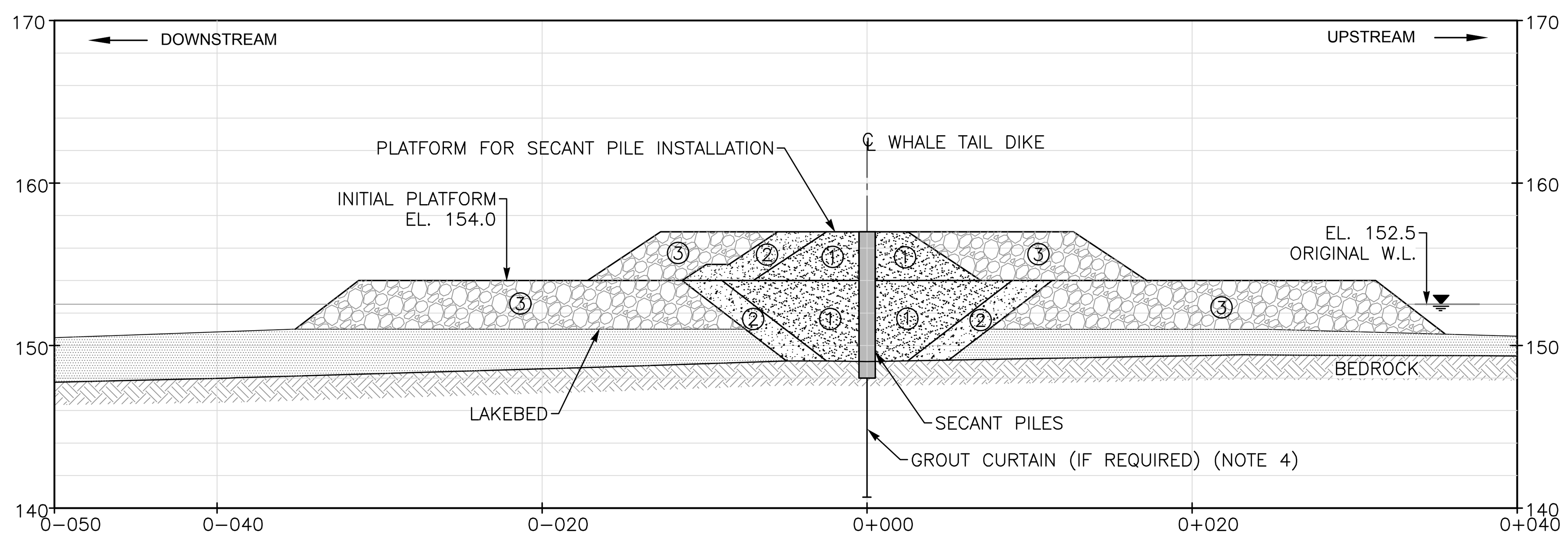
3- BACKFILL BETWEEN INITIAL PLATFORMS
SCALE 1: 300



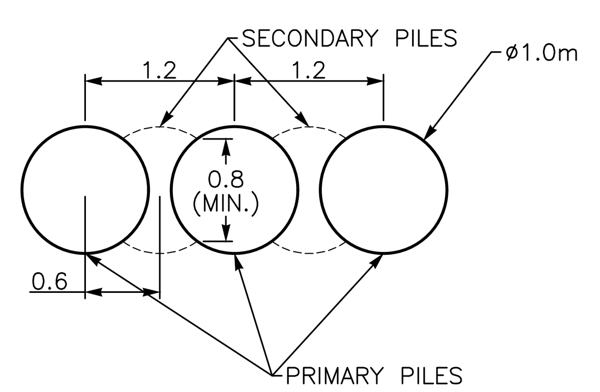
4- CONSTRUCTION OF WTD UP TO 157.0m AND DYNAMIC COMPACTION OF THE FINE FILTER BY HEAVY TAMPING
SCALE 1: 300



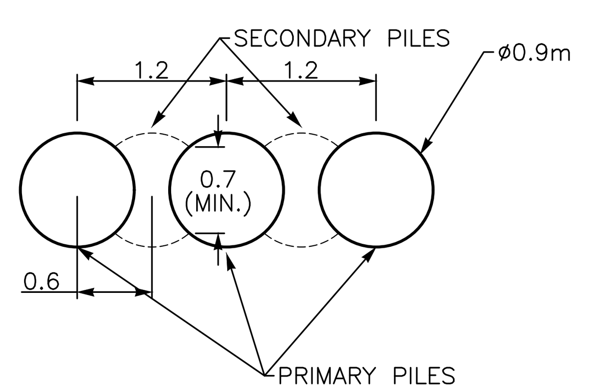
5- SUGGESTED LIFT SEQUENCE PLATFORM ELEVATION 154.0m TO 157.0m
SCALE 1: 300



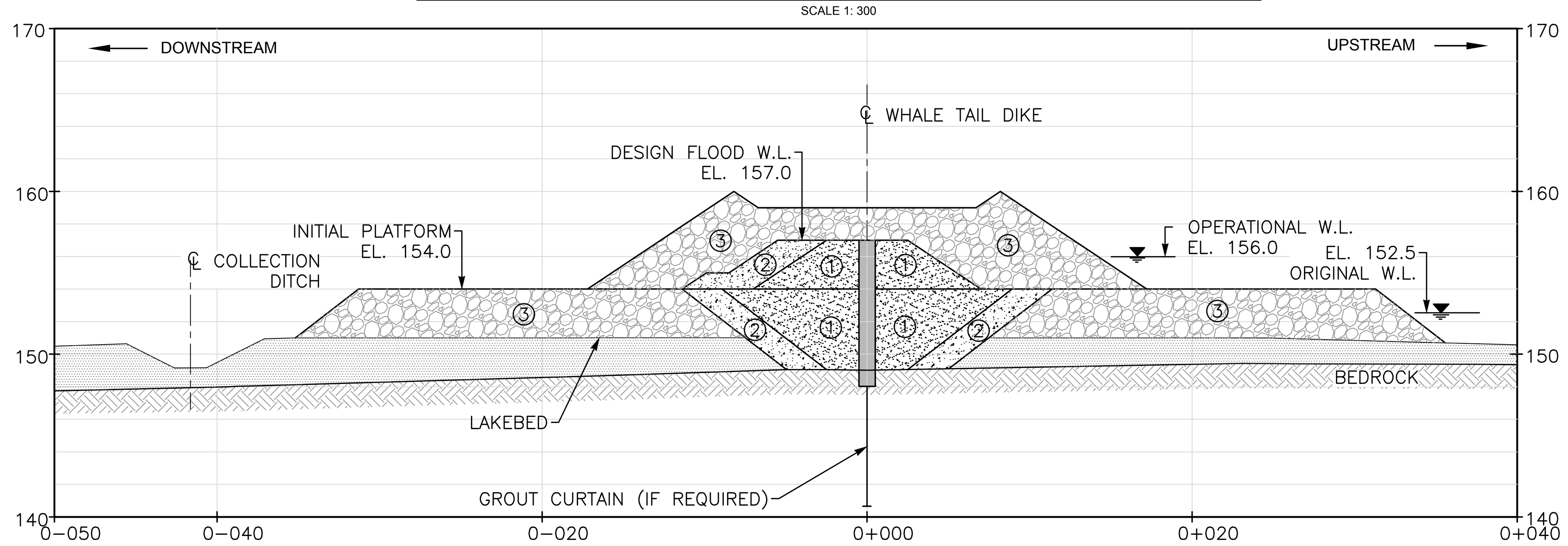
6 - INSTALLATION OF SECANT PILES AND GROUT CURTAIN IF REQUIRED
SCALE 1: 300



7 - PLAN OF SECANT PILES IN OVERBURDEN
SCALE 1:50



8 - PLAN OF SECANT PILES IN BEDROCK
SCALE 1:50



9- CONSTRUCTION OF WHALE TAIL DIKE TO ITS FINAL CONFIGURATION
SCALE 1: 300

LEGEND FOR FILL MATERIAL

- 1 FINE FILTER
- 2 COARSE FILTER
- 3 ROCKFILL

NOTES:

1. THIS DRAWING ILLUSTRATES CONSTRUCTION STAGES OF WHALE TAIL DIKE, TYPICAL FOR SECTION F–F.
2. WORKS SHOWN ON THIS DRAWING SHALL BE EXECUTED IN ACCORDANCE WITH THE APPLICABLE TECHNICAL SPECIFICATIONS.

ISSUE REGISTER

ISSUE No	REV.	DATE (Y/M/D)	PURPOSE OF ISSUE
04	00	2018-05-07	ISSUED FOR CONSTRUCTION
03	PC	2018-01-15	ISSUED FOR MDRB COMMENTS
02	PB	2017-12-22	ISSUED FOR CLIENT COMMENTS
01	PA	2017-10-17	ISSUED FOR INTERNAL COMMENTS

REVISION REGISTER

No	REVISION DESCRIPTION	DATE (Y/M/D)	*	**

INITIALS: * DESIGNED ** APPROVED

REFERENCE DRAWINGS

PROFESSIONAL SEAL

REGISTERED PROFESSIONAL ENGINEER — A.A. ARBAIZA GUZMAN — LICENSEE — 2018-05-07 — NT/NU

REGISTERED PROFESSIONAL ENGINEER — D-Y JALBERT — LICENSEE — 2018/05/07

PERMIT TO PRACTICE
SNC-LAVALIN INC.
Signature ______ P.Eng.
Date 2018-05-07
PERMIT NUMBER: P 260
The Association of Professional Engineers, Geologists and Geophysicists of NWT/NU

SNC·LAVALIN
Mining & Metallurgy
5500, des Galeries Blvd., bur. 200, Quebec (Quebec), Canada G2K 2E2
Telephone: (418) 621-5500, Fax: (418) 621-8887

PREPARATION	APPROVAL
DESIGNED: A. ARBAIZA, P. ENG.	PROJECT DISCIPLINE ENGINEER: G. HAILE, ENG.
DRAWN: D. LAN	PROJECT ENGINEERING MANAGER: Y. JALBERT, P. ENG.
CHECKED: Y. JALBERT./L. MACPHIE	CLIENT
DATE: 2018-05-07	PERMIT TO PRACTICE: D. LEAHY, P. ENG.
SCALE: AS SHOWN	

CLIENT: AGNICO EAGLE

PROJECT: AGNICO EAGLE – MEADOWBANK DIVISION AMARUQ PROJECT

TITLE: WHALE TAIL DIKE CONSTRUCTION STAGES

PROJECT No	SUBDIVISION	SUBJECT	SERIAL	REV.
651298	2500	4G DD	0010	00

A1-HOR-FRAME-EN (SI)

LAST SAVE: 2018/05/07 – 3:26pm
PATH: T:\proj\651298 Detailed Eng WM-Geotech Amaruq\40_DD_DESSINS\00\651298-2500-4GDD-0010-00.dwg

Appendix B

Technical specifications

Note: The Appendix 1 (construction drawings) of the technical specifications has been removed since the drawings are presented in Appendix A of this report. The Appendices 3 to 7 were not presented in the PB version of the technical specifications.

TITLE: **TECHNICAL SPECIFICATIONS FOR THE CONSTRUCTION OF WHALE TAIL DIKE**

CLIENT: AGNICO-EAGLE MEADOWBANK DIVISION

PROJECT: DETAILED ENGINEERING OF WATER MANAGEMENT AND GEOTECHNCAL INFRASTRUCTURE AT AMARUQ MINE

PREPARED BY : Angie Arbaiza, P.Eng, Yohan Jalbert, P.Eng and Tom Xue,P.Eng

REVIEWED BY : Les MacPhie, Getahun Haile, P.Eng and Pierre DeCourval

APPROVED BY : Yohan Jalbert, P. Eng.

Index of revisions

<table>
<tr><th colspan="4">Revision</th><th rowspan="2">Revised pages</th><th rowspan="2">Remark</th></tr>
<tr><th>No.</th><th>Prep.</th><th>App.</th><th>Date</th></tr>
<tr><td>PA</td><td>AA/YJ/TX</td><td></td><td>22/03/18</td><td>All</td><td>Internal review</td></tr>
<tr><td rowspan="2">PB</td><td>TX</td><td>PDC</td><td rowspan="2">30/03/18</td><td>Section 5.0</td><td rowspan="2">Issued for AEM comments</td></tr>
<tr><td>AA/YJ</td><td>LM/GH</td><td>All excepted 5.0</td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td></td><td></td><td></td><td></td><td></td></tr>
</table>

INSTRUCTION TO PRINT CONTROL: (Indicate X where applicable)

[ ] Entire Criteria revised. Reissue all pages

[ ] Reissue revised pages only

STAMP THE CRITERIA AS FOLLOWS:

[ ] Released for internal review

[X] Issued for comments and approval

[ ] Released for bid

[ ] Released for construction (installation specifications only)

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

1.0 WORK DESCRIPTION

1.1 Description of the project

Agnico Eagle Mines Limited, Meadowbank Division ("AEM") will be developing the Whale Tail Pit, a satellite deposit on the Amaruq property, as a continuation of current mine operations and milling at the Meadowbank Mine. The Amaruq 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 AEM 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 planning to extend the life of the mine by constructing and operating the Whale Tail Pit. As an amendment to the existing operations at the Meadowbank Mine, it is subject to an environmental review established by Article 12, Part 5 of the Nunavut Land Claims Agreement (NLCA).

The Whale Tail Dike (WTD) is a temporary dike that will be used to dewater a portion of the Whale Tail Lake and exploit gold resources in the Whale Tail Pit. WTD is located on a shallow plateau of the lake floor with an approximate 2 m depth of water. This plateau is located between deeper sections of the lake with water depths of about 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 the water level at which time discharge will occur in a new South Whale Tail Diversion Channel located west of the property.

1.2 Work included

The Work shall include mobilization of all necessary equipment and materials as well as providing supervision, technical personnel (including surveyors) and skilled labour for its execution.

A detailed work procedure describing how to execute the earthworks, the cutoff wall and the grout curtain shall be prepared by the Contractor for approval by the Owner and the Designer.

The Work includes but is not limited to the following items:

- The preparation of all documentation that the Contractor is required to provide prior to the beginning of the Work.
- Site preparation including snow, ice and boulder removal and proper disposal.
- Key trench drilling, blasting, stripping, excavation and grading.
- Granular fill loading, hauling, placement and compaction.
- Sampling and testing.

If judged necessary by the QA Engineers and/or the QC representatives, or the applicable representatives, additional tests shall be performed by an external laboratory.

1.3 Instrumentation

The technical specifications and drawings for the installation of the instrumentation will be issued later in the project and are presently excluded from the scope of work of the Contractor.

1.4 List of drawings

The list of construction drawings is presented in Table 1-1 and the drawings are included in Appendix 1.

Table 1-1: List of drawings

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

2.0 GENERALITIES

2.1 Unit system

Unless indicated otherwise, Amaruq's coordinate system is used, all elevations are tied to the UTM Zone 14, NAD83 (CSRS), and the metric unit system (SI) is used.

2.2 Norms and codes

Whenever mention is made of a standard or regulation, it is understood that the reference is to the most recent issue of the said standard or regulation unless specifically mentioned.

The Contractor may suggest the application of alternative standards provided that the resulting final product is at least equal in quality to that specified.

The standards presented in the following table shall be respected during the execution of the works.

Table 2-1: Standards and codes

Activity/Tests	Norm/Code
Soils – Grain size analyses	ASTM D422 - 63(2007), Standard Test Method for Particle-Size Analysis of Soils.
Weight of bentonite mud and cement-bentonite slurries	ASTM D4380-84, Standard Test Method for Density of Bentonite Slurries
Marsh cone viscosity of muds and cement bentonite slurries	ASTM D6910-04, Standard Test Method for Marsh Funnel Viscosity of Clay Construction Slurries.
Concrete : constituents and execution of the works	CAN/CSA A23.1/A23.2-M, Concrete materials and methods of concrete construction.
GU Cement	CSA-A3001, Cementitious materials for use in concrete.
Grout Mix	ASTM C150/C150M-11, Standard Specification for Portland Cement.
Grout Mix	ASTM C494, Standard Specification for Chemical Admixtures for Concrete.

Grout Mix	CSA-A23.1, Concrete Materials and Methods of Concrete Construction
Grout Mix	CAN/CSA-A3001, Cementitious Materials Compendium.
Grout Mix	CSA-A23.2-1B, Viscosity, Bleeding, Expansion and Compressive Strength of Flowable Grout.
Bedrock	ISRM (1981), Determination of some engineering properties of weak rocks.

2.3 Definition of terms and stakeholders

Below is a list of the stakeholders engaged in the Whale Tail Dike construction process:

- The **Owner,** represented by the AEM Geotechnical Supervisor, is responsible for the execution and coordination of the entire work. AEM will also be the contractor for the instrument installation (out of the current scope) and for provision, transport and dumping (at specific locations on the dike) of all the rockfill (Material Type 3) required for dike construction.
- The **Designer** is represented by SLI. The SLI **Resident Engineer** on site will act as the Designer's Representatives, as well as the QA Engineers..
- The **Contractor** is represented by SANA, the surveyors and all its subcontractors.
- Quality Assurance (**QA**) is represented by SLI for the entire Work.
- Quality Control (**QC**) is represented by:
  1. AEM which will be responsible for Quality Control (QC) during earthworks.
  2. SANA which will be responsible for Quality Control (QC) during dynamic compaction.
  3. SANA which will be responsible for Quality Control (QC) during cutoff wall installation.
  4. SANA which will be responsible for Quality Control (QC) during grout curtain operations.

2.4 Responsibility chart

The responsibilities of each stakeholder are defined as follows:

2.4.1 Owner's representative (AEM)

- Primary point of contact for the Designer, QA Engineer, QC representatives and the Contractor.
- Direct and review work and monitoring of construction.
- Share data with QA Engineers and QC representatives including but not limited to layout, scope limit control and data collection for as-built drawings and report.
- Review quantities.
- Coordination, daily interaction with QA and QC personnel.
- Follow-up and update the construction schedule.
- Prepare as-built report, including testing results, drawings and reports.
- Planning coordination meetings when required.
- Confirm the waste disposal area.
- Plan or approve platforms to stockpile material.
- The Owner shall supply, install, maintain and operate efficient and practical communication systems between the grout mixing unit, the grouting supervisor and the grouting operators at hole collars.

2.4.2 Quality Control representative (AEM and SANA)

- Inspection and documentation of work procedures to ensure the works meet the lines, grades and the specifications.
- QC testing as required by the specifications (Appendices 2 and 3).
- Prepare daily report.
- Work under the supervision of the Owner's Representative as applicable.

2.4.3 Quality Assurance Engineers (SLI)

- Inspection, documentation and review of work procedure to ensure that the work meets the specifications and the Design.
- Request additional testing if required by the specifications and review of QC testing and procedures.
- Review survey data.
- Prepare QA report to be included in AEM's as-built report.

2.4.4 Contractor (SANA):

- Construction of the Whale Tail Dike in such a way that the work meets the requirements of the drawings and the specifications.
- Carry out all survey and stake out.
- Supervise all its sub-contractors if any.
- Share all collected data with Owner's representatives, QA Engineers and/or QC representatives.

2.4.5 Designer (SLI)

- Review and approve documentation requested from the Contractor prior to the beginning of the Work.
- Be informed of the advancement of the Work.
- Make design change(s) when required.
- Send a sealed technical memorandum within appropriate timeframe to confirm the design change(s).
- Approve design change(s) submitted by the Contractor when required.

2.5 Line of communications

The line of communications defined by AEM is presented in the Appendix 6.

2.6 Work method and equipment

The Contractor shall present to the Owner's representative its working methods with the specific equipment and procedures one month prior to the beginning of the Work. The complete list of documentation to be provided prior the beginning of the Work is presented in Section 2.144.

2.7 Subsurface conditions

Based on the geotechnical investigations (carried out from 2015 to 2018), the subsurface profile generally consists of sand and gravel overburden with cobbles and boulders and/or glacial till overlying weathered bedrock. The bedrock encountered in the boreholes varies from greywacke (sedimentary) to diorite (intrusive). Both lithologies are deformed with an oblique foliation structure varying from weak to very strong in intensity. The most dominant structures are the foliations and then the veinlets. The Rock Quality Designation (RQD) values along the dike foundation vary significantly with a typical values in the range of 25 to 70%. Based on joint spacing and RQDs, and according to ISRM (1981), the bedrock can be characterized as close to very closely jointed with poor to fair quality.

Based on the information available, the Contractor shall make its own interpretations and/or perform its own investigation at no cost to the Owner.

2.8 Lines, grades and tolerances

1. Lines and grades shall be obtained from the drawings in Appendix 1.
2. The Contractor's surveyor shall be responsible for all staking and other survey requirements such as lines and grades specified or shown on the drawings.
3. Lines and grades are subject to modification by the Designer's representatives (when justified) and additional lines and grades may be required as the work progresses.
4. The minimum dimensions of excavation and fill areas are presented on the drawings. Settled areas shall be corrected by placing additional appropriate material to the dimensions presented on the drawings.
5. The tolerance on lines and grades is 0.1 m.
6. As-built drawing(s) will be prepared by AEM.

2.9 Additional drawings

The SLI representative may provide additional drawing(s) if considered necessary. These drawings shall form part of the contractual document.

2.10 Water Quality Monitoring Program

The Contractor shall be equipped with all the necessary pumps, hoses, parts, accessories, and labour to complete the Water Quality and Monitoring Plan as requested and presented in Appendix 5.

2.11 Land, lake, environment and infrastructure protection

1. The Contractor shall limit traffic to the area inside the boundary established by the Owner and his representative.
2. Fires are not allowed on site.
3. The Contractor shall make sure that all personnel under his responsibility will do everything possible to protect the environment.
4. Unless approved by the Owner, once construction is completed, no material shall be left on the ice cover of Whale Tail Lake.
5. All excavated materials must be disposed of as directed by the Owner or his representative.

6. Unless approved by the Owner, all excavated snow shall be disposed of as per AEM's Environmental Management Plan.

2.12 Site cleanup

1. The Contractor is responsible for the cleanup and removal of garbage and other foreign materials from the construction site to the satisfaction of the Owner or his representative.

2.13 Health and Safety

1. All construction work shall be conducted in accordance with AEM's sustainable development and Health and Safety standards and regulations.
2. All personal protection equipment appropriate for the work shall be used by all workers.
3. Detailed work procedures for every construction task shall be provided by the Contractor and approved by the Owner or its representative.
4. A Detailed Job Safety Analysis (JSA) shall be completed for each construction task and submitted by the Contractor to the Owner for approval.
5. A daily coordination meeting shall be held by the Contractor, QA Engineers, as well as by the QC and AEM representatives to discuss planning and safety.

2.14 Documentation to be provided by the Contractor

At least thirty (30) days prior to the beginning of the Work, the Contractor shall present the following documents:

1. Location of the stockpile area he plans to use as well as the location of borrow sources and access roads.
2. Technology and methods planned to be used to prepare bathymetric maps.
3. A list of specific tools and equipment which will be used to reach bedrock during the excavation of the key trench.
4. Procedure and technique for placement of fill material by lifts from elevation 154.0 m to 157.0 m.
5. The proposed method of compaction of the fine filter including elevations of working platforms to perform dynamic compaction, crane and pounder details, impact grid spacing, work area, Quality Control program and safety procedures.
6. A detailed schedule of the Work.
7. A secant pile installation program that includes the proposed sequence and timing of pile installation that is consistent with strength requirements for adjacent piles.

8. The CB mix the Contractor proposes including additives, details of the materials, the mix proportions and the lab test results.
9. A waste management plan for the secant piles (for water and spoil).

3.0 EARTHWORKS

3.1 Work sequence

1. The method of construction shall be adapted to minimize risks of contamination within stockpiles and / or backfill material.
2. The passage of trucks and other equipment shall be adapted to minimize ruts in the working areas. The Contractor shall grade the affected surfaces when required.

3.2 Site preparation

3.2.1 General

1. The Contractor's attention is drawn to the fact that the work will be executed (in part) during arctic winter and that the work will be carried out in a protected area (in an environmental context). Special care shall be taken to ensure the safety of all employees, to avoid damage to the land and to avoid breaking the ice covering the lake outside the designated working area.
2. Prior to the start of any work, the temporary thermistor strings installed during the geotechnical investigations in 2015 and 2017 shall be removed with care, handled, and stored as directed by the Owner or his representative.
3. Where required, all permanent thermistor strings installed in the past will be buried or destroyed after taking the last reading.

3.2.2 Access roads

1. The Contractor is responsible for the maintenance of the access roads he uses.
2. If required, the Contractor shall submit to the Owner or its representative full details of all temporary construction roads, ramps and access planned for the construction. Details related to these temporary works shall include location, alignment, required safety berm or traffic signs, period of use, materials used, water management from runoff water and plans for their removal. These temporary works will have to be approved prior the beginning of their construction.
3. The Contractor shall maintain in good condition all existing or new access roads used for the execution of the work, such as the access roads connecting the work area to stockpiles and waste dump areas, to the satisfaction of the Owner or his representative.
4. All temporary access roads shall be constructed on top of the existing ground. No stripping or excavation shall be undertaken unless approved by the Owner or his representative.
5. The Contractor shall supply and install all required traffic signs and safety equipment to ensure workers' safety at the construction site for the complete duration of the work.

6. Access road maintenance shall be planned and executed in such a way that workers' safety is not compromised. Access roads shall be kept clean of snow and, if required, sprinkled with abrasive materials such as gravel to the satisfaction of the Owner or his representative.
7. When required, a dust control (water or other approved environmentally safe product) shall be applied to minimize dust in the work area.
8. Once the construction work is complete, all temporary access roads shall be removed and the material disposed of as directed by the Owner.

3.2.3 Clearing and stripping

1. The Contractor shall clear snow, ice and boulders from the ground surface within the Whale Tail Dike footprint area prior to start the work.
2. The material removed shall be stockpiled separately in areas approved by the Owner or its representative.

3.3 Water management during construction

The Water Quality Monitoring Program during the Whale Tail Dike construction is presented in Appendix 5. The Contractor shall manage the Total Suspended Solids (TSS) by the installation of turbidity barriers and pumping and treating turbid water. The Specifications for these two activities are not in the scope of work of this document but shall be performed by the Contractor with care to minimize risks of construction delays.

3.4 Foundation preparation and approval

3.4.1 General

1. The definition of the bedrock profile is an important aspect of the Work. Extensive effort shall be deployed to define and clean the bedrock surface.
2. Prior to anyexcavation, a complete bathymetry and on land survey of the dike footprint shall be carried out. The Contractor shall include in the documentation to be presented, the technology and methods he plans to use to carry out the bathymetric survey in the expected conditions with highly turbid water.
3. The QC representative may occasionally request that additional soil stripping and removal of snow and ice be carried out in areas outside the dike footprint.
4. Excavation shall be done with appropriate equipment and tools such as a backhoe equipped with teeth or hydraulic attachment (tarmac), dozer with ripper in dry sectors or any other tool that will allow reaching the bedrock in the key trench. A list of these specific tools and equipment shall be presented in the documentation that shall be provided.

5. Excavated materials shall be set aside separately or stockpiled in areas approved by the Owner or its representative.
6. All survey, approval and visual inspection forms prepared by the Owner and QC representatives and/or QA Engineers shall be signed by all stakeholders.
7. The QC/QA representatives may, from time to time, request additional excavation.
8. Final key trench depth as well as longitudinal and lateral extensions shall meet the dimensions shown on the drawings and shall be approved by the Owner and the QC/QA representatives.
9. Approval of the foundation by the Owner or his representative shall be done prior to placing any backfill.
10. All piles bottoms shall be checked for the presence of loose or soft materials before placing the CB mix. The Contractor shall remove any such loose or soft material which is likely to affect the performance of the pile.

3.4.2 Excavation into Whale Tail Lake

1. Between the construction platforms, the Contractor shall excavate all material until refusal to further excavation is reached.
2. The equipment used to excavate between the two platforms shall make the necessary effort to reach the bedrock surface and remove it, debris, soil and boulders and provide a clean surface to the satisfaction of the QC representatives or the QA Engineers.
3. Additional stripping, excavation and bathymetric survey shall be carried out until the foundation is approved.
4. For the sectors where frozen conditions are encountered, additional time may be required to thaw the foundation before performing stripping and excavation.
5. Excavated material shall be disposed in the Waste Rock Storage Facility (WRSF) or any other designated area approved by the Owner.

3.4.3 Blasting Whale Tail Dike abutments

1. The Contractor shall be responsible for the blast hole depth, spacing, and pattern prior to blasting of the frozen overburden. The Contractor shall use a drilling pattern with blast hole depths that will produce a key trench configurations as shown on the Drawings. The Contractor`s attention is drawn to the requirement that blasting of the ice-rich till at the east abutment shall be carried out with care to minimize cracking of frozen ground below excavation level.
2. If possible, the Contractor shall monitor bedrock depth for each blast hole. Blast hole pattern shall be defined according to the bedrock depth encountered.

3. The Contractor shall put all possible effort to limit vibrations within the dike foundation.
4. Blast monitoring to evaluate the vibrations shall be done with blast seismographs. Locations of the blast seismographs will be provided by the Owner and / or the Designer.
5. The material shall be excavated immediately after the blast.
6. The foundation shall be approved before placing any backfill.
7. Blasted materials from the east abutment shall not be used for the Whale Tail Dike construction and shall be disposed in the Whale Tail WRSF.
8. Blasted material (above elevation 153.0 m) from the west abutment shall be used for the construction if it meets the requirements specified in Section 3.7.
9. Foundation of the blasted areas shall be inspected and approved prior to any backfill placement. The approval shall be given once the absence of ice-rich till in the foundation has been validated.

3.5 Stockpile and disposal areas

1. A temporary stockpile area can be created at the west abutment of the Whale Tail Dike. However, the Contractor may propose a different stockpile area and a method to manage the runoff from the area. This proposed area shall be presented in the documentation to be provided to the Owner for approval (Section 2.14).
2. The designated waste disposal area is located in the Whale Tail WRSF. Any other waste disposal area shall be proposed in the documentation to be provided (Section 2.14) for approval by the Owner.
3. The Contractor shall develop its stockpiles to facilitate drainage and minimize segregation.

3.6 Use of borrow sources

3.6.1 General conditions

1. Prior to developing borrow sources, the Contractor shall present in the documentation (Section 2.14) complete information on the location and the details of the execution plan including tonnage, limitations, water management and rehabilitation.
2. The Contractor shall develop borrow sources at sites selected by the Owner and properly manage these fill sources to generate the required materials for the project in such a manner that all of the fill materials derived meet the material quality requirements of the technical documents.
3. The Contractor shall carry out all blasting operations that may be required at the eskers in accordance with AEM’s Standard Practice Instruction.

4. The Contractor shall proceed with the generation of fill materials such that the materials can be inspected, classified and placed immediately at the designed locations. With the exception of the rockfill (Material Type 3), the Contractor is responsible for planning the production of each fill material and its final placement in designated areas so as to avoid as much as possible double handling.
5. All materials unsuitable for fill shall go to designated stockpiles or spoil dumps. The Contractor shall develop and maintain separate stockpiles and separate spoil dumps for each material being excavated.
6. The Contractor may, if required, process the unsuitable material by drying, thawing, dewatering, screening, raking or by any other means to make the material stable and suitable prior to incorporating it into the fill.
7. All other materials not meeting the gradation specifications and index properties as indicated in Section 3.7 shall be stockpiled in spoil dump(s) approved by the Owner.
8. The Contractor shall adequately compact, trim and drain stockpiles and spoil dumps as may be necessary to maintain them in a stable condition and to such slopes and levels as the Owner shall direct. They shall be kept free draining at all times with edges trimmed to stable batters and surfaces free of ponded water.

3.6.2 Temporary stockpiles

1. The locations of all temporary stockpiles shall be proposed prior to the beginning of the Work and included in the documentation to be provided by the Contractor (Section 2.14).
2. The runoff from stockpiles shall be collected and drained or pumped towards authorized sectors for treatment before release to the environment.

3.7 Fill materials

3.7.1 General

1. Only sound and suitable materials meeting the requirements of this document and approved by the QC representatives and/or QA Engineers shall be used.
2. Great care shall be taken to limit particle segregation during placement. Occasionally, the the QC representatives and/or the QA Engineeers may ask the Contractor to modify its construction procedures to meet this requirement.
3. Fill materials shall be free from all organic matter or other deleterious, unapproved, unstable or unsuitable materials, such as ice and/or snow, frozen fill, or peat.
4. Unless approved by the Owner, or the QA Engineer and the QC representatives, all fill materials shall only be obtained from stockpiles or sources identified at the beginning of the construction works.

5. All materials shall be manufactured from good quality non acid generating (NAG) sources.
6. Stockpiling, loading, and placement of fill materials shall be carried out in a way that minimizes segregation.

7. If needed, road surfacing shall consist of 0.3 m of < 50 mm crushed rock placed on the full width of the dike crest at final elevation.

3.7.2 Definitions

1. "Sound" or "Suitable" fill materials are defined as being free from deleterious matter, having a gradation which permits compaction or placement to a stable state, and having the characteristics specified for the particular materials when all necessary handling, re-handling, processing, and reprocessing have taken place.
2. "Unstable" or "Unsuitable" fill materials are defined as being too wet, containing oversized or segregated particles, organic or other deleterious matter, such as ice or snow, or having poor characteristics which may result in undesirable settlement or other movement of the fill or within the fill, or otherwise not meeting the requirements of the specifications. However this definition permits thawing, drying, dewatering, watering, screening, raking and any other processing or reprocessing to make the material stable and suitable prior to incorporating it into the fill.

3.7.3 Fine filter

1. The material to be used for the fine filter of Whale Tail Dike shall be uniform clean sand (Material Type 1) and shall meet the gradation limits specified below in Table 3-1.

Table 3-1: Fine filter gradation

Sieve (mm)	Passing (%)
20	100
4.75	70-100
2	49-100
0.850	21-95
0.425	8-78
0.25	3-51
0.150	2-25
0.106	2-15
0.075	1-10

2. The Contractor shall avoid excessive handling of this material to prevent particle segregation.

3. For the material to be placed below elevation 154.0 m, special care shall be taken in the technique of placement. The placement below water shall be done with the bucket of an excavator.
4. For the material to be placed above elevation above 154.0 m, the Contractor shall include in its documentation (Section 2.14) a technique for the placement of the fill by lift thicknesses of 0.5 m up to elevation 157.0 m.
5. The fill above elevation 154.0 m can be compacted with a vibratory roller compactor. If this technique of compaction is used, the material shall be compacted to at least 95 percent of Standard Proctor density. If required, a test pad for compaction performance shall be done with the roller compactor.
6. The fine filter above elevation 154 m may have to be placed in freezing temperatures. In these conditions, the fill shall be unfrozen and shall be compacted before it freezes. In addition, any snow and ice accumulated on the previous lift shall be removed before placement of the new lift.

3.7.4 Coarse filter

1. The material to be used for the coarse filter of Whale Tail Dike shall be a well graded clean granular fill (Material Type 2) and shall meet the gradation limits specified below in Table 3-2.

Table 3-2: Coarse filter gradation

Sieve (mm)	Passing (%)
75	100
50	85-100
20	50-85
4.75	23-42
2	13-28
0.850	7-19
0.425	0-11
0.25	0-8
0.150	0-5
0.106	0-3
0.075	0-2

2. The Contractor shall avoid excessive handling of the coarse filter to prevent particle segregation.
3. The material below elevation 154.0 m shall be placed with an excavator bucket prior the fine filter placement.

4. The material above elevation 154.0 m shall be placed in a a maximum lift thicknesses of 0.5 m prior to compaction.
5. Placement and compaction of the fill must be performed to the satisfaction of the QC representatives and the QA Engineers.
6. The coarse filter above elevation 154 m may have to be placed in freezing temperatures. In these conditions, the fill shall be unfrozen and shall be compacted before it freezes. In addition, any snow and ice accumulated on the previous lift shall be removed before placement of the new lift.

3.7.5 Rockfill

1. The material to be used for the initial platforms (up to elevation 154.0 m), for the shell of Whale Tail Dike and the safety berms, shall be a well graded crushed rock < 1000 mm (Material Type 3) from a quarry, pre-production or production zone. After placement it shall meet the gradation limits specified below in Table 3-3.

Table 3-3: Rockfill gradation

Sieve (mm)	Passing (%)
1000	100
300	40-100
150	20-70
75	11-50
50	8-40
20	0-22
4.75	0-7

2. Oversize rockfill up to 1500 mm is allowed in the exterior 15 m wide strip of the rockfill platforms where the platform thickness is greater than 2.0 m.
3. As much as possible, finer rockfill for the platforms shall be placed in the fill zone closer to the key trench.
4. For the platforms, the Contractor shall dump the rockfill on the horizontal surface of the platforms and use a dozer to advance the front of the rockfill and thus limit segregation.
5. For rockfill above elevation 154 m, the maximum allowable lift thickness is 2.0 m.
6. For rockfill above elevation 154 m the Contractor shall use heavy equipment traffic to promote its compaction. In addition, each lift shall be compacted with at least 4 passes of a heavy dozer or equivalent.

7. Placement and compaction of fill must be performed to the satisfaction of the QC representatives and the QA Engineers.

3.7.6 Non-conforming materials

1. Where and when directed by the QC representatives or the QA Engineers, the Contractor shall excavate and/or remove all unsuitable materials to the designated spoil or dump.

3.8 Dynamic compaction

3.8.1 Definitions

1. The crane, usually track-mounted, is a device that lifts the tamper to the specified height with a single cable and allows it to free-fall. The crane must be manufactured or modified to allow free spooling of the drum.
2. The tamping grid spacing is the horizontal distance between the centers of impact locations.
3. A crater is the depression formed in the fill following single or multiple impacts of the tamper.

3.8.2 General

1. The scope of work of the dynamic compaction is to densify the core of Whale Tail Dike in the shortest time possible. The secant pile wall shall be drilled into densified fine filter (Section 3.7.3).
2. The objective of densification is to minimize local caving below the end of the casing during drilling for the secant piles, which is comparable to slurry wall installation experience acquired with the dewatering dikes at Meadowbank.
3. The Contractor shall be responsible for all activities related to the dynamic compaction including the survey of each crater location, their backfilling and their densification and levelling.
4. In the documentation to be provided, the Contractor shall submit the work plan he proposes including elevations of working platforms to perform the dynamic compaction, crane and pounder dimensions, grid spacing for the impact points for each phase, Quality Control program and safety procedures.
5. The Contractor shall accurately lay out the proposed impact points using wire flags or other methods acceptable to the QC representatives and the QA Engineers.
6. The Contractor shall ensure that he has sufficient material to fill the craters within a reasonable time.

7. The Contractor shall present a separate technical report summarising the results of the crater filling activity with a layout drawing.
8. No dynamic compaction shall be allowed within 50 m radius of the closest filled secant pile.
9. The Contractor shall take special care to protect workers from projected rocks or cobbles due to dynamic compaction and shall implement all safety measures required to minimize risks.
10. The Contractor shall present on a daily basis its short term schedule (2 days) for the work area planned. If required, and for a restricted area, a flagman shall be present to control the traffic.
11. Craters shall be leveled on a daily basis by the Contractor to prevent ponding of water at their bottoms. Any standing water in the craters shall be removed by the Contractor prior to leveling/backfilling of the craters. Unsuitable or deleterious materials shall be removed from craters by the Contractor as directed by the Engineer prior to leveling.
12. The Contractor shall level and proof roll all crater locations at the end of each work day. Additionally, proof rolling during post-improvement surface compaction shall identify any soft spots at the site. Proof rolling shall consist of a minimum of 4 passes of a vibratory roller.

3.8.3 Compaction criteria

1. No STP or pressure meter tests will be required to evaluate the densification achieved by the compaction.
2. The minimum compaction criterion required shall be similar to what has been achieved for the dewatering dikes at Meadowbank where the objective was to ensure the stability of the open trench during the construction of the slurry wall.
3. The optimal densification requirement of the core of WTD shall be developed on site by the Contractor and approved by the Designer's representative.
4. The Quality Control required during the densification shall measure crater depth and volume to estimate the degree of densification due to volume change.
5. The energy of compaction induced during the Work shall be estimated.

3.9 Adverse conditions

The Contractor shall not carry out any excavation, placing or compaction of fill materials when conditions are such that in the opinion of the QC representative and the QA engineer, the quality of the work or adjacent works would be adversely affected. After any operation has been stopped owing to adverse conditions, it shall not be re-started without the approval of the QC representatives and or QA Engineers.

3.10 Acceptance

It is the Contractor's responsibility to submit for acceptance by the Owner's Engineer works or sections of works completed to the lines and grades shown on the drawings.

4.0 CUTOFF WALL

4.1 General

1. The Contractor is responsible for providing all the required equipment (drills, mixers, silos, pipes heaters, spares, etc) to complete the Work as shown in the drawings.
2. The Contractor shall have adequate drilling tools and chisels to drill into any soil conditions including large boulders and frozen ground. The Contractor shall be aware of the site conditions and be familiar with the geotechnical investigation completed in the sector.
3. The cutoff wall of Whale Tail Dike shall be composed of Cement-Bentonite (CB) secant piles. At the west abutment, the secant piles will tie in to a CB slurry wall.
4. At the west abutment, the CB slurry wall shall be completed before the adjacent secant piles are constructed. The secant pile wall shall overlap the CB slurry wall by 1.5 m horizontally as shown on the drawings. The last secant pile shall be located where bedrock surface is at elevation 155.5 m.
5. After the approval of fills constructed to elevation 157 m, the Contractor shall prepare setting out drawings showing the location of each pile to be installed and submit the drawings to the Designer for approval within 7 days prior to construction.
6. The Contractor shall be responsible for the accuracy of the location and positioning of each pile. Any error in setting out and any consequential loss to the Owner shall be assumed by the Contractor.
7. Upon completion of all piling works, the Contractor shall produce as-built Drawings showing the positions of all piles as installed. The positions of piles shall be verified and endorsed by the surveyor.
8. The Contractor shall ensure not to damage the completed piles through his method of working. The Contractor shall submit in the documentation (Section 2.14) a pile installation program. The proposed sequence and timing of pile installation shall be such that the installation work shall not cause any damage to adjacent piles. Piling work shall not commence until approval has been obtained.
9. Spoil shall be disposed in the Whale Tail WRSF area. If the Contractor wants to dispose the spoil elsewhere, he shall submit his proposal to the Owner for approval.
10. Prior to the beginning of the Work, the Contractor shall submit a waste management plan for the secant piles that shall include spoil and waste water.
11. The Contractor shall be responsible for having all equipment, facilities and labour to dispose of spoil and waste water produced during secant pile Work.
12. The Contractor shall be responsible for insulating all the facilities and equipment to be able to work in winter.

13. The pile diameter selected by the Contractor shall be subject to the approval of the Designer.

4.2 CB design mix criteria

The cement-bentonite (CB) mix shall meet the compressive strength, permeability and constructability requirements. The mix design criteria for fresh and cured CB slurry mix for the cutoff wall are presented in the following table.

Table 4-1: Criteria for CB mix design

Characteristic	Requirement
Permeability (cm/sec)	≤ 10^{-6} after 28 days of curing
Unconfined compressive strength (UCS), (kPa)	≥ 200 after 28 days of curing
Minimum Early-Strength	Minimum UCS of 50 kPa after 7 days of curing
Marsh Viscosity (seconds)	≤ 80 over 8 hours of curing
Density (g/cm3)	≥ 1.2

4.3 Materials

4.3.1 Storage

1. The Contractor shall store the material in accordance with the manufacturer's recommendations.
2. The Contractor shall make a complete inventory of the stock on weekly basis and provide the information to the Owner.

4.3.2 Cement

The cement to be used for preparing the CB slurry shall be ordinary Portland cement GU (10) conforming to the CSA-A3001-13 standard.

4.3.3 Bentonite

The bentonite to be used for preparing the CB slurry shall be the BARAKADE 90 that meets API 13A, Section 9 specifications.

4.3.4 Water

Water used for CB mix preparation shall be fresh and obtained from an approved local source free of oil, silt, soluble chloride, organic matter, acid, alkali and other undesirable substances in conformance with ASTM C1602.

The current laboratory testing program used tap water to prepare the CB slurry mixes tested. The actual CB slurry mix on site shall be prepared by using water from Whale Tail Lake. The Contractor shall perform its own tests to confirm the proposed mix design is in accordance with the specifications.

4.3.5 Additives

The additive to be used for the CB slurry shall be the ARBO S01_P (Sodium Lignosulfonate), from KemTek Industries Inc.

Some additives are sensitive to freezing temperatures. If possible, non-sensitive material shall be used.

The use of any additive shall be approved by the Environmental department of the Owner.

4.3.6 Proposed Cement-Bentonite mix

Lab testing has been done to establish a mix design that satisfies the design criteria and the resulting mix ratios are given in Table 4-2.

Table 4-2: CB design mix ratios

Material	Value (by weight)
Cement (Cement/Water)	0.4
Bentonite (Bentonite/Water)	4.6 %
Additive (Additive/Cement) (ARBO S01P – Sodium Lignosulfonate/Cement)	0.5 %

The mixture to be used for 1 cubic meter of slurry is:

- 875 liters of water
- 350 kg of cement
- 40 kg of bentonite
- 1.75 kg of additive (ARBO S01P – Sodium Lignosulfonate)

However, the Contractor shall submit its CB mix design or any other additives to the Designer for approval prior the beginning of the Work.

4.4 Execution of CB cutoff wall

4.4.1 Working platform

Prior to the beginning of the Work, the Contractor shall verify that the width of the dike is suitable for operations and traffic during construction.

4.4.2 Slurry trench

1. A trench in the fine filter shall be excavated at the west abutment as shown on the drawings. The trench shall expose the bedrock surface from elevation 157.0 m (the west end of the trench) to a point 1.5 m east of the location where bedrock surface elevation is155.5 m (the east end of the trench).
2. The final depth of the trench shall be measured and provided to the QC representatives and/or the QA Engineers for approval.
3. The trench shall be clean of debris and of all loose materials.
4. The slurry shall be placed by pumping immediately after the trench excavation is approved by the QC representatives and/or the QA engineers.

4.4.3 Drilling and CB backfilling for secant piles

1. The secant piles shall be installed in a carefully controled manner to ensure that the required pile location tolerances and vertical alignment requirements are achieved.
2. The secant pile diameter and spacing shall be selected to achieve a continuous wall while taking into account the required tolerances for pile location and vertical alignment (Section 4.4.4). The Contractor shall submit the selected pile diameter and spacing to the Owner and Designer for approval.
3. Before advance of the casings for the secant piles, the verticality and location of the casings shall be checked to ensure that they meet the specified tolerances. The Contractor shall have all of the required instruments to carry out these measurements. During the advancement of the casings, the Contractor shall make frequent measurements of verticality of the casings.
4. Casings for the secant piles shall be advanced through the fine filter zone to bedrock. During the advancement and cleaning of the casings through the saturated zone of the fine filter (which occurs below lake level), the Contractor shall take measures to prevent piping of the fine filter into the casings as directed by the QC representatives and/or the QA Engineers. These measures could include maintaining a water head in the casings equivalent to lake level or maintaining a fine filter plug at the bottom of the casings of sufficient length to prevent piping until the bedrock surface is reached. When the casings reach bedrock, they shall be advanced so that the casing is seated in bedrock. Total length of the casing and the encountered bedrock depth shall be written in the daily report of the driller.
5. After seating of the casing in the bedrock and cleaning out of the fine filter material, a one meter long socket shall be formed in the bedrock and a final cleaning shall be carried out.

6. The CB slurry shall be placed into each pile by tremie. Volume of slurry for every pile shall be monitored, as well as the date, the slurry sequential number to link with the appropriate test, the settlement of slurry below platform level and other informative data.
7. The secondary piles shall not be bored into the primary piles unless the CB reaches the specified minimum early strength of 50 kPa UCS. The strength testing shall be carried out by the Contractor.
8. The Contractor shall submit a detailed description of his proposed methodology for installation of the secant piles taking into account the factors listed above and other relevant issues.

4.4.4 Tolerances

1. The maximum permitted deviation of pile locations shall be 50 mm in any direction.
2. The maximum permitted deviation of the finished pile from the vertical at any level is 0.75 %. The Contractor shall demonstrate to the satisfaction of the Owner`s Engineer that the pile verticality is within the allowable tolerance.
3. If tolerances are exceeded, the Contractor shall submit a remedial proposal, for review and approval by the QC representatives and/or the QA Engineers, describing the method proposed to rectify the problem.

5.0 DRILLING AND GROUTING

5.1 General

5.1.1 Scope of work

The Work described in this Section includes the supply of all labour, equipment and materials required for the execution of drilling and grouting of the grout curtain at the Whale Tail Dike. The Drilling and Grouting consists of all works required to seal fractures in the bedrock foundation by drilling and grouting holes from the crest of the dike as shown on the drawings and as specified herein or as required by the QC representative.

The Work includes but is not limited to the following:

- Rotary drilling (tricone) or core drilling (diamond drill) with casing installation thru the secant pile wall of the dike down to bedrock.
- Drilling in the underlying bedrock foundations.
- Hole washing.
- Supply and use of inclinometers to measure hole inclination.
- Execution of grout curtain in bedrock foundations.
- Supply of cement, water and admixtures for grout.
- Water pressure testing.
- Backfilling and finishing of drilled holes and casings.

The drilling and grouting program shown on the drawings and this specification is tentative only, subject to changes following results of primary grout holes.

5.1.2 Extent of work

1. Drilling with and without casing, hole cleaning, water pressure testing, pressure grouting, hole backfilling as well as all works specified herein will be required along the section of the Whale Tail Dike where the foundation rock temperature is above freezing, as shown on the drawings and as specified by the QC representative during the course of the Work.
2. Cored (diamond drill) and rotary/percussion check holes in the bedrock foundations will be required at locations to be specified by the QC representative during the course of the Work.
3. The drilling and grouting works shall be executed from the crest of the dike after completion of the secant pile wall and will require installation of casing thru the secant piles down to the bedrock.

5.1.3 Definitions

1. Stable Grout: grout with less than 5 % bleed after 2 hours.
2. Curtain Grouting: consists of one or more lines of holes drilled and grouted at variable depth and spacing to produce an impervious curtain in the structure's foundation.
3. Split-spacing Method: means a procedure of progressively completing a grout curtain by drilling and grouting additional holes located midway between holes which have previously been drilled and grouted.
4. Stage Grouting: means the grouting of a hole by stages either from top to bottom (downstage method) or from the bottom upwards (upstage method). In general the upstage method will be used when bedrock conditions permit.
5. Stage: denotes a section of a hole in which grouting is being carried out. Stage length is in general 5 m. Shorter or longer stages may be directed where drilling water is lost, in cases of artesian flow from the hole, collapse of the hole or for any other reason determined by the QC representative.
6. Successful Connection: defines all the operations necessary to install a packer assembly which can resist the specified grouting or water testing pressure without leaks and without loss of pressure during grouting or the execution of a water pressure test.
7. Grouting Pressure: means the grout pressure measured at the collar of the hole while grouting the hole.
8. Effective Pressure: means the grout pressure effective at the packer elevation while grouting a stage in a hole.
9. Thermistor: instrument used to monitor the temperature of the rock foundation before grouting.
10. Frozen rock: rock which has a temperature of 0ºC or less, as measured with thermistors.

5.1.4 Submittals

At least 30 days prior to the start of the drilling and grouting works, the Contractor shall submit the following for the Designer Engineer's Approval:

1. Manufacturer's specifications and material safety data sheets of the proposed products.
2. A grout methodology statement including details of drilling, water testing and grouting procedures, the proportions of the proposed grout mix, the type of admixtures proposed and list of all equipment to be used in water pressure testing, batching, transporting and pumping of the grout, inclusive of proposed standby equipment.
3. Proposed Work schedule detailing the number of shifts and size of crews.
4. Test results of the proposed grout.

5. Specifications and photographs of mixing, pumping and monitoring units.
6. Characteristics and dimensions of the shelter proposed to perform grouting works, including heating systems.
7. All equipment calibration certificates shall be provided prior to the start of the Work. All equipment shall be calibrated based on the manufacturer’s recommendations or as specified herein or as required by the QC representative.
8. The Contractor shall submit progress records documentation on a daily basis to the Owner.
9. The Contractor shall submit as-built grouting drawings after the completion of the Work to the Owner.

5.1.5 Qualifications

The Contractor shall maintain on site a grouting specialist-field supervisor to monitor, supervise and direct all phases of grouting. The responsibilities of the specialist- supervisor shall be:

1. To supervise and direct the personnel carrying out the drilling and grouting Work.
2. To obtain and maintain the supply of grouting material such as to ensure continuous grouting operations.
3. To ensure that all personnel adhere to all safety and environmental requirements, and to ensure that all equipment and documentation for these requirements are in place before the start of the Work.
4. To obtain and maintain in good working order all equipment used for drilling, cement mixing, water pressure testing and grouting, and to ensure the equipment is of sufficient capacity for the Work to be performed.
5. To ensure that drilling, water pressure testing, cement mixing, carrying and injecting the grout are carried out properly in accordance with the specification and the approved procedures.
6. To approve the detailed working procedures and to approve the equipment and materials brought on to site.
7. To complete all quality control documentation and progress records required and submit this documentation on a daily basis.

5.2 Materials for grouting

5.2.1 Grout Mix

The grout mix for the grouting Work specified herein is a stable grout mix consisting of cement, water and admixtures. The grout mix shall be developed by the Contractor based on the characteristics specified in this Section, which will be confirmed or modified according to the field tests results. The grout mix design shall be subject to the QA Engineer's Approval.

The Contractor may use bentonite in the grout mix if deemed required to achieve a more viscous and cohesive grout.

Generally, only two stable grout mixes shall be used for all the grouting work, a low mobility grout and a high mobility grout, unless otherwise directed by the QC representative and required by actual rock conditions.

5.2.2 Mixing water

Mixing water for grout shall be fresh, clean and free from injurious amount of oil, acid, alkali, salts, organic matter and other deleterious substances, all in conformity with the requirements of CSA-A23.1 and CSA-A23.2. Mixing water shall be at a temperature of not less than 10^{o}C and not more than 25^{o}C.

5.2.3 Cement

Cement shall be Portland type HE (type 30) and shall comply with CSA-A3001. Cement shall be obtained from one manufacturing source. Partially hydrated cement shall not be used in grout.

5.2.4 Admixtures and bentonite

Admixtures such as superplasticizers, Viscosity Modifying Admixtures and accelerating agents shall be required. All admixtures shall comply with ASTM C494. All admixtures shall be from the same supplier to avoid incompatibilities and they shall be used according to the manufacturer's recommendations. The Contractor shall submit admixture datasheets to the Owner for Approval. Bentonite shall be finely ground (less than 200-mesh), premium grade sodium montmorillonite.

5.3 Handling, storage and disposal of grouting materials

Grout materials shall be stored and handled as recommended by the manufacturer and in accordance with all regulations, codes and ordinances.

Spilled, spoiled or opened, as well as unused materials shall be disposed of in accordance with all regulations in an area designated by the Owner.

5.4 Equipment

All drilling and grouting equipment supplied and used in the Work shall be of a type, efficiency and mechanical condition suitable for executing adequately the drilling, water pressure testing, and grouting operations and shall be subject to the Designer's Approval.

5.4.1 Drilling equipment

1. The holes shall be drilled from the crest of the dike along the center line of the secant pile wall. The section of holes thru the secant piles shall be executed by means of rotary drilling (tricone) or core drilling equipment using only water for flushing the hole. Near perfect hole verticality is required thru the secant pile wall. The section of holes in the bedrock foundation shall be executed by drilling equipment using only water for flushing the hole. Use of air to remove cuttings from a hole shall not be permitted, unless authorized by the Owner. Use of airlift technique shall be required to remove water and unset grout from holes when instructed by the QC representative.
2. All drilling equipment shall produce holes free of irregularities to avoid leakage of water or grout around the packers.
3. Inclinometers shall be used to measure the inclination of holes; the holes to measure will be designated by the QC representative. Deviations from vertical in the secant piles section of the holes shall not exceed 0,5 % of drilled length. Deviations from vertical in the bedrock section of the holes shall not exceed 5 % of drilled length.
4. Grout holes in bedrock foundations shall have a minimum diameter of 50 mm and maximum length of 20 m.
5. Cored (diamond drill) investigation holes in the bedrock foundation shall be carried out using NQ double-tube core barrels (48 mm diameter core size) or larger.
6. Steel casings shall be installed while drilling thru the secant piles to bedrock foundation. The steel casings will be left in place and backfilled with casing grout.

5.4.2 Water pressure testing equipment

1. Water pressure testing shall be carried out with dedicated equipment.
2. Water pressure testing equipment shall be capable of delivering 200 liters per minute at a maximum pressure of 8 bars.

5.4.3 Grouting Equipment

1. The grouting equipment shall be capable of adequately supplying, mixing, pumping and injecting grout mixes as specified herein or as required by the QC representative.

2. The equipment shall be of adequate size and capable of supplying an uninterrupted flow of grout at the rate of 20 litres per minute at the maximum pressure of 20 bars measured at the collar of the grout hole.
3. The pumps shall be of the variable speed, progressing cavity (Mono or Moyno) type and shall be able to regulate pressure and flow rate with precision from zero to the maximum allowed.
4. The grout-mixing unit shall be capable of obtaining the required design grout mixes as determined from the test results. The grout-mixing unit shall be capable of reproducing the required grout mixes with an accuracy of 2% of the specified design mixes. The mixer shall be high speed, high-shear with rotation speed of between 1200 and 1500 RPM, allowing colloidal suspension of the cement particles. The grout-mixing unit shall include graduated cylinders and a digital scale of ±0,1 g accuracy. The required capacity of the grout-mixing unit is 200 litres of grout per batch.
5. Retention tanks shall have a minimum capacity of 500 litres, shall be mechanically operated with paddles rotating at about 100 RPM and shall be designed to keep the mixed grout agitated and in suspension. Holding tanks shall be provided with 2,5 mm sieve to screen solids in the grout return line.
6. Hoses and supply lines used for water pressure testing and grouting shall be rated to 25 bars safe working pressure and shall have a minimum inside diameter of 25 mm. Fittings and connections shall be rated to 25 bars safe working pressure and shall include safety chains.
7. Packers shall be of pneumatic type, capable of sealing the drill hole in bedrock as well as the lower end of the casing, and capable of withstanding the maximum grouting pressure prescribed without leakage. Both single and double packers are required. The inside diameter of the tubes carrying the grout mix shall not be less than 19 mm. Double packers shall be used for water pressure testing and shall be capable of isolating 2 m to 5 m long sections of the grout hole, without leakage. The Contractor shall maintain on site, at all times, a sufficient quantity and variety of packers to carry out grouting and water pressure testing.
8. Flow meters in the water supply lines shall be graduated in litres and 0.1 litres, without bypass, in order to measure the water volume in the mixer accurately. The flow meter shall be capable of measuring flows as low as 0.3 liter per minute with a precision of ±0.1 litre per minute. The flow meter capacity shall be 100 litres per minute.
9. Pressure gauges shall be Bourdon type, available in different pressure ranges including 0 to 5 bars, 0 to 10 bars and 0 to 20 bars. Pressure gauges shall be protected from grout using gauge savers. An adequate number of spare pressure gauges shall be available on site. A pressure gauge shall not be used for more than two shifts after which it shall be cleaned and calibrated using a reference gauge supplied by the Contractor. All pressure gauges shall be numbered for identification. All pressure gauges shall have a minimum

window diameter of 75 mm. Pressure gauges shall be installed at each pump discharge and at each hole collar.

10. Water heaters shall be used to heat mix water.
11. Thermometers shall be permanently installed in the tanks and mixers. Other thermometers shall be provided to measure temperature of the return wash water.

5.4.4 Data Recording System

The Contractor shall provide an automated data recording system capable of continuous recording of pressure, flow rate and total volume injected for each stage. The recorded data shall be shown on the monitor screen and printed as a grouting curve representing pressure (Y axis) versus volume (X axis). The monitor screen shall be visible to personnel located in the grouted hole collar and pressure control valve area. The equipment shall be able to include the pressure of the grout column above the stage grouted in order to display the effective pressure applied at the stage elevation. The equipment shall allow the QA and QC representatives to access all data by connection to a laptop computer.

5.4.5 Arrangement and operation of grouting equipment

The grouting equipment shall be arranged to provide continuous grout circulation throughout the circuit and permit accurate pressure control by operation of valves.

Plugging of the equipment and lines shall be prevented by maintaining a continuous flow of grout and by periodic flushing with water.

5.4.6 Contractor's site Laboratory

The Contractor shall have on site a laboratory to undertake necessary testing. Equipment essential to undertake routine testing and to design grout mixes includes (the list is not exhaustive): mud balance, Marsh cones, cube moulds, cylindrical moulds, set of sieves, necessary glass hardware (test tubes, thermometers, beakers).

5.5 Execution

5.5.1 Drilling

1. Grout holes shall be drilled along the center line of the secant pile wall, as shown on the drawings or as required by the QC representative. The set out of hole locations shall be established by survey tied to the project control points and approved by the QC representative prior to drilling. The split-spacing method of grouting shall be used; primary holes shall be drilled and grouted prior to drilling secondary holes and secondary holes shall be drilled and grouted prior to tertiary holes.

2. The drilling method in the secant piles section of the holes shall be rotary (tricone) or cored (diamond drill) with water flushing and installation of casing; the casing must be socketed a depth of 200 mm minimum in bedrock. The drilling method in the bedrock section of the holes shall be rotary/percussion or cored with water flushing. The minimum diameter of grout holes in the bedrock section shall be 50 mm.
3. The maximum permitted inaccuracy of grout hole casing positioning is 50 mm in any direction.
4. The alignment of drill hole casings and drill holes shall be verified at all times. Maximum deviation of a drill hole casing shall not exceed 0.75% of its drilled length. Maximum deviation of a drill hole shall not exceed 5% of its drilled length in the bedrock section.
5. Drilling of all holes shall be carried-out using clean water as the drilling fluid. The use of grease or other lubricants on the drill rods used is not permitted.
6. Holes shall not be drilled within 6m of a hole grouted in the preceding 24 hours.
7. On completion of drilling and immediately before grouting, each grout hole shall be flushed with clean water injected at the bottom of the hole for a minimum period of 5 minutes or for such time as is required to clean the hole. Wash water velocity shall be sufficient to raise drill cuttings to the surface. A hole shall be considered clean when the return wash water is clear. Washing shall be interrupted whenever there is evidence that, due to poor rock quality, washing is eroding the hole walls.
8. Water used for washing and water pressure testing shall be at a temperature of not less than 7ºC and not more than 25ºC.
9. Foundation conditions may require special washing procedures in order to remove loose deposits in larger fissures intercepted by the grout hole. When instructed by the QC representative, this may require the drilling in advance of adjacent holes in order to perform normal washing as well as higher pressure air and water washing with the aim of ejecting loose materials prior to grouting.
10. If caving occurs during the drilling of a grout hole, the drilling operation shall be interrupted and the section of the hole where the caving occurred shall be washed and grouted. Drilling shall resume after initial set of the grout.
11. Each hole shall be protected from clogging or obstruction by means of a temporary cap or other suitable device at the collar. Any hole obstructed shall be cleaned out or grouted and another hole shall be drilled by the Contractor.

5.5.2 Water pressure testing

1. Water pressure tests shall be conducted before and after grouting with the objective of evaluating the change of hydraulic conductivity of the foundation bedrock.

2. Water pressure tests shall be done with a single packer or with a double packer arrangement to isolate maximum 5 m long sections in the hole.
3. Water pressure tests shall not be performed within 12 m of a hole grouted within the preceding 24 hours.
4. The Lugeon procedure shall be used. The flow rate shall be measured at five pressure steps for a period of 5 minutes each. The pressure to apply for each step shall be:
    - 1st step: 1/3 of maximum pressure.
    - 2nd step: 2/3 of maximum pressure.
    - 3rd step: maximum pressure.
    - 4th step: 2/3 of maximum pressure.
    - 5th step: 1/3 of maximum pressure.
5. In case of zero absorption at step 3, the 4th and 5th steps shall be cancelled.
6. The maximum water pressure for each test shall be the maximum pressure defined for the grouting.
7. A water pressure test report shall be provided to the QC representative at the end of each test.
8. Water pressure tests shall be performed independently of grouting operations in holes chosen by the QC representative.

5.5.3 Grout mix design

1. The grout mix used to backfill casings shall have a W/C (Water/Cement) ratio by weight of 2 to 1 with 7.5% bentonite (% of cement weight). 7 day UCS value of 1 MPa. Cement shall be Portland (HE) Type 30.
2. Two grout mixes, both with identical WC ratios but with differing amounts of additives, shall be developed. The aim is to obtain 2 grout types with differing viscosity and mobility.
3. The grout mix used for curtain grouting shall be typically composed of the following materials:
    - Water.
    - Portland Cement (HE) Type 30.
    - Superplasticizer: percentage by weight of cement as specified by the manufacturer.
    - Viscosity modifying agent, accelerating agent, bentonite and other additives, if required: percentage by weight of cement as specified by the manufacturer.

- W/C (Water/Cement) ratio by weight: 0.6 to 0.8/1 with target of 0.7, as determined by results of trial mixing at site and quality control testing of mix viscosity and other parameters.

5.5.4 Field acceptance criteria

Prior to commencing grouting operations, a Field Acceptance Trial involving the actual equipment, materials and procedures shall be conducted to verify their adequacy. The grouting specialist-supervisor shall be on site for the Field Acceptance Trial. The Field Acceptance Trial shall cover the following:

1. Calibration, verification of gauges, scales, water meters and grouting monitoring/ data recording equipment.
2. Calibrating procedure for the automated monitoring/data recording system pressure transducer and flow meter.
3. Batching, mixing and pumping of a trial mix through the circulating line system.
4. Quality control tests and grouting records: parameters such as density, viscosity, cohesion, bleeding and initial setting time shall be measured and recorded for each trial mix.

5.5.5 Grouting operation

1. Grouting operations shall be done from the dike crest and shall consist of creating, in the foundation bedrock under the secant pile wall, a single row grout curtain. Grouting shall be done using the split-spacing method.
2. Grouting operations shall be performed based on measured ground reaction to injection; continuous digital recording of grouting pressures, flow rates and injected volumes are required.
3. After washing a hole, it shall be grouted using the upstage grouting method with 5 m stages at the specified pressures for each depth. Shorter stages may be required by the QC representative according to the bedrock condition.
4. For the uppermost grouting stage of each hole, the packer shall be inflated at the lower extremity of the casing in order to ensure grouting of the topmost section of bedrock near the secant pile-bedrock contact.
5. No grouting shall take place if the temperature of the rock to be grouted is below 0°C.
6. The temperature of the grout at any point shall be between 5°C and 20°C.

7. Grouting of any stage shall continue uninterrupted until refusal occurs or the instructed volume limit for that stage is reached, unless otherwise instructed by the QC representative.
8. Any loss of pressure or sudden increase in grout take shall be immediately reported to the QC representative.
9. The maximum amount of grout per meter of stage length shall be 500 liters unless otherwise instructed by the QC representative.
10. Secondary and higher order grout holes shall be required on either side of an existing grout hole if the grout take of any stage in that hole is higher than 200 liters per meter.
11. The split-spacing method shall be used for curtain grouting; primary holes shall be drilled and grouted prior to drilling the secondary holes. Secondary holes shall be drilled and grouted prior to drilling the tertiary holes, and so on for quaternary holes. Primary, secondary and tertiary holes are mandatory; quaternary holes are if required. The spacing of primary holes shall be 12 m.
12. Holes shall normally be grouted one at a time in a sequence to obtain the desired consolidation or control of seepage. If during the grouting, grout is found to flow from another hole or holes adjacent to the one being grouted, the QA and QC representative shall be immediately informed. Such hole or holes shall be grouted simultaneously with the original grout hole, unless otherwise instructed by the QC representative. The grouting of interconnecting holes shall be completed at the pressures specified for grouting of the various depths of these holes.
13. A grout batch which has been in circulation for over 2 hours shall be disposed of.
14. High Grout Take

    If high grout take is observed without noticeable pressure increase, the Contractor may, with the QC representative's consent:

    - Use the low mobility grout.
    - Add accelerating agent and or viscosity modifying agent to the mix.
    - Eliminate the superplasticizer.
    - Reduce the grout flow rate.
    - Discontinue the grouting momentarily.

    If none of the above procedures permits reaching the specified pressure, grouting of the hole will be discontinued. After the grout has set, the hole shall be redrilled, and grouting resumed. Additional holes in the immediate vicinity of the hole showing the abnormal high absorption may be required.
15. Grouting Pressures

- Grouting pressures shall be applied gradually. The Contractor shall install a control device to limit the grout pressure. The control device shall stop the pump automatically when the specified pressure for the stage being grouted is reached in order to avoid hydrojacking.
- Initial specified pressures at the mid-point of a stage shall be 0.2 bar per meter measured vertically from the working platform to the top of bedrock, plus 0.5 bar per meter from the bedrock surface. The minimum pressure at mid-stage shall be 1 bar above the hydrostatic head from the water level, and the maximum shall be 15 bars. In general, the pressure shall be as high as possible but compatible with security against uplift and the mechanical resistance of the rock.

16. Refusal Criteria

- When the maximum pressure has been reached, the maximum pressure shall be maintained constantly for 10 minutes. Grouting of the stage is then considered complete if the flow to maintain this pressure is less than 5 litres per minute per 5 m of stage length, measured over a 10 minute period.
- After the grouting of a hole is complete, the pressure shall be maintained by suitable valves until the grout pressure has decreased to zero or the grout has set sufficiently so that it will be retained in the hole

17. Hole Backfilling

- After refusal is met for all stages of a hole, the hole shall be backfilled from the bottom up with the specified casing grout, using a pipe lowered to the bottom of the hole.

18. Drill Check holes

- Rotary/percussion drill holes or cored drilling (diamond drill) may be required by the QC representative in specific areas. It shall be carried out using holes with different directions and orientations compared to the grout holes. These holes will be subject to the water pressure testing and grouting as directed by the QC representative.
- Following the results of the check holes, either additional check holes or additional grout holes may be required by the QC representative.

19. Quality Control Testing

At least twice a day, the Contractor shall take grout samples at the location of the hole to verify if it conforms to the requirements of this Section.

During the grouting operations, testing of the grout shall be conducted by:

- Baroid mud scale to verify specific-gravity/unit weight;
- Marsh flow cone to verify the viscosity;
- Grout temperature readings at the holding tank;

- Grout bleeding after 2 hours;
- Cohesion measurement using the Lombardi plate method;
- Initial setting time.

Accurate quality control records shall be kept.

At least once a week, or as required by the QC representative, two sets of brass cube moulds (3 cubes/mould) shall be supplied for the purpose of casting grout cubes from any batch requested by the QC representative.

Results of the cube tests shall be used as a guide to benchmark the quality of the grout material batched.

5.5.6 Records

During the progress of the Work, the Contractor shall maintain on site at all times a complete set of documents indicating clearly and accurately, as the Work progresses, all changes, revisions and additions to the Work.

6.0 QUALITY CONTROL AND QUALITY ASSURANCE PROGRAM

1. A QA / QC approval form is presented in the Appendix 3 and the tasks for QA / QC are presented in Table 2.
2. The Contractor shall be entitled to be represented during all field tests carried out by the QC representative in order to determine whether fill materials meet the requirements of the specifications.
3. The QC representatives will notify the Contractor of any test but the QC representatives shall not be required to wait for the arrival of the Contractor prior to the start of the test.
4. The Contractor shall provide assistance when required for collecting and handling the samples.
5. Sampling or testing required by the QC representatives shall be executed by the Contractor without delay. All samples and tests shall be taken or performed in accordance with the appropriate standard, approved by the QC representative, and shall meet the requirements of the present document.
6. Visual inspections of excavation and sources of fills will be carried out by the QC representatives on a regular basis to ensure that the excavation work and fill materials meet the requirements of the document.

APPENDIX 2

APPENDIX 2

SUMMARY OF QUALITY CONTROL AND ASSURANCE PROCEDURES DURING WORK

ITEMS	ELEMENT	RESPONSABILITIES OF THE QA REPRESENTATIVES			RESPONSABILITIES OF THE QC REPRESENTATIVES		
		TASK	FREQUENCY	FORM TO FILL	TASK	FREQUENCY	FORM TO FILL
Survey	General : Section 2.8	• Survey validation. • Verification of the lifts thickness. • Visual inspection and assessment of grade limits.	Periodically	Daily report.	• Ensure that the surveyors use the updated information and have a good understanding of the Project and meet the requirements. • Survey all the works. • Implementation of works and verification of lines and dimensions. • Use initial and final grade limits for quantity calculations.	Continuously	Raw file (txt) and processed file (dwg). Daily report.
Borrow sources	Earthworks : Section 3.6	• Verifiy area has good drainage. • Use good material and optimize quantities.	Daily	Daily report.	• Establish a dewatering system, if required • Ensure that the material is appropriate (gradation) • Minimize losses and segregations • Manage excavations • Make sure to keep safe slopes • Coordination to change the borrow source if required, adapt the exploitation by sorting out boulders with the AEM operator.	Continuously 1 sample / shift (min)	Daily report Gradation Report
Stockpiles	Earthworks : Section 3.5	• Verify that stockpile configuration minimizes segregations and facilitates good drainage.	Daily	Daily report.	• Ensure that the drainage is planned and maintained • Plan a stockpile configuration that minimizes segregation	Continuously 1 sample / shift (min)	Daily report Gradation Report
Storage	Instrumentation / Cement/ Bentonite/additives	Visual inspection	Weekly	Daily report	• Visual inspection • Storage in conformity with the manufacturer's specifications • Count the quantity of cement / bentonite used on a daily basis • Count the complete stock inventory on a weekly basis	Weekly Weekly Daily Weekly	Daily report
Drill & Blast	Overburden / Bedrock: Section 3.4	Verify that drill and blast limits will satisfy the specifications. Confirm that there is no ice-rich till left in place	Once / drill pattern	Daily report	• Follow the driller and adapt the pattern (including limits and depth if required) based on the depth of bedrock. • Monitor blast set-up and readings.	Once / drill pattern	Daily report
Foundation approval (footprint)	Snow / Ice / Boulder: Section 3.4	Visual inspection for foundation approval Confirm that there is no ice-rich till left in place	Periodically prior to filling of footprint	QA/QC approbation form	• Visual inspection to detect any unsuitable material and coordination with Contractor to ensure specifications are met. • Manage the clearing limits with the surveyor	Continuously	QA/QC approbation form
Foundation approval (Key trench)	Bedrock / Ice poor Till: Section 3.4	Visual inspection for foundation approval and verify the excavation limit with the surveyor. Compare with bathymetry and expected limits.	When required and prior to fill placement	QA/QC approbation form	Visual inspection to detect any unsuitable material / unfrozen water. Coordination with the Contractor to remove all undesirable material. Verify the excavation limits with the surveyor and bathymetry.	Continuously	QA/QC approbation form

SNC·LAVALIN

ITEMS	ELEMENT	RESPONSABILITIES OF THE QA REPRESENTATIVES			RESPONSABILITIES OF THE QC REPRESENTATIVES		
		TASK	FREQUENCY	FORM TO FILL	TASK	FREQUENCY	FORM TO FILL
Placement of material	Fine filter: Section 3.7.3	• Verify and approve the lab compilation file. • Verify technique of placement. • Approve test pad procedures and results. • Take samples of in-place material.	Daily Daily Once 1 sample / day	• Daily Report • Daily Report • QA/QC approbation • Lab compilation files and test reports.	• Realize the standard Proctor test on fine filter • Visual inspection and testing: • Grain Size distribution • In situ density of lifts	Grain size distribution: 1 / 1 000 m³ Density : 1 / 100 m³	• Daily report • Lab compilation files and test reports.
		• Review and approve the method developed by the Contractor for the dynamic compaction (controlled by energy). • Review energy deployed during compaction.	Once Daily	• Approval form • Daily report • Test compilation file	• Drop weight of the tamper as required to achieve sufficient compaction (dynamic compaction) • Perform the number of passes required to achieve sufficient compaction (roller compaction) • Visual inspection of each lift – ensure specifications are met.	Continuously	Daily report QA / QC approval form.
	Coarse filter: Section 3.7.4	• Verify and approve the lab compilation file. • Take samples of in-place material.	Periodically 2 samples	• Approval form. • Daily report • Lab compilation files and test reports.	Grain Size distribution	1 / 5 000 m³	• Daily report / QA / QC approval form. • Lab compilation file and test reports.
		Placement control on site	Periodically	• Daily report • Approval form	Perform the number of passes required to achieve sufficient compaction.	Continuously	Daily report / QA / QC approval form.
	Rockfill: Section 3.7.3	Placement control on site	Periodically	• Daily report • Approval form	Perform the number of passes required to achieve sufficient compaction.	Continuously	Daily report / QA / QC approval form.
CB Mix: Section 4.3 and 5.2	Bentonite Certification	Check bentonite powder certification (API 13A)	1 per shipment	• Daily report • Approval form	•		•
	Cement Certification	Check cement powder certification (CSA-A3001-13)	1 per shipment	• Daily report • Approval form	•		•
	Water	Check water quality	Periodically	• Daily report • Approval form	• Temperature of water	One test per mix	• Daily report / QA / QC approval form. • Lab compilation files and test reports.
	Bentonite	• Verify and approve the lab compilation file. • Placement control on site Bentonite content (from batch records)	Periodically	• Daily report • Approval form	Testing: • Density by mud balance • Marsh cone viscosity (spelling error) •	One test per mix	• Daily report / QA / QC approval form. • Lab compilation files and test reports.
	CB slurry	• Approve the depth of casings and verify cuttings • Take in-place samples to make external lab testing • Cement content (from batch records)	Periodically	• Daily report • Approval form	Perform mixes with adequate mix ratio. Tests from the mixing plant • Density by mud balance. • Marsh cone viscosity (spelling error) • Bleeding • Temperature of slurry	• Five tests per shift for the slurry from the supply conduit and two tests per shift for the slurry from the trench. • Strength tests: daily for 7 days, then	• Daily report / QA / QC approval form. • Lab compilation files and test reports.

SNC·LAVALIN

ITEMS	ELEMENT	RESPONSABILITIES OF THE QA REPRESENTATIVES			RESPONSABILITIES OF THE QC REPRESENTATIVES		
		TASK	FREQUENCY	FORM TO FILL	TASK	FREQUENCY	FORM TO FILL
					• Vane tests from the secant pile Molded samples of the CB slurry from the trench shall be prepared and properly cured and stored for: • UCS Tests • Permeability Tests	weekly thereafter or more frequently depending on test results. 12 molds for every 100 m	
CB cutoff wall: Section 4.0	Pile casing	Placement control on site	Periodically	• Daily report • Approval form	• Inclination to be verified • Implementation and survey of piles	Continuously (every pile)	• Daily report / QA / QC approval form.
	Loss of slurry	Placement control on site	continuously	• Daily report • Approval form	• Check top level of the cutoff wall, compare volume of slurry added to the pile with theoretical pile volume	Continuously (every pile)	• Daily report / QA / QC approval form.
	Cracking of top of wall	Placement control on site	Periodically	• Daily report • Approval form	• Check top of cutoff wall for cracks	Periodically (every pile)	• Daily report / QA / QC approval form.
	Key-in depth into bedrock	Placement control on site	Periodically	• Daily report • Approval form	• Check elevation for each pile	Continuously (every pile)	• Daily report / QA / QC approval form.
	Keying between primary and secondary piles	Placement control on site	Periodically	• Daily report • Approval form	• Check overlap between primary and secondary piles, or check spacing between primary piles	Continuously (every pile)	• Daily report / QA / QC approval form.
	Cleaning of trench and pile bottom	Placement control on site	Periodically	• Daily report • Approval form	• Check the washing fluid clarity prior placing CB mix.	Continuously (every pile)	• Daily report / QA / QC approval form.

Appendix C

Thaw settlement analysis

Title of document: **THAW SETTLEMENT ANALYSES AT WHALE TAIL DIKE**

Client: **AGNICO EAGLE MINES LIMITED**

Project: **AMARUQ WHALE TAIL DIKE DETAILED ENGINEERING**

Prepared by: Mathieu Durand-Jézéquel, Jr. Eng., M. Sc.

Reviewed by: Getahun Haile, P.Eng., M.Sc.A.

Approved by: Yohan Jalbert, P.Eng.

REVISION INDEX

Revision				Pages Revised	Remarks
#	Prep.	App.	Date		
PA	M. D.-J.		2018-04-06		
PB	M. D.-J.	GH	2018-04-29	All	Issued for Client comments

NOTICE TO READER

This document contains the expression of the professional opinion of SNC-Lavalin Inc. ("SNC-Lavalin") as to the matters set out herein, using its professional judgment and reasonable care. It is to be read in the context of the agreement dated October 4th, 2017 (the "Agreement") between SNC-Lavalin and Agnico Eagle Mines Limited (the "Client") and the methodology, procedures and techniques used, SNC-Lavalin's assumptions, and the circumstances and constraints under which its mandate was performed. This document is written solely for the purpose stated in the Agreement, and for the sole and exclusive benefit of the Client, whose remedies are limited to those set out in the Agreement. This document is meant to be read as a whole, and sections or parts thereof should thus not be read or relied upon out of context.

SNC-Lavalin has, in preparing estimates, as the case may be, followed accepted methodology and procedures, and exercised due care consistent with the intended level of accuracy, using its professional judgment and reasonable care, and is thus of the opinion that there is a high probability that actual values will be consistent with the estimate(s). Unless expressly stated otherwise, assumptions, data and information supplied by, or gathered from other sources (including the Client, other consultants, testing laboratories and equipment suppliers, etc.) upon which SNC-Lavalin's opinion as set out herein are based have not been verified by SNC-Lavalin; SNC-Lavalin makes no representation as to its accuracy and disclaims all liability with respect thereto.

To the extent permitted by law, SNC-Lavalin disclaims any liability to the Client and to third parties in respect of the publication, reference, quoting, or distribution of this report or any of its contents to and reliance thereon by any third party.

TABLE OF CONTENTS

List of figures

List of tables

List of Appendix

1.0 Introduction

1.1 Context

Agnico Eagle Mines Limited, Meadowbank Division ("AEM") is at a detailed design stage to develop the 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 (Figure 1-1). The property was acquired by Agnico Eagle Mines Limited in April 2013 subject to a mineral exploration agreement with Nunavut Tunngavik Incorporated.

Figure 1-1: Whale Tail Pit Mine Site General Location

1.2 Abbreviations

The abbreviations used in this technical note are as follows:

AEM = Agnico Eagle Mines Limited, Meadowbank Division

PFS = Prefeasibility Study

WTD = Whale Tail Dike

1.3 Previous Studies

In 2016 and 2017, SNC-Lavalin Inc. ("SNC-Lavalin") was retained by AEM to develop the permitting level engineering of the water management infrastructure. These documents were produced at a PFS engineering level. The documents produced are listed below:

- Factual report for 2016 field work (AEM, 2016)
- Permitting level engineering report for geotechnical and water management infrastructure (SNC-Lavalin, 2016a)
- Sensitivity analysis of seepage through the foundation of WTD (SNC-Lavalin, 2016b)
- Preliminary stability analysis of WTD (SNC-Lavalin, 2017d)
- Updated seepage analysis of the WTD (SNC-Lavalin, 2017f)
- Construction technique optimisation of WTD (SNC-Lavalin, 2017b)
- Preliminary thermal analysis of WTD during the mine operation (SNC-Lavalin, 2017e)
- 2017 geotechnical investigation at Amaruq (SNC-Lavalin, 2017a)
- Whale Tail Dike secant pile cut-off wall preliminary design (SNC-Lavalin, 2017g)
- Preliminary design of Mammoth Dike (SNC-Lavalin, 2017c).

Following the permitting level engineering, SNC-Lavalin was retained in September 2017 to develop the detailed engineering of the water management infrastructure. This document is one of a series of deliverables as part of the detailed engineering. It includes a literature review on thaw consolidation and a discussion on the state of the art in the assessment of thaw consolidation settlements.

1.4 Scope of Work

The main objective of this study was to assess the effects of thaw settlement on the water management infrastructure, the most important of which is the WTD. More specifically, the potential degradation of the permafrost under the abutments of the WTD due to the projected rise of the lake level upstream of the dike was a concern since it is expected that a thaw front will penetrate the ground and melt the ice-rich material(s), potentially causing subsidence.

The scope of work includes the following items:

- A review of literature on the thaw consolidation phenomenon with a focus on practical cases in permafrost conditions;
- A review of all the parameters required for the assessment of geotechnical properties of thawed permafrost;
- Thermal analyses to determine the rate of thaw;
- Calculation of settlements over time.

2.0 Thaw Consolidation Process

2.1 Introduction

According to van Everdingen (2005), thaw consolidation is a "time-dependent compression resulting from thawing of frozen ground and subsequent draining of excess water". In permafrost regions, thaw processes are initiated as a result of temperature changes at the surface, which can be due to:

- Seasonal variations;
- Uninsulated space heating;
- Surface flooding by warmer water (Panday & Corapcioglu, 1995).

According to Leroueil *et al*. (1991), such thermal changes can be attributed to pipeline construction (Watson, Rowley, & Slusarchuk, 1973; McRoberts, Law, & Moniz, 1978; Nelson, Luscher, Rooney, & Stramler, 1982), dike construction (Brown & Johnston, 1970; Keil, Neilsen, & Gupta, 1973), building construction (Gur'yanov, 1975) or excavation work (Tremblay & Doré, 1988). Thawing is also associated with disturbance of vegetation or construction activity (Lesage & Wang, 2008). Degradation of the permafrost may be caused in several ways and may occur in a few days or over many years (Crory, 1973).

Ice in frozen ground is contained in several forms, ranging from individual particles spread into the soil matrix, to lenses of various thicknesses up to massive deposits. When ground ice melts, there is a change in void ratio due to the drainage of the unfrozen water. The volume reduction causes subsidence in the thawed zone of the permafrost, which is referred to as "thaw settlement". If thaw consolidation occurs in an ice-rich soil, thawing of ice rich soils (in particular fine-grained soils), can generated excess pore pressure (EPWP), which in turn lead to loss of strength and stiffness of the thawed zone (Saarelainen, 1999) as well as settlement as the EPWP dissipates. Brown & Johnston (1970) mention that prediction of thaw and settlement is important for two main reasons:

- The amount and rate of settlement, which depend on the depth and rate of thaw, are of interest in assessing the need for and the scheduling of future maintenance;
- The stability of the foundation, which is dependent on the rate at which thaw water is redistributed or escapes.

2.2 Theoretical Principles

Van Everdingen (2005), based on the work by Morgenstern & Nixon (1971) and Andersland & Anderson (1978), states that "if during thaw, the flow of water from the thawed ground is unimpeded, then the variation of thaw settlement with time is controlled solely by the position of the thawing front. If the thawed ground is not sufficiently permeable, and flow is impeded, however, the rate of settlement with time is also controlled by the compressibility and permeability of the thawed ground. In the case of thawing fine-grained soils, if the rate of thaw is sufficiently fast, water is released at a rate exceeding that at which it can flow from the soil, and pore pressures in excess of hydrostatic will be generated. These excess pore pressures may cause severe instability problems in slopes and foundation soils. It has been found that excess pore pressures and the degree of consolidation in thawing soils depend primarily on the thaw consolidation ratio".

Morgenstern & Nixon (1971) defined the thaw consolidation ratio R as

$$R = \frac{\alpha}{2\sqrt{c_v}}$$

where α is the constant relating depth of thaw to the square root of time as given by the Neumann solution (Carslaw & Jaeger, 1947), and c_v is the coefficient of consolidation. The theory assumes that the parameter α for the movement of a thaw plane downwards correctly describes the progress of the thaw front in permafrost (Carslaw & Jaeger, 1947; Morgenstern & Smith, 1973), that is

$$X = \alpha\sqrt{t}$$

where X denotes the depth of the thaw front, and t denotes time.

The stability of permafrost soils which are actively thawing is intimately related to the excess pore pressure maintained in the soil during thawing. When the thaw consolidation ratio R is large (i.e. greater than unity), severe excess pore water pressures are maintained in the thawing soil, and the settlements are considerably impeded. A large value of R indicates that the rate of thawing is large compared with the ability of the soil to discharge the excess melt water (Nixon, 1973). In other words, the ratio R is a measure of the balance between the rate of generation of excess pore fluids, and the ability of the soil to expel these fluids from the pore space (Nixon & Morgenstern, 1973).

If P_0 denotes the stress applied at the ground surface (the weight of a dike, for example), and X denotes the depth measured from the ground surface, two dimensionless parameters can be introduced: the normalized depth $z = x/X(t)$ and the excess pore pressure $u(z,t)/P_0$. Figure 2-1 shows that those two parameters are linked by the thaw consolidation ratio R.

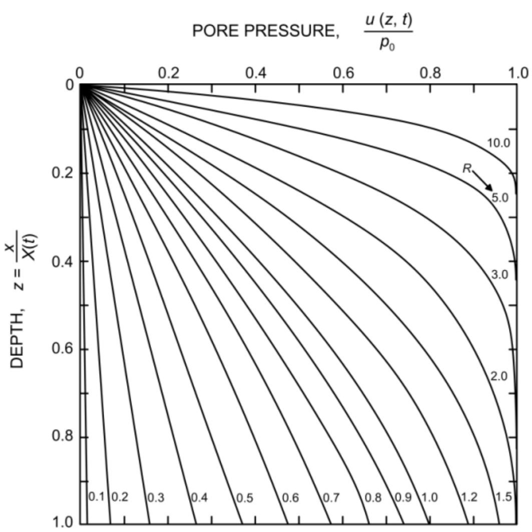


Figure 2-1: Excess pore pressures for a weightless material (after Morgenstern & Nixon (1971))

Figure 2-1 allows the assessment of the excess pore water pressure at any depth between the thaw front and the ground surface, knowing the thaw consolidation ratio of the soil. The higher the ratio R, the greater the pore pressure at a given depth. The higher the pore pressure, the lower the effective stress, which can lead to thawed soil instability. Once the excess pore pressures are known, the effective stress and therefore the available shearing resistance may be calculated (Nixon, 1973).

Once thaw consolidation is completed, the only settlement that occurs is due to gravity drainage according to the Terzaghi's one dimensional consolidation theory (Panday & Corapcioglu, 1995), which is:

$$\frac{\partial \varepsilon}{\partial t} = -\frac{k}{\gamma_w}\frac{\partial^2 u}{\partial z^2}$$

where the rate of settlement $\partial \varepsilon / \partial t$ is proportional to the distribution of pore pressures u in the soil at a depth z, k is the hydraulic conductivity and γ_w is the density of water. By defining the constrained modulus of elasticity M as the ratio of vertical effective stress σ_v' to laterally confined strain ε_v:

$$\partial \sigma_v' = M \partial \varepsilon_v,$$

the consolidation equation can be written as

$$\frac{\partial \sigma'}{\partial t} = -\frac{\partial u}{\partial t} = -\frac{kM}{\gamma_w}\frac{\partial^2 u}{\partial z^2}.$$

By assuming that k, M and gam are constant, the coefficient of consolidation c_v can be written as

$$c_v = \frac{kM}{\gamma_w}$$

or alternatively

$$c_v = \frac{k}{m_v \gamma_w}$$

where the coefficient of volume compressibility m_v is the reciprocal of the constrained modulus of elasticity of the unconsolidated porous medium in the inelastic range, that is

$$m_v = \frac{1}{M} = \frac{\Delta e}{\Delta \sigma (1 + e_0)}$$

where e_0 denotes the initial void ratio.

3.0 Litterature Review

It has been widely recognized that thaw settlement caused by permafrost degradation coupled with global warming and large scale engineering activities in cold regions is one of the main causes of damage to infrastructures (Cheng & Yang (2006); Qi *et al*., (2007); Wu *et al*., (2002; 2008)). Wang & Liu (2015) mention that thaw settlement is even a greater threat when ice-rich permafrost is involved. That is because a large amount of pore water is drained out of the soil with subsequent decrease in volume, thus causing large settlement (Yao, Qi, & Wu, 2012).

Qi *et al*. (2012) studied the thaw consolidation phenomenon of permafrost under a roadway embankment and found that the consolidation of the thawed permafrost layers are controlled by several factors, including the load, the characteristics of the drainage medium and time. Wang *et al*. (2016) mention that in China for instance, on the Qinghai–Tibet highway, embankment thaw settlement usually occurs in the warm season and ceases in the cold season (Liu *et al*., (2002); Qi *et al*., (2007)), which implies that thaw settlement does not take place over the whole year and occurs periodically as pavement temperature rises above 0 °C. Moreover, Qi *et al*. (2012) mention that “in the initial operation years of the studied highway embankment section, degree of consolidation tends to increase. As the underlying permafrost keeps thawing, the characteristic drainage length increases and the effective consolidation time decreases continuously, the newly thawed layer cannot finish consolidation in the same year. Some residual consolidation time is needed for consolidation to accomplish. Up till the permafrost layer thaws completely, with accumulation of pore water pressure the residual consolidation time may accumulate to a high level. It will therefore take a fairly long time for consolidation to finish, and thus settlement of the embankment will continuously develop for a considerable time. This can well explain the phenomenon that in some regions where permafrost is already completely thawed while settlement is still under development”.

3.1 Thaw Settlement Under Dams and Dikes

Sayles (1987) mentions that in practice, the design of dams and dikes on permafrost can be divided into two types:

- The frozen type, where the embankments and their foundations are maintained frozen during the life of the structure; and
- The thawed type, where the embankments usually are designed assuming that the embankment will remain unfrozen and its permafrost foundation will thaw during either the construction of the operation of the structure.

3.1.1 Frozen embankments

By definition, a frozen core dam is a dam designed as a water retaining structure incorporating an impermeable frozen fill zone and frozen foundation (Miller, Kurylo, & Rykaart, 2013). Some authors report the use artificial and/or natural freezing method(s) for water retaining dike construction in North America (Kitze & Simoni, 1972; Andersland & Anderson, 1978). The latter authors concluded that for one particular remote site, where the permafrost is deep and saline, and suitable low-permeability borrow material is scarce, a conventional unfrozen dam was unsuitable. They designed a frozen core dam to accommodate these challenging foundation conditions. Many dams were constructed by the frozen method in Russia in the 1960s as well (Semenov, 1967; Biyanov, 1969; 1970; Trupak, 1970). Andersland & Ladanyi (2004) indicate that frozen embankments are more suitable for sites where permafrost is continuous and the foundation would become unstable if thawed. In fact, many factors should be taken into account for embankment design for a particular site, as summarized by Sayles (1984; 1987):

- Service type: retain water continuously or intermittently;

- Water retained by embankment: contact width, depth, temperature, and chemical composition;
- Site climatic conditions: regional and local, especially temperature;
- Existing permafrost: temperature, extent in area and depth;
- Material sources: availability of local materials and logistics involving fabricated construction materials;
- Safety; consequences to life and property in the event of embankment failure;
- Environment: effects of construction and operation of water-retaining structure;
- Solar radiation: orientation of the downstream embankment face;
- Frost action: problems on the embankment crest and downstream slope; and
- Costs: economics relative to a particular design (Andersland & Ladanyi, 2004).

Both artificial and natural techniques can be used to maintain an embankment in the frozen state. If the climate is very cold (mean annual temperature of -8 °C or colder), natural winter cooling can be sufficient up to a depth of 10 m for a snow-free surface (Tsytovich, 1975; Sayles, 1984). Frozen earth dams with heights up to 25 m have been constructed using artificial refrigeration, like blowing cold air through a system with pipes within the embankment (Sayles, 1984; Semenov, 1967). BGC Engineering (2014) reports the use of thermosyphon groups at the talik-permafrost contacts beneath the dike abutments for preservation and enhancement via additional cooling of marginal and warm permafrost in this area.

3.1.2 Unfrozen embankments

"The design for an unfrozen embankment founded on thawing permafrost is most suitable for sites where the foundation materials are thaw-stable; i.e. where the thawing strengths of the earth materials provide an adequate factor of safety against shear failure, and deformations resulting from thawing will not endanger the integrity of the embankment. This requirement usually restricts the use of the thawing foundation design to sites where permafrost soil is ice-poor (i.e. free from segregated ice) or where reasonably sound bedrock can serve as the foundation and the permeability of the thawed foundation is tolerable. At sites where only a portion of the foundation contains ice-rich permafrost at shallow depths, this ice-rich portion is usually thawed before placing the embankment, or the frozen soil is excavated to a predetermined depth (Gluskin & Ziskovich, 1973) and replaced" (Sayles, 1987).

MacPherson *et al*. (1970) suggest a method of estimating the depth of excavation so that thaw settlement can be limited to a predetermined amount h during the operation of the embankment:

$$H = \frac{h(1 + G_s w)}{G_s w - e_f}$$

where H is the depth of high ice content soil layer, G_s is the specific gravity of soil solids and e_f is the final void ratio. Curves for h = 5 feet of settlement and final void ratios e_f = 0 and e_f = 0.3 are plotted in Figure 3-1.

If the study showed that if the ice-rich soil layer plots above any one of the above curves or other curve selected for a project at particular site, then excavation by a depth of 'd' is required until the point defining the reduced thickness reaches the curve. MacPherson *et al*. (1970) recognized there are several simplifying assumptions:

- The curves are based on a theoretical final void ratios;
- No allowance has been made for lateral movement;
- No allowance has been made for settlement within the dike itself or below the permafrost-affected layer;

- In many areas it is necessary to consider individually several strata within the foundation.

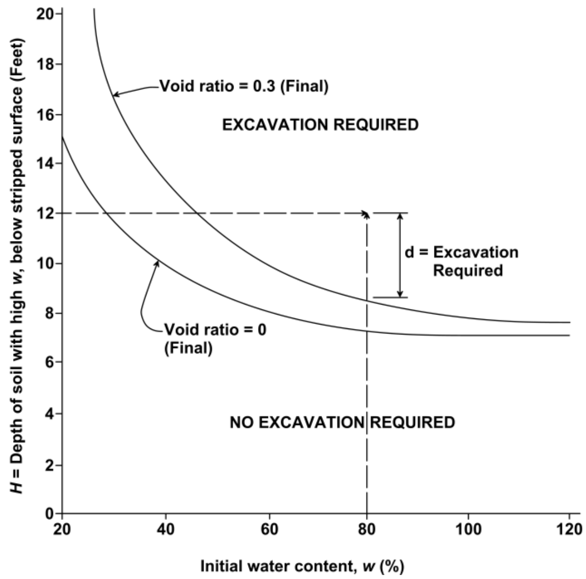


Figure 3-1: Curves showing conditions for 5 feet of settlement for final void ratios of 0 and 0.3 (after MacPherson *et al*., 1970)

Nevertheless, MacPherson *et al*. (1970) consider that the results will still err on the conservative side and provide a guide to the selection of excavation requirements.

Sayles (1987) mentions that “where the permafrost is not removed and the foundation is expected to thaw during the life of the structure, the embankment design is similar in many respects to that of a water retaining embankment located in a temperate climate when the effects of thawing on the consolidation, permeability and strength are taken into account. However, special consideration is given to certain elements of the embankment. One such element is the impervious zone, which must be constructed of self-healing soils (Gupta, Marshall, & Badke, 1973) so that this zone can remain "impervious" even if cracking occurs during the settlement of the foundation. Soils that become stiff and brittle when compacted in the impervious zone are avoided. Other design provisions that are often included to accommodate the anticipated settlement are: the use of flatter embankment slopes; overbuilding the height of the embankment by an amount equal to the anticipated settlement; and periodically rebuilding portions of the embankments that settle more than a tolerable limit (Johnston, 1969; Macpherson, Watson, & Koropatnick, 1970). Prethawing followed by preloading to consolidate the foundation before placement of the embankments has also been suggested as a means of reducing foundation settlements (Gluskin & Ziskovich, 1973). The concept of utilizing sand drains in a thawing foundation to increase the rate of consolidation and, hence, quickly improve the shear resistance and stability of embankments has been used successfully (Johnston, 1965; 1969; Macpherson, Watson, & Koropatnick, 1970). In addition, analytical methods have been developed for estimating the rates of thaw and settlements of dikes on permafrost during operations using simple heat conduction and heat balance equations for a one-dimensional transient condition that takes into account heat from water seepage (Brown & Johnston, 1970).”

Brown & Johnston (1970) studied the construction of dikes on permafrost. They mentioned that prediction of thaw settlement is important for two main reasons:

- The amount and rate of settlement, which depend on the depth and rate of thaw, are of interest in assessing the need for and the scheduling of future maintenance;
- The stability of the foundation, which is dependent upon the rate at which thaw water is redistributed or escapes, is also related to rate of thaw.

In the case of a pervious dike, it was recognized that thaw settlement would occur as a result of heat transfer between the warm water of the forebay and the frozen ground under and adjacent to the dike. The rate of thaw of underlying permafrost depends on the temperature and amount of water flowing through a dike as well as on the thermal properties of the ground. Thawing under the entire dike begins as soon as the water level on the upstream side of the dike rises, whereas with an impervious dike thawing commences only from the upstream face and is delayed under the main body of the dike. Because of the lateral penetration of thaw in an impervious dike, settlement and stability considerations are more complex for it than for a pervious dike, where thawing progresses downward at essentially the same rate under the whole dike. “When thaw progresses laterally and at unequal rates under the dike and adjacent forebay, significant differential settlements and cracking can be expected, only tolerated with difficulty by a typical, compacted, impermeable core material” (Brown & Johnston, 1970).

Brown & Johnston (1970) concluded that “it is essential, therefore, for predicting thaw rates, that adequate field investigations be carried out to provide the required information. The most important quantities to be determined are total moisture content (ice plus water) and mean annual ground and water temperature. Next, but somewhat less important, is a determination of the thermal conductivity of the unfrozen soil”.

“In Canada, the literature does not record the construction of an embankment designed as a frozen structure on permafrost but it does reveal that several small dikes (Johnston & MacPherson, 1981) and a waste impoundment (Thornton, 1974) were designed and constructed as the thawed type of embankment on permafrost. In these designs, the amount of thaw consolidation that would occur in the foundation was estimated, and the embankment height was increased to accommodate the anticipated settlement. As an alternative to overbuilding the dikes, the design heights can be maintained by periodically adding embankment material to the crest of the dikes as settlement progresses. In some instances, vertical sand drains (Johnston, 1969; Macpherson, Watson, & Koropatnick, 1970) can be installed in the permafrost foundation beneath the embankment to reduce pore pressures and hence increase the shearing resistance of the soil while thawing occurs […]. The differential settlements associated with this type of design can lead to transverse cracking of the embankments. To accommodate the cracking, the embankments constructed in Canada were constructed of soils that are selfhealing in nature (Johnston & MacPherson, 1981). Essential to this type of design is a continuous observation program throughout the life of the structure” (Sayles, 1987).

Investigations carried out in the Soviet Union have shown that most of the total settlement is the result of thawing of the ice lenses and inclusions, and occurs at about the same rate as the frozen ground thaws. The remainder, which is a much smaller portion of the total settlement, results from consolidation of the thawed soil and takes place at rates dependent on the drainage and compressibility characteristics of the soil. Studies by Tsytovich (1965), Malyshev (1966) and Zaretskii (1968), among others, well illustrate the complexity of thaw consolidation and the difficulty of predicting total settlement.

3.2 Assessing Thaw Settlement

Panday & Corapcioglu (1995) took the following parameters into account in their thaw settlement prediction model:

- Pore pressure;
- Temperature;
- Degree of unfrozen water content and ice content;
- Porosity.

More specifically, they used the physical coefficients and constants listed in Table 3-1, where θ_w denotes the volumetric water content, θ_i denotes the volumetric ice content, n denotes porosity, S_w denotes the degree of unfrozen water saturation, p denotes pressure and T denotes temperature.

Table 3-1: Parameters used in a thaw-subsidence model (after Panday & Corapcioglu, 1995)

Parameter	Value
Density of water	$\rho_w = 1{,}000$ kg/m³
Density of ice	$\rho_i = 920$ kg/m³
Density of soil solids	$\rho_s = 2{,}330$ kg/m³
Acceleration due to gravity	$g = 9.81$ m/s²
Heat capacity of water	$C_w = 4{,}180$ J/kg/°C
Heat capacity of ice	$C_i = 2{,}044.6$ J/kg/°C
Heat capacity of soil	$C_s = 1{,}445.7$J/kg/°C
Thermal conductivity of water	$\lambda_w = 0.59356$ J/m/s/°C
Thermal conductivity of ice	$\lambda_i = 0.7106$ J/m/s/°C
Thermal conductivity of soil	$\lambda_s = 2.926$ J/m/s/°C
Latent heat of fusion for water	$L = 0.34*10^6$ J/kg
Coefficient of compressibility of soil	$m_v = 0.017$ cm²/N
Thaw-settlement parameter	$A_0 = 0.28$
Hydraulic conductivity of medium	$K = 2.47*10^9 \exp(44\theta_w)$ cm/h
Hydraulic conductivity in frozen zone	$K_{\text{frozen}} = K/(10^{10\theta_i})$
Specific retention curve	$n.S_{w_1}(p) = 0.02418/[0.186 + (-0.7p)^{2.56}] + 0.286$
Phase-composition curve	$n.S_{w_2}(T) = \exp(1.1T)$
Thermal liquid diffusivity	$D_{MT} = 4 * 10^5(-5.6S_w^2 + 28S_w + 5)$ m²/s

According to Ushkalov *et al*. (1963), is it necessary to know four main physical characteristics of the soils in cold regions geotechnical problems:

- Unit weight of mineral particles;
- Total water content with respect to the dry weight of the soil;
- Amount of unfrozen water, and;
- Unit weight of frozen undisturbed soil.

These characteristics are of great importance because they determine thermophysical and mechanical properties of the ground. Brown & Johnston (1970) stressed that the most important quantities to be determined are total moisture content (ice plus water) and mean annual ground and water temperature when assessing thaw settlement under a dike. Next, but somewhat less important, is the determination of the thermal conductivity of the unfrozen soil. This can be measured directly in the laboratory or estimated from soil type and unfrozen moisture content.

Tsytovich (1958) mentions that changes in voids ratio e are caused by two factors: coefficient of thawing and coefficient of consolidation. Those two factors are taken into account by Watson *et al*. (1973) where they calculate the total thaw-settlement using

$$S = A_0 X + m_v P X + \frac{m_v \gamma' X^2}{2}$$

where A_0 is the thaw settlement/strain parameter, X is the depth to thaw front from original surface, P is surcharge load and γ' is submerged unit weight of thawed soil (assuming that the water table in the thawed material is at the surface). Leroueil *et al*. (1991) obtained similar stress-strain curves in their oedometer tests, and can be described by

$$\varepsilon_v = A_0 + m_v \sigma'_v$$

where ε_v is the axial strain and σ'_v is the effective vertical stress. The thaw-settlement parameter A_0[1] can represent a large part of the total settlement. Figure 3-2 show the A_0 parameter as a function of the frozen bulk density and gravimetric water content on low-plasticity frozen silts. From Figure 4-1, one can observe that the higher the density or the lower the water content of the frozen soil, the larger the settlement/strain parameter.

Andersland & Ladanyi (2004) presented two simple methods for evaluating thaw settlement. The first relationship (Crory, 1973) is expressed in terms of soil dry densities:

$$\frac{\Delta H}{H_f} = 1 - \frac{\rho_{d_f}}{\rho_{d_{\mathrm{th}}}}$$

where $\Delta H / H_f$ is the vertical strain, and where ρ_{d_f} and $\rho_{d_{th}}$ are the frozen and thawed dry densities of the soil, respectively.

The second method is an empirical relationship developed by Ladanyi (1994) and modified by Nixon & Ladanyi (1978) as follows:

$$\frac{\Delta H}{H_f} = 0.90 - 0.868\left(\frac{\rho_f}{\rho_w} - 1.15\right)^{1/2} \pm 0.05$$

[1] $\Delta e = \Delta e/(1+e_0)$, where Δe and e_0 are the change in void ratio and is the initial void ratio respectively.

where ρ_f is the frozen bulk density and ρ_w is the density of water. Thaw settlement for silts and sands from several sites along the Mackenzie River can be expressed by the aforementioned relationship (Speer, Watson, & Rowley, 1973; Johnston, 1981; McRoberts, Law, & Murray, 1978; Johnson, McRoberts, & Nixon, 1984) as well as for low-plasticity clays (Keil, Neilsen, & Gupta, 1973).

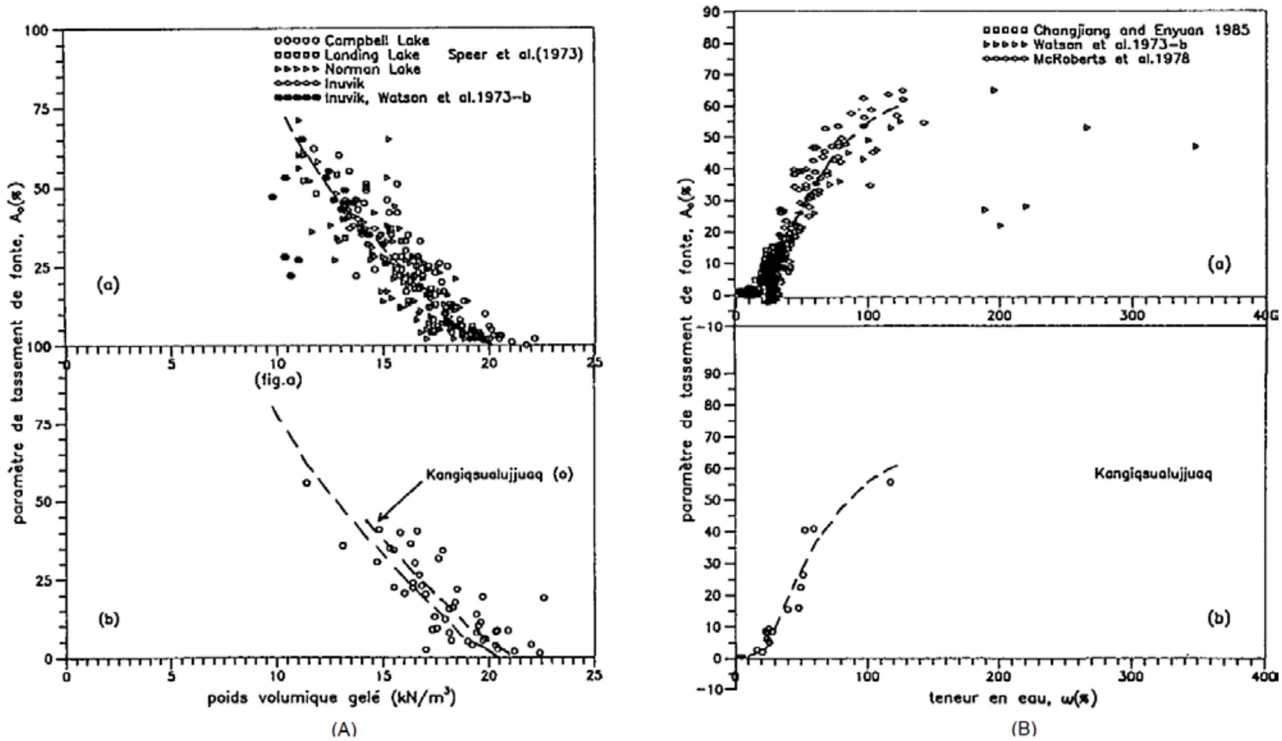


Figure 3-2: Thaw-settlement parameter vs frozen bulk density (A) and vs gravimetric water content (B) (after Leroueil *et al*., 1991)

4.0 Thaw Settlement Phenomenon at Whale Tail Dike

As part of the whole water management infrastructure at Amaruq, there will be an important dewatering dike that is required to enable extraction of the mineral in an open pit on the downstream/north side of the dike, known as the Whale Tail Dike (WTD). The WTD, which is a rock fill dike with a cement bentonite cutoff wall keyed 1 m into bedrock, is located on a shallow plateau of the lake bottom with an approximate 2 m depth of water. This plateau is located between deeper sections of the lake with water depths of about 12 m. Once in operation, the downstream side of the dike will be dewatered (see Figure 4-1) and the upstream side of the dike will allow a 3.5 m rise of the lake level prior to being discharged into Mammoth Lake via the South Whale Diversion Channel to be built southwest of the WTD. The rising the lake level on the upstream side of the WTD, will change the thermal regime of the flooded lands and could degrade the underlying permafrost.

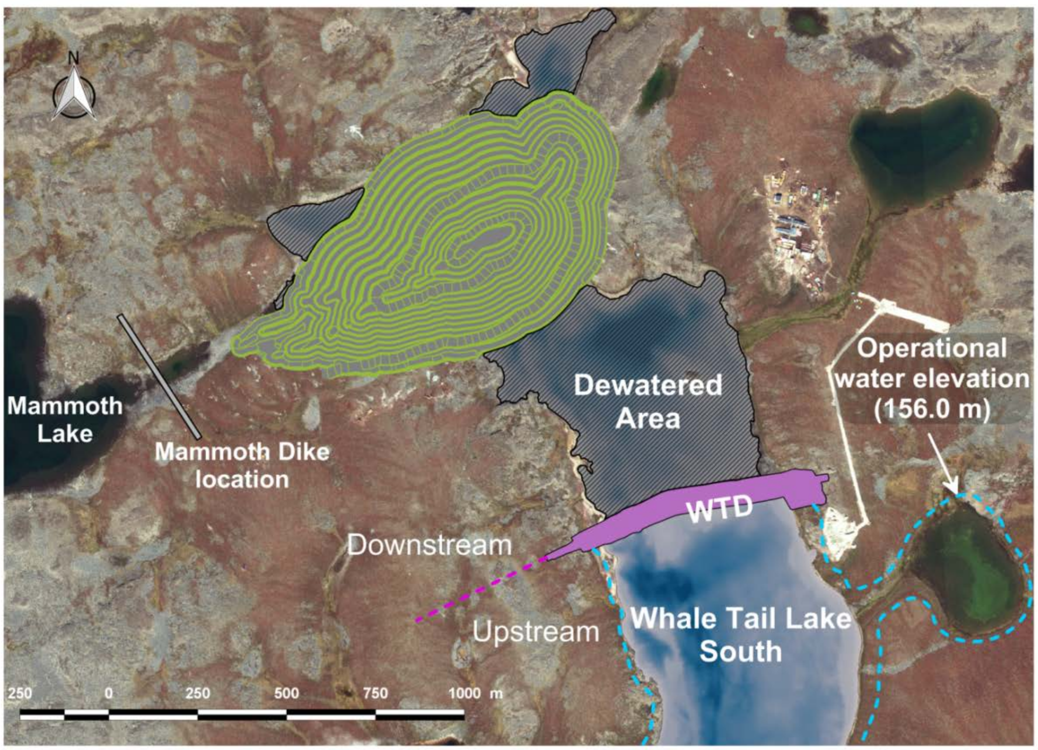


Figure 4-1: General location of the WTD

SNC-Lavalin (2017e) showed the presence of a talik under Whale Tail Lake to a depth of approximately 110 mbgs[2]. This means that a rise of the lake level on the upstream side by 3.5 m will not cause thaw settlements in most of the current footprint of the lake, since the lake bed deposits are already unfrozen. However, the permafrost at the abutments is expected to thaw due to flooding of the banks once the operational water level of the dike is reached. Thaw settlements will be observed following the construction of the dike, especially if the underlying soil contains thick ice lenses. Moreover, if the thaw front progresses rapidly in the

[2] Mgbs = metres below ground surface.

permafrost, local instability may occur. It is therefore important to assess the magnitude and consequences of thaw settlements under both abutments of Whale Tail Dike.

4.1 Available Data

The available geotechnical and geothermal properties (water content, dry density, frozen and unfrozen thermal conductivity, etc.) are very limited hence do not allow us to carry out a comprehensive thaw settlement study. As a result a few reasonable assumptions and data extrapolations had to be made.

4.1.1 Geothermal data

The soils beneath WTD are monitored by thermistor strings, which allow us to obtain a thermal profile of the longitudinal section along the dike. In total, 16 boreholes were drilled within or near the footprint of WTD in which temporary or permanent thermistor strings were installed. They are distributed as follows:

- 3 temporary thermistor boreholes in each of the abutments;
- 4 permanent thermistors and 3 temporary thermistor boreholes in Whale Tail Lake; and
- 3 temporary thermistor boreholes in the bank, south of WTD east abutment.

All the temporary boreholes cannot be monitored at the same time. In fact, 3 thermistor strings are available to monitor each of the temporary boreholes. They can be carried around and used to monitor soil temperature in several locations. Figure 4-2 shows the location of thermistors in the WTD area.

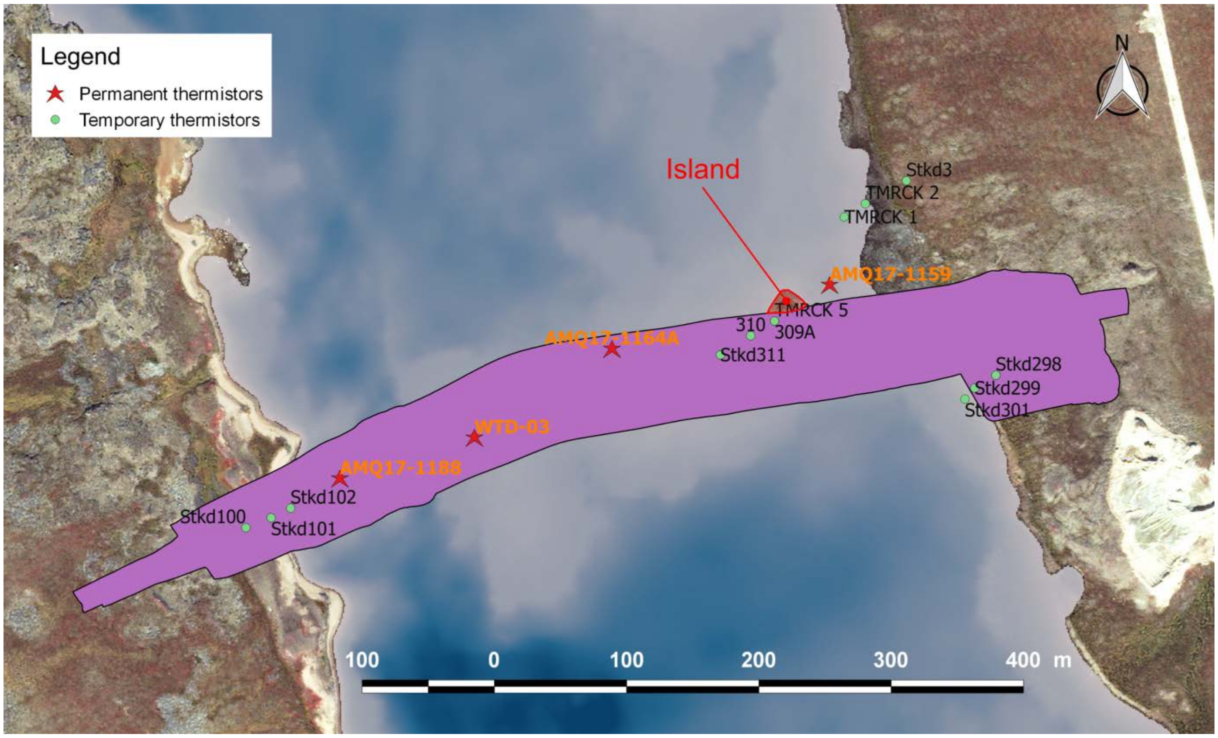


Figure 4-2: Temporary and permanent thermistors location along WTD

Among the 16 boreholes shown on Figure 4-2, temperature data from 13 boreholes (10 temporary and 3 permanent) are available for the period between April and October 2017.

Temperature data show that permafrost exists on the shallower portion of the lake near the east abutment. In fact, SNC-Lavalin (2017g) pointed out that the presence of the island near the east bank has a major influence on the thermal regime because it acts as a natural thermosyphon. This explains why the thermistor AMQ17-1159 shows the presence of permafrost at -5 °C between the island and east abutment, and thermistors 309A, 310 and Stkd311 show the presence of permafrost at -3 °C west of the island. However, the two other thermistors in Whale Tail Lake (AMQ17-1164A and AMQ17-1188) show the presence of a talik which is at least 40 m deep.

The 3 temporary thermistors on the west abutment show that the overburden is underlain by permafrost. The temperature of the permafrost increases with the proximity of the lake, which acts as a heat source. Temporary thermistor strings located near the east abutment of the WTD (Stkd299 and Stkd301) show the same behaviour as well. This means that the boundary between permafrost and unfrozen soil is in the water near the shore on the west side, and between AMQ17-1164A and Stkd311 on the east side. The temperature profile along WTD is showed on Fig 5-3.

Updated technical report on thermal analysis showed that the esker at the west abutment should thaw in approximately 3 to 4 years after the construction of the dike, while the ice-rich till under the upstream part of the embankment at the east abutment should thaw within 30 years of operation (SNC-Lavalin, 2018). This information makes it possible to assess thaw settlements at both abutments over time. Moreover, the assumption that unless mitigated the whole thickness of the overburden will thaw at both abutments during the operational life of the structure was confirmed by the thermal analysis.

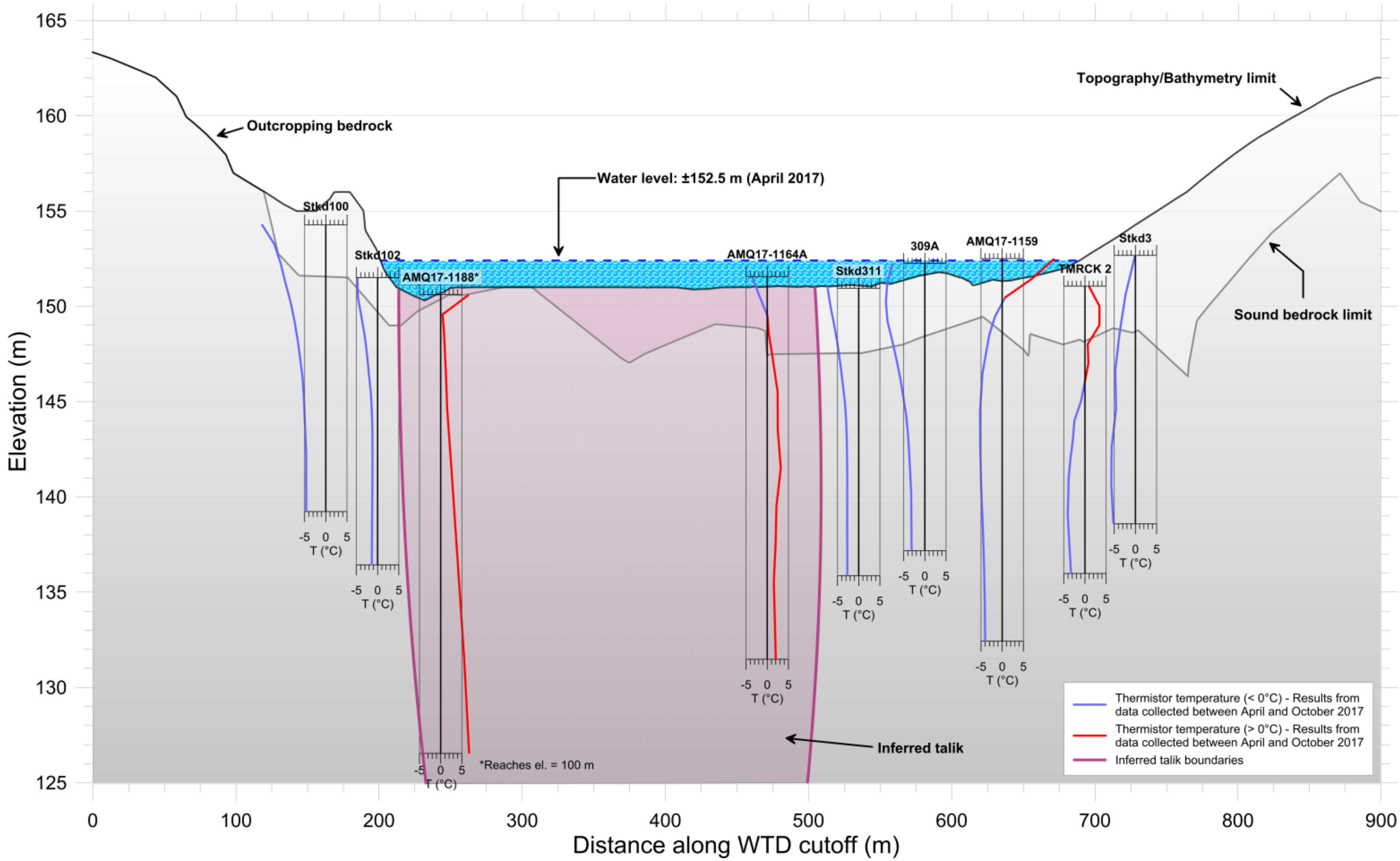


Figure 4-3: Thermal profile along the WTD alignment

4.1.2 Geotechnical properties and abutments earthwork

Since the east and west abutments have distinct soil strata and overburden thicknesses, the geotechnical properties are unique for each abutment. Moreover, the construction method at each abutment will differ in terms of the amount of soil that will be excavated or other mitigating measures that may be appropriate. For those reasons, the west and east abutments are treated separately. Figure 4-4 shows the location of the samples taken at the WTD abutments.

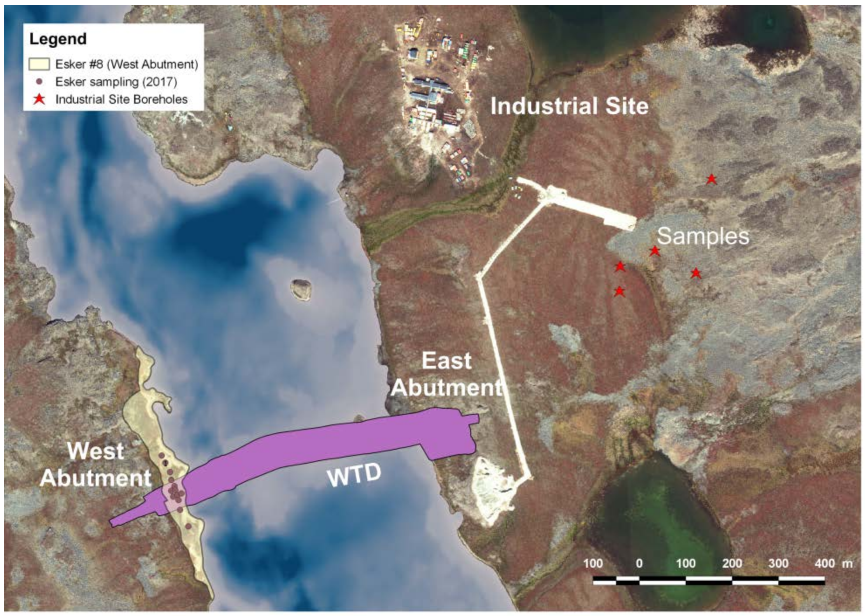


Figure 4-4: Sampling location in WTD abutments

4.1.2.1 West abutment

As shown on Figure 4-4, the west abutment partially sits on an esker along the shore, while the west end of the dike directly sits on exposed bedrock. Thaw settlement might be an issue in the esker area if it contains massive buried ice or thick segregated ice lenses. However, there is no information indicating the presence of ice in the esker, hence thaw settlement should not be a problem because the esker is essentially a medium to coarse-grained sand, which will only undergo elastic settlement and essentially no time dependant consolidation. Appendix A shows the gradation curves of the esker samples that were taken at the west abutment.

During the construction of WTD scheduled for summer 2018, the whole esker located west of Whale Tail Lake will be excavated up to the elevation of 153 m. The sand thus excavated will be used as a borrow material for the construction of the dike. Following the excavation, only 1.5 metres at most of the esker material will remain in place above the bedrock surface at the west abutment of WTD, which should cause negligible thaw settlements.

4.1.2.2 East abutment

The situation at the east abutment of the WTD differs greatly from the other side of the lake because the overburden is much thicker and consists of a well-graded till with higher fines content. Only 5 soil samples were taken on the east side of the lake, and are located about 500 m inland as shown on Figure 4-4. Since those samples are the only information available, it is assumed that the soil gradations and water contents are representative of the condition at the east abutment, even though the dike is located at a lower elevation. Five (5) water contents and three (3) gradation curves were produced from samples collected approximately from depths of 1 to 8 metres. The water contents are shown on Figure 4-5.

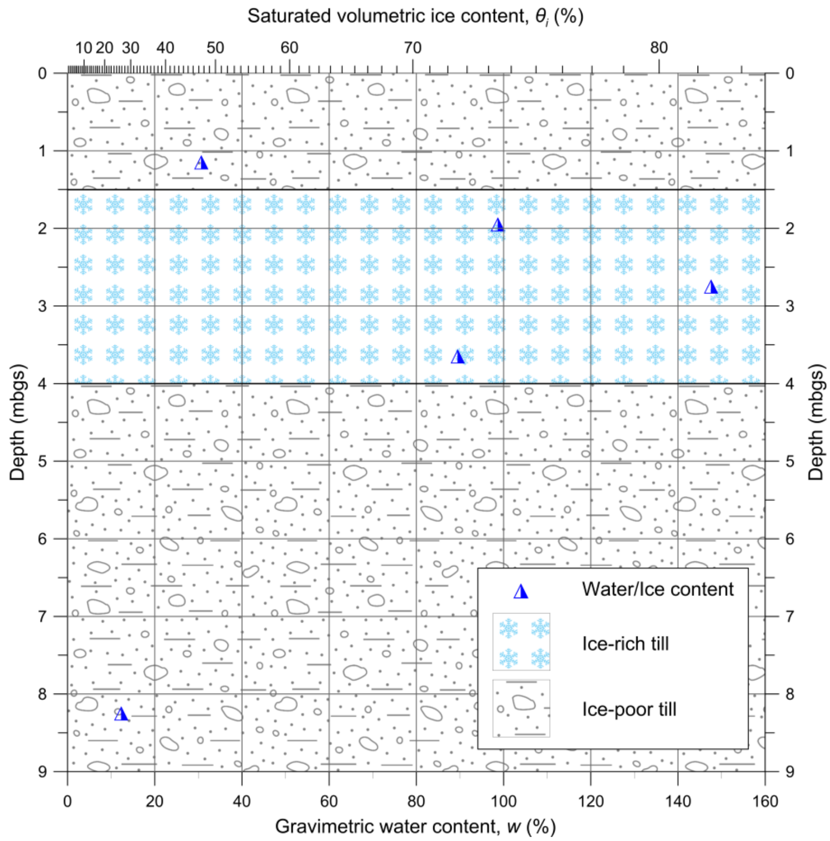


Figure 4-5: Measured gravimetric water contents and computed volumetric ice contents on till sampled 500 m inland of the east abutment of WTD

The Y axes are the actual depth at which the samples were collected. Even though the samples were collected at different locations, the surface elevation stayed pretty much the same for all the sampling locations. The bottom X-axis shows the gravimetric water content that were measured on thawed samples, that is

$$w = \frac{W_w}{W_s}$$

where W_w is the weight of the soil solids and W_s is the weight of water. Assuming that the active layer is saturated (S_r = 1) and that the specific gravity of soil solids G_s = 2.7, the volumetric ice content θ_i can be calculated as

$$\theta_i = \frac{V_i}{V_t}$$

where V_i is the volume of ice and V_t is the total volume of soil.

According to Arenson *et al*. (2007), a frozen soil can be described as “ice-rich” from a volumetric ice content of 50-60 %. Based on the data shown on Figure 4-5, the active layer in this area is approximately 1.5 to 2 metres thick, under which lies a layer of ice-rich soil that is about 2.5 metres thick. It is SNC-Lavalin’s understanding that the top of the active layer has low to intermediate ice content, but sits on a frozen layer that contains potential segregated ice lenses, under which is ice-poor till. It is interesting to point out that the soil sample which has the largest water/ice content also has the greatest amount of clay size particles (10%), for a total fine particle content of 44 %. The deepest soil sample with the lowest water content also has the lowest amount of fine particles (17 %). All the test results are included in Appendix A.

For the thaw settlement analyses we have assumed that the unfrozen soil above the ice-rich till will be excavated before the construction of the dike. More specifically, the first 2 metres of till will be excavated down to a minimum elevation of 153 m. In the east abutment area, is expected that 2 metres of ice-rich till be remain in place, which may subside during thawing.

The main concern at this location is the thawing of the 2-metre-thick ice-rich till layer in terms of settlements, and not in terms of stability. Because the soil is a well-graded till, pore water pressure generated caused at the thaw front should be discharged rapidly. A typical value for the coefficient of volume compressibility m_v of a glacial till can be estimated at 0.150*10^{-3} m^2/kN (Head, 1982). A plastic material such as clayey silt would require a stability analysis on its own, whereas the only concerns in the case of the east abutment are the potential settlements caused by the melting of ice in the ice-rich layer of till.

The thaw settlement parameter A_0, as described in section 3.2, can account for a large part of the calculated settlement, especially when the frozen bulk unit weight is low due to the presence of a significant volumetric ice content. Values of A_0 for many frozen soils were retrieved from the literature (Watson, Slusarchuk, & Rowley, 1973; Johnston, 1981; Leroueil, Dionne, & Allard, 1991), which varied between 0.32 and 0.43. In order to stay on the conservative side and predict the maximum thaw settlements, a value of A_0 = 0.43 is used in the current study.

4.1.3 Summary of input parameters used in the thaw settlement assessment

Following the information presented in section 4.1.2, the geotechnical properties of the materials at the east abutment are summarized in Table 4-1.

Table 4-1: Geotechnical properties of the till material at the east abutment

Parameter	Ice-rich till (frozen)	Ice-rich till (thawed)[c]	Ice-poor till (thawed)[a]
Water content, w (%)	112.4[b]	33.3 [c]	16.5
Degree of saturation, S (%)	100	100	100
Specific gravity of soil solids, G_s	2.7	2.7	2.7
Void ratio, e	3.31	0.90	0.45
Porosity, n	0.84	0.47	0.31
Bulk density, ρ (kg/m^3)	1,395	1,895	2,175
Dry density, ρ_d (kg/m^3)	627	1,421	1,866

[a]The frozen properties of the ice-poor till layer can be calculated using phase change relationships (Andersland & Ladanyi, 2004).
This value represents the average of two lab test results (Appendix A).

[b] Average of lab test results (Appendix A) into ice rich till

[c] It is assumed that the thawed void ratio of the ice-rich till material corresponds to its loosest state (Andersland & Ladanyi, 2004).

Table 4-2 presents a summary of the calculation parameters for the assessment of thaw settlements at the east abutment.

Table 4-2: Input parameters used in the calculation of thaw settlements in the ice-rich till layer at the east abutment

Parameter	Ice-rich till	Ice-poor till	Source
Thaw settlement parameter, A_0 (%)	43	0	Leroueil *et al.* (1991)
Depth of thawed layer, X (m)	2.0	3.0	Thermal model, SNC-Lavalin (2018)
Coefficient of volume compressibility, m_v (m^2/kN)	0.15×10^{-3}	0.15×10^{-3}	Head (1982)
Effective surcharge load*, P (kPa)	116	124	See Figure 4-6

*The effective vertical stress caused by the construction of the embankment was calculated following Das (2011).

The effective surcharge load caused by the weight of the rockfill embankment was calculated using the geometry shown in Figure 4-6. The point of calculation for the thaw settlement as shown on Figure 4-6 was based on the worst case scenario, having the maximum embankment height on the one hand, while being close to the heat source (the water from the lake) on the second hand. The thermal model showed that the overburden should not thaw close to the cutoff wall due to the great distance from the lake.

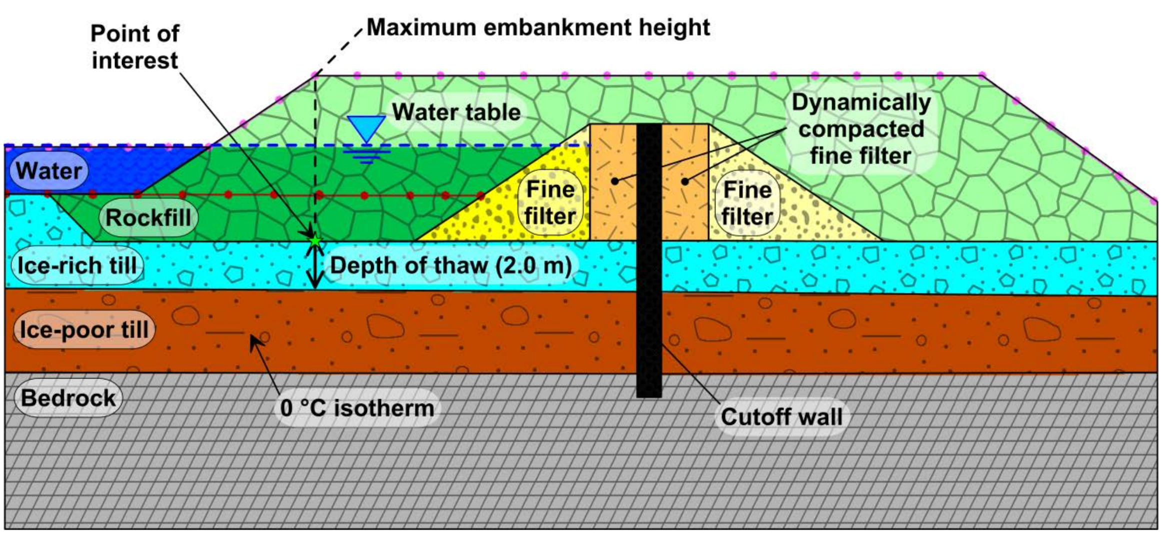


Figure 4-6: Point of interest at which thaw settlement is evaluated at the east abutment

The current section mentions parameters for the assessment of thaw settlement at the east abutment only. The same methodology was applied at the west abutment as well, but it was found that the sandy material of the esker is thaw stable and would not need further investigation. As mentioned in the next section, the ice-rich till layer at the east abutment contributes to the majority of the computed settlements, so the focus is on the thaw settlement of the two-metre-thick ice-rich layer.

4.2 Computation of Thaw Settlements

In order to assess the total thaw settlements following the construction of the Whale Tail Dike, three different methods mentioned in section 3.2 were used. They are summarized in Table 4-3.

Table 4-3: Computed thaw settlements at the east abutment

Method		Equation	Computed thaw settlement (m)	
			Ice-rich till	Ice-poor till
A	Empirical method (Watson *et al.*, 1973)	$S = A_0X + m_vPX + \frac{m_v\gamma' X^2}{2}$	0.90	0.06
B	Dry density method (Crory, 1973)	$\frac{\Delta H}{H_f} = 1 - \frac{\rho_{d_f}}{\rho_{d_{th}}}$	1.12	0.08
C	Statistical method (Nixon & Ladanyi, 1978)	$\frac{\Delta H}{H_f} = 0.90 - 0.868\left(\frac{\rho_f}{\rho_w} - 1.15\right)^{1/2} \pm 0.05$	1.04	0.10

The three different methods show similar values of thaw settlements computed for each till layer, with an average settlement of 1.02 m for the ice-rich and 0.08 m in the ice-poor till layers respectively. The ice-rich till layer contributes to about 93 % of the total settlements due to its very high volumetric ice content. These results are valid for the point of interest shown in Figure 4-6, and can be coupled with the thermal analysis to estimate the thaw settlement at this specific location over time. The three methods are compared in Figure 4-7.

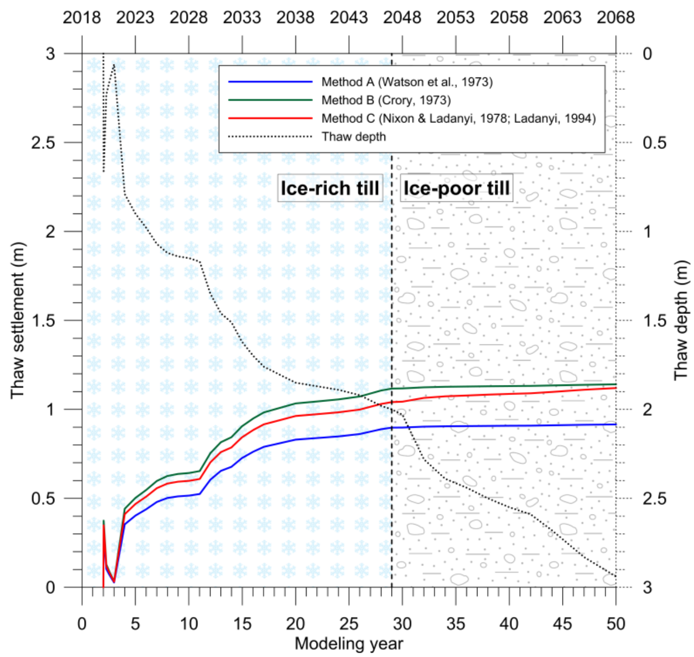


Figure 4-7: Computed thaw settlement over time for three different methods

The thaw front penetration is shown with the dotted line on the right Y-axis, while the thaw settlements for the three different methods are shown with the colored lines on the left Y-axis. The settlements occur rapidly after the construction of the dike, and reach approximately 0.40 m after 5 years of operation. The settlements continue to increase while the two-metre-thick ice-rich till layer thaws, up until the 29th year of operation where the thaw front reaches the interface between the ice-rich and the underlying ice-poor layers. The thaw front penetration rate then increases because the ice-poor till material is easier to thaw due to its lower ice content, thus lower latent heat required for the melting of the ice. For the same reasons, the thaw settlement progression is nearly negligible in the ice-poor layer. The model shows that no matter which method is used, the great majority of the thaw settlements should occur in the first 30 years of operation.

Using only one point of interest makes it possible to assess the settlements over time at a specific location by using the data from the thermal analysis at the Whale Tail Dike. However, by fixing the temperature distribution in the dike area at one point in time, a longitudinal profile of the settlements can be obtained. The main interest of having a geometric profile of the computed settlements is to evaluate if significant subsidence could occur near the cutoff wall, which might jeopardize its integrity. The maximum computed thaw settlement profile as a function of distance from the cutoff wall after 50 years of mining operations is shown in Figure 4-8.

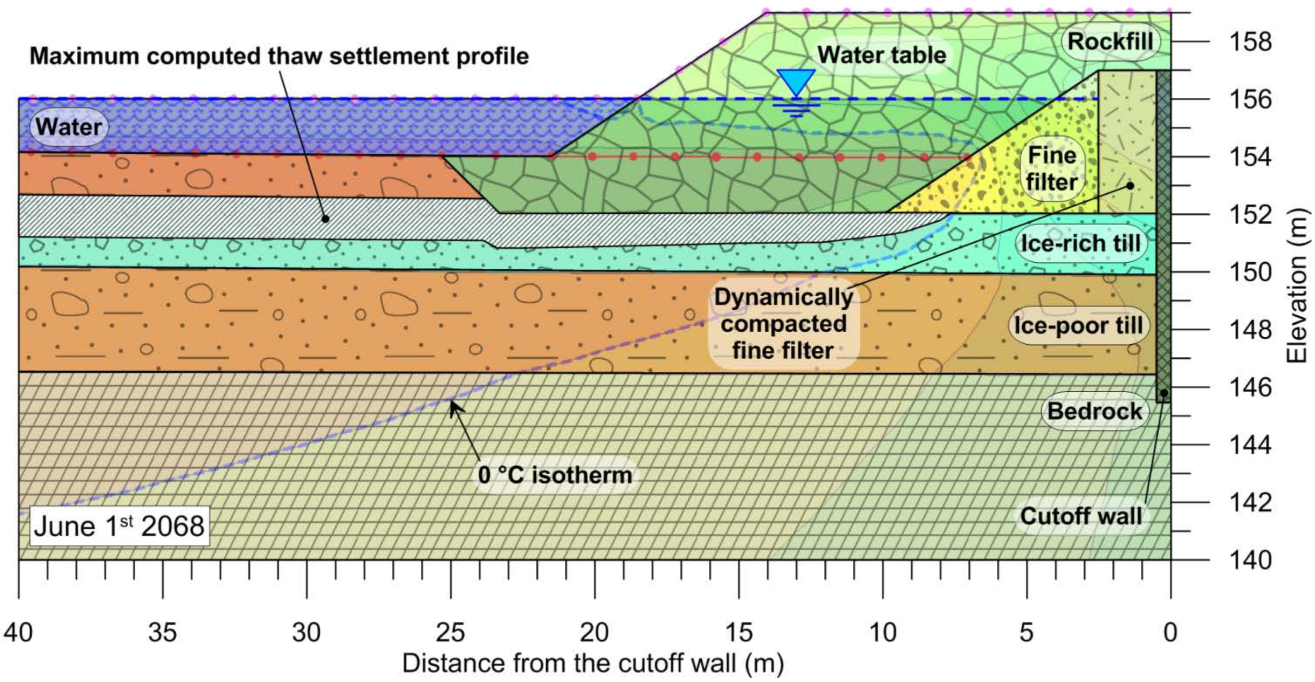


Figure 4-8: Thaw settlement profile after 50 years of operation

In order to facilitate the understanding of the Figure 4-8, the thaw settlement profile is represented in the ice-rich till material since it will contribute to the great majority of the settlements. The settlements shown on the shaded area is on scale with the vertical scale shown on the right of the figure. The thermal model shows that the ice-rich till layer will start to thaw approximately 7.5 m from the cutoff wall, which means that very little settlement should occur close to the wall. The whole ice-rich layer should be thawed at a distance of 12 m from the cutoff wall after 50 years of operation, and the maximum computed settlement at that location is approximately 1.1 m. The maximum settlements occur more than 25 m from the cutoff wall, where the overburden is not excavated. At that location, the ice-rich layer is 2.5 metre-thick and the whole overburden is thawed, which contribute to total thaw settlements of 1.5 m.

It is interesting to point out that the total thaw settlements calculate in Table 4-3 are conservative because it was assumed in the calculation that the ice-poor till layer was completely thawed (X = 3.0 m as mentioned in Table 4-2). It can be observed in Figure 4-8 that the point of interest under the maximum embankment height shows that only the top metre of the ice-poor till has thawed after 50 years of operation. For those reasons, it can be assumed that only the ice-rich till layer will contribute to thaw settlements within the footprint of the dike.

4.3 Segregation Potential of the Till Material

Damage due to frost action can be caused by the presence of segregated ice lenses in soils, creating either excessive deformations or strength-weakening following melting (Konrad, 1999). Deformations following the thawing of the frozen till were evaluated in section 4.2. However, these numbers are based on very limited field data, and rely on data on samples collected about 500 m from the east abutment. The segregation potential (“SP”) parameter was calculated before and after dike construction for completeness in the study.

As shown in the laboratory test results in Appendix A, the ice-rich till contains 10 % of clay and 34 % of silt and falls under the USCS classification SM (silty sand). The ice-poor till only contains 3 % of clay size particles and 14 % of silt, and its USCS classification is GW-GM (well graded gravel with some silt). The frost susceptibility of those two materials can be assessed with the chart presented in Figure 4-9.

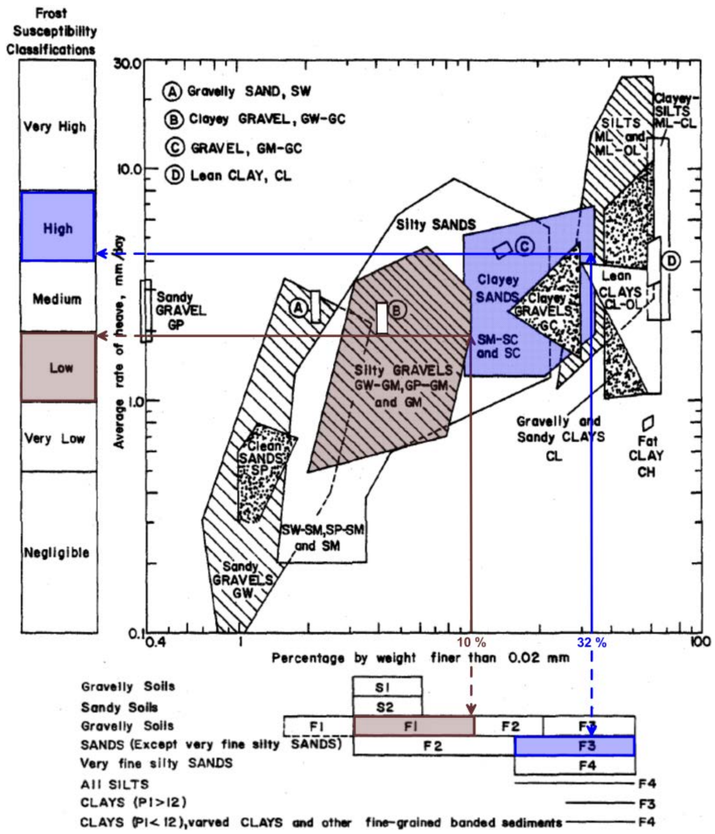


Figure 4-9: Frost susceptibility classifications for the ice-rich till (blue) and ice-poor till (brown) (after the Department of the Army and Air Force USA (1985))

The ice-rich till material can be classified as highly frost susceptible, whereas the ice-poor till material can be classified in the low frost susceptibility category due to its low clay content. Moreover, the U.S. Corps of Engineers Frost Design Soil Classification places the ice-poor till in the lowest category of frost susceptibility (F1), whereas the ice-rich till is in the F3 category. Soils are listed in four categories, F1 to F4, in approximate increasing order of frost susceptibility and loss of strength during thaw. It is expected nonetheless that the ice-rich till layer is highly frost susceptible as demonstrated by its very high water contents, which indicate the presence of massive ice in the stratum.

However, the chart presented in Figure 4-9 is open to interpretation because it is only based on the grain size distribution (the percentage by weight of soil particles finer than 0.02 mm). Konrad (2005) established a relationship between the SP at zero overburden pressure ("SP_0") and the average particle dimension of the fines fraction $d_{50}(\mathrm{FF})$. For the ice-rich till material with a $d_{50}(\mathrm{FF})$ = 7.1 µm, the value of SP_0 is estimated to be about 290 mm^2/°C*d, indicating a highly frost-susceptible soil.

For a glacial till classified as SC, Konrad (1999) found that the value of SP decreased rapidly with the overburden pressure. By applying the weight of the embankment of the ice-rich till, the value of SP drops from 95 mm^2/°C*d to 6 mm^2/°C*d (Figure 4-10) which means that the soil should not be frost susceptible once the embankment is built.

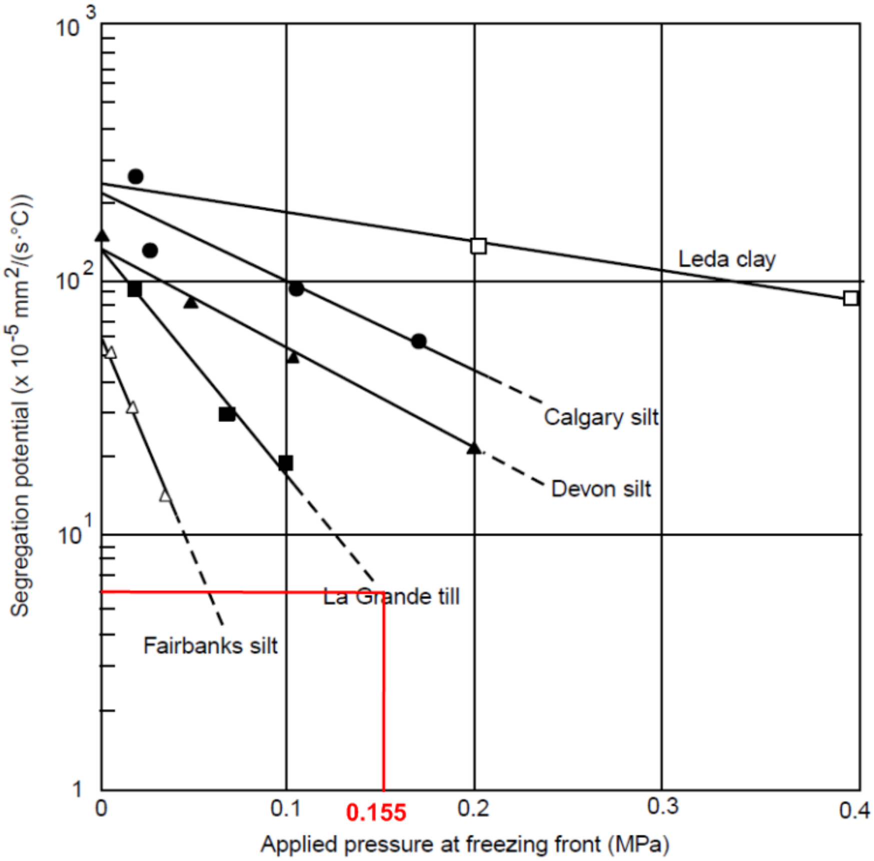


Figure 4-10: Segregation potential – pressure relationships for various soils (Konrad, 1999)

5.0 Conclusions and Recommendations

5.1 Conclusions

Comprehensive thaw settlement analyses were carried out at the Whale Tail Dike east abutment to quantify potential thaw settlements that may occur after the construction of the infrastructure. The main conclusions of this study are:

- No significant thaw settlements should occur at the west abutment since the esker is inferred to be a thaw-stable material with no presence of buried massive ice;
- The near-majority of the thaw settlements at the east abutment will occur following the thawing of the ice-rich till layer, which should take approximately 30 years under the upstream rockfill embankment of the dike;
- Significant thaw settlements could be observed upstream of the cutoff wall, where the total deformation could reach 1.1 m after 50 years of operation;

5.2 Recommendations

The study presented in this technical note lead to the following recommendations:

- In the thermal analysis report of the Whale Tail Dike, SNC-Lavalin (2018) recommended the construction of a thermal berm at the east abutment to preserve the permafrost condition close to the cutoff wall and in order to prevent or keep to the minimum possible thaw settlement within 10 to 15 metres from the cutoff wall. This design feature has been adopted as shown on the drawings;
- Removing the layer of ice rich till would minimize expected settlement to occur. It is recommended to remove completely this layer within Whale Tail Dike footprint.
- Field instrumentation, especially thermistor strings and survey monuments, should be installed in the upstream part of the dike to monitor the thermal regime of the underlying permafrost and thaw settlements;
- Field work should be carried out at the abutments to confirm that the assumptions made in that study reflect what is observed in the field.

6.0 References

AEM. (2016). *Factual Report for 2016 Field Work.* 640387-1000-4GER-0001-01, December 23rd 2016.

Andersland, O. B., & Anderson, D. M. (Eds.). (1978). *Geotechnical Engineering for Cold Regions.* McGraw-Hill, New York, N.Y., 576 p.

Andersland, O. B., & Ladanyi, B. (1994). *An introduction to frozen ground engineering.* Chapman and Hall, New York.

Andersland, O. B., & Ladanyi, B. (2004). *Frozen Ground Engineering - Second Edition.* John Wiley & Sons, 363 p.

Arenson, L. U., Springman, S. M., & Sego, D. C. (2007). The Rheology of Frozen Soils. *Applied Rheology, 17*(1), 1-14.

BGC Engineering Inc. (2014). *A21 Dike - 2014 Design Report Update.* Report No 1207002-101 for Diavik Diamond Mines Inc., November 2014.

Biyanov, G. F. (1969). *Construction of a Water Storage Dam on Permafrost; Discharge and Blocking up the River During Construction of the Uilyui Hydro-Electric Power Station.* Technical Translation (National Research Council Canada); nº NRC-TT-1353, 30 p.

Biyanov, G. F. (1970). *Experience in construction and operation of low-head dams on permafrost soils.* Hydrotechnical Construction (U.S.S.R.), No. 9, pp. 12-17, Transl. by Consultants Bureau, New York, NY, for ASCE.

Brown, W. G., & Johnston, G. H. (1970). Dikes on permafrost: predicting thaw and settlement. *Canadian Geotechnical Journal, 7*(4), 365-371.

Carslaw, H. S., & Jaeger, J. C. (1947). *Conduction of Heat in Solids.* Clarendon Press, Oxford.

Cheng, G. D., & Yang, C. S. (2006). Mechanics related with frozen ground in construction of Qinghai–Tibet railway. *Mech. Eng., 28*(3), 1-8 (in Chinese with English Abstract).

Crory, F. E. (1973). Settlement associated with the thawing of permafrost. *In Proceedings of the Second International Conference on permafrost, Yakutsk*, (pp. 599-607). Washington, D.C. National Academy of Sciences.

Das, D. M. (2011). *Principles of Foundation Engineering.* Cengage Learning.

Department of the Army and Air Force USA. (1985). *Pavement Design for Seasonal Frost Conditions. Tech. Manual TM 5-818-2/AFM 88-6, chap. 4.* Washington, D.C.: U.S. Government Printing Office.

Gluskin, Y. E., & Ziskovich, V. Y. (1973). Dam construction in the Northern USSR (in Russian). *Permafrost: USSR Contribution to the Second International Conference, Yakutsk.* Washington, D.C.: National Academy of Sciences, 1987. USA Cold Regions Research and Engineering Laboratory, Translation 438.

Gupta, R. C., Marshall, R. G., & Badke, D. (1973). Instrumentation for dikes on permafrost, Kettle Generating Station. *Canadian Geotechnical Journal, 10*(3), 410-427.

Gur'yanov, I. E. (1975). Relations of calculated and actual settlement of a thawing foundation bed. *Soil mechanics and foundation engineering, 12*(1), 29-33.

Head, K. H. (1982). *Manual of Soil Laboratory Testing: Permeability, Shear Strength and Compressibility Tests* (Vol. 2). London: Pentech Press.

Johnson, T. C., McRoberts, E., & Nixon, J. F. (1984). Design implications of subgrade thawing. In R. L. Berg, & E. A. Wright (Eds.), *Frost Action and Its Control* (pp. 54-103). New York: Technical Council on Cold Regions Engineering, ASCE.

Johnston, G. H. (1965). *Permafrost studies at the Kelsey Hydroelectric Generating Station, research and instrumentation.* Division of Building Research, National Research Council of Canada, Technical Paper No. 178.

Johnston, G. H. (1969). Dykes on permafrost, Kelsey Generating Station, Manitoba. *Canadian Geotechnical Journal, 6*(2), 139-157.

Johnston, G. H. (Ed.). (1981). *Permafrost: Engineering Design and Construction.* John Wiley & Sons, Canada Ldt., Toronto, 540 p.

Johnston, G. H., & MacPherson, J. G. (1981). Dams and reservoirs. In G. H. Johnston (Ed.), *Permafrost Engineering Design and Construction* (pp. 393-413). New York: John Wiley.

Keil, L. D., Neilsen, N. M., & Gupta, R. C. (1973). Thaw-consolidation of permafrost dyke foundation at Long Spruce generating station. *26th Canadian geotechnical conference*, (pp. 134-141). Toronto.

Kitze, F. F., & Simoni, O. W. (1972). *An earth fill dam on permafrost, Hess Creek Dam, Livengood, Alaska.* U.S. Army, CRREL, Tech. Rpt. 196, 56 p.

Konrad, J.-M. (1999). Frost susceptibility related to soil index properties. *Canadian Geotechnical Journal, 36*(3), 403-417.

Konrad, J.-M. (2005). Estimation of the segregation potential of fine-grained soils using the frost heave response of two reference soils. *Canadian Geotechnical Journal, 42*(1), 38-50.

Ladanyi, B. (1994). *La conception et la réhabilitation des infrastructures de transport en régions nordiques.* Manual prepared for the Ministry of Transportation of Québec.

Leroueil, S., Dionne, G., & Allard, M. (1991). Tassement et consolidation au dégel d'un silt argileux à Kangiqsualujjuaq. *Canadian Geotechnical Journal, 28*(5), 678-689.

Lesage, K., & Wang, B. (2008). Experimental studies of thaw consolidation of fine grained permafrost soils from the Mackenzie Valley. *GeoEdmonton'08*, (pp. 911-918). Edmonton, Canada.

Liu, Y. Z., Wu, Q. B., Zhang, J. M., & Sheng, Y. (2002). Deformation of highway roadbed on permafrost regions of the Qinghai–Tibet Plateau. *J. Glaciol. Geocryol., 24*(1), 10-15 (in Chinese with English Abstract).

Macpherson, J. G., Watson, G. H., & Koropatnick, A. (1970). Dikes on permafrost foundations in northern Manitoba. *Canadian Geotechnical Journal, 7*(4), 356-364.

Malyshev, M. V. (1966). Calculation of the settlement of foundations on thawing soil. In *Osnovaniya, Fundamenty i Mekhanika Gruntov, No. 4* (pp. 20-24).

McRoberts, E. C., Law, T. C., & Moniz, E. (1978). Thaw settlement studies in the discontinuous permafrost zone. *3rd International conference on permafrost*, (pp. 701-706). Edmonton.

McRoberts, E. C., Law, T. C., & Murray, T. (1978). Creep test on undisturbed ice-rich till. *Proc. 3rd Int. Conf. on Permafrost. 1*, pp. 539-545. Edmonton, Alberta: National Research Council of Canada.

Miller, M., & Rykaart, M. (2016). Frozen core dam performance: four years post construction. *CDA Annual Conference*, (pp. 1-11). Halifax, NS, Canada.

Miller, M., Kurylo, J. B., & Rykaart, M. (2013, April 15-17). *Mine Water Solutions.* Retrieved 11 06, 2017, from http://2013.minewatersolutions.com/wp-content/uploads/2013/05/24-M-Miller-et-al-Frozen-Dams-in-Permafrost-Regions.pdf

Morgenstern, N. R., & Nixon, J. F. (1971). One-dimensional Consolidation of Thawing Soils. *Canadian Geotechnical Journal, 8*(4), 558-565.

Morgenstern, N. R., & Smith, L. B. (1973). Thaw-Consolidation Tests on Remoulded Clays. *Canadian Geotechnical Journal, 10*(1), 25-40.

Nelson, R. A., Luscher, U., Rooney, J. W., & Stramler, A. A. (1982). Thaw strain data and thaw settlement predictions for Alaskan soils. *4th International conference on permafrost*, (pp. 912-917). Fairbanks.

Nixon, J. F. (1973). Thaw-Consolidation of Some Layered Systems. *Canadian Geotechnical Journal, 10*(4), 617-631.

Nixon, J. F., & Ladanyi, B. (1978). Thaw consolidation. In O. B. Andersland, & D. M. Anderson (Eds.), *Geotechnical Engineering for Cold Regions* (pp. 164-215). New York: McGraw-Hill.

Nixon, J. F., & Morgenstern, N. R. (1973). The Residual Stress in Thawing Soils. *Canadian Geotechnical Journal, 10*(4), 571-580.

Panday, S., & Corapcioglu, M. Y. (1995). Solution and Evaluation of Permafrost Thaw-Subsidence Model. *Journal of Engineering Mechanics, 121*(3), 460-469.

Qi, J. L., Sheng, Y., Zhang, J. M., & Wen, Z. (2007). Settlement of embankments in permafrost regions in the Qinghai–Tibet Plateau. *Nor. J. Geogr., 61*, 49-55.

Qi, J., Yao, X., & Yu, F. (2013). Consolidation of Thawing Permafrost Considering Phase Change. *KSCE Journal of Civil Engineering, 17*(6), 1293-1301.

Qi, J., Yao, X., Yu, F., & Liu, Y. (2012). Study on thaw consolidation of permafrost under roadway embankment. *Cold Regions Science and Technology, 81*, 48-54.

Saarelainen, S. (1999). Thaw-weakening of frost-susceptible subgrades. *Proceedings of the Twelfth European Conference on Soil Mechanics and Geotechnical Engineering*, *2*, pp. 1353-1358. Amsterdam, Netherlands.

Sayles, F. H. (1984). Design and performance of water-retaining embankments in permafrost. *4th Int. Conf. on Permafrost*, (pp. 31-42). Fairbanks, Alaska. Final Proc. Washington, D.C.: National Academy Press.

Sayles, F. H. (1987). *Embankment dams on permafrost: Design and Performance Summary, Bibliography and an Annoted Bibliography.* U.S. Army Cold Regions Research and Engineering Laboratory Special Report 87-11.

Semenov, N. G. (1967). *Construction of a dam in the permafrost zone.* Hydrotechnical Construction (U.S.S.R.), No. 9, pp. 14-15, Transl. by Consultants Bureau, New York, NY, for ASCE.

SNC-Lavalin. (2016a). *Permitting level engineering report for geotechnical and water management infrastructure.* 627215-1000-40ER-0004-01, February 2016.

SNC-Lavalin. (2016b). *Sensitivity analysis of seepage through the foundation of WTD.* 640387-4000-4WER-00, December 22th, 2016.

SNC-Lavalin. (2017a). *2017 Geotechnical Investigation, Factual Report.* 645003-2000-4GER-0001-00, July 31st 2017.

SNC-Lavalin. (2017b). *Construction technique optimization of WTD.* 640387-6000-4GER-0001-00, February 2017.

SNC-Lavalin. (2017c). *Preliminary design of Mammoth Dike.* 645003-3000-4GER-0001-01, August 16th 2017.

SNC-Lavalin. (2017d). *Preliminary stability analysis of WTD.* 640387-2000-4GER-0001-00, February 2017.

SNC-Lavalin. (2017e). *Thermal Analysis at Whale Tail Dike.* 640387-2000-4GER-0002-00, March 2017.

SNC-Lavalin. (2017f). *Update of the hydrogeological seepage analysis of WTD.* 645003-1000-4WER-0001-00, July 12th 2017.

SNC-Lavalin. (2017g). *Whale Tail Dike secant pile cutoff wall preliminary design.* 645003-3000-4GER-0003-PB, July 14th 2017.

SNC-Lavalin. (2018). *Thermal analysis at Whale Tail Dike.* Report No. 651298-2100-4GER-0001_PB, February 2018.

Speer, T. L., Watson, G. H., & Rowley, R. K. (1973). Effects of ground-ice variability and resulting thaw settlements on buried warm-oil pipelines. *North Am. Contrib., 2nd Int. Conf. on Permafrost*, (pp. 746-752). Yakutsk, USSR. Washington, D.C.: National Academy of Sciences.

Thornton, D. E. (1974). *Waste Impounding Embankments in Permafrost Regions: The Oxidation Pond Embankment, Inuvik, N.W.T.* Task Force on Northern Oil Development, Environmental-Social Committee, no. 74-10, pp. 159-193.

Tremblay, C., & Doré, G. (1988). Airport network and housing construction programmes in northern Quebec, Canada. *5th International conference on permafrost*, (pp. 1500-1506). Trondheim, Norway.

Trupak, N. G. (1970). *Construction of earth dams on permafrost soils.* Hydrotechnical Construction (U.S.S.R.), No. 9, pp. 8-11, Transl. by Consultants Bureau, New York, NY, for ASCE.

Tsytovich, N. A. (1958). *Bases and foundations on frozen soil.* Highway Research Board Special Report No. 58.

Tsytovich, N. A. (1965). Consolidation of thawing soils. *Proc. Int. Conf. Soil Mech. Found. Eng. 6th, Montreal, Quebec*, *I*, pp. 390-394.

Tsytovich, N. A. (1975). *The Mechanics of Frozen Ground.* (G. K. Swinzow, & G. P. Tschebotarioff, Eds.) New York: Scripta/McGraw-Hill. (In Russian: Moscow: Vysshaya Shkola Press, 1973).

Ushkalov, V. O., Pchelintsev, A. M., Yefimov, A. I., & Dementyev, A. I. (1963). Engineering Geocryological Research. *Proceedings of the Permafrost International Conference*, (pp. 400-404). 11-15 November 1963. Lafayette, Indiana.

van Everdingen, R. O. (2005). *Multi-Language Glossary of Permafrost and Related Ground-Ice Terms.* International Permafrost Association.

Wang, S., & Liu, F. (2015). A Hypoplasticity-Based Method for Estimating Thaw Consolidation of Frozen Sand. *Geotechnical and Geological Engineering, 33*(5), 1307-1320.

Wang, W., Wang, L., Yu, F., & Wang, Q. (2016). One Dimensional Thaw Consolidation Behaviors with Periodical Thermal Boundaries. *KSCE Journal of Civil Engineering, 20*(4), 1250-1258.

Watson, G. H., Rowley, R. K., & Slusarchuk, W. A. (1973). Performance of warm pipeline buried in permafrost. *2nd International conference on permafrost*, (pp. 759-766). Yakutsk, North American Contribution volume.

Watson, G. H., Slusarchuk, W. A., & Rowley, R. K. (1973). Determination of some frozen and thawed properties of permafrost soils. *Canadian geotechnical journal, 10*(4), 592-606.

Wu, Q. B., Liu, Y. Z., & Zhang, J. M. (2002). A review of recent frozen soil engineering in permafrost regions along Qinghai-Tibet Highway, China. *Permafr. Periglac. Process., 13*(3), 199-205.

Wu, Q. B., Lu, Z. J., Zhang, T. J., Ma, W., & Liu, Y. Z. (2008). Analysis of cooling effect of crushed rock-based embankment of the Qinghai-Xizang Railway. *Cold Reg. Sci. Technol., 53*(3), 271-282.

Yao, X., Qi, J., & Wu, W. (2012). Three dimensional analysis of large strain thaw consolidation in permafrost. *Acta Geotechnica, 7*(3), 193-202.

Zaretskii, Y. K. (1968). Calculation of the settlement of thawing soil. In *Osnovaniya, Fundamenty i Mekhanika Gruntov, No. 3* (pp. 3-6).

Appendix A

Laboratory test results

SNC·LAVALIN

2740, Chemin Sullivan
Val-d'Or (Qc) J9P 0B9
Téléphone : (819) 824-6894
Télécopieur : (819) 824-3762

ESSAIS SUR MATÉRIAUX
SOMMAIRE DES ESSAIS

Soumis à : M. Yohan Jalbert, ing. SNC-Lavalin inc. (Mines et Métallurgie Mondiales) Développement minier durable Montréal (Québec) H2Z 1Z3	**Dossier n°** : 645003 **Date** : 2017-06-28 **Vos références** **Référence N° :**
Entrepreneur :	**Référence N° :**
Projet : Prelim. Water Management Amaruq	**Référence N° :**
Localisation : Projet Amaruq, Nunaviq	

RENSEIGNEMENTS GÉNÉRAUX

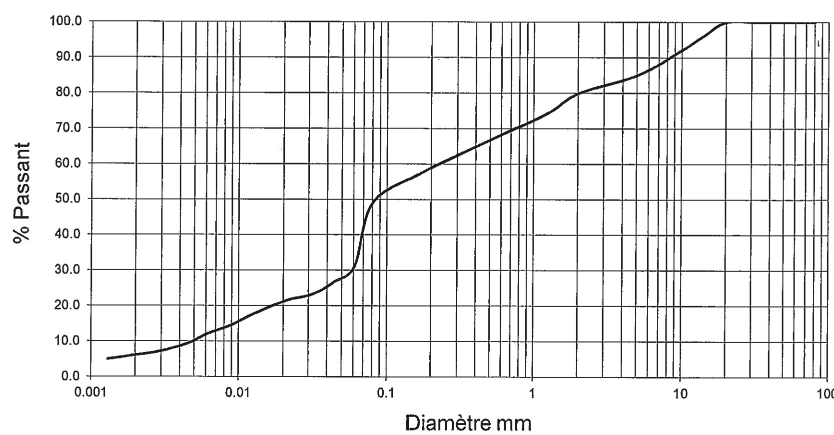
Numéro d'échantillon : 16-SG-09156	**Prélevé par** : Orbit Garand (calibre HQ3)
Type de matériaux : Till	**Source** : Sols naturels en place
Calibre du matériaux : Non spécifié	
Usage proposé : Indéterminé	
Lieu de prélèvement : Site industriel ; GT-002-02 (1,0 à 1,3 m)	
Date de prélèvement : 2017-06-13	**Date de réception :** 2017-06-16

GRANULOMÉTRIE LC 21-040

Diam. (mm)	% Passant
80	100
56	100
40	100
31.5	100
20	100
14	96
10	92
5	85
2	80
1.25	74
0.630	69
0.315	63
0.160	57
0.080	49.0
0.061	31.1
0.044	26.7
0.032	23.3
0.021	21.4
0.012	17.5
0.009	14.5
0.006	12.1
0.005	9.5
0.003	7.4
0.002	6.5
0.001	5.0

SÉDIMENTOMÉTRIE ASTM D-422

Courbe granulométrique



Résultats		
mm	Matériaux	Proportion
x > 5,0	Gravier	15
5,0 > x > 0,080	Sable	36
0,080 > x > 0,002	Silt	43
0,002 > x	Argile	6
	Total:	100

REMARQUE : Pour les calculs, la densité a été estimée à 2,7 Teneur en eau : 30,7 %
Échantillon prélevé et transmis à notre laboratoire par le client. Échantillon livré à l'état dégelé.

Vérifié par : Rémi Guillemette, ing.

Chargé de projet : Nicolas Masson

SNC·LAVALIN

3306, boul. Saint-François
Jonquière, (Québec), G7X 2W9
Téléphone : (418) 547-5716
Télécopieur : (418) 547-0374

SOLS ET GRANULATS
SOMMAIRE DES ESSAIS

Soumis à	M. Yohan Jalbert, ing. SNC-Lavalin inc (Mines et Métallurgie Mondiales) Développement minier durable Montréal, Québec, H2Z 1Z3	**Dossier N°**	645003
		Date	2017-06-27
Entrepreneur	-		
Projet	Prelim Water Management Amaruq		
Localisation	Mines et métallurgie / Développement minier durable		

RENSEIGNEMENTS GÉNÉRAUX

No échantillon	16-SG-09157	**Prélevé par**	Client
Type de matériau	Till	**Source**	Sols naturels en place
Calibre du matériau	Non spécifié	**Date de l'essai**	2017-06-20
Usage proposé	Indéterminé		
Lieu de prélèvement	Site industriel ; GT-002-003 (1,6 à 2,3 m)		
Date de prélèvement	2017-06-13	**Date de réception**	2017-06-16

GRANULOMÉTRIE (LC 21-040)

Tamis	% passant	Exigences min.	Exigences max.
112 mm	100		
80 mm	100		
56 mm	100		
40 mm	90		
31,5 mm	90		
20 mm	84		
14 mm	78		
10 mm	74		
5 mm	69		
2,5 mm	64		
1,25 mm	59		
630 µm	54		
315 µm	49		
160 µm	44		
80 µm	36,5		

MODULE DE FINESSE 3,13

ESSAIS DIVERS

ESSAIS DIVERS		Résultats	Exigences min.	Exigences max.
Teneur en eau %	(LC 21-201)	98,7		

PROCTOR MODIFIÉ (NQ 2501-255) **Méthode :**
Masse volumique sèche maximale : **kg/m³**
Teneur en eau optimale : **%**

REMARQUE * Un astérisque accompagne tout résultat individuel non conforme lorsque les exigences sont spécifiées.
Échantillon prélevé par Orbit Garand (carottier de calibre HQ3) et transmis à notre laboratoire de Val-d'Or par le client. Échantillon arrivé au laboratoire à l'état dégelé

Vérifié par : Rémi Guillemette , ing.

Chargé de projet : Nicolas Masson

SNC • LAVALIN

2740, Chemin Sullivan
Val-d'Or (Qc) J9P 0B9
Téléphone : (819) 824-6894
Télécopieur : (819) 824-3762

ESSAIS SUR MATÉRIAUX
SOMMAIRE DES ESSAIS

Soumis à	: M. Yohan Jalbert, ing. SNC-Lavalin inc. (Mines et Métallurgie Mondiales) Développement minier durable Montréal (Québec) H2Z 1Z3	**Dossier n°**	: 645003
		Date	: 2017-06-28
		Vos références	
Entrepreneur :		**Référence N° :**	
		Référence N° :	
Projet	: Prelim. Water Management Amaruq	**Référence N° :**	
Localisation	: Projet Amaruq, Nunaviq		

RENSEIGNEMENTS GÉNÉRAUX

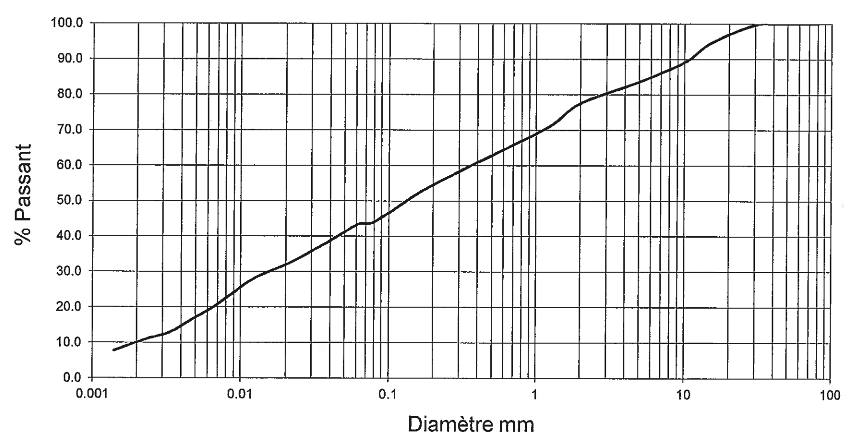
Numéro d'échantillon	: 16-SG-09158	**Prélevé par**	: Orbit Garand (calibre HQ3)
Type de matériaux	: Till	**Source**	: Sols naturels en place
Calibre du matériaux	: Non spécifié		
Usage proposé	: Indéterminé		
Lieu de prélèvement	: Site industriel ; GT-002-04 (2,5 à 3,0 m)		
Date de prélèvement	: 2017-06-13	**Date de réception**	: 2017-06-16

GRANULOMÉTRIE LC 21-040

Diam. (mm)	% Passant
80	100
56	100
40	100
31.5	100
20	97
14	94
10	89
5	84
2	78
1.25	71
0.630	65
0.315	59
0.160	52
0.080	44.1
0.062	43.5
0.045	40.0
0.032	36.5
0.021	32.4
0.012	28.2
0.009	24.0
0.006	19.8
0.005	16.3
0.003	12.8
0.002	10.9
0.001	7.7

SÉDIMENTOMÉTRIE ASTM D-422

Courbe granulométrique



Résultats		
mm	Matériaux	Proportion
x > 5,0	Gravier	16
5,0 > x > 0,080	Sable	40
0,080 > x > 0,002	Silt	34
0,002 > x	Argile	10
	Total:	100

REMARQUE : Pour les calculs, la densité a été estimée à 2,7 Teneur en eau : 147,6 %
Échantillon prélevé et transmis à notre laboratoire par le client. Échantillon livré à l'état dégelé.

Vérifié par : ______________________
Rémi Guillemette, ing.

Chargé de projet : ______________________
Nicolas Masson

SNC • LAVALIN

3306, boul. Saint-François
Jonquière, (Québec), G7X 2W9
Téléphone : (418) 547-5716
Télécopieur : (418) 547-0374

SOLS ET GRANULATS
SOMMAIRE DES ESSAIS

Soumis à	: M. Yohan Jalbert, ing. SNC-Lavalin inc (Mines et Métallurgie Mondiales) Développement minier durable Montréal, Québec, H2Z 1Z3	**Dossier N°**	: 645003
		Date	: 2017-06-27
Entrepreneur	: -		
Projet	: Prelim Water Management Amaruq		
Localisation	: Mines et métallurgie / Développement minier durable		

RENSEIGNEMENTS GÉNÉRAUX

No échantillon	: 16-SG-09159	**Prélevé par**	: Client
Type de matériau	: Till	**Source**	: Sols naturels en place
Calibre du matériau	: Non spécifié	**Date de l'essai**	: 2017-06-26
Usage proposé	: Indéterminé		
Lieu de prélèvement	: Site industriel ; GT-002-005 (3,3 à 4,0 m)		
Date de prélèvement	: 2017-06-13	**Date de réception**	: 2017-06-16

GRANULOMÉTRIE (LC 21-040)

Tamis	% passant	Exigences min.	Exigences max.
150 mm	100		
112 mm	100		
80 mm	100		
56 mm	100		
40 mm	100		
28 mm	100		
20 mm	87		
14 mm	85		
10 mm	81		
5 mm	75		
2,5 mm	69		
2,0 mm	69		
1,25 mm	63		
0,630 mm	57		
0,315 mm	52		
0,160 mm	46		
0,080 mm	37,6		

MODULE DE FINESSE 2,7

ESSAIS DIVERS

ESSAIS DIVERS		Résultats	Exigences min.	Exigences max.
Teneur en eau %	(LC 21-201)	89,5		

PROCTOR MODIFIÉ (NQ 2501-255) **Méthode :**
Masse volumique sèche maximale : **kg/m³**
Teneur en eau optimale : **%**

REMARQUE * Un astérisque accompagne tout résultat individuel non conforme lorsque les exigences sont spécifiées.
Échantillon prélevé par Orbit Garand (carottier de calibre HQ3) et transmis à notre laboratoire de Val-d'Or par le client. Échantillon arrivé au laboratoire à l'état dégelé

Vérifié par : Rémi Guillemette , ing.

Chargé de projet : Nicolas Masson

SNC·LAVALIN

2740, Chemin Sullivan
Val-d'Or (Qc) J9P 0B9
Téléphone : (819) 824-6894
Télécopieur : (819) 824-3762

ESSAIS SUR MATÉRIAUX
SOMMAIRE DES ESSAIS

Soumis à : M. Yohan Jalbert, ing.
SNC-Lavalin inc. (Mines et Métallurgie Mondiales)
Développement minier durable
Montréal (Québec) H2Z 1Z3
Entrepreneur :
Projet : Prelim. Water Management Amaruq
Localisation : Projet Amaruq, Nunaviq

Dossier n° : 645003
Date : 2017-06-28
Vos références
Référence N° :
Référence N° :
Référence N° :

RENSEIGNEMENTS GÉNÉRAUX

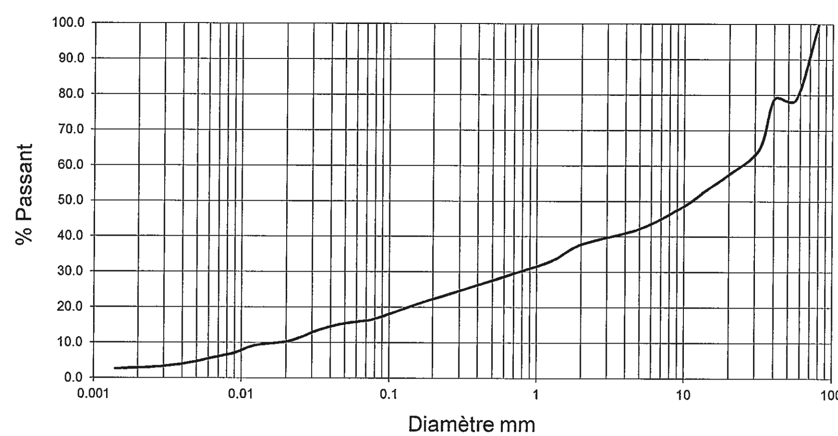
Numéro d'échantillon : 16-SG-09160
Type de matériaux : Till
Calibre du matériaux : Non spécifié
Usage proposé : Indéterminé
Lieu de prélèvement : Site industriel ; GT-005-03 (7,65 à 8,85 m)
Date de prélèvement : 2017-06-13
Prélevé par : Orbit Garand (calibre HQ3)
Source : Sols naturels en place
Date de réception : 2017-06-16

GRANULOMÉTRIE LC 21-040

Diam. (mm)	% Passant
80	100
56	79
40	79
31.5	65
20	58
14	53
10	49
5	42
2	38
1.25	33
0.630	29
0.315	25
0.160	21
0.080	16.8
0.062	16.0
0.045	15.1
0.032	13.4
0.021	10.5
0.012	9.2
0.009	7.1
0.006	5.8
0.005	4.4
0.003	3.5
0.002	3.0
0.001	2.7

SÉDIMENTOMÉTRIE ASTM D-422

Courbe granulométrique



Résultats		
mm	Matériaux	Proportion
x > 5,0	Gravier	58
5,0 > x > 0,080	Sable	25
0,080 > x > 0,002	Silt	14
0,002 > x	Argile	3
	Total:	100

REMARQUE : Pour les calculs, la densité a été estimée à 2,7 Teneur en eau : 12,3 %
Échantillon prélevé et transmis à notre laboratoire par le client. Échantillon livré à l'état dégelé.

Vérifié par : ______________________
Rémi Guillemette, ing.

Chargé de projet : ______________________
Nicolas Masson

Appendix D

Stress analyses

Title of document: **Whale Tail Dike Stress Analyses**

Client: **AGNICO EAGLE LIMITED**

Project: **Amaruq Whale Tail Dike Detailed Design**

Prepared by: Tezera Firew Azmatch/Ruijie Chen

____

Reviewed by: Getahun Haile

____

Approved by: Yohan Jalbert

____

REVISION INDEX

Revision				Pages Revised	Remarks
#	Prep.	App.	Date		
PA	TFA/RC	GH/YJ	2018-02-14		Comments for internal review
PB	TFA/RC	GH/YJ	2018-04-27		Issued for client's review

NOTICE TO READER

This document contains the expression of the professional opinion of SNC-Lavalin Inc. ("SNC-Lavalin") as to the matters set out herein, using its professional judgment and reasonable care. It is to be read in the context of the agreement dated October 4th 2017 (the "Agreement") between SNC-Lavalin and Agnico Eagle Mines Limited (the "Client") and the methodology, procedures and techniques used, SNC-Lavalin's assumptions, and the circumstances and constraints under which its mandate was performed. This document is written solely for the purpose stated in the Agreement, and for the sole and exclusive benefit of the Client, whose remedies are limited to those set out in the Agreement. This document is meant to be read as a whole, and sections or parts thereof should thus not be read or relied upon out of context.

SNC-Lavalin has, in preparing estimates, as the case may be, followed accepted methodology and procedures, and exercised due care consistent with the intended level of accuracy, using its professional judgment and reasonable care, and is thus of the opinion that there is a high probability that actual values will be consistent with the estimate(s). Unless expressly stated otherwise, assumptions, data and information supplied by, or gathered from other sources (including the Client, other consultants, testing laboratories and equipment suppliers, etc.) upon which SNC-Lavalin's opinion as set out herein are based have not been verified by SNC-Lavalin; SNC-Lavalin makes no representation as to its accuracy and disclaims all liability with respect thereto.

To the extent permitted by law, SNC-Lavalin disclaims any liability to the Client and to third parties in respect of the publication, reference, quoting, or distribution of this report or any of its contents to and reliance thereon by any third party

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

1.1 General

Agnico Eagle Mines Limited, Meadowbank Division ("AEM") is proposing to develop the Whale Tail Pit, a satellite deposit found in the Whale Tail Lake, on the Amaruq property, as a continuation of current mine operations and milling at the Meadowbank Mine. Therefore, the construction and operation of the Whale Tail Pit Project (the Project) would extend the life of the process operational facilities at Meadowbank Mine.

The Amaruq 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 (Figure 1-1). The property was acquired by AEM in April 2013 and is subject to a mineral exploration agreement with Nunavut Tunngavik Incorporated.

A permitting level study for developing the water management infrastructure for the Amaruq project was completed in 2016 (SNC-Lavalin, 2016a). As part of this water management infrastructure, there is an important dewatering dike that is required to enable mineral extraction in an open pit, located in the northern part of Whale Tail Lake. This dike, named the Whale Tail Dike (WTD), is located on a shallow plateau of the lake floor with an approximate 2 m depth of water. The WTD incorporates a cement-bentonite (CB) secant pile cutoff wall. The preliminary design for a CB secant pile cutoff wall which will be keyed into bedrock was developed in 2017 during the feasibility level study (SNC-Lavalin, 2017a). Based on the design, the lake, on the downstream side of the WTD will be dewatered and the water level on the upstream side will rise by 3.5 m during operation.

Based on the thermistor measurement results (SNC-Lavalin, 2017b) as well as the preliminary thermal modeling results of the Feasibility Study (SNC-Lavalin, 2017c), a talik is present underneath the lake at the proposed WTD area and permafrost is present at the abutments of the WTD. Construction of the WTD, downstream dewatering and upstream lake level rising will change the ground thermal regime of the area and result in permafrost thaw and settlement at the abutments.

Stress analyses were carried out for the WTD to evaluate the stress-strain response of the CB cutoff wall due to permafrost thaw and settlement.

The analyses were carried out based on available geotechnical information (SNC-Lavalin, 2017b), one dimensional thermal analyses results, as well as assumptions where applicable. Detailed design for the WTD and the CB mix are in progress; thus this report presents a summary of the preliminary analyses.

1.2 *Scope of Work*

Construction of the WTD and the operation will change both the thermal and stress conditions of the foundations. Stress analyses using SIGMA/W from GeoStudio developed by Geo-Slope was used to assess the soil-structure interaction between the WTD and the underling soil. The cutoff wall needs to be designed to sustain stresses caused by the deformation of the embankment. The scope of work for the stress analyses includes the following:

- Review available data;
- Identify at least one critical cross-section and its associated geometry to build the model;
- Evaluate the most critical condition for the stress analyses, which should be representative of spring condition, when soils have a high water content;
- Use both thaw settlement and stress analyses to assess the impact of thawing permafrost under the abutment of WTD; and
- Assess the tensile and compressive stresses in the cutoff wall induced by changes in the thermal regime of the foundation.

The results of these analyses will assist in developing appropriate design criteria for the CB mix design.

2.0 FOUNDATION CONDITIONS

The project is located in a continuous permafrost region with a mean annual air temperature of about -11°C. Talik is present underneath lakes. An aerial photo of the WTD area and the proposed WTD is provided on Figure 2-1.

The thermistor measurements from the field investigation performed in 2016 and 2017 indicate that both abutments of the WTD are frozen, whereas the section between the abutments (referred to on the figures as centre section) of the dike foundation is in talik.

The subsurface condition established by the site investigation carried out in 2017 (SNC-Lavalin, 2016c, 2017b) indicates that the foundation of the WTD mainly consists of till and cobbles overlying bedrock except the west abutment where an esker is encountered

The talik is mainly composed of sandy gravel to gravelly sand with varying content of silt ranging from silty to traces of silt. At the east abutment, the ice rich and ice poor tills are composed respectively of sand and silt with some gravel and sandy gravel with some silt. Based on the in-situ thermal profile established as part of the site investigation, the till in WTD site exists under the following three different conditions depending on the location:

- Frozen Ice-Rich Till: This unit exists at the east abutment of the WTD. It has volumetric and gravimetric moisture contents of content of about 75% and 112.4% respectively and is about 2.5

m thick. Under the base case, it is assumed that the upper part of the ice rich till (IRT) will be removed by blasting or equivalent methods leaving only about a 2 m thick ice-rich till layer. This ice rich till layer is expected to thaw since the rise of the water level impacts the thermal regime of the foundation. The thawing is expected to occur gradually over the life of the WTD leading to about 1 m of total and a significant differential settlement between the upstream and downstream sides of the CB cutoff wall.

The WTD design has considered three options to address the potential impact of the ice-rich till:

- The first option is to construct a thermal cover so that the thermal regime of the ice-rich till would not be affected by the rise in water level. The thermal cover would help to keep the ice-rich till frozen and protect it from thawing.
- The second option is not to provide the thermal cover, thus allowing the 2 m thick ice-rich till to thaw when the water level rises resulting in differential settlement.
- The third option is to remove all the ice-rich till by blasting or equivalent methods. This would avoid the issues related to thawing of the ice-rich till. But the impact of blasting on the development of cracks leading to seepage has to be investigated.

- Frozen Ice-Poor Till: This unit exists in the east abutment above and below the IRT of WTD. This unit is also expected to thaw as the rise of the water level impacts the thermal regime of the foundation. Gradual thawing of this foundation unit would occur over the life of the WTD and would contribute to the total and differential settlement that would take place.
- Talik: This unit exists between the east and west abutments of the WTD underneath the lake.

Supplementary field investigation carried out following the 2017 investigation indicates that the esker located at the west abutment is generally not ice-rich. The bedrock encountered at the WTD area varies from greywacke (sedimentary) to diorite (intrusive). The depth of bedrock below the ground surface varies from approximately 0 m near the west abutment to a maximum of approximately 10 m at the east abutment (SNC-Lavalin, 2017a).

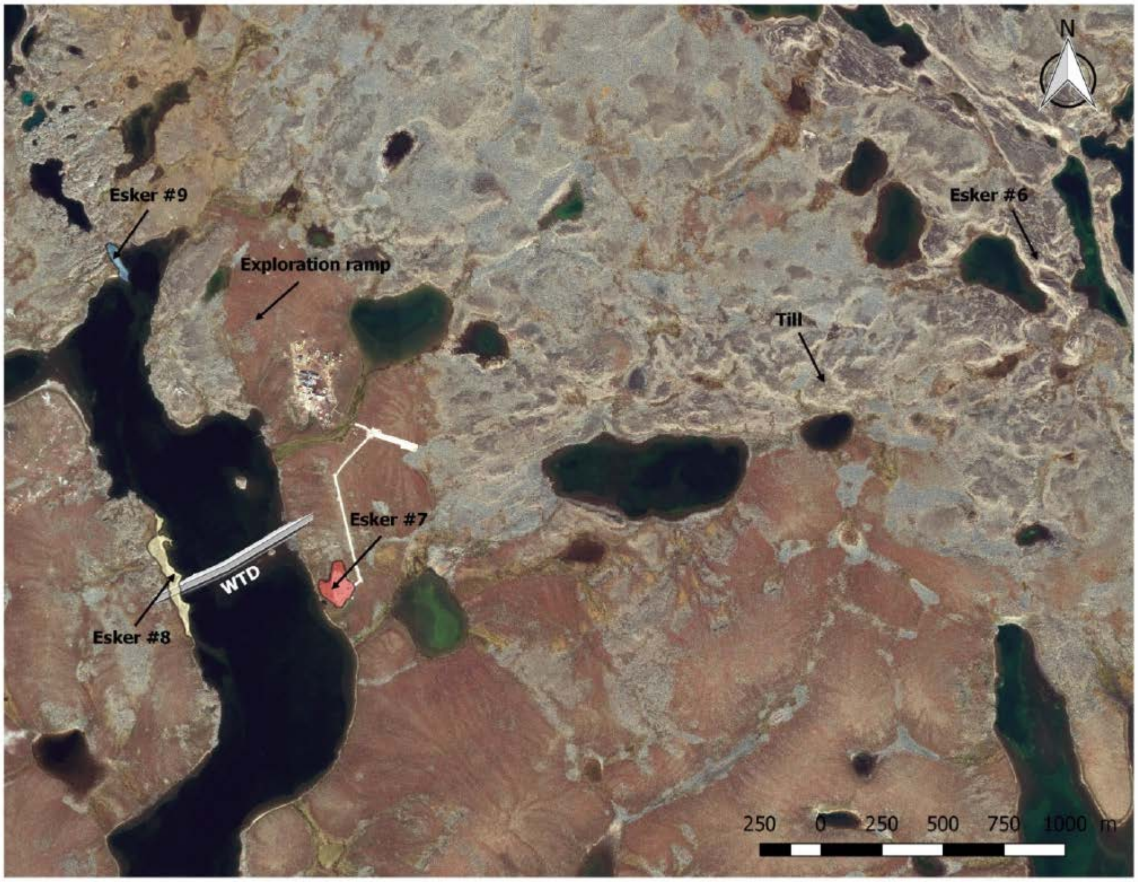


Figure 2-1: Proposed WTD and Aerial View of the Area

3.0 STRESS ANALYSES

3.1 General

Two-dimensional finite element stress-deformation analyses were carried out to assess the stresses and deformations in the CB cutoff wall of the WTD using SIGMA/W (version 8.16), a computer program developed by Geo-Slope International Inc.

3.2 Section Locations and Cases Analysed

3.2.1 Section Locations

Two cross-sections, one at the east abutment, named East Abutment Section and the other one in the middle of the WTD alignment, named Centre Section, were selected for the stress analyses as shown on Figure 3-1. A brief description of each cross-section is presented below.

East Abutment Section

This section is selected to represent the conditions at the east abutment, where for the base case, it is assumed that 2 m thick ice-rich till will be left in place. This case represents the least favorable foundation condition since thawing of the ice-rich till unit is expected to result in differential settlement of the dike due to the fact that the settlement on the upstream side will be more than that on the downstream side. The model section for the east abutment is shown on Figure 3-2.

Raising of the upstream water level would result in thawing and settlement of the foundation on the upstream side. With assumed 2 m ice-rich till left in place after excavation, a total settlement of about 1 m on the upstream side and differential settlement between upstream and downstream of the dike are expected. On the downstream side the major settlement will come following the dewatering of the lake and resulting in increase in vertical stresses. Settlement due to the weight of the dike fill will be more or less the same on both sides of the CB wall soon after construction except at the abutments. The settlements are expected to generate downdrag forces and bending stresses on the CB cutoff wall, which need to be evaluated.

As stated in Section 2 above, three options are being considered to address the issue related to the ice-rich till. The following three possible scenarios exist based on the three options:

- Scenario 1 – This scenario assumes that a thermal cover, which is designed for protection of the frozen ground from thawing, would be placed near the end of winter such that the thermal regime of the foundation and abutment of the WTD would not be affected by the rise in water level. This option has been verified by thermal modeling (SNC-Lavalin, 2018) but ignoring seepage, the effect of which we consider to be negligible on account of the fact the cover will be built in winter and expected to fully freeze.
- Scenario 2 – This scenario assumes that no thermal cover would be constructed and the 2 m ice-rich till left in place and the ice-poor till underneath will completely thaw during operation.
- Scenario 3 – This scenario involves the removal of the ice rich till by blasting or equivalent techniques.

The stress analyses were carried out only for Scenarios 1 and 2.

Centre Section

This section is representative of the conditions in the middle/centre of the WTD where talik is present. The construction of the dike and the raising of the upstream water level would not change the unfrozen foundation condition upstream of the WTD. However, the talik downstream of the WTD will freeze following dewatering of the downstream side, SNC-Lavalin (2018). The model for the central section of the WTD is presented on Figure 3-2.

The surcharge pressure due to the weight of the dike will result in settlement of the talik in this sector. These settlements are expected to cause drag stresses on the CB cutoff wall. Hence, the CB cutoff wall must accommodate the imposed stresses.

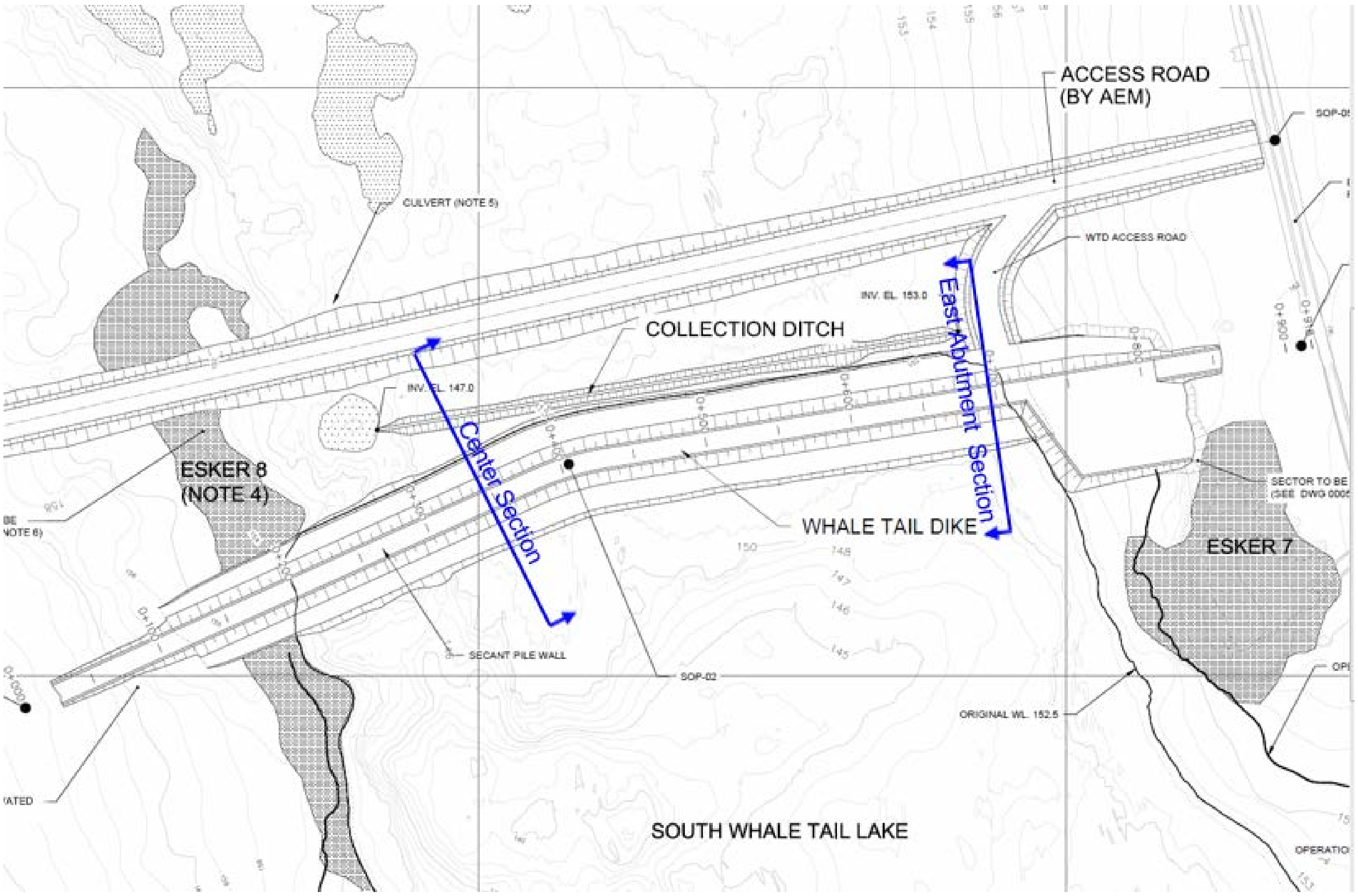
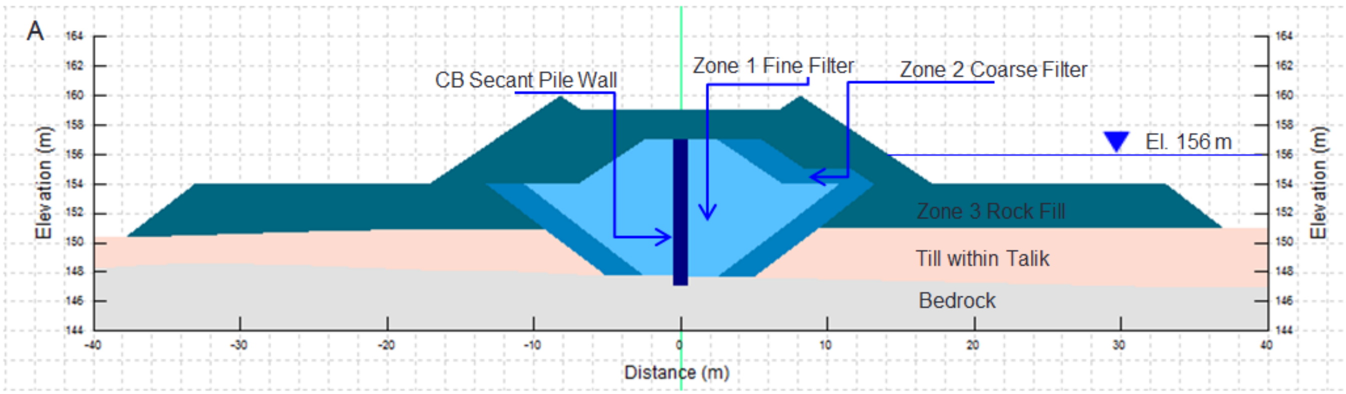


Figure 3-1: Plan View of WTD and Location of Cross-Sections

Centre Section



East Abutment Section

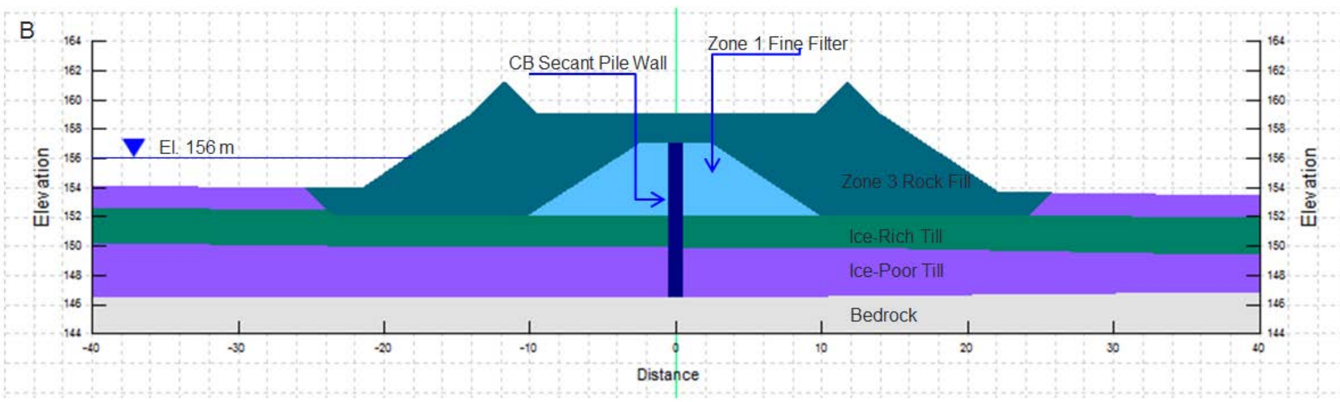


Figure 3-2: Center and East Abutment Dike Sections

3.2.2 Cases Analysed

The cases considered for center and the east abutment sections are described below.

3.2.2.1 Center Section

For the center section the WTD founded on talik was analyzed.

3.2.2.2 East Abutment

As described in section 3.2.1 of this technical note, there are three possible scenarios for the section at the east abutment. Of the three scenarios, only the two, referred to in the above section as Case 1 and Case 2, were considered in the analyses. The two scenarios considered for the east abutment are:

- Scenario 1: This scenario assumes that for the construction of the WTD, a thermal cover, which is designed for protection of the frozen ground from thawing, would be placed such that the thermal regime of the foundation and abutment of the WTD would not be affected by the rise in water level.
- Scenario 2: This scenario assumes that for the construction of the WTD, no thermal cover would be constructed and about a 2 m thick ice-rich till left in place and the ice-poor till underneath will completely thaw during operation.

3.3 *Material Parameters*

The material parameters for the stress analyses are selected based on literature review, laboratory test results (for CB samples only), and assumptions or SNC-Lavalin's experience with similar materials. Details of the parameter selection are discussed below.

3.3.1 CB Cutoff Wall

The secant pile cutoff wall would be constructed with CB mix. The design criteria for the proposed CB mix which are presented in the Whale Tail Dike Secant Pile Cutoff Wall Preliminary Design report (SNC-Lavalin, 2017a) are as follows:

- Permeability of 10^{-8} m/s or lower; and
- Strength of the CB piles at 0.5 MPa after 28 days.

During preliminary laboratory testing, it is noted that three CB mix samples, MIX 1, MIX 2 and MIX 3 with different cement-bentonite ratios (presented in Table 3.1 below) were developed in SNC-Lavalin laboratory. The unconfined compressive strength (UCS) test results for the three mix samples after 28 days of curing are summarized in Table 3-2. The results indicate that out of the three mixes, MIX 1 has the highest strength whereas MIX 3 has the lowest strength. Mix 1 (referred to herein as upper-bound CB Mix) has a USC of 644 kPa and a modulus of elasticity of 94 MPa and Mix 3 (referred to herein as lower-bound CB Mix) has a USC of 124 kPa and modulus of elasticity of 33 MPa. These two mixes were selected for the stress analyses to bracket the strength and stiffness limits.

Laboratory test data was not available to estimate the Poisson's ratio of the CB cutoff wall. A Poisson's ratio of 0.2 was selected as a base case parameter for the CB cutoff wall based on the observation that both Mix 1 and Mix 3 samples showed insignificant lateral deformation at failure under unconfined compressive strength test (as shown in Figure B.1 in Appendix B). Sensitivity analysis was also carried out with a Poisson's ratio 0.35.

Table 3-1: Cement-Bentonite Mix Ratios for Phase I Laboratory Testing

Material	Mix 1	Mix 2	Mix 3
Cement (C/W)	0.4	0.32	0.25
Bentonite (B/W, in %)	5	5	5

Note. Proportions are estimated by weight of material.
Mix 1 was proposed by Bauer and is considered as the Base Case Mix

Table 3-2: UCS Test Results for MIX 1, MIX 2 and MIX3 at 28 Days Age

CB MIX No.	28 day UCS Test Results Summary		
	Compressive Strength (kPa)	Axial Strain	Modulus of Elasticity (MPa)
MIX 1	644	0.35%	94
MIX 2	301	0.26%	59
MIX 3	124	0.19%	33

3.3.2 Ice-Rich Till

Ice-rich till exists at the eastern abutment of the dike. In a frozen state this material is a strong foundation unit and is modelled with a modulus of elasticity of 200 MPa, cohesion of 200 kPa and Poisson's ratio of 0.2 (Andersland and Ladanyi, 2003). In a thawed state, the ice-rich till unit becomes very soft due to the excess pore water generated when the ice melts. The parameters for the thawed ice-rich till were selected based on literature review (Jiang et. al., 2015) as well as model calibration based on a dike settlement of approximately 1 m which was estimated by thaw-settlement analyses. Two sets of parameters were considered for thawed ice-rich till, which are:

- ❑ Reasonable upper-bound modulus of elasticity of 450 kPa with a Poisson's ratio of 0.15, and
- ❑ Reasonable lower-bound modulus of elasticity of 250 kPa with a Poisson's ratio of 0.30.

The material parameters applied for the stress analyses are presented in Table 3-3 and discussed below.

Table 3-3: Material Parameters used for the stress-deformation analyses

Material	Unit Weight (kN/m3)	Strength Model	Friction Angle (º)	Total Cohesion, Cu (kPa)	Poisson's Ratio	Modulus of Elasticity, E (MPa)
CB Cutoff Wall	12	Elastic-Plastic	NA	62 (Note 1)	0.2	33 (Note 1)
				322 (Note 2)	0.2	94 (Note 2)
Frozen Ice Poor Till	21	Hyperbolic	0	200	0.2	200
Frozen Ice Rich Till	14	Hyperbolic	0	200	0.2	200
Thawing Ice-Poor Till	21	Elastic-Plastic	0	60	0.45	30
Thawing Ice-Rich Till	14	Elastic-Plastic	0	10	0.30	250
	14	Elastic-Plastic	0	10	0.15	450
Till within Talik	21	Elastic-Plastic	32	NA	0.45	30
Zone 1 Fine Filter	18	Hyperbolic	30	NA	0.3	30
Zone 2 Coarse Filter	18	Hyperbolic	32	NA	0.3	50
Zone 3 Rockfill	18	Hyperbolic	40	NA	0.3	100

Note 1: The strength and modulus of elasticity are based on UCS test results of MIX 3 at 28 days with an axial strain of 0.35%, representing the lower bound strength of the CB tested.

Note 2: The strength and modulus of elasticity are based on UCS test results of MIX 1 at 28 days with an axial strain of 0.35%, representing an upper bound strength of the CB tested.

3.4 *Assumptions*

Some of the assumptions made for the stress analyses have been discussed in earlier sections. The major assumptions are summarized below:

- The CB secant pile wall will be keyed into competent bedrock and foundation settlement along the CB wall is negligible.
- Part of the ice-rich till at the east abutment will be excavated prior to the placement of the dyke fill and the construction of the CB secant pile and slurry walls. The maximum thickness of ice-rich till left in place is assumed to be 2 m.
- Thawing of the foundation takes place gradually over the design life of the dike (20 years) with the thickness of the thawed layer increasing with time. However, the stress analysis assumed that thawing would take place instantaneously.

- The material parameters for the thawing ice-rich till were selected assuming that the dike settlement is primarily due to it. The ice-rich till softens when melted as a result of excess pore water generation. The thawed ice-rich till is modelled as a soft material with low modulus of elasticity.
- It is conservatively assumed that the frozen till upstream of the CB cutoff wall dike at the east abutment will be fully thawed during operation (20 years) even though the thermal analyses indicate about 30 yrs. However, it is assumed that the frozen till downstream of the CB cutoff wall at the east abutment will remain frozen.
- In the center sector, the talik upstream of the dike will remain unfrozen. The talik downstream of the dike will freeze after dewatering. However, the current analyses assumed that downstream talik will also remain unfrozen to be on the conservative side for the stress analyses.

4.0 ANALYSES RESULTS AND DISCUSSIONS

4.1 *Centre Section*

This section represents the middle WTD sector which is founded on talik. The stress-deformation analyses results for the centre section are presented on Figures A.1 to A.6 in Appendix A, and a summary of the results is provided in Table 4.1 below.

4.1.1 Lower-Bound CB Mix 3

The analyses results indicate that the maximum vertical deformation at this section is 0.05 m as shown in Figure A.1A. The analyses results indicate that the CB wall is deforming vertically down with the soil mass surrounding it.

For the CB wall with this mix (MIX 3) that has the lower-bound modulus of elasticity of 33 MPa, the predicted deflection of the CB wall is negligible (Figure A.2A). The predicted maximum compressive stress is 223 kPa (Figure A.3), which is higher than the UCS of the CB mix, which is 124 kPa. Hence, this mix does not satisfy design requirements for compressive stress. However, the CB cutoff wall is confined by adjacent fill materials and hence the actual in-situ compressive strength will be higher than the UCS. Therefore, if this mix is to be retained for economic reasons, it is recommended to carry out compressive strength tests under confined conditions.

The analyses results indicated that the tensile stresses in the CB wall are negligible. Hence, only minor surficial cracks, which would not affect the integrity of the CB secant pile wall, may develop.

The maximum normal strain, which corresponds to the maximum compressive stress is 0.85% as shown on Figure A-5C.

4.1.2 Upper-Bound CB Mix 1

The analyses results indicate that the maximum vertical deformation at the section is 0.045 m as shown in Figure A.1B. The analyses results indicate that the CB wall (which is stiffer than the surrounding soils) is resisting the vertical deformation in the soil mass around it as indicated by the local peak around the centerline of the dike as shown on Figure A.1B.

For the CB wall with this mix (MIX 1) that has the upper-bound modulus of elasticity of 94 MPa, the predicted deflection of the CB wall is negligible (Figure A.2B). The predicted maximum compressive stress is 471 kPa (Figure A.4) and is lower than the UCS of the CB mix, which is 644 kPa. This would provide a factor of safety of 1.37.

Stress-deformation analyses were also carried out by modelling the CB wall as a structural beam. The predicted deflection and stresses from these analyses were of lower magnitude as shown in Table 4.1.

The analyses results indicated that the tensile stresses in the CB wall are negligible. Hence, only minor surficial cracks, which would not affect the integrity of the CB secant pile wall, may develop.

The maximum normal strain, which corresponds to the maximum compressive stress, is 0.6% as shown on Figure A-6C.

Table 4-1: Stress Analyses Results for Center Section

Description		CB Wall with E=33 MPa, Cu=62 kPa (Lower-bound Value from Laboratory Test on Mix 3)	CB Wall with E=94 MPa, Cu=322 kPa (Upper-bound Value from Laboratory Test on Mix 1)
Max. Vertical Dike Deformation (m)	CB Wall	0.05 (Fig. A.1)	0.045 (Fig. A.1)
	Beam	0.05	0.05
Max. CB Wall Deflection (mm)	CB Wall	2.5 (Fig. A.2)	1.9 (Fig. A.2)
	Beam	2.6	2.5
Max. Tensile Stress (kPa)	CB Wall	negligible	negligible
	Beam	negligible	negligible
Max. Compressive Stress (kPa)	CB Wall	223 (Fig. A.3)	471 (Fig. A.4)
	Beam	98	133
Maximum normal strain (%)	CB Wall	0.85 (Fig. A.5)	0.6 (Fig. A.6)
	Beam	0.40	0.25

4.2 East Abutment Section

4.2.1 Scenario 1 with Thermal Cover

The results for Scenario 1 with a thermal cover are presented on Figures A.7 to A.12 in Appendix A and a summary of the results is presented in Table 4.2 below.

4.2.1.1 Lower-Bound Strength CB Mix 3

The analyses results indicate that the maximum vertical deformation in the dike is 0.02 m as shown in Figure A.7A.

For the CB wall with this mix (MIX 3) that has the lower-bound modulus of elasticity of 33 MPa, the predicted deflection of the CB cutoff wall is negligible (Figure A.8A). The predicted maximum compressive stress is 239 kPa (Figure A.9), which is higher than the UCS of the soil determined from laboratory test (i.e., 124 kPa). However, the CB cutoff wall is confined by adjacent fill materials and hence the actual compressive strength could be higher than the UCS.

The analyses results indicated that the tensile stresses in the CB wall are negligible. Hence, only minor surficial cracks, which would not affect the integrity of the CB secant pile wall, may develop.

The maximum normal strain is 0.43% (Figure A-11C), which corresponds to a compressive stress of 133 kPa (Figure A-9C). The normal strain corresponding to the maximum compressive stress (Figure A-9C) is 0.08% (Figure A-11C).

4.2.1.2 Upper-Bound Strength CB Mix 1

The analyses results indicate that the maximum vertical deformation in the dike is 0.02 m on the upstream side of the CB cutoff wall as shown on Figure A.7 B, which is essentially the same as for the CB MIX 3. The analyses results indicate that the CB wall (which is stiffer than the surrounding soils) is resisting the vertical deformation in the soil mass surrounding it as indicated by the local peak in the figure.

For the CB wall with this mix (MIX1) that has the upper-bound modulus of elasticity of 94 MPa, the predicted deflection of the CB wall is negligible (Figure A.8B). The predicted maximum compressive stress is 249 kPa (Figure A-10B) and is lower than the UCS of the soil (i.e., 644 kPa). This would provide a factor of safety of 2.59.

Stress-deformation analyses were also carried out by modelling the CB wall as a structural beam. The predicted deflection and stresses from these analyses were of lower magnitude as shown in Table 4.2.

The analyses results indicated that the tensile stresses in the CB wall are negligible. Hence, only minor surficial cracks, which would not affect the integrity of the CB secant pile wall, may develop.

The maximum normal strain, which corresponds to the maximum compressive stress, is 0.28% (Figure A-12C).

Table 4-2: Summary of Stress-Deformation Analyses Results on East Abutment Section with Thermal Cover – Scenario 1

Description		CB Wall with E=15 MPa, Cu=62 kPa (Lower-bound Value from Laboratory Test on Mix 3)	CB Wall with E=94 MPa, Cu=322 kPa (Upper-bound Value from Laboratory Test on Mix 1)
Max. Vertical Dike Deformation (m)	**CB Wall**	0.02 (Fig. A.7)	0.02 (Fig. A.7)
	Beam	0.02	0.02
Max. CB Wall Deflection (mm)	**CB Wall**	0.7 (Fig. A.8)	0.6 (Fig. A.8)
	Beam	0.7	0.6
Max. Tensile Stress (kPa)	**CB Wall**	negligible	negligible
	Beam	negligible	negligible
Max. Compressive Stress (kPa)	**CB Wall**	239 (Fig. A.9)	249 (Fig. A.10)
	Beam	125	177
Maximum normal strain (%)	**CB Wall**	0.43 (Fig. A.11)	0.28 (Fig. A.12)
	Beam	0.27	0.16

4.2.2 Scenario 2 without Thermal Cover

The stress-deformation analyses results for Scenario 2 are presented in Figures A.13 to A.24 in Appendix A, and a summary of the results is presented in Table 4.3 below.

4.2.2.1 Lower-Bound CB Mix 3

The analyses results indicate that the maximum vertical deformation in the dike is about 1 m on the upstream side of the CB cutoff wall as shown on Figure A.13 A and A.19A. There is negligible vertical dike deformation on the downstream side of the CB cutoff wall.

For the CB wall with this mix (MIX 3) that has the lower-bound modulus of elasticity of 33 MPa, and over the range of parameters considered for the ice-rich till:

- The maximum predicted deflection of the CB wall varies from 90 mm to 115 mm (Figures A.14 and A.20).

- The predicted maximum compressive stress varies from 526 kPa to 556 kPa (Figures A.15 and A.21). The predicted compressive stresses are higher than the UCS of the CB mix, which is 124 kPa.
- The predicted maximum tensile stress varies from 249 kPa to 265 kPa (Figures A.15 and A.21). At the moment, there is no tensile strength test data on this mix and hence no comparison can be made with the predicted tensile stresses.
- The maximum normal strains are about 5% in compression and vary from 1.3% to 1.7% in tension (Figures A.17 and A.23).

4.2.2.2 Upper-Bound CB Mix 1

The analyses results indicate that the maximum vertical deformation in the dike is about 1 m on the upstream side of the CB cutoff wall as shown on Figure A.13 B and Figure A.19B. There is negligible vertical dike deformation on the downstream side of the CB cutoff wall.

For the CB wall with this mix (MIX 1) that has the upper-bound modulus of elasticity of 94 MPa, and over the range of parameters considered for the ice-rich till:

- The maximum predicted deflection of the CB wall ranges from 61 mm to 82 mm (Figures A.14 and A.20).
- The predicted maximum compressive stress ranges from 352 kPa to 365 kPa (Figures A.16 and A.22). The predicted compressive stresses are lower than the UCS of the CB mix, which is 644 kPa. This would provide a factor of safety of 1.76 to 1.83.
- The predicted maximum tensile stress ranges from 263 kPa to 361 kPa (Figures A.16 and A.22). At the moment, there is no tensile strength test data on this mix and hence no comparison can be made with the predicted tensile stresses.
- The maximum normal strains are about 0.7% in tension and vary from 1% to 1.4% in compression (Figures A.18 and A.24).

The stress-deformation analyses were also carried out by modelling the CB wall as a structural beam. The predicted deflection and stresses from these analyses were of lower magnitude as shown in Table 4.3 and Figure A.25 to A.28.

Table 4-3: Summary of Stress-Deformation Analyses Results on East Abutment Section without Thermal Cover – Scenario 2

		CB Wall with E=33 MPa, Cu=62 kPa (Lower-bound Value from Laboratory Test on Mix 3)		CB Wall with E=94 MPa, Cu=322 kPa (Upper-bound Value from Laboratory Test on Mix 1)	
		Thawed Ice-rich Till modelled with		Thawed Ice-rich Till modelled with	
Description		**E=250 kPa, v = 0.30**	**E=450 kPa, v = 0.15**	**E=250 kPa, v = 0.30**	**E=450 kPa, v = 0.15**
Max. Vertical Dike Deformation (m)	**CB Wall**	1.03 (Fig. A.13)	0.94 (Fig. A.19)	1.02 (Fig. A.13)	0.93 (Fig. A.19)
	Beam	1.01 (Fig. A.25)	0.92 (Fig. A.27)	1.01 (Fig. A.26)	0.92 (Fig. A.28)
Max. CB Wall Deflection (mm)	**CB Wall**	115 (Fig. A.14)	90 (Fig. A.20)	82 (Fig. A.14)	61 (Fig. A.20)
	Beam	64 (Fig. A.25)	45 (Fig. A.27)	51 (Fig. A.26)	34 (Fig. A.28)
Max. Tensile Stress (kPa)	**CB Wall**	265 (Fig. A.15)	249 (Fig. A.21)	361 (Fig. A.16)	263 (Fig. A.22)
	Beam	109 (Fig. A.25)	57 (Fig. A.27)	255 (Fig. A.26)	175 (Fig. A.28)
Max. Compressive Stress (kPa)	**CB Wall**	526 (Fig. A.15)	556 (Fig. A.21)	365 (Fig. A.16)	352 (Fig. A.22)
	Beam	245 (Fig. A.25)	292 (Fig. A.27)	107 (Fig. A.26)	138 (Fig. A.28)
Maximum normal strain (%)	**CB Wall**	4.6 (1.7)* (Fig. A.17)	5 (1.3)* (Fig. A.23)	1.4 (0.7)* (Fig. A.18)	1 (0.7)* (Fig. A.24)
	Beam	0.4 (0.2)* (Fig. A.25)	0.40 (0) (Fig. A.27)	0.2 (0.4)* (Fig. A.26)	0.1 (0.2)* (Fig. A.28)
* Values in ( ) are maximum tensile normal strains. The values outside ( ) are maximum compressive normal strains.					

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 *Conclusions*

Stress-deformation analyses were carried out on WTD to assess the stresses and deformations in the CB cutoff wall. Two representative cross-sections were considered for the analyses – a section along the center sector of the WTD where it is founded on talik, and a section along the east abutment where it is founded on ice-rich till and ice-poor till. A range from lower bound to upper bound strength values from the UCS test results on CB mixes (MIX 1 to 3) was selected for the CB cutoff wall and ice-rich till (in the east abutment). The major conclusions from the stress analyses are as follows:

- **Center Section:** The stress analyses results for the Center Section indicate that the tensile stresses and deflection in the cutoff wall are insignificant. For this section, the analyses for the cutoff wall with CB MIX 1 which provides the upper-bound strength indicate a factor of safety of about 1.4 for compressive stresses. Whereas the analyses for the cutoff wall with CB MIX 3 which has the lower-bound strength may not meet design requirement for compressive stresses.
- **East Abutment Section:**
  - The results from Scenario 1, where thermal cover is applied for the East Abutment Section, indicate that the cutoff wall with CB MIX 1 would have very small deflection and that the factor of safety value for compressive stress is 2.59. However, for the cutoff wall with MIX 3, the predicted compressive stresses are higher than the unconfined compressive strength and hence this mix may not meet design requirements.
  - The results from Scenario 2, where no thermal cover is applied at the East Abutment Section, indicate that the cutoff wall with CB MIX 1 would have very small deflection and that the factor of safety value for the compressive stress ranges from 1.76 to 1.83. However, for the cutoff wall with MIX 3, the predicted compressive stresses are higher than the unconfined compressive strength and hence this mix may not meet design requirements.

For the East Abutment Section, it is also planned to remove the full thickness of the ice-rich till by blasting. Under such a scenario, the settlement of the foundation soils would be very small compared to the settlement in the presence of a 2 m thick ice-rich till. Hence, the stresses induced on the CB cutoff wall would also be lower.

5.2 Recommendations

The CB cutoff wall in the WTD will be confined by adjacent fill materials and hence the actual compressive strength would be higher than the strength provided by the unconfined compression test. It is recommended that compressive strength tests under the expected confining pressures within WTD be conducted to check the confined compressive strength.

It is not clear whether the cutoff wall with the current three mixes would satisfy tensile strength requirements since there are no tensile strength test data available for these mixes. It is recommended that tensile strength tests be conducted on the CB mix samples. However, this will be required only if the 2 –m thick ice-rich till is left in place without a thermal cover. The tensile strength test will not be required if the ice-rich till is fully removed by blasting or if an effective thermal cover that prevents thawing of the ice-rich till can be put in place. The tensile strength test can be carried out in either of the following ways:

- Brazilian tensile strength test

- Four-point bending test
- Triaxial test

Differential settlement due to thawing of the ice-rich till at the east abutment induces significant tensile stresses in the CB cutoff wall which may impact the integrity of the cutoff wall. Hence, it is recommended to either completely remove the ice-rich till at the east abutment or provide an effective thermal cover that avoids thawing of the ice-rich till.

The stress-deformation analyses in this report assumed a Poisson's ratio of 0.2 for the CB cutoff wall. Sensitivity analyses results using a Poisson's ratio of 0.35 (presented in Appendix B) indicated that the results from the stress-deformation analysis are not affected significantly by the value of the Poisson's ratio, except for Scenario 2 of the east abutment where thawing of the ice-rich till would result in significant deformation. Hence, it is recommended to carry out tests to determine the Poisson's ratio of the CB cutoff wall for the actual CB mix.

The stress-deformation analyses reported here-in are preliminary and were performed to assess the influence of the mix design on the behaviour of the CB cutoff wall composed of the trial CB mixes, Mix 1 and Mix 3. In order to evaluate the performance of the WTD during its operational life, a more detailed stress-deformation analysis needs to be carried out using additional laboratory test results on the CB mix to be adopted for the construction of the cutoff wall.

6.0 REFERENCES

Andersland, O.B. and Ladanyi B., 2003. Frozen Ground Engineering. John Wiley & Sons and American Society of Civil Engineers.

SNC-Lavalin, 2015. SNC-Lavalin Inc. Scoping Study Report of Whale Tail Pit Project. Infrastructures and Water Management, 627215-0000-4GER-0001, May 2015.

SNC-Lavalin, 2016. Permitting Level Engineering Report for Geotechnical and Water Management Infrastructure, 627215-10000-40ER-0004-01, February 2016.

SNC-Lavalin, 2017a. Preliminary Studies for Water Management and Geotechnical Infrastructure at Amaruq, Whale Tail Dike Secant Pile Cutoff Wall Preliminary Design, 645003-3000-4GER-0003, October, 2017

SNC-Lavalin, 2017b. 2017 Geotechnical Investigation, Factual report, July, 2017

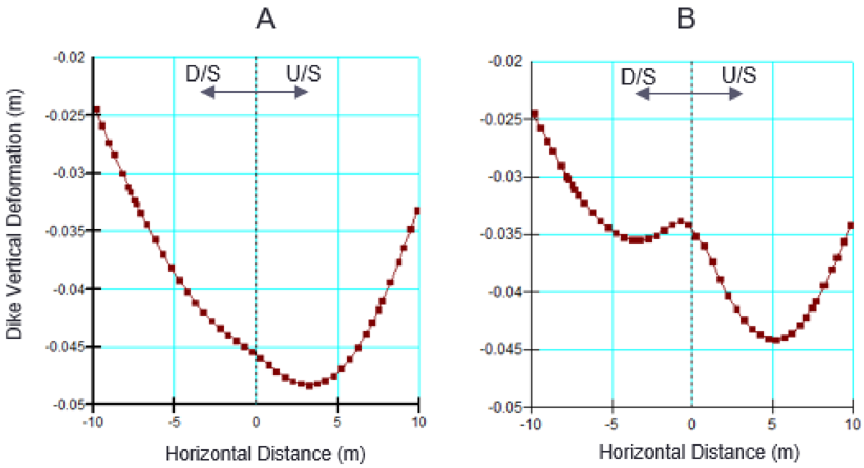
SNC-Lavalin, 2017c. Thermal Analyses at While Tail Dyke, 640387-2000-4GER-0002-00, March, 2017

SNC-Lavalin, 2018. Report 651298-2100-4GER-001. Report of Thermal analysis at Whale Tail Dike, February, 2018.

Jiang, H., Shan, W and Hu, Z., 2015, Freeway Extension Project Island Permafrost Section Foundation Deformation Characteristics, Engineering Geology for Society and Territory, V I, 243-247

APPENDIX A
Stress Analyses Results

Figure A-1: Center Section - Dike Vertical Deformation: A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa



Note: D/S = downstream, U/S = upstream

Figure A-2: Center Section - CB Cutoff Wall Deflection: A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa

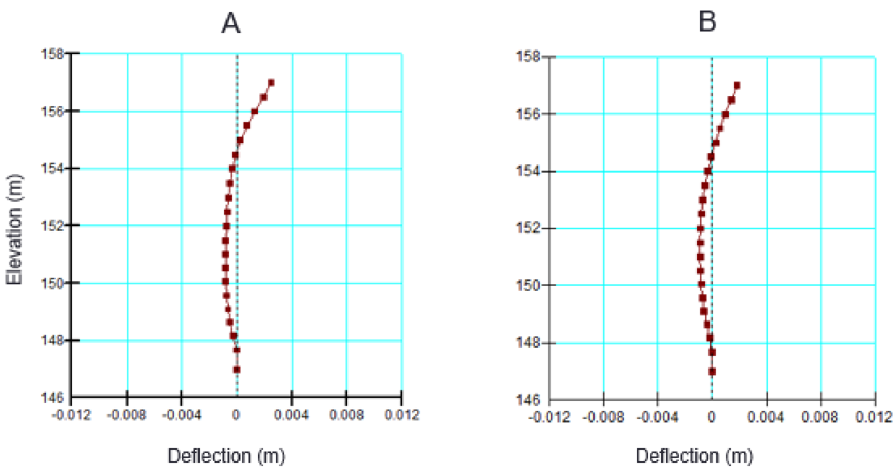


Figure A-3: Center Section - Variation of compressive stress with elevation on CB Cutoff Wall with E =33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

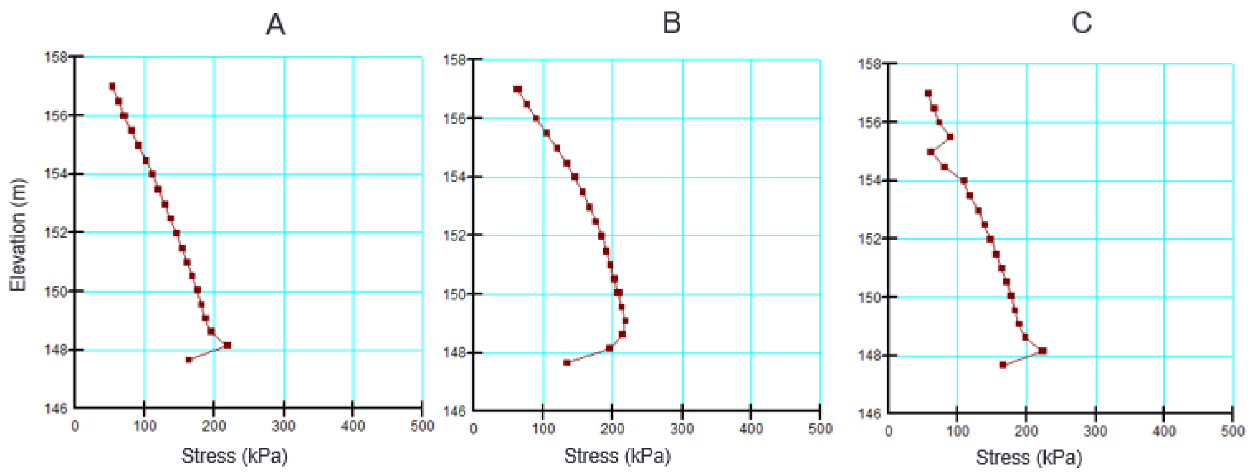


Figure A-4: Center Section - Variation of compressive stress with elevation on CB Cutoff Wall with E =94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

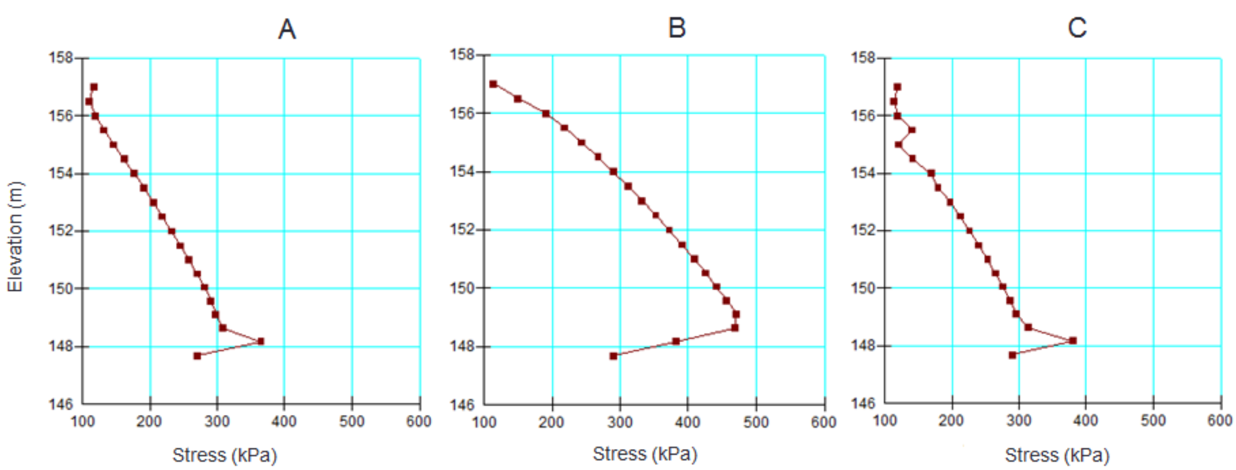


Figure A-5: Center Section - Variation of Normal Strains with elevation in CB Cutoff Wall with E =33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

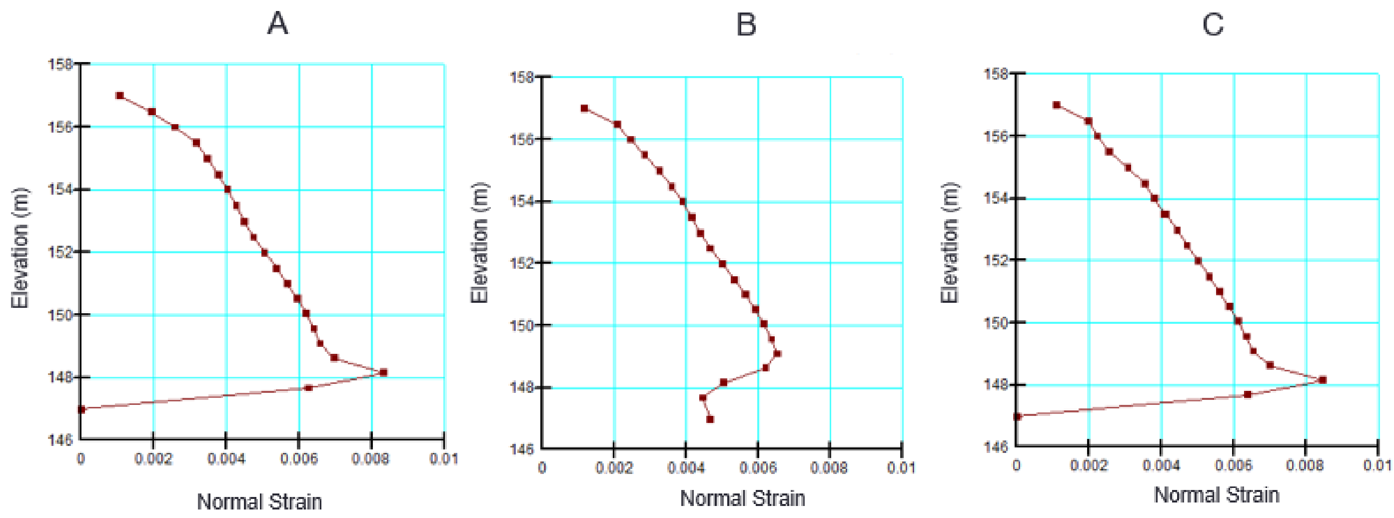


Figure A-6: Center Section - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

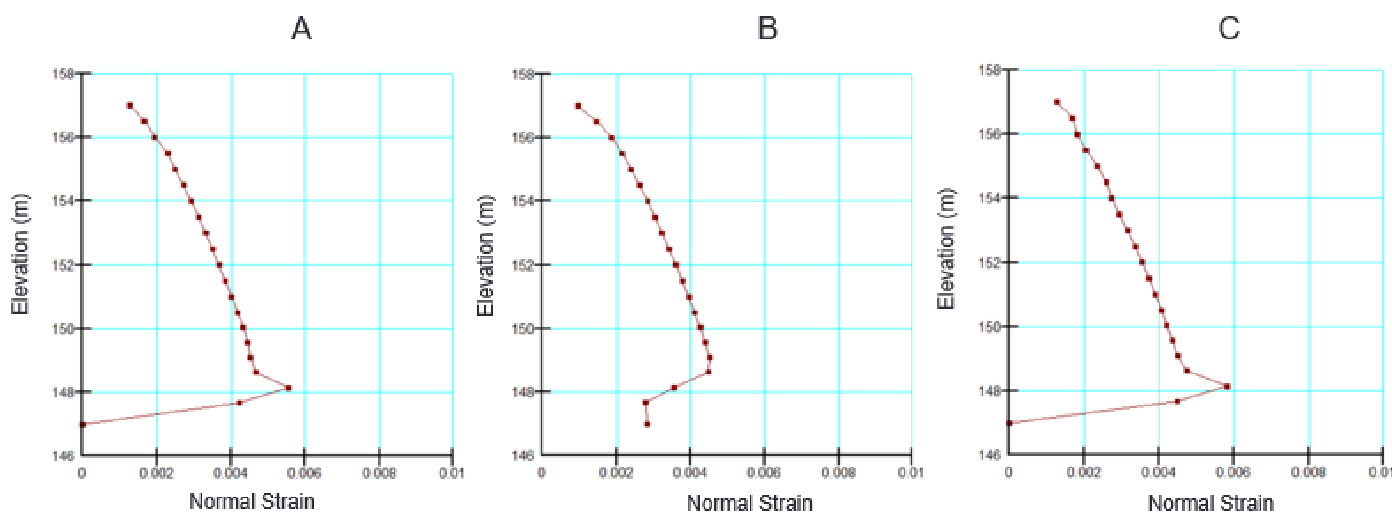
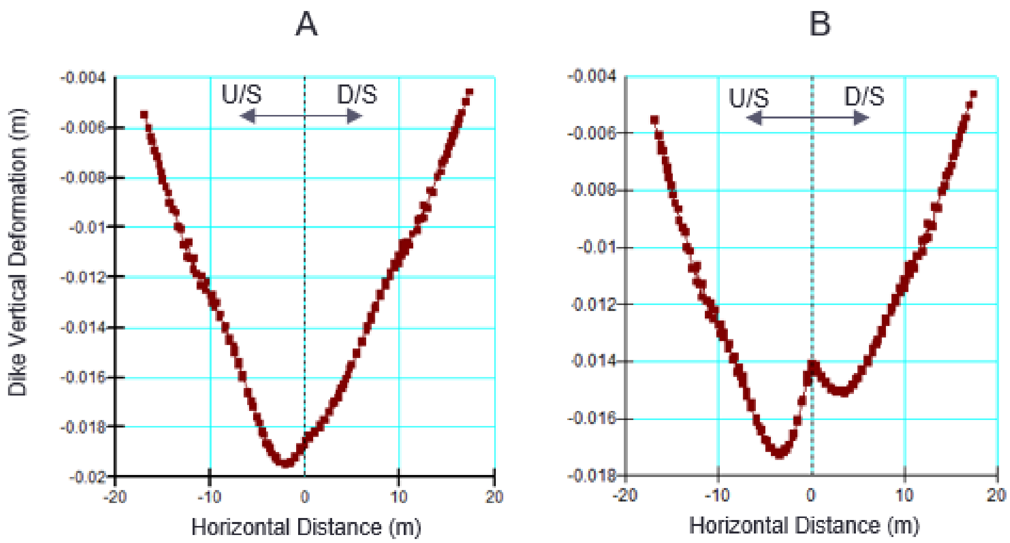


Figure A-7: East Abutment Section with Thermal Cover - Dike Vertical Deformation: A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa



Note: D/S = downstream, U/S = upstream

Figure A-8: East Abutment Section with Thermal Cover - CB Cutoff Wall Deflection: A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa

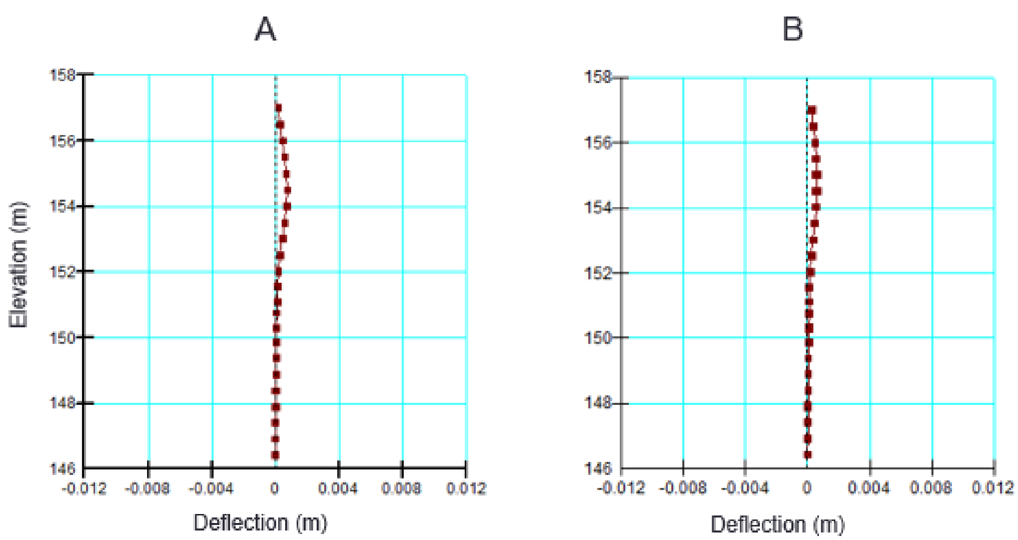


Figure A-9: East Abutment Section with Thermal Cover - Variation of compressive stress with elevation on CB Cutoff Wall with E =33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

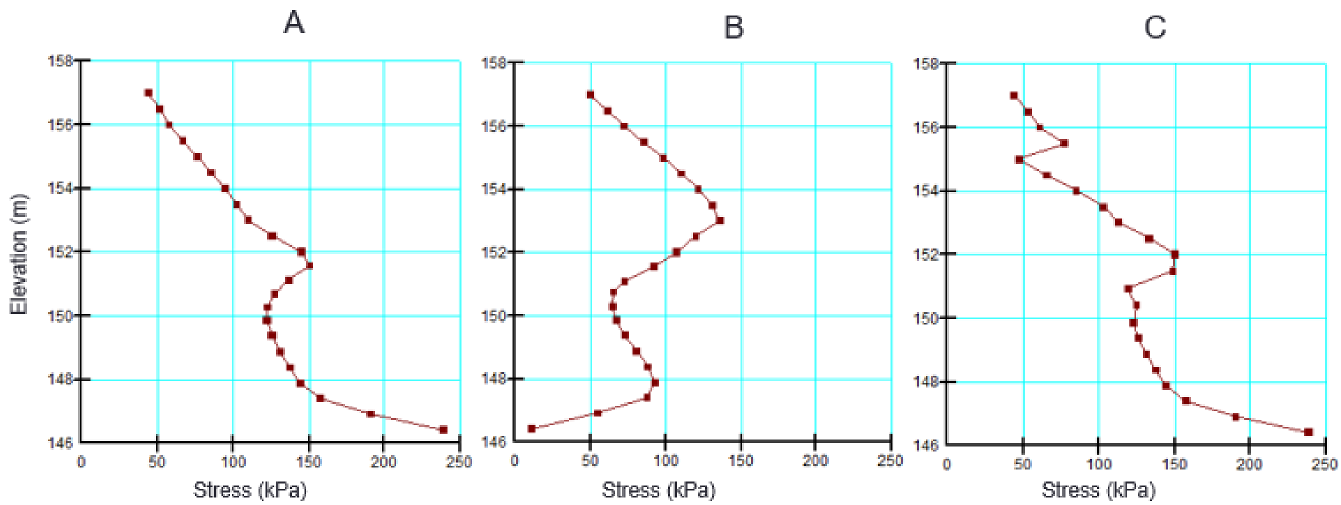


Figure A-10: East Abutment Section with Thermal Cover - Variation of compressive stress with elevation on CB Cutoff Wall with E =94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

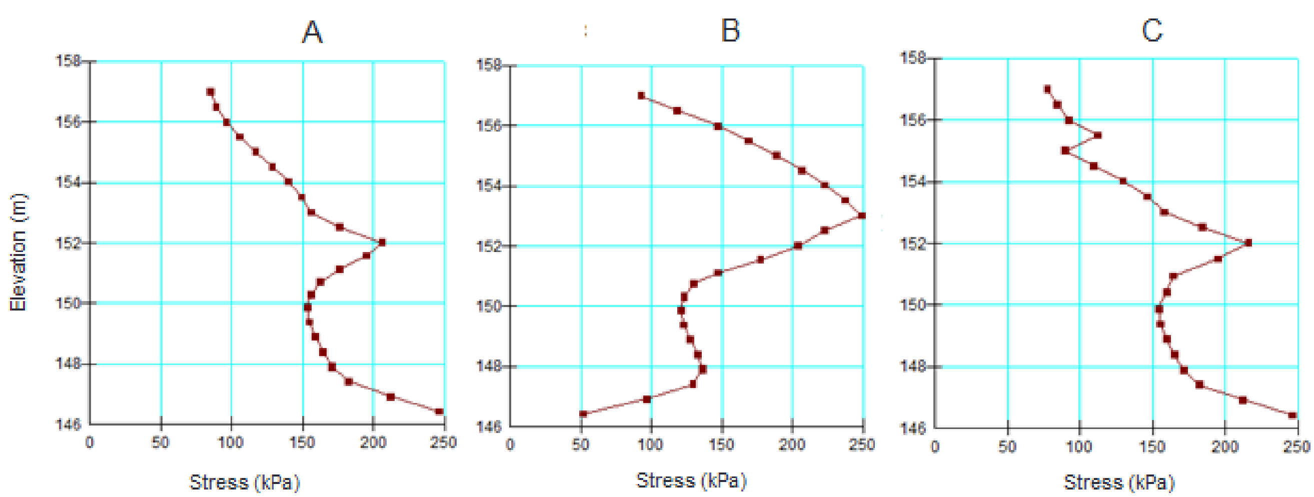


Figure A-11: East Abutment Section with Thermal Cover - Variation of Normal Strains with elevation in CB Cutoff Wall with E =33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

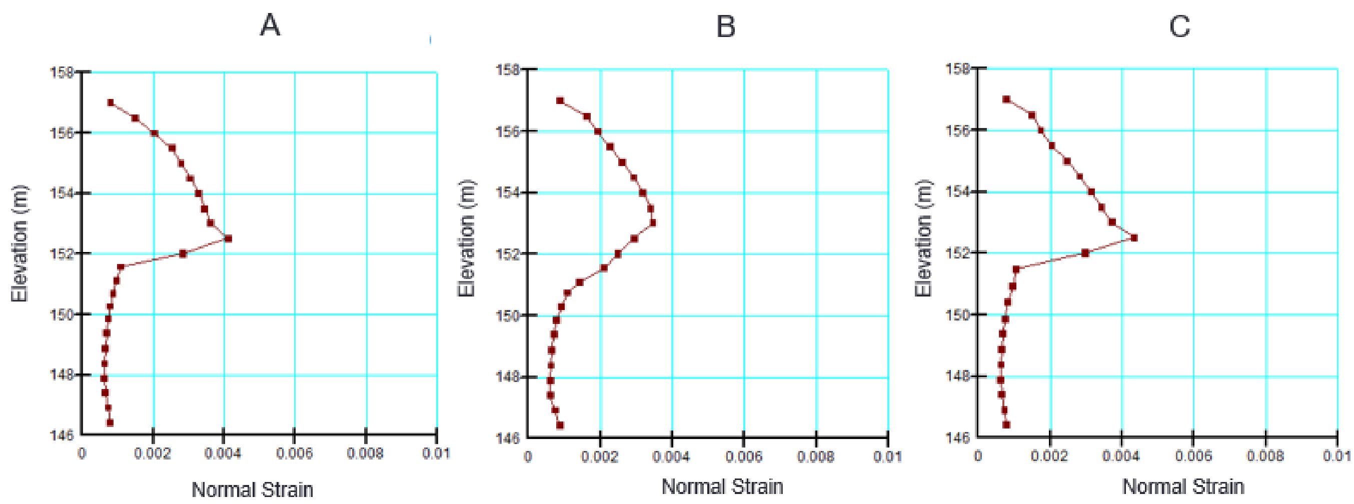


Figure A-12: East Abutment Section with Thermal Cover - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

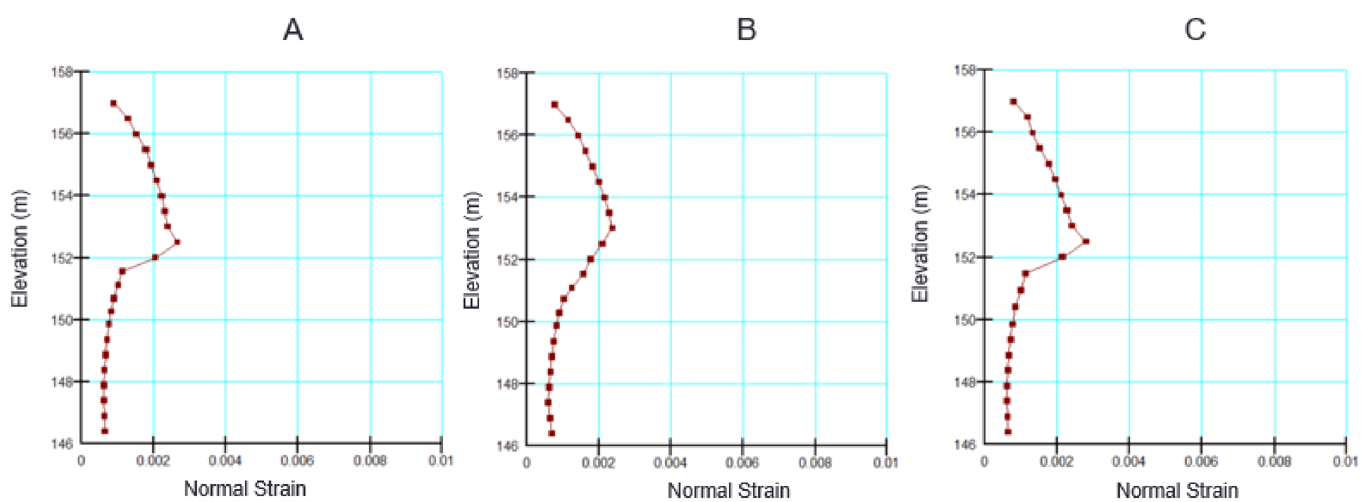
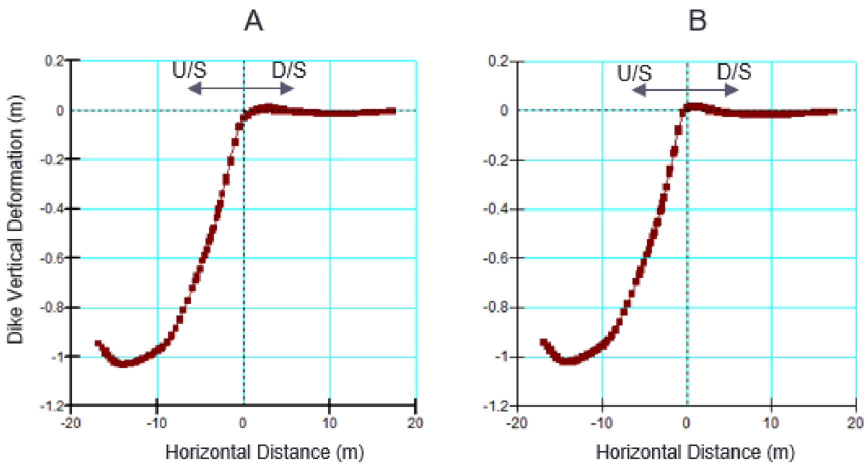


Figure A-13: East Abutment Section without Thermal Cover - Dike Vertical Deformation (with ice-rich till of E=250 kPa and v=0.30): A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa



Note: D/S = downstream, U/S = upstream

Figure A-14: East Abutment Section without Thermal Cover - CB Cutoff Wall Deflection (with ice-rich till of E=250 kPa and v=0.30): A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa

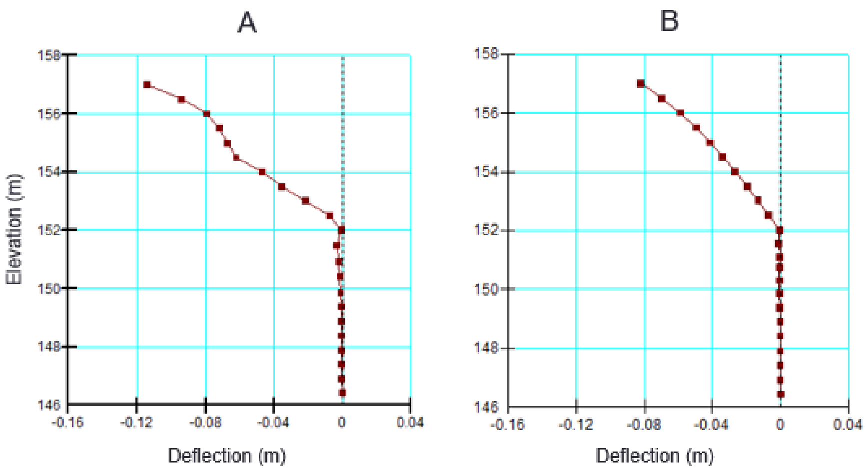
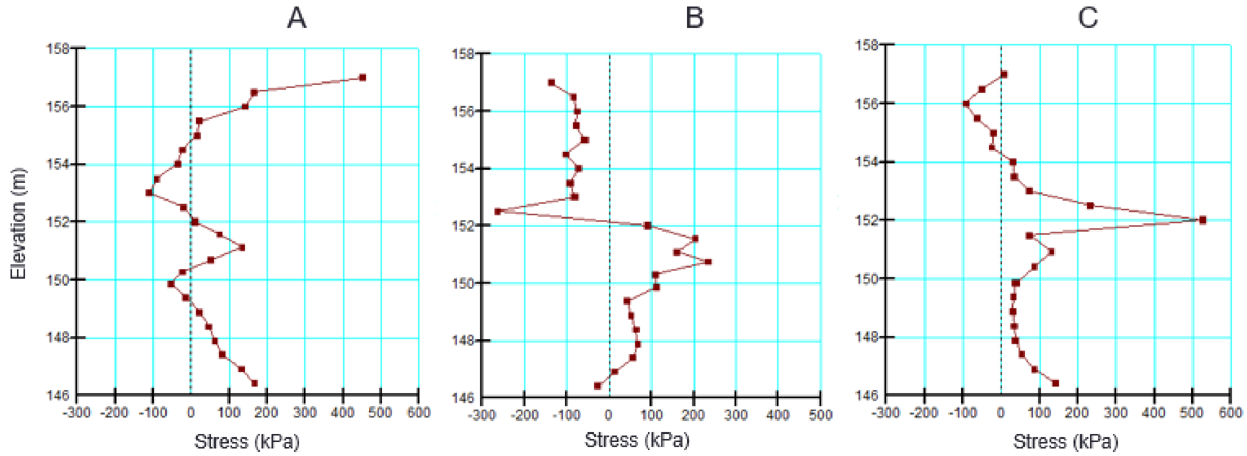
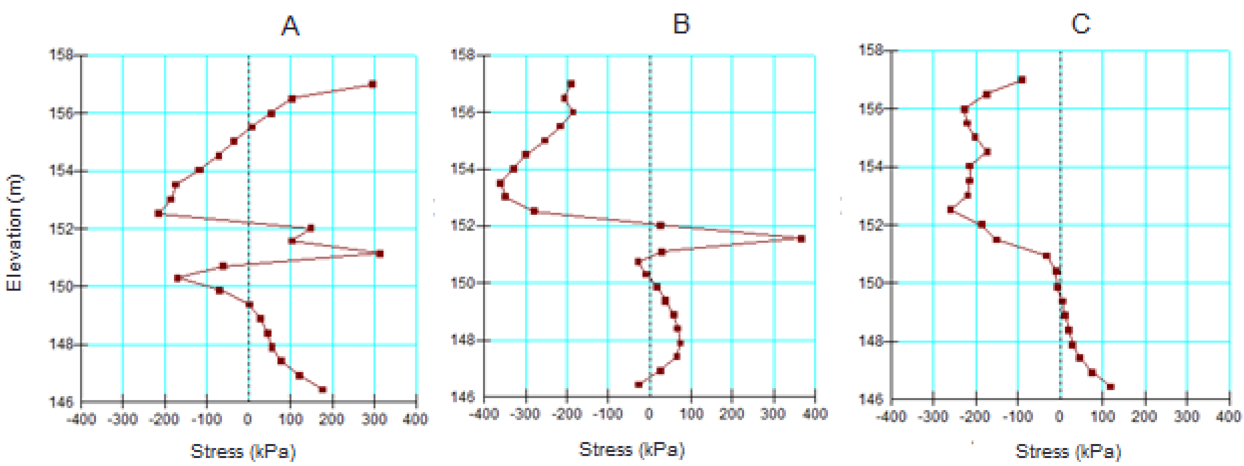


Figure A-15: East Abutment Section without Thermal Cover - Variation of tensile/compressive stress with elevation on CB Cutoff Wall with E =33 MPa & Cu = 62 kPa (with ice-rich till of E=250 kPa and v=0.30): A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.



Note: negative values indicate tensile stresses and positive values indicate compressive stresses

Figure A-16: East Abutment Section without Thermal Cover - Variation of tensile/compressive stress with elevation on CB Cutoff Wall with E =94 MPa & Cu = 322 kPa (with ice-rich till of E=250 kPa and v=0.30): A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.



Note: negative values indicate tensile stresses and positive values indicate compressive stresses

Figure A-17: East Abutment Section without Thermal Cover (with ice-rich till of E=250 kPa and v=0.30) - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

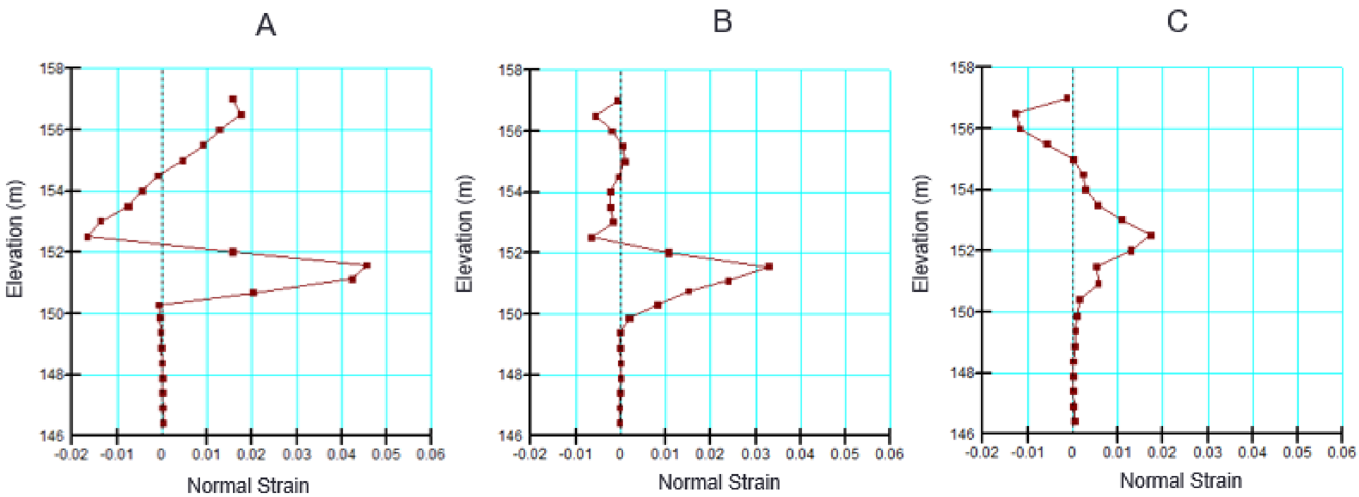


Figure A-18: East Abutment Section without Thermal Cover (with ice-rich till of E=250 kPa and v=0.30) - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

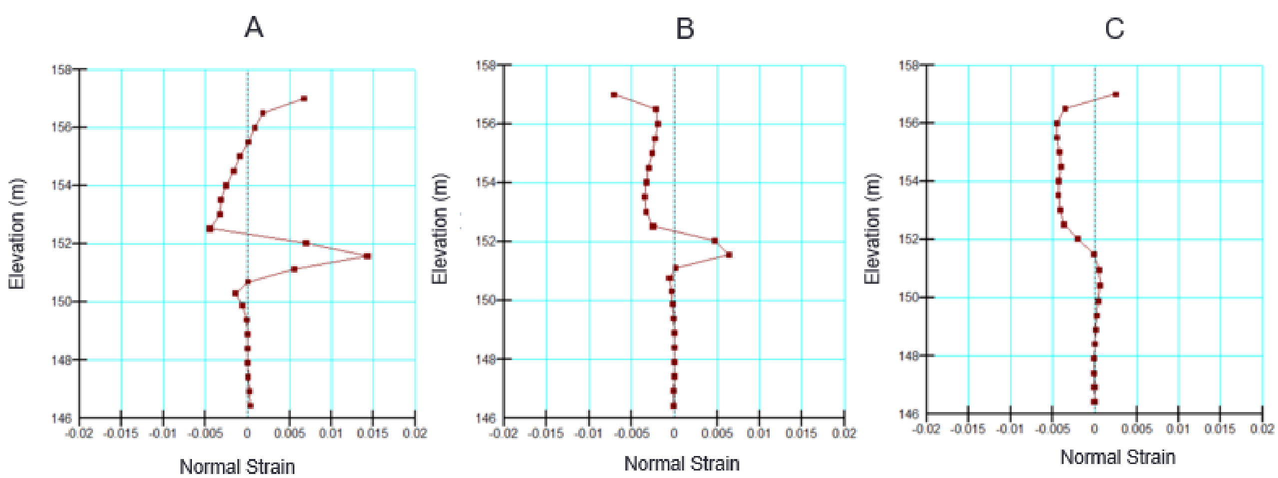
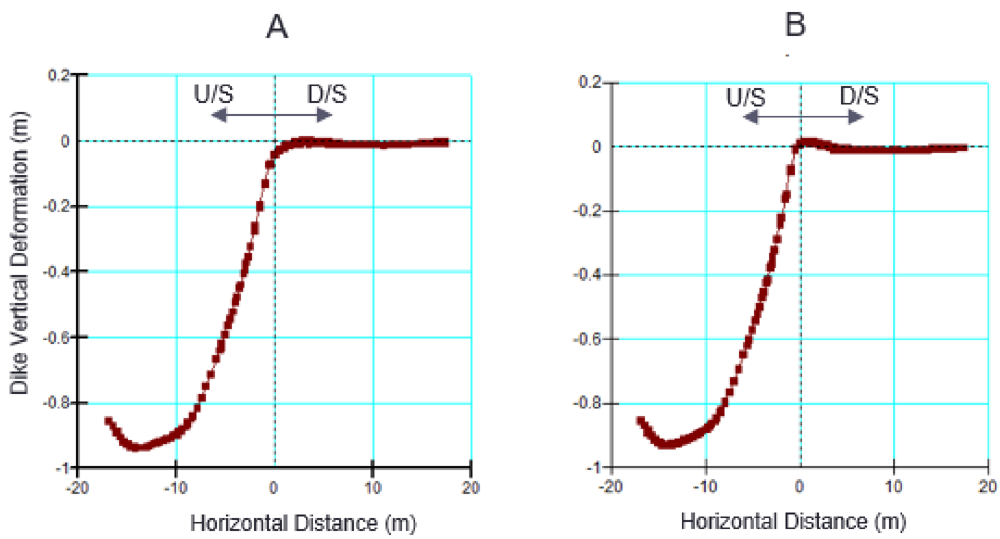


Figure A-19: East Abutment Section without Thermal Cover - Dike Vertical Deformation (with ice-rich till of E=450 kPa and v=0.15): A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa



Note: D/S = downstream, U/S = upstream

Figure A-20: East Abutment Section without Thermal Cover - CB Cutoff Wall Deflection (with ice-rich till of E=450 kPa and v=0.15): A) CB Cutoff Wall with E=33 MPa and Cu =62 kPa , B) CB Cutoff Wall with E=94 MPa and Cu =322 kPa

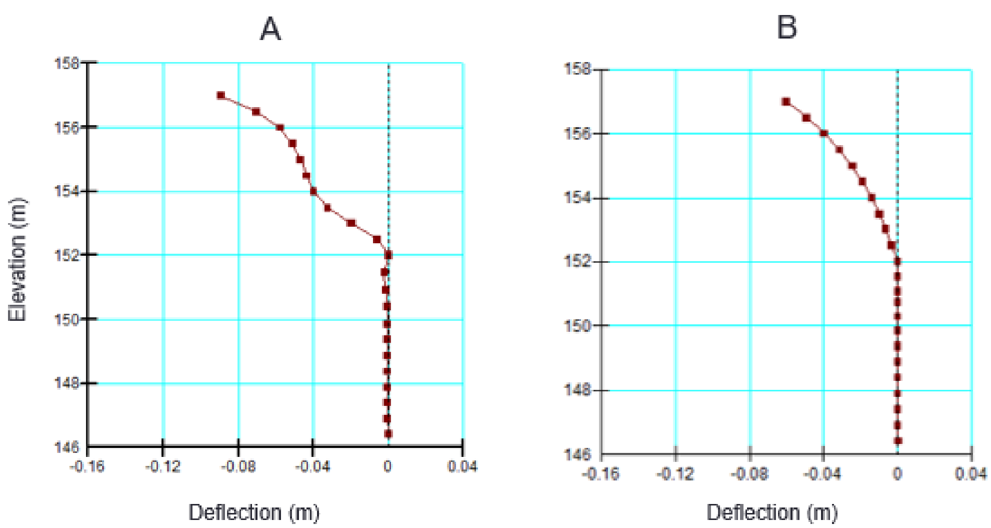
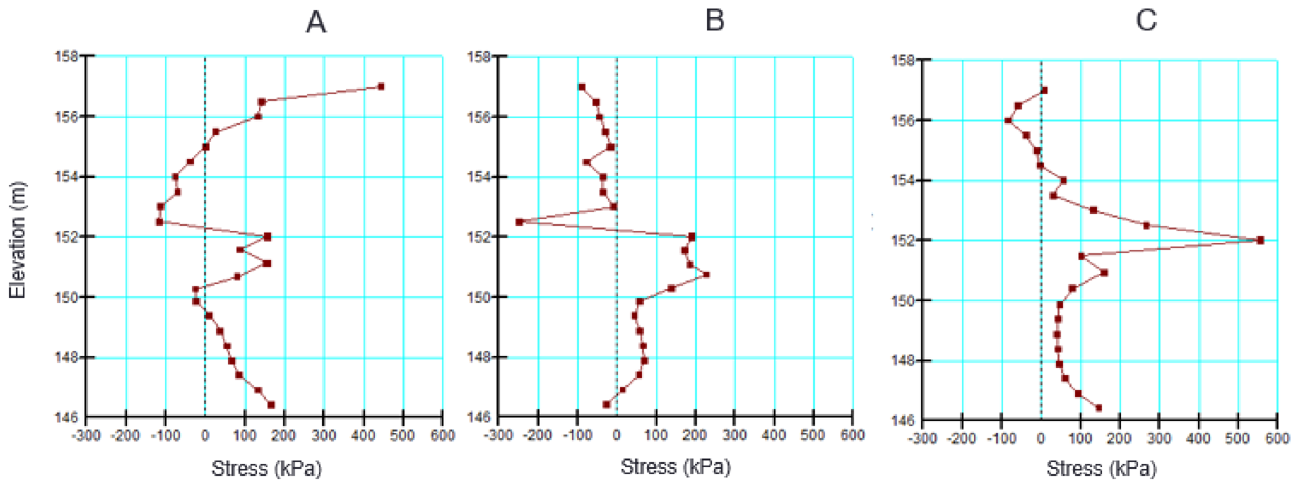
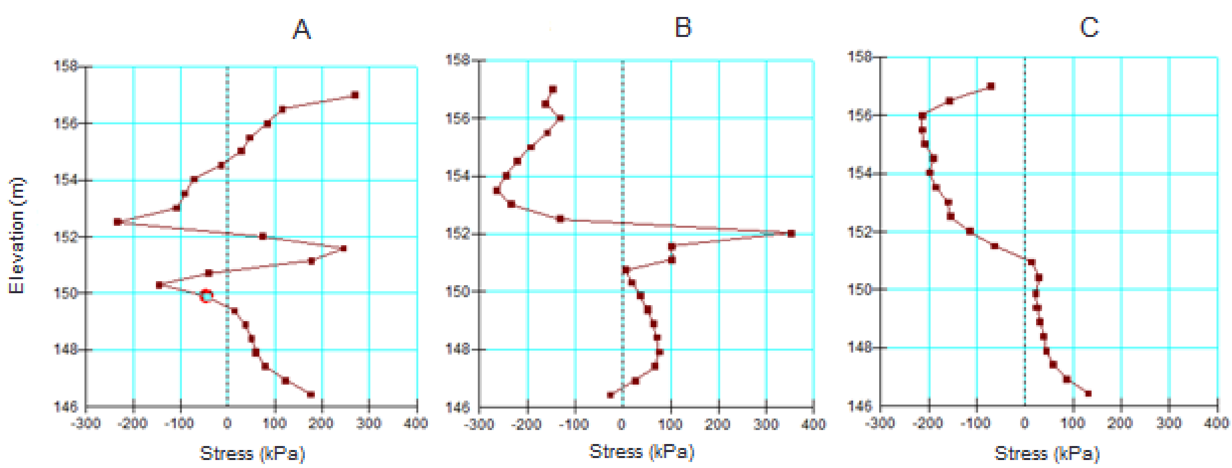


Figure A-21: East Abutment Section without Thermal Cover - Variation of tensile/compressive stress with elevation on CB Cutoff Wall with E =33 MPa & Cu = 62 kPa (with ice-rich till of E=450 kPa and v=0.15): A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.



Note: negative values indicate tensile stresses and positive values indicate compressive stresses

Figure A-22: East Abutment Section without Thermal Cover - Variation of tensile/compressive stress with elevation on CB Cutoff Wall with E =94 MPa & Cu = 322 kPa (with ice-rich till of E=450 kPa and v=0.15): A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.



Note: negative values indicate tensile stresses and positive values indicate compressive stresses

Figure A-23: East Abutment Section without Thermal Cover (with ice-rich till of E=450 kPa and v=0.15) - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 33 MPa & Cu = 62 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

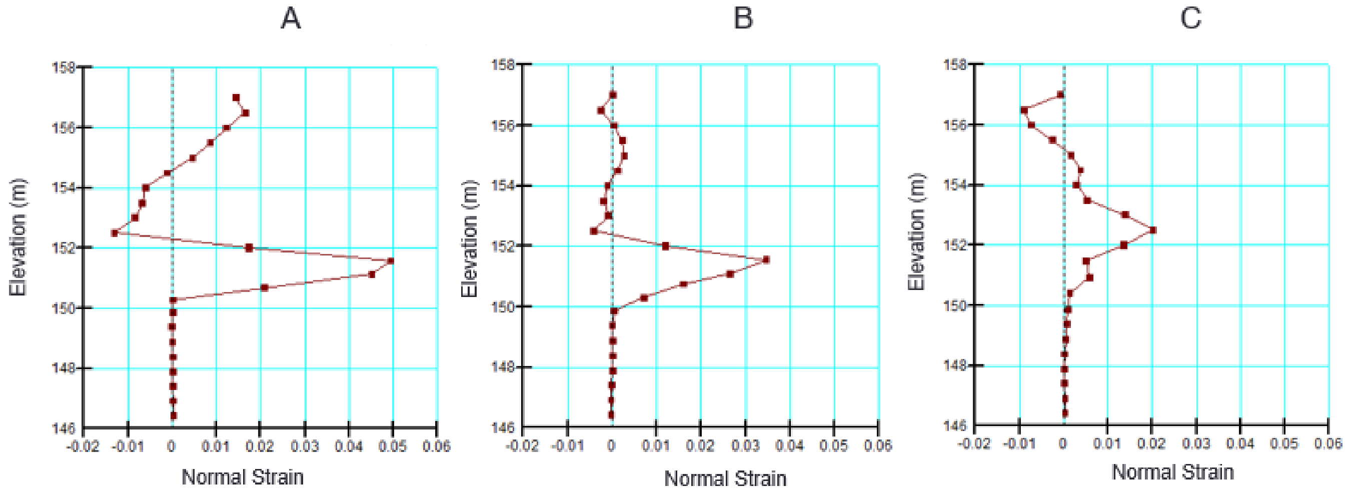


Figure A-24: East Abutment Section without Thermal Cover (with ice-rich till of E=450 kPa and v=0.15) - Variation of Normal Strains with elevation in CB Cutoff Wall with E = 94 MPa & Cu = 322 kPa: A) Upstream side of the wall, B) Along Centerline of the wall, C) Downstream side of the wall.

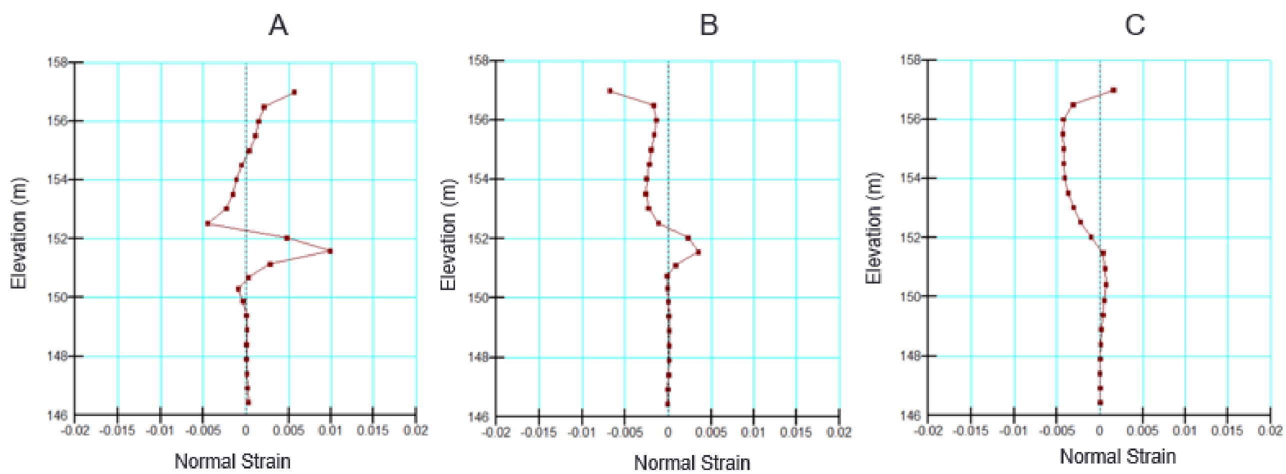


Figure A-25: East Abutment Section without Thermal Cover - Stress-Deformation Analysis Resluts with the CB wall modelled as a structural beam with E = 33 MPa (with ice-rich till of E=250 kPa and v=0.30): A) Bending moment variation with elevation, B) CB Cutoff Wall deflection, C) tensile/compressive stresses in the CB wall, D) Vertical dike deformation

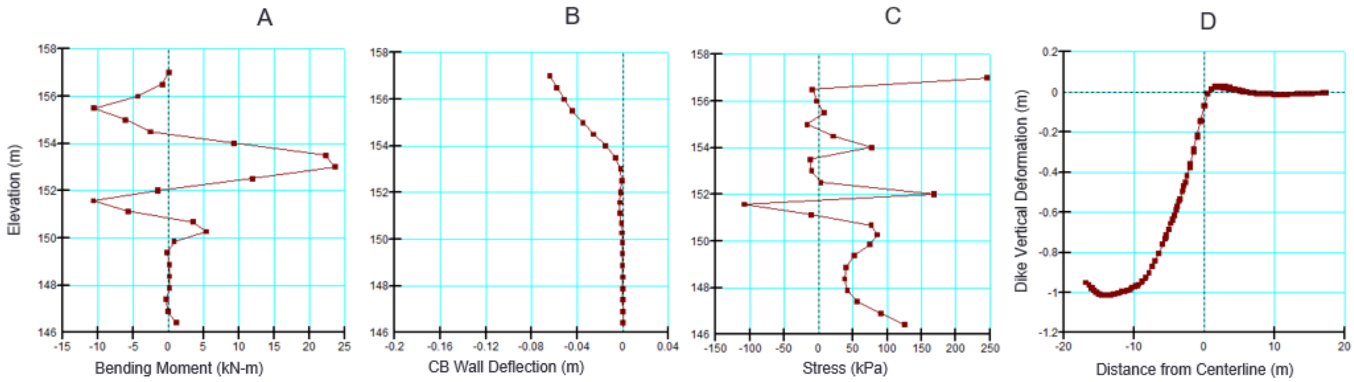


Figure A-26: East Abutment Section without Thermal Cover - Stress-Deformation Analysis Resluts with the CB wall modelled as a structural beam with E = 94 MPa (with ice-rich till of E=250 kPa and v=0.30): A) Bending moment variation with elevation, B) CB Cutoff Wall deflection, C) tensile/compressive stresses in the CB wall, D) Vertical dike deformation

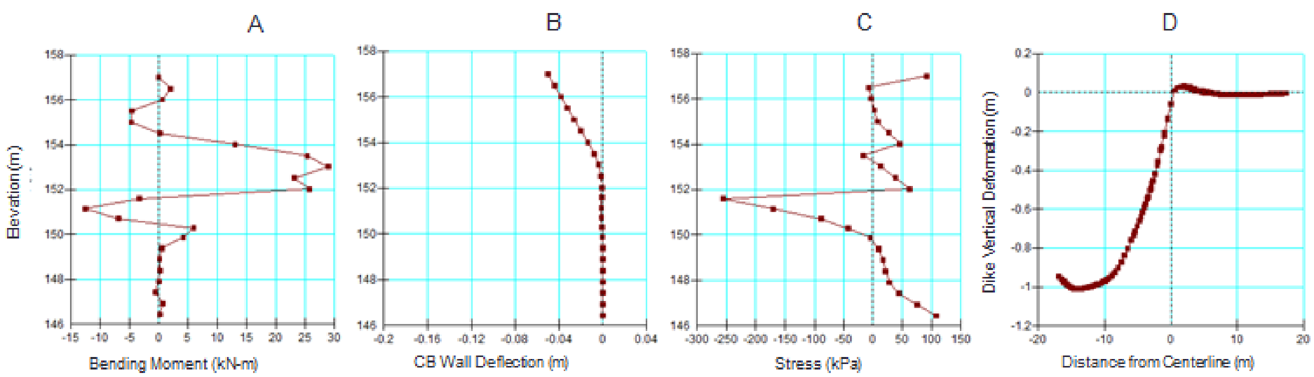


Figure A-27: East Abutment Section without Thermal Cover - Stress-Deformation Analysis Resluts with the CB wall modelled as a structural beam with E = 33 MPa (with ice-rich till of E=450 kPa and v=0.15): A) Bending moment variation with elevation, B) CB Cutoff Wall deflection, C) tensile/compressive stresses in the CB wall, D) Vertical dike deformation

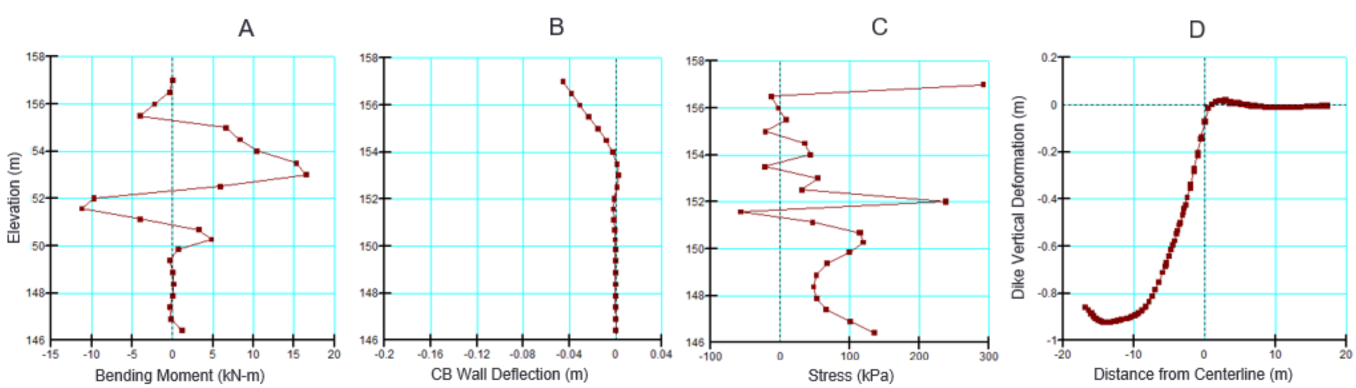
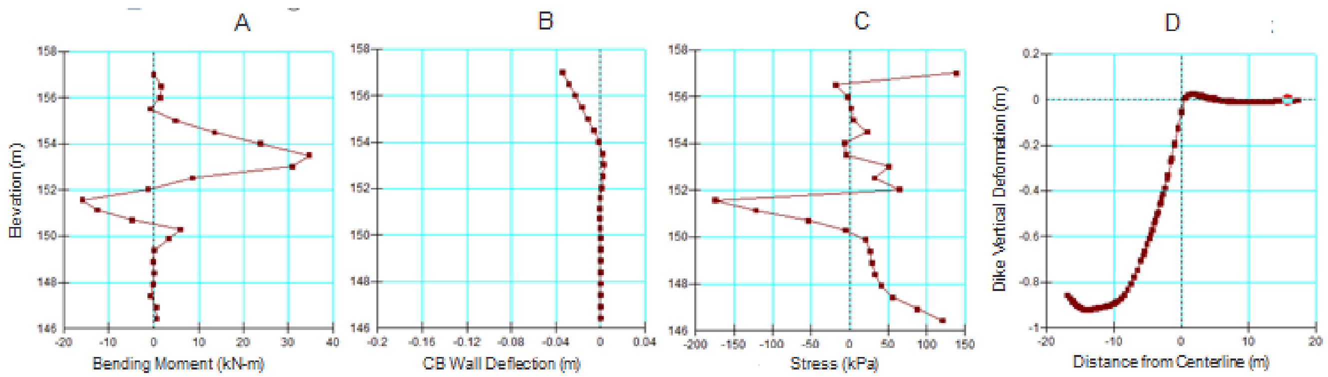


Figure A-28: East Abutment Section without Thermal Cover - Stress-Deformation Analysis Resluts with the CB wall modelled as a structural beam with E = 94 MPa (with ice-rich till of E=450 kPa and v=0.15): A) Bending moment variation with elevation, B) CB Cutoff Wall deflection, C) tensile/compressive stresses in the CB wall, D) Vertical dike deformation



APPENDIX B
Sensitivity Analysis for Poisson's Ratio of CB Cutoff Wall

Table B.1 Summary of Poisson's Ratio sensitivity analysis results for Center Sector of the Dike

Description	CB Wall with E=33MPa, Cu=62kPa (Lower-bound Value from Laboratory Test on Mix 3)	CB Wall with E=94MPa, Cu=322kPa (Upper-bound Value from Laboratory Test on Mix 1)
Max. Vertical Dike Deformation (m)	Negligible change	Negligible change
Max. CB Wall Deflection (mm)	Negligible change	Negligible change
Max. Compressive Stress (kPa)	223 [247]	471 [504]
Max. Tensile Stress (kPa)	Negligible change	Negligible change
Maximum normal strain (%)	Negligible change	Negligible change
[x] – are values with Poisson's ratio of CB cutoff wall assumed to be 0.35, and values outside bracket are for base case assumption of Poisson's ratio of CB cutoff wall as 0.20		

Table B.2 Summary of Poisson's Ratio sensitivity analysis results for East Abutment Section with Thermal Cover

Description	CB Wall with E=33MPa, Cu=62kPa (Lower-bound Value from Laboratory Test on Mix 3)	CB Wall with E=94MPa, Cu=322kPa (Upper-bound Value from Laboratory Test on Mix 1)
Max. Vertical Dike Deformation (m)	Negligible change	Negligible change
Max. CB Wall Deflection (mm)	Negligible change	Negligible change
Max. Compressive Stress (kPa)	239 [234]	249 [266]
Max. Tensile Stress (kPa)	Negligible change	Negligible change
Maximum normal strain (%)	Negligible change	Negligible change
[x] – are values with Poisson's ratio of CB cutoff wall assumed to be 0.35, and values outside bracket are for base case assumption of Poisson's ratio of CB cutoff wall as 0.20		

Table B.3 Summary of Poisson's Ratio sensitivity analysis results for East Abutment Section without Thermal Cover

Description	CB Wall with E=33 MPa, Cu=62 kPa (Lower-bound Value from Laboratory Test on Mix 3)		CB Wall with E=94 MPa, Cu=322 kPa (Upper-bound Value from Laboratory Test on Mix 1)	
	IRT with E = 250 kPa, v = 0.30	IRT with E = 450 kPa, v = 0.15	IRT with E = 250kPa, v =0.30	IRT with E = 450kPa, v = 0.15
Max. Vertical Dike Deformation (m)	Negligible change	Negligible change	Sensitivity analysis not carried out	Negligible change
Max. CB Wall Deflection (mm)	Negligible change	Negligible change	Sensitivity analysis not carried out	Negligible change
Max. Compressive Stress (kPa)	526 [567]	556 [653,625,575]	Sensitivity analysis not carried out	352 [417]
Max. Tensile Stress (kPa)	265 [361]	249 [387,326,300]	Sensitivity analysis not carried out	263 [307]
Maximum normal strain (%)	Negligible change	Negligible change	Sensitivity analysis not carried out	Negligible change

[x] – are values with Poisson's ratio of CB cutoff wall assumed to be 0.35, and values outside bracket are for base case assumption of Poisson's ratio of CB cutoff wall as 0.20

[x,y,z) are values with assumed Poisson ratio values of 0.35, 0.30, and 0.25, respectively for the CB cutoff wall.

Figure B.1: CB samples at failure under Unconfined Compressive Strength Test (a) Mix 1 and (b) Mix 3

(a) Mix 1 (b) Mix 3