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Lower Churchill Project

ENGINEERING REPORT
North Spur Stabilization Works – Design Report

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1 INTRODUCTION

1.1 GENERAL

As part of the Lower Churchill Project (LCP), the Muskrat Falls Hydroelectric Development is located on the Churchill River, about 291 km downstream of the Churchill Falls Hydroelectric Development which was developed in the early 1970's. The installed capacity of the Muskrat Falls facility will be 824 MW (4 units of 206 MW each).

Access to the hydroelectric development is from the south bank of the Churchill River, and the access to the North Spur is on the north side of the river, from the Trans Labrador Highway. The existing access road to the North Spur will be upgraded over a distance of about 3 km.

General layout of the hydroelectric development is shown in drawing MFA-SN-CD-2800-CV-GA-0002-01.

1.2 GENERAL CHARACTERISTICS OF THE NORTH SPUR

The North Spur is a deposit of marine and estuarine sediments which naturally provides a partial closure of the Churchill River valley at the Muskrat Falls site. This natural closure is one of the economically attractive features of this site and needs to be preserved for the life of the project.

This natural dam is about 1 kilometre long between the rock knoll in the south and the kettle lakes in the north, which represent natural boundaries of the North Spur, in terms of both seepage and stability. Main features of the North Spur are presented in an aerial photo taken in 1988, Figure 1-1.

Figure 1-1 : North Spur – 1988 Aerial Photo

The early studies for the Muskrat Falls site recognized the importance of the North Spur as part of the reservoir retention works. A major slide on the downstream face of the Spur, in November 1978 (Figure 1-1), revealed the fragility of this natural deposit and its susceptibility to groundwater level variations and toe erosion from ice accumulation in the bay downstream. Preserving the integrity of the Spur is fundamental to the viability of the project and this factor has been included from the outset. The studies and evaluation of the stability and integrity of this natural dam during the stabilization works, and during and after reservoir impoundment is presented in this report.

1.3 REPORT OBJECTIVES

This engineering report deals fundamentally with the main geological and geotechnical aspects of the North Spur.

The Muskrat Falls hydroelectric project will affect the stability and integrity of the North Spur. The impact of the new development on the Spur has been studied and the appropriate stabilization measures have been developed and are presented in detail in the subsequent sections.

The main objectives of this report are:

- Gather the main findings of the studies and geotechnical investigations performed in order to prepare the documents for bidding;
- Present the summary of the existing data, their interpretation and the design criteria adopted for the design of the stabilization works,
- Present additional information for use in the understanding and application of the drawings and technical specifications;
- Help inspectors to understand the quality control measures to be undertaken during construction;
- Provide information necessary for the inspection and maintenance of the structures when in operation.

This document will help the technical staff become familiar with the design criteria for the structures and provides information about the considerations that led to the final design. The document will also be a guide to the inspection and monitoring by staff during construction and operation.

This document does not include all the requirements contained in the technical specifications and drawings, although it highlights several important elements of those documents. This document is to be used to facilitate the interpretation of the impact of changed conditions on the design to assist the project team in adapting the design to actual conditions encountered, should they vary from the design assumptions.

1.4 REPORT ORGANISATION

Section 2 of the report presents a review of general data, including site description, geography, regional geology, seismicity and climatology.

A summary of the field work and engineering studies performed is presented in Section 3 in chronological order.

An interpretation of geological and hydrogeological conditions of the North Spur is presented in Section 4. Soil stratigraphy based on available data from physical, mechanical and hydrogeological properties of the soils are presented. Existing hydrogeological conditions such as aquifers and groundwater regimes are also presented in this section. The impacts of the project on the stability of the North Spur and the consequences of reservoir impoundment and potential problems are also identified in this section.

Section 5 describes the proposed stabilization works. The issues identified in previous sections are addressed. The requirements of the design criteria are established and the solutions are presented.

Section 6 includes construction provisions for the stabilization works.

Section 7 presents the appurtenant structures such as roads and laydown areas.

Relevant references are provided at the end of this document for more detailed information.

2 GENERAL SITE DATA

2.1 SITE DESCRIPTION

The Muskrat Falls site is located on the Churchill River, about 30 km upstream from Happy Valley-Goose Bay in Labrador. The two falls, about 1 km apart, cause a drop of about 14 m in the Churchill River water level from 17 m at the upstream side to 3 m at the downstream side of the falls. A rock knoll with a top elevation of 142 m is located at the North side of the falls and is connected to the left bank of the river by a spur of land which is referred to as the “North Spur”.

The Churchill River actually flows in a bedrock channel south of the rock knoll. Past continental glaciations followed by glacial, fluvio-glacial and marine activities formed the overburden in the Muskrat Falls area.

2.2 GEOGRAPHY

2.2.1 TOPOGRAPHY

The Churchill River valley, for the most part, runs through a wide valley characterized by extensive surface terraces of overburden material. The overburden consists of sands overlying silty sands, silty and clayey soils. The sediments also extend to the north of a rock knoll forming a narrow spur of land. Although these infill sediments form a natural dam that has effectively changed the river to flow to the south of the rock knoll, erosion and landslides within the spur has been an on-going feature of the area and are still active.

The main terrace of the Spur is at an elevation of about 60 m and represents an ice-contact stratified drift deposit. The deposit is characterized by the presence of kettles, local slumping and abrupt stratigraphy change within the first 50 m to 70 m of depth.

At higher elevations in the valley (towards north or south), there is initially a succession of smaller terraces and then glacial till with locally exposed bedrock.

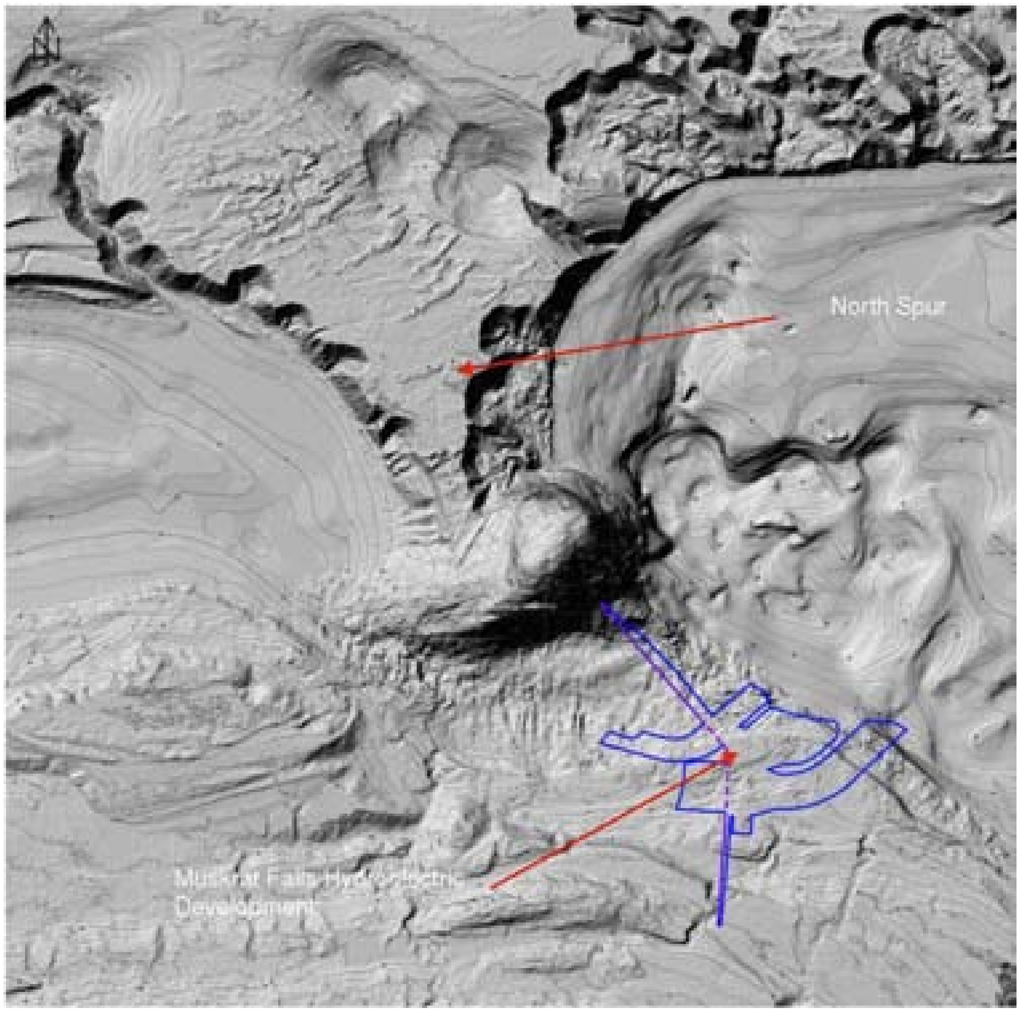
Above an elevation of approximately 250 m, the land primarily consists of Precambrian age bedrock with various types of glacial deposits, the most notable being glacial till drumlins just to the north of the Muskrat Falls.

Minor topographic features, which exist on the main valley terrace, include sand dunes and kettles, and perhaps even strand lines. In some areas, the relative surface tranquility is broken by mass wasting features which, on a few occasions, may approach 1 km^2.

The boundaries of the North Spur are defined by the rock knoll in the south, the three kettle lakes in the north, and the Churchill River in the west and east. The crest of the North Spur in the north-south direction is about 1,000 m long and in the kettle lakes area the crest of the North Spur is about 1,000 m wide. In the south limit, close to the rock knoll, the width of the crest of the North Spur reduces to a minimum of 80 m.

A LiDAR (Light Detection and Ranging) image of the Muskrat Falls site and the footprint of the related structures are shown in Figure 2-1.

Figure 2-1: Ground Topography and River Bathymetry Based on LiDAR Image



2.2.2 VEGETATION

The vegetation is relatively sparse in the center of the terrace. In the kettle lakes area, near the rock knoll, in the upstream slope and old landslide areas on the downstream slope the vegetation is denser. The steep slope on the downstream side, south of the kettle lakes outlet, has less vegetation due to continuous sloughing.

The different vegetation existing in the North Spur can be seen in the aerial photo taken in 1988, Figure 1-1.

2.3 REGIONAL GEOLOGY

Previous studies reported that the region through which the Churchill River flows from Churchill Falls to Lake Melville is underlain by a variety of ancient, eroded Precambrian gneisses, which during the early Tertiary were covered by the sea. During the Pliocene, the area was uplifted and the river, descending from the interior plateau, incised a valley trough the marine sediments while its course was directed by the depressions and fault lines in the underlying rock. There followed in the Pleistocene at least four incursions of glaciations with intervening periods of complete deglaciation when the sea reoccupied the valley which had been depressed under the burden of ice. With the final retreat of the ice, the land began to rebound towards its present level and the deep marine deposits were covered by extensive outwash and littoral sands.

This alternation of glaciations, inundation and riverine and estuarine processes scoured the valley and laid down a complex of sediments, terraces, deltas, moraines, drumlins and other geomorphic features. In some places, winds created dunes on the exposed sand plains. As the land continued to rise, successive deltas and terraces were formed as the river cut deeper and deeper into the overburden, sometimes to bedrock, as it began to form the present valley.

2.3.1 PLEISTOCENE GEOLOGY

The recent history for the North Spur sedimentation process starts after de-glaciation. Observed soil sequence and stratigraphy process are typical of East Canadian valley. The Pleistocene stratigraphic process is very similar to the Saguenay valley in Québec province (Bouchard et al, 1983). The stratigraphic sequence is as follows.

First, over the rock, glacier placed a till deposit of variable thickness. This till deposit is not always present in the stratigraphic sequence.

With the progressive ice melt, fluvio-glacial deposit and ice water contact deposit were placed. In the Churchill River valley, these fluvio-glacial deposits are encountered in the deep buried valley and sometimes on the margin of the valley. Sand, gravel and cobbles particles are common in that generally pervious material. In the deep Churchill River buried valley (under the Spur of land), this deposit can have a thickness up to 160 m. The ice in the valley continued to melt but on the highland, sediment transportation progress and river energy brought material in the valley (depression) or along the contact between the valley wall and the glacial ice (Kame Terrace formation).

From a certain point in geological time, sea water filled the lowered valley and fine sediments (clay and silt deposit) covered the underlying sand and gravel layer (under the North Spur, this strata is called Lower Clay deposit). The deep water fine material, (Lower Clay) reached a thickness of about 50 m in the vicinity of the Spur.

Isostatic rebound of the land led to a decrease of the water thickness. Therefore, the water energy for the main and lateral water courses (Churchill River, Lower Brook, Upper Brook, etc.) increased and the deposition changed from clayey silt (during the period of less energy) to sandy silt to silty sand (during the period of more energy). This stratified deposit is called Stratified Drift.

The last strata in the Churchill River is a deltaic sand, (beach sand). It is a shallow water deposit. The top of the sand deposit was remolded by wind forming sand dunes. (Very evident in the northern part of the Spur)

After and during the final deposition steps with the rebound progress, water courses started to erode the deposits, digging the actual Churchill Valley. At Muskrat Falls, the level of the river is vertically controlled by the upper fall but the lateral erosion process continues and causes landslides on both side of the river. This phenomenon is also acting on the downstream side of Muskrat Falls.

2.3.2 BEDROCK GEOLOGY

The Muskrat Falls area is located in the Grenville province of the Precambrian Shield of Canada. Bedrock in the Muskrat Falls is mainly crystalline metamorphic rock composed of granitic gneisses with local presence of amphibolites and pegmatite stringers.

The bedrock exposure in the North Spur is limited to the rock knoll in the south. The elevation of the bedrock surface decreases from south to north, to about elevation -200 m in the kettle lakes area. From the kettle lakes to the north, the bedrock surface elevation increases to ground surface (elevation 200 m) north or the Trans-Labrador Highway.

2.4 SEISMI CITY

An assessment of the seismic hazard potential on the proposed development sites on the Lower Churchill Project was conducted in 2008 [Ref.1].

This assessment included a brief outline of the geological characteristics of the proposed sites of the LCP hydro development and the analysis made of the potential for a natural earthquake and/or a seismic event induced by reservoir impoundment to occur on the sites.

As result of this assessment, the peak ground acceleration (PGA) was reported as 0.09g for an annual exceedance probability level of 1/10,000. The soil constituting the North Spur was classified as a NEHRP D site, as per the standard NEHPR (National Earthquake Hazard Reduction Program) site classification scheme, with an amplification factor of 1.23 leading to an Earthquake Design Ground Motion (EDGM) of 0.11g.

In these studies, it was concluded that there is no evidence of seismic activities in the Muskrat Falls area in recent geological time and the identified tectonic features seemed to be inactive. The earthquake hazard analyses were conducted based on the probabilistic method. No evidence of recent movement has been observed along the fault line identified in the area of the project; therefore, the analysis relied on specific seismic sources around Labrador. Basic parameters of ground motion such as amplitude, frequency of vibration, Peak Ground Acceleration (PGA) and Peak Ground Velocity (PGV) were calculated.

The Reservoir Induced Seismicity (RIS) were studied with two different approaches. It was concluded to be unlikely that the reservoir could trigger an induced earthquake comparable to a M3 or M4 event on the Richter scale.

An updated seismic hazard assessment was performed for the Muskrat Falls site in 2014 [Ref. 2].

The new assessment considered the effects of major uncertainties on the hazard at the Muskrat Falls site and incorporated up-to-date information on seismicity and Ground Motion Prediction Equations (GMPEs), which have evolved considerably over the last 10 years.

The analysis determined the expected ground motions over a range of probability levels. The ground motions were calculated for stiff soil to rock site conditions. These site conditions corresponded to a NEHRP B/C boundary in the standard NEHRP site classification scheme.

As a conclusion, based on the result of the probabilistic analyses, the probability of 1/10,000, the expected peak ground acceleration (PGA) from natural earthquakes for the reference site condition (NEHRP B/C) at Muskrat Falls is approximately 0.06g.

2.5 CLIMAT OLOGY

The Labrador plateau experiences a continental type of climate with a wide range of temperatures. The Lower Churchill River basin climate is moderated by the Atlantic Ocean and experiences more of a maritime climate.

The climatological data relevant to the Muskrat Falls site are presented in the climatological data report [Ref. 3] which is based on Environment Canada recordings at Goose Bay A- station 8501900. The data presented in this section are extracted from that report.

Meteorological data such as temperature, precipitation, visibility, wind speed, wind direction and pressure are recorded at that station.

2.5.1 TEMPERAT URE

The average annual temperature is 0°C, with recorded extremes of -39.4°C and 37.8°C. Table 2-1 summarizes the monthly variations of temperature. The minimum, maximum and average values of temperatures are presented on a monthly basis in this table.

Table 2-1: Temperature Monthly Variation – Data from 1971 to 2000

Month	Min. recorded temp. (°C)	Average min. daily temp. (°C)	Average temp. (°C)	Average max. daily temp. (°C)	Max. recorded temp. (°C)
January	-38.9	-23.3	-18.1	-12.9	11.2
February	-39.4	-21.9	-16.3	-10.6	10.6
March	-35.6	-15.4	-9.6	-3.7	16.4
April	-29.7	-6.6	-1.7	3.3	21.2
May	-15.0	-0.3	5.1	10.5	32.1
June	-4.2	5.2	11.0	16.8	36.2
July	0.1	9.7	15.4	20.9	37.8
August	0.0	9.0	14.5	19.9	35.3
September	-6.7	4.5	9.2	13.9	30.0
October	-17.0	-1.5	2.4	6.2	22.8
November	-26.1	-8.1	-4.5	-0.8	16.7
December	-36.7	-18.3	-13.9	-9.4	11.7
Per year	**-39.4**	**-5.6**	**-0.5**	**4.5**	**37.8**

2.5.2 PRECIPITATION

The average annual precipitation is reported as 950 mm, of which 55% is rainfall and 45% occurs as snowfall. Average precipitation per month is shown in Table 2-2.

Return period rainfall rates are shown in Table 2-3. The frost free period occurs from June to September.

Table 2-2: Average Precipitation – Data from 1971 to 2000

Month	Total (mm)[1]	Rainfall (mm)	Snowfall (cm)	Days with rainfall > 0.2 mm	Days with snowfall > 0.2 mm
January	64.6	1.9	80.2	1.3	16.2
February	55.1	3.3	62.6	1.2	13.0
March	69.6	5.3	75.8	2.5	13.9
April	65.4	19.3	52.3	5.3	11.0
May	66.2	47.0	19.9	12.2	5.2
June	95.8	92.1	3.2	16.4	1.2
July	113.8	113.8	0.0	18.8	0.03
August	98.8	98.8	0.0	17.7	0
September	95.2	92.3	2.6	17.7	0.93
October	80.1	59.6	22.1	12.7	6.4
November	75.6	20.3	62.0	5.2	12.1
December	69.0	5.7	78.3	2.0	15.7
Per year	**949.0**	**559.5**	**458.8**	**113.1**	**95.6**

[1]Equivalent water in case of snow

Table 2-3: Return Period Rainfall Rates – Data from 1961 to 2007

Duration	2 yrs (mm/h)	5 yrs (mm/h)	10 yrs (mm/h)	25 yrs (mm/h)	50 yrs (mm/h)	100 yrs (mm/h)
5 min	47.1 ±6.0	66.8 ±10.1	77.9 ±13.6	96.4 ±18.3	108.6 ±21.9	120.7 ±25.6
10 min	34.1 ±5.2	51.1 ±8.7	62.4 ±11.8	76.6 ±15.9	87.2 ±19.0	97.7 ±22.1
15 min	27.1 ±4.2	41.0 ±7.1	50.3 ±9.7	62.0 ±13.0	70.7 ±15.6	79.3 ±18.2
30 min	17.3 ±2.8	26.4 ±4.6	32.4 ±6.3	40.0 ±8.4	45.6 ±10.1	51.2 ±11.8
1 h	11.0 ±1.4	15.5 ±2.3	18.4 ±3.1	22.2 ±4.2	25.0 ±5.0	27.7 ±5.8
2 h	7.2 ±0.8	9.7 ±1.3	11.3 ±1.7	13.4 ±2.3	15.0 ±2.8	16.5 ±3.2
6 h	4.0 ±0.4	5.2 ±0.6	6.0 ±0.8	7.0 ±1.1	7.7 ±1.3	8.4 ±1.5
12 h	2.7 ±0.2	3.3 ±0.4	3.8 ±0.5	4.4 ±0.6	4.8 ±0.8	5.2 ±0.9
24 h	1.7 ±0.1	2.1 ±0.2	2.4 ±0.3	2.8 ±0.4	3.1 ±0.5	3.3 ±0.6

2.5.3 WIND

Average wind data including the speed and the most frequent directions are presented in Table 2-4.

Table 2-4: Average Wind – Data from 1971 to 2000

Month	Average speed (all directions) (km/h)	Most frequent direction	Maximum hourly wind observed Speed (km/h)
January	16.9	SW	84
February	15.9	W	77
March	16.3	W	77
April	15.3	NE	65
May	14.3	NE	77
June	14.6	NE	58
July	13.5	NE	64
August	13.6	W	69
September	14.7	SW	72
October	15.5	SW	74
November	16.6	W	81
December	17	SW	81

2.5.4 HYDR AULICS

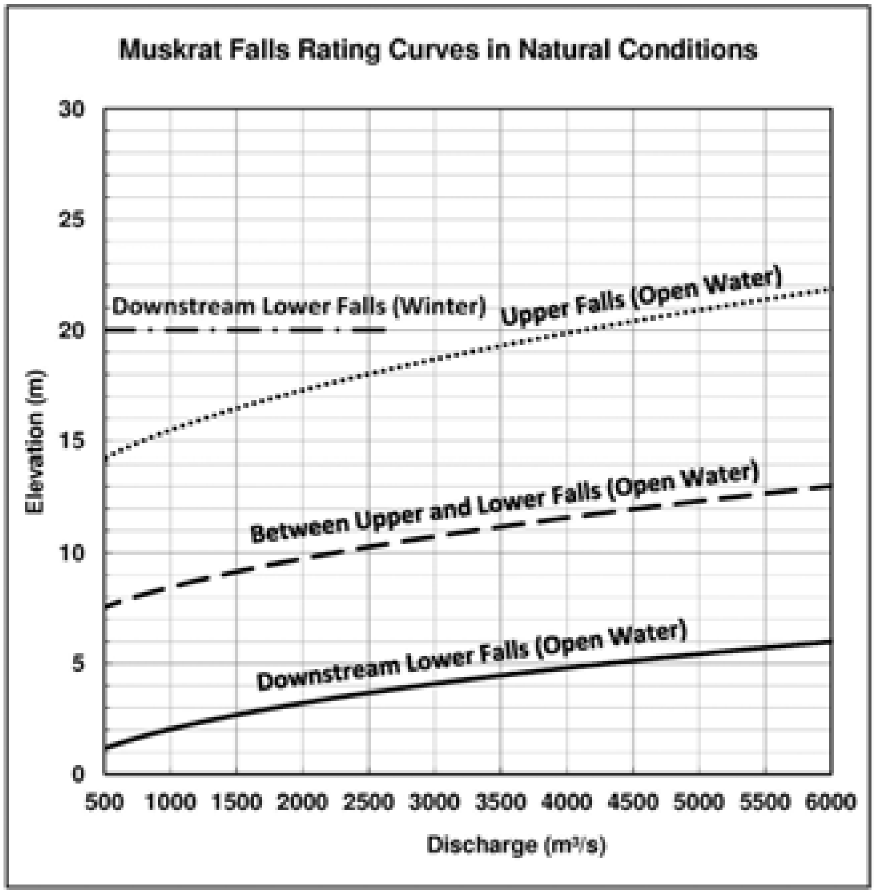
The rating curves for water levels in the Churchill River, upstream and downstream of the Muskrat Falls site, in natural conditions, are presented in Figure 2-2 [Refs. 4 and 5]. Considering an average flow rate of 1,830 m^3/s, the average water levels at the upstream and downstream sides of the Spur are about 17 m and 3 m, respectively.

The reference water levels, during and after construction, are as follows:

- Maximum diversion head pond level = 25 m
- Full Supply Level (FSL) = 39 m
- Low Supply Level (LSL) = 38.5 m
- Probable Maximum Flood (PMF) (Upstream) = 45.1 m

- Probable Maximum Flood (PMF) (Downstream) = 12.5 m

Figure 2-2 : Muskrat Falls Site – Natural Conditions Rating Curves



3 REVIEW OF FIELD INVESTIGATION PROGRAMS AND ENGINEERING STUDIES

3.1 INTRODUCTION

From 1965 to 2013 several field investigation programs have been performed on the North Spur. Different engineering solutions for the stabilization works of the North Spur have been studied during the same period until the engineering for final design in 2013.

The outcomes of the field works and interpretations presented in the different engineering studies shaped the basis of the current knowledge of the geology, stratigraphy and hydrogeology of the North Spur. Location of all inventoried field investigation works is presented in drawing MFA-SN-CD-2800-GT-PL-0012-01. Extent of each investigation campaign is briefly reviewed in the following sections. The installation, maintenance and refurbishment of the instruments and pump system is also reviewed.

The different engineering studies of the North Spur stabilization works carried out between 1965 and 1998 are reviewed in this section, to show the evolution of the design of time as more information became available. The final design, undertaken in 2013, is presented in more details in Section 5. Several independent technical evaluations and reviews of the final design were performed by external experts and advisors; these are also reviewed at the end of Section 5.

Additional specialized studies were carried out in 2013 and 2014. General details of those studies are presented in that section.

Table 3-1 summarizes all the field works, engineering studies and technical evaluations performed on the North Spur.

The work performed, such as field investigations, dewatering system installation, assessment and maintenance, instrumentation installations and readings, engineering studies and external advisors technical evaluations are reviewed in chronological order.

Table 3-1: North Spur – Summary of Field Works and Engineering Studies

Year	Company	Reference
1965	Acres Canadian Bechtel	Lower Churchill River Muskrat Falls Development Engineering Assessment November 1965 [Ref. 6]
1966	Acres Canadian Bechtel	Lower Churchill River Muskrat Falls and Gull Island Sites, February 1966 [Ref. 7]
1966	Norwegian Geotechnical Institute	Laboratory Works September and October 1966 [Ref. 8]
1976	Lower Churchill Consultants	Muskrat Falls Development, Geotechnical Review of 1965 Layout, June 1976 [Ref 9]
1977	Acres Consulting Services Limited	Muskrat Falls Development, Engineering Assessment January 1978 [Ref. 10]
1979	Newfoundland and Hydro Labrador	Site Visit, Muskrat Falls Geological Report, Photos for Cores and Site Area, October 1979 [Ref. 11]
1979	SNC-LAVALIN Newfoundland Ltd	Muskrat Falls Power Development & 345 KV Transmission Intertie to Churchill Falls Vol. I to IV, Field Works and Engineering Study, March 1980 [Refs. 12 to 15]
1981	SNC-LAVALIN Newfoundland Ltd	Muskrat Falls Dewatering System Construction Report and Engineering Assessment March 1982 [Ref. 16 and 17]
1996	Acres International Limited	Muskrat Falls Hydroelectric Project Dewatering System Assessment and Rehabilitation, February 1997 [Ref. 18]
1997	Acres International Limited	Muskrat Falls- Standpipe Piezometer Installation Program Report, February 1998 [Ref 19]
1998	SNC-AGRA	Muskrat Falls Hydroelectric Development Final Feasibility Study Volumes 1 and 2, January 1999 [Refs. 20 and 21]
2007	HATCH	MF1260 – Assessment of Existing Pumpwell System July 2008 [Ref. 22]
2009	HATCH	MF1271 – Evaluation of Existing Wells, Pumps and Related Infrastructure in the Muskrat Falls Pumpwell System March 2010 [Ref 23]
2009	HATCH	MF1272 – Installation of New Piezometers in the Muskrat Falls Pumpwell System, April 2010 [Ref. 24]
2013	AMEC Environment & Infrastructure	Geotechnical Investigation Report 2013 Field Investigations- North Spur for Nalcor Energy – Lower Churchill Project, November 2013 [Ref. 25]

Year	Company	Reference
2013	Advisory Board	Advisory Board Meetings # 1 and 2, April, October 2013 [Refs. 26 and 27]
2013	MWH Americas Inc.	Interim Independent Engineering's report, November 2013 [Ref. 29]
2013	HATCH	Cold Eye Review of Design and Technical Specifications, September 2013 [Ref. 28]

3.2 1965 FIELD INVESTIGATIONS

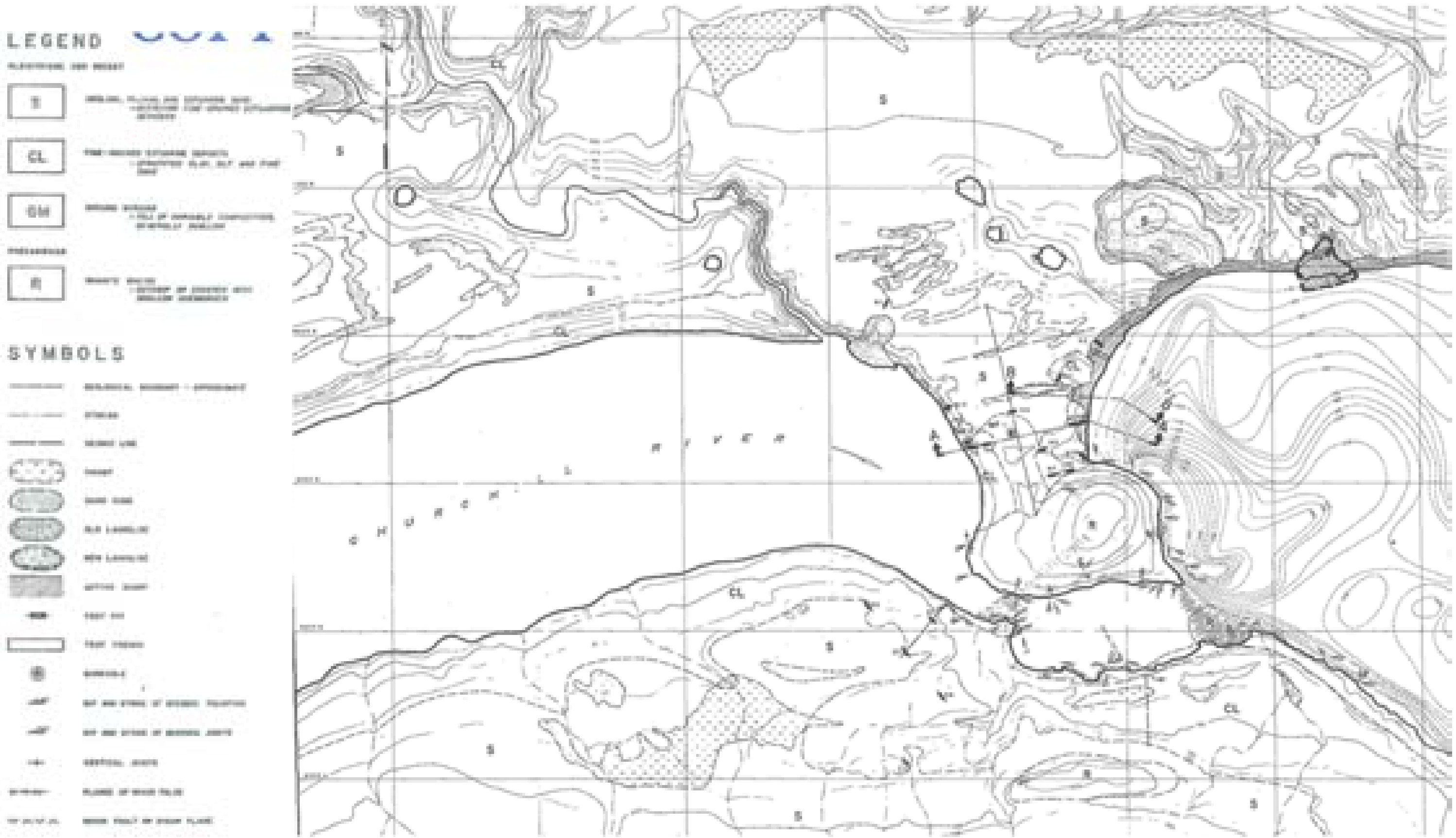
In 1965 a field investigation program was performed in the Muskrat Falls site in order to review and update a previous report on the Lower Churchill Project [Ref. 6]. Geological conditions on both banks of the river at Muskrat Falls were established from surface mapping.

The 1965 field work included geological surveys, seismic investigations, diamond drilling, test pitting, topographic surveys and mapping, river soundings and laboratory testing of drill and test pit samples.

A total of 2 boreholes, including MF-1 on the crest of the Spur and MF-2 on the upstream slope, were drilled and 13 test pits and 1 trench were dug in different areas of the North Spur in August 1965.

The stratigraphy of the overburden layers was interpreted from the boreholes and the bedrock level was estimated from the geophysical surveys. A geological plan of the site and borehole locations and approximate test pit locations are shown in Figure 3-1.

Figure 3-1 : 1965 Investigation Works and Geological Plan



Available data from boreholes are presented in Table 3-2.

Table 3-2 : 1965 Boreholes

Borehole No.	Elevation (m)	
	Ground surface	End of Hole
MF-1	59.44	-10.66
MF-2	24.38	-62.18

3.3 1965 ENGINEERING STUDY

In 1965 a review and update of a previous report on the Lower Churchill Project was performed to allow the start of a full engineering program including basic design suitable for estimating purposes [Ref. 7].

During the study it was mentioned that any scheme of the hydroelectric development at the site of Muskrat Falls must include treatment of the North Spur to ensure its stability.

Main features of the design for the treatment of the North Spur were, trimming and slope protection of the existing slopes, buttress of free draining granular material on the downstream side of the Spur, construction of a protective groin, construction of inverted filters on the downstream area, filling of the deep hole downstream of the Spur and reduction of the eddy motion in the river.

3.4 1966 LABORATORY WORKS

In September and October 1966, several laboratory analyses were performed on clay samples retrieved during the 1965 field work [Ref. 8]. Main objectives of this study aimed to investigate the possible nature of the bonds acting between particles of the Canadian clay.

3.5 1976 ENGINEERING STUDY

In 1976 a geotechnical review of the 1965 layout was carried out in the light of research performed on the behaviour of marine clays in Canada [Ref. 9]. In order to supplement the existing data, a limited test program was carried out on samples retrieved in the past field works.

The study concluded that certain design aspects of the previous study were possibly overly conservative and thus, the stabilization measures required to preserve the integrity of the North Spur could be considerably less than those envisioned in the past.

The proposed scheme for the stabilization measures of the North Spur involved treatment of the foundation soils as well as stabilizing the slopes including a line of relief wells, partial excavation of the crest of the neck, a weighting berm and a drainage blanket on the downstream slope.

It was also concluded that all elements of the design must contemplate that construction sequences are such that no activity will cause reduction in the existing stability of the Spur.

Layout and sections of the proposed stabilization works are presented in Figure 3-2 and Figure 3-3.

Figure 3-2 : 1976 Stabilization Works – Layout

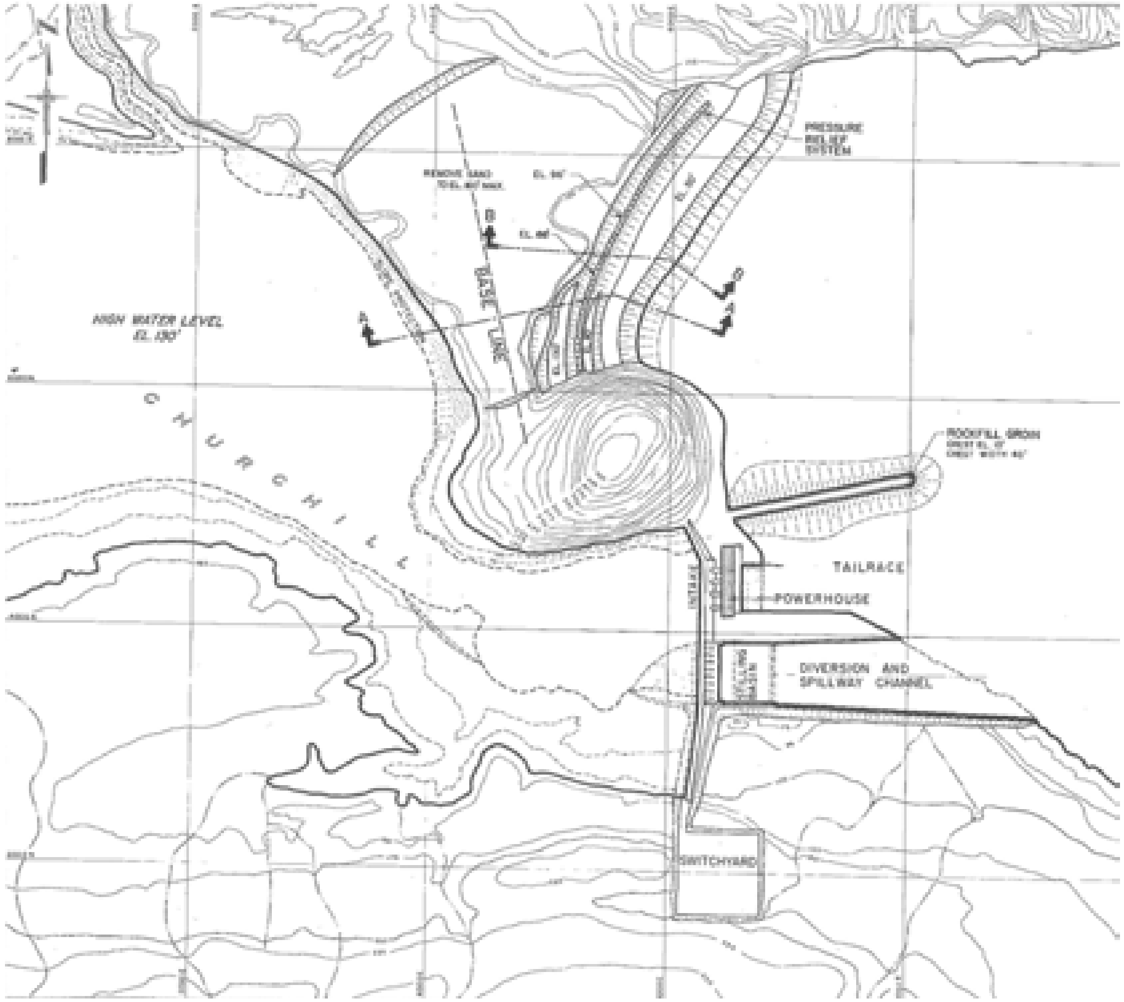
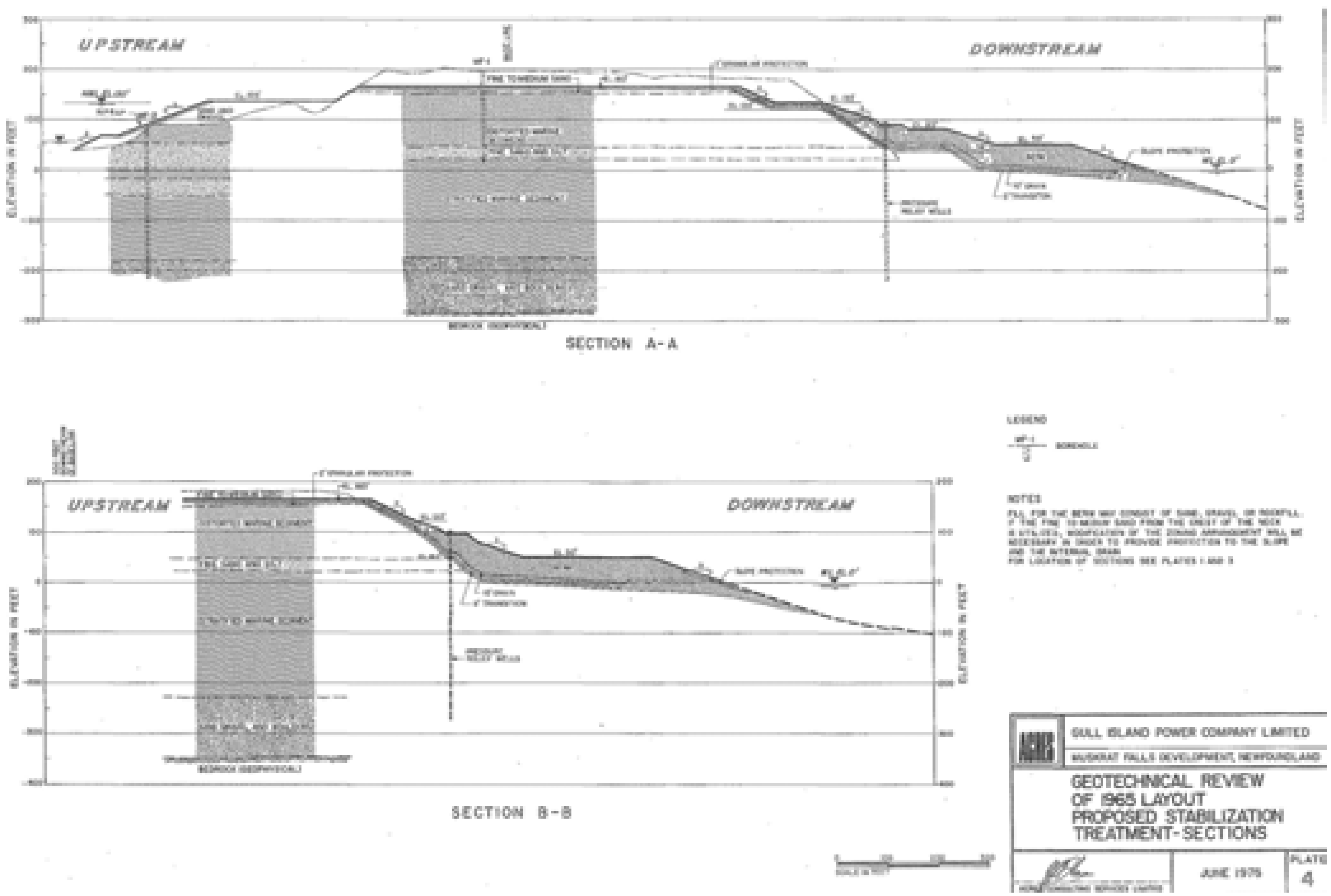


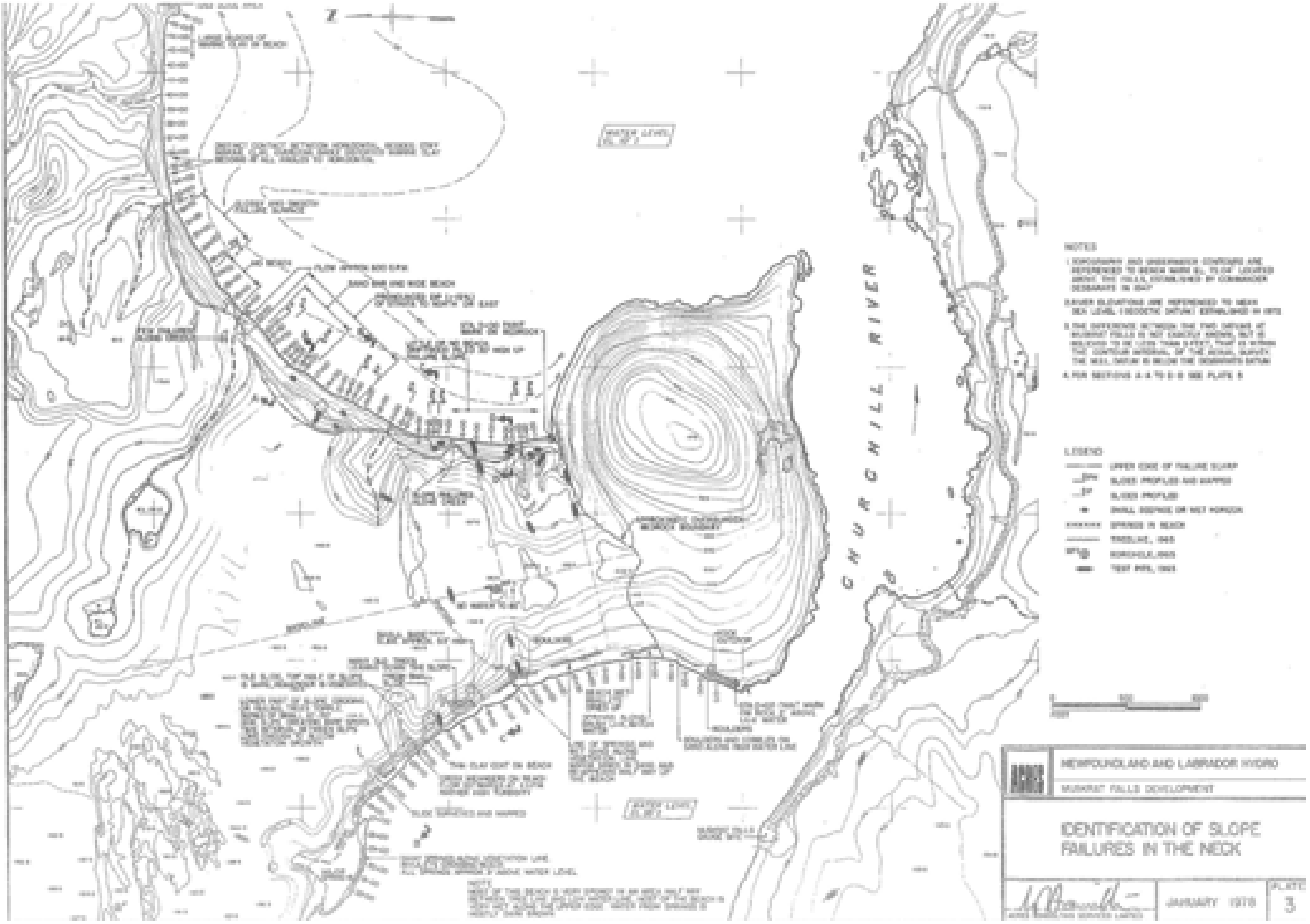
Figure 3-3 : 1976 Stabilization Works – Cross Sections



3.6 1977 FIELD INVESTIGATIONS

In 1977, a detailed study of clay slide areas in the vicinity of the site was performed [Ref. 10]. The site work included a surficial field examination and the retrieval of samples for further laboratory testing. The main objectives of this campaign were to identify and document as many of the surface and other features of the landslides as possible. Main findings of these works are presented in Figure 3-4.

Figure 3-4 : 1977 Field Works



3.7 1977 ENGINEERING STUDY

The field work carried out in 1977 and the related findings concerning the soil properties of the North Spur confirmed that the treatment proposed in the 1976 review was satisfactory.

Some improvements were made to the previous layout. The proposed stabilization measures of the North Spur studied in 1977 comprised a line of relief wells along the downstream slope, partial removal of the upper sand layer, construction of a weighting berm on the downstream slope and construction of an impervious blanket on the upstream slope.

Layout and sections of the proposed 1977 stabilization works are presented in Figure 3-5 and Figure 3-6.

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Figure 3-5 : 1977 Stabilization Works – Layout

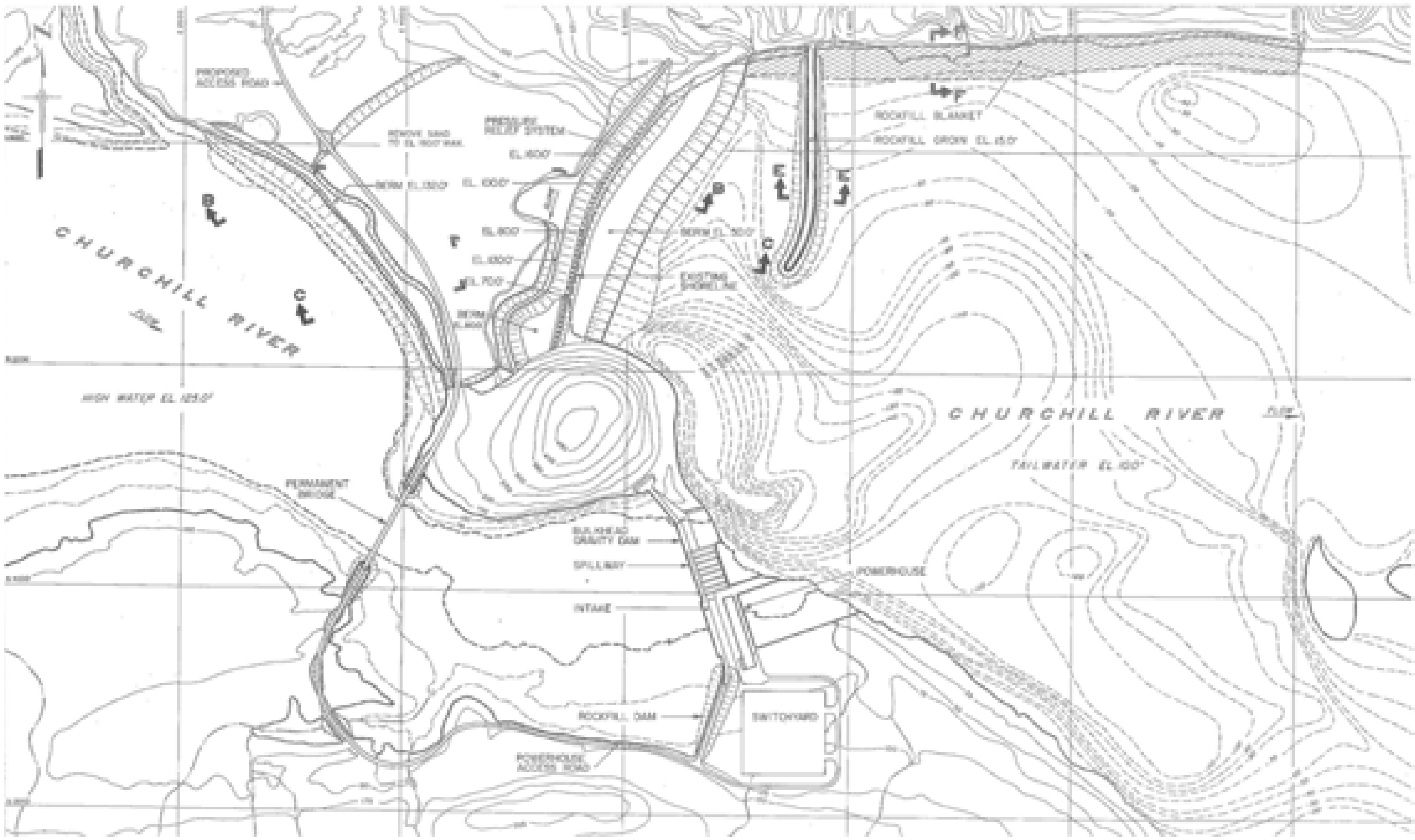
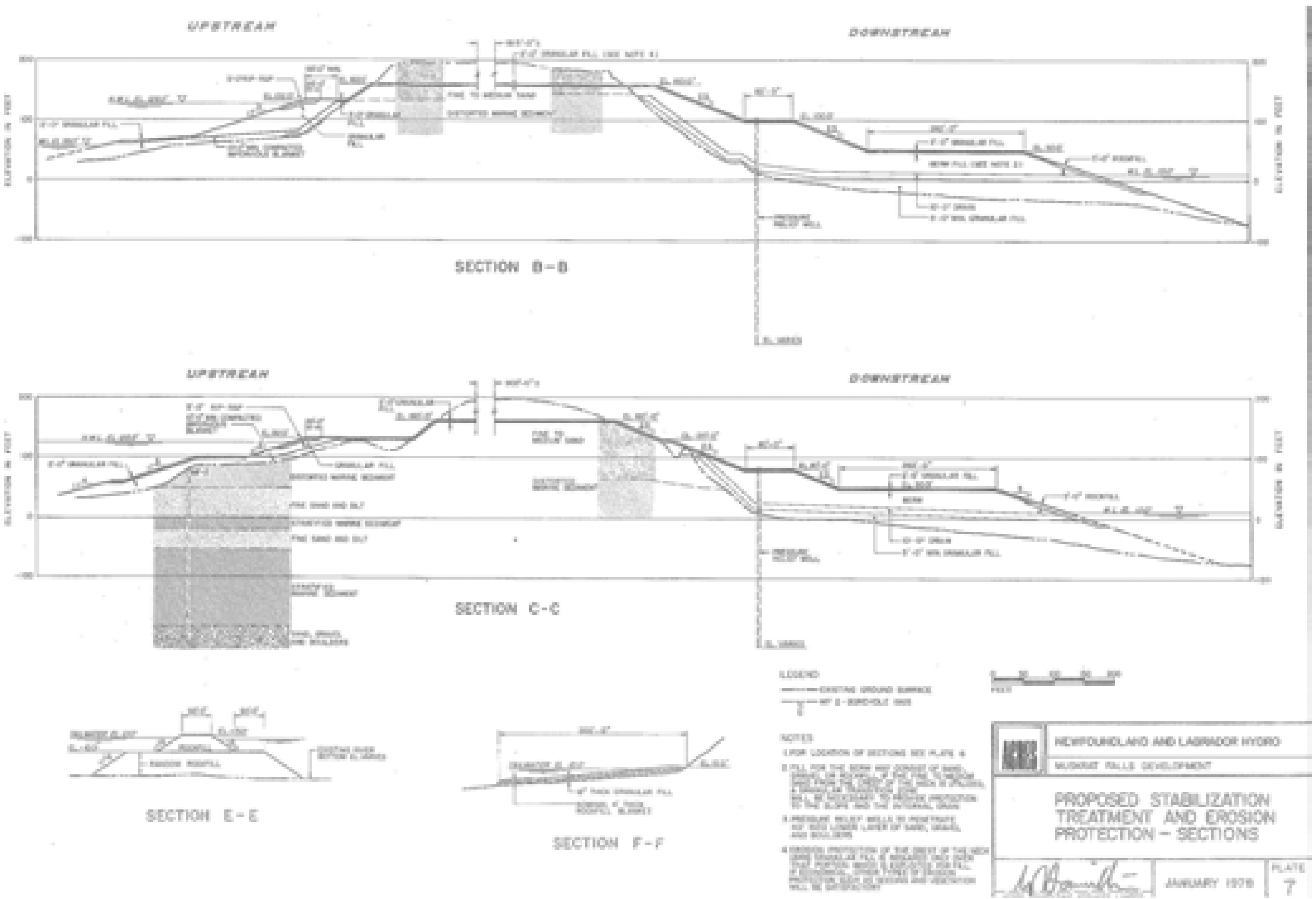


Figure 3-6 : 1977 Stabilization Works – Cross Sections



3.8 1979 FIELD INVESTIGATIONS

In November 1978 a large landslide occurred on the downstream side of the North Spur. The slide removed about 1 million m^3 of soil from the neck of the Spur and reduced the width of the crest of the North Spur by almost 100 m. Location and extent of the 1978 slide can be observed in Figure 3-7 by comparing aerial photos taken in 1951 and 1988.

Figure 3-7: 1978 Landslide

Extensive field investigations were conducted in 1979 to understand the stratigraphy, the groundwater conditions and main causes for slope instabilities in the North Spur, the effects of reservoir impoundment on its overall stability and to assess the water tightness of the Spur and the availability of the construction materials.

The field work included:

- Drilling of 42 boreholes, including 3 boreholes for piezometer installations in the lower aquifer;

- In-situ testing and sampling;
- Installation of piezometers and monitoring of water levels;
- Test pitting in the borrow areas;
- Geophysical surveys over land including seismic refraction surveys and over water including bathymetric, side scan sonar and seismic refraction surveys;
- Pumping tests in the lower aquifer.

Summary of 1979 boreholes, including piezometer installation are summarized in Table 3-3. Layout of the investigations is shown in Figure 3-8.

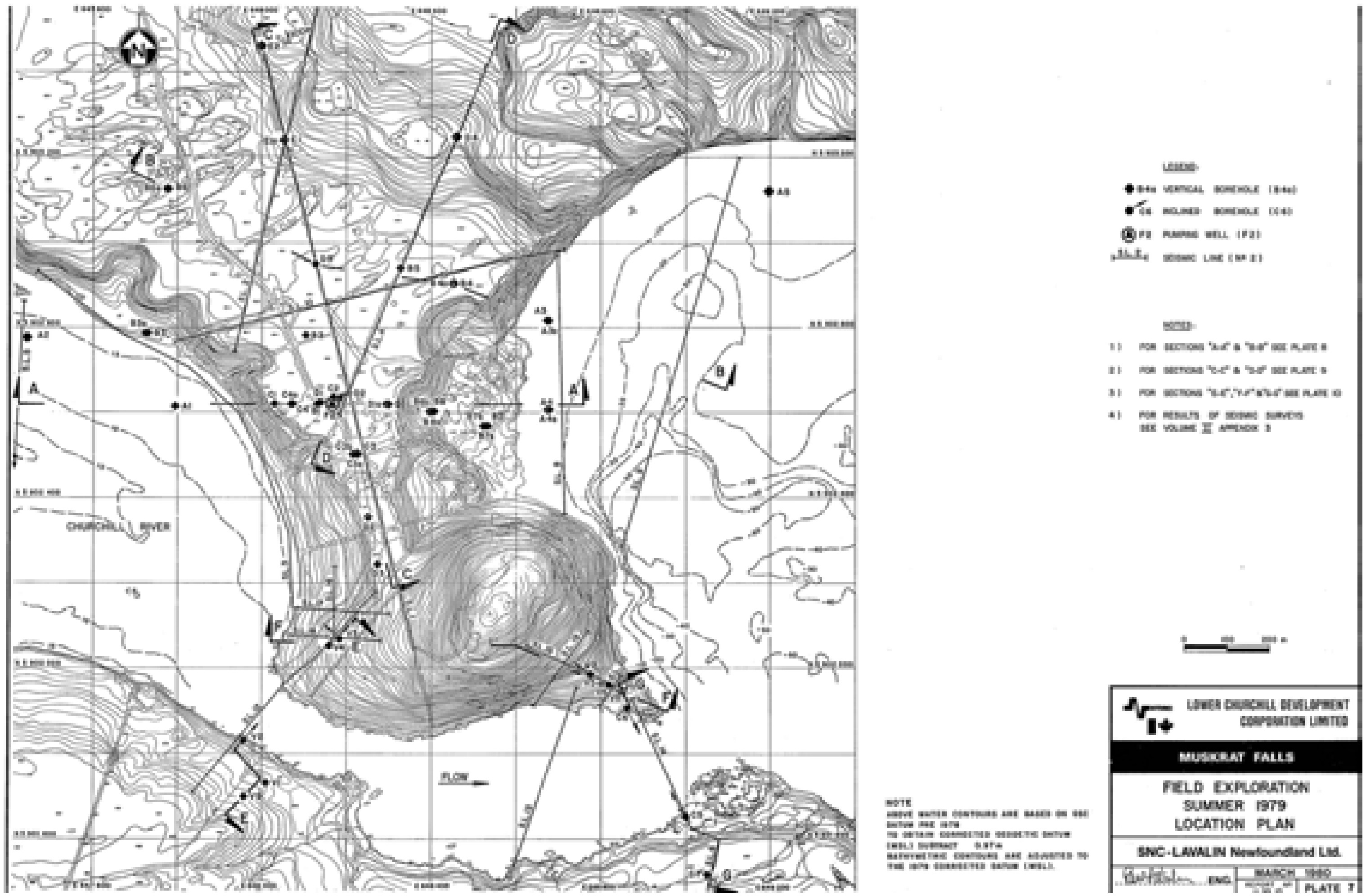
Table 3-3: 1979 Field Investigations – Boreholes Summary

Borehole No.	Elevation (m)				
	Ground surface	End of Hole	Bedrock	Piezometer tip	Groundwater Table
A1	13.8	-102.2	-95.2	-94.0	7.0
A2	13.5	-138.9	-137.9	-76.5	13.5
A3	-5.6	-24.0	-	-	-
A3a	-5.6	-8.6	-	-	-
A4	-5.3	-25.3	-	-	-
A4a	-5.3	-13.1	-	-	-
A5	-5.5	-26.5	-	-	-
B1	59.3	-89.7	-	-78.7	
B2	59.4	-97.6	-	48.0 -97.0	49.5 5.6
B3	20.0	-136	-	-115	6.8
B3a	20.0	-9.0	-	-8.0	20.0
B4	57.4	-17.2	-	43.0 11.0	Dry 13.1
B4a	57.4	13.1	-	31.0	31.4
B5	57.6	-23.8	-	42.0 18.0 -13.	Dry 31.2 25.0
B6	60.0	-15.2	-	25.0	39.6

Borehole No.	Elevation (m)				
	Ground surface	End of Hole	Bedrock	Piezometer tip	Groundwater Table
				-18.0	24.8
B6a	60.0	45.2	-	46.0	Dry
B7	10.7	-66.3	-	-65.0	4.3
B7a	10.7	-16.7	-	0.0 -16.0	8.7 10.7
B7b	10.7	-8.5	-	-8.5 5.7	9.4 6.2
B8	24.6	-74.4	-	-67.0	
B8a	24.6	3.0	-	3.5	20.6
B8b	24.6	9.2	-	10.0	24.0
C1	59.9	-82.9	-	-82.5	5.9
C2	59.1	-115.5	-114.6	-	-
C3	59.1	-68.9	-	34.0 -68.5	Dry 5.2
C3a	59.1	4.1	-	4.1	27.5
C3b	59.1	44.7	-	46.0	49.5
C4	59.4	-111.6	-105.6	22.9 15.4 -30.0	26.3 25.4 9.6
C4a	59.4	32.4	-	-	-
D1	60	-79.6	-	-17.0	27.6
D1a	60	-25.2	-	32.0 15.0 -2.0	Dry 31.2 28.2
D2	60.1	-17.7	-15.5	40.0 15.0 -14.5	42.5 26.0
D3	61.9	18.2	32.3	55.0	Dry
D4	26.1	-35.9	-	21.0 -8.0 -27.0	23.7 24.8
D5	58.8	-185	-176.2	-92.0	7.9
E1	44.6	-135.8	-	-132.	4.4
E1a	44.6	24.6	-	25.0	27.6
E2	60.1	-96.4	-	-96.0	37.7

Borehole No.	Elevation (m)				
	Ground surface	End of Hole	Bedrock	Piezometer tip	Groundwater Table
E2a	60.1	38.1	-	48.0 38.0	48.9 47.7
P1	59.17	-81.3	-	Lower aquifer	5.6
P2	60.13	-78.5	-	Lower aquifer	5.5
F2	59.2	-108.7	-	Lower aquifer	5.4

Figure 3-8 : 1979 Field Investigations



Sampling of overburden was carried out at selected depth intervals. In-situ tests and laboratory tests were conducted and reported. One laboratory permeability test of the intermediate aquifer unit was performed on a sample retrieved from a borehole.

Direct shear tests, triaxial tests and consolidation tests were carried out in the laboratory.

Details of the test pitting works are presented in more details in Section 6.

During the 1979 field works, pumping tests were carried out in the lower aquifer and water quality tests were performed during the tests.

A list of seismic lines performed on the North Spur during the 1979 investigations is presented in Table 3-4.

Table 3-4: 1979 Field Investigation – Summary of Seismic Refraction Surveys

Seismic Line No.	Length (m)
SL-1	1650
SL-2	1050
SL-3	1070
SL-4	975
SL-8	600
SL-10	960

3.9 1979 ENGINEERING STUDY

The need for permanent stabilization works on the North Spur, in order to serve as a reservoir retention structure for the Muskrat Falls Hydroelectric project, was recognized in the previous engineering studies. The stability of this natural dam was identified as crucial to the feasibility of the project.

Results of the investigation works carried out in 1979 allowed to better understand the stratigraphic and hydrogeological conditions of the North Spur. It was recognized in the engineering report that the natural conditions could be characterized by a complex stratigraphy, perched water tables and the presence of erodible and sensitive materials.

The engineering study stated that stability works have to be done to preserve the stability of the North Spur. Stabilization measures would include mainly, lowering of the groundwater level inside the North Spur, downstream erosion protection, surface run-off control and upstream slope protection.

In order to sufficiently improve the stability of the downstream slope, it was established that the ground water level should be lowered from elevation 30 m to at least elevation 15 m.

Alternative seepage interception systems including, a diaphragm cut-off wall along the ridge of the Spur with a length of about 900 m and a maximum depth of about 70 m; drainage wells bored from the surface to intercept a dewatering tunnel excavated within the marine clay strata and drainage wells equipped with submersible pumps discharging into a header pipe system were studied. It was concluded from these studies that the drainage well scheme proposed was the most economic and highly reliable system.

The main features or the studied stabilization works were a line of relief wells in the crest of the Spur along the downstream slope, a secondary line of defense comprised of inclined gravity drains, permanent drainage of the Spur in the area of the Kettle Lakes, relieving weight by partial excavation on the slope crest, rockfill protection in the downstream area and a free draining berm with inverted filter in the upstream slope.

As a complement of the stabilization works mentioned above and in order to assess the effectiveness of the drainage and slope treatment measures in the North Spur, it was concluded that the area must be properly instrumented and monitored both prior to and after the proposed remedial measures were undertaken. Layout and cross-sections of the proposed 1979 stabilization works are presented in Figure 3-9 and Figure 3-10.

Figure 3-9 : 1979 Stabilization Works – Layout

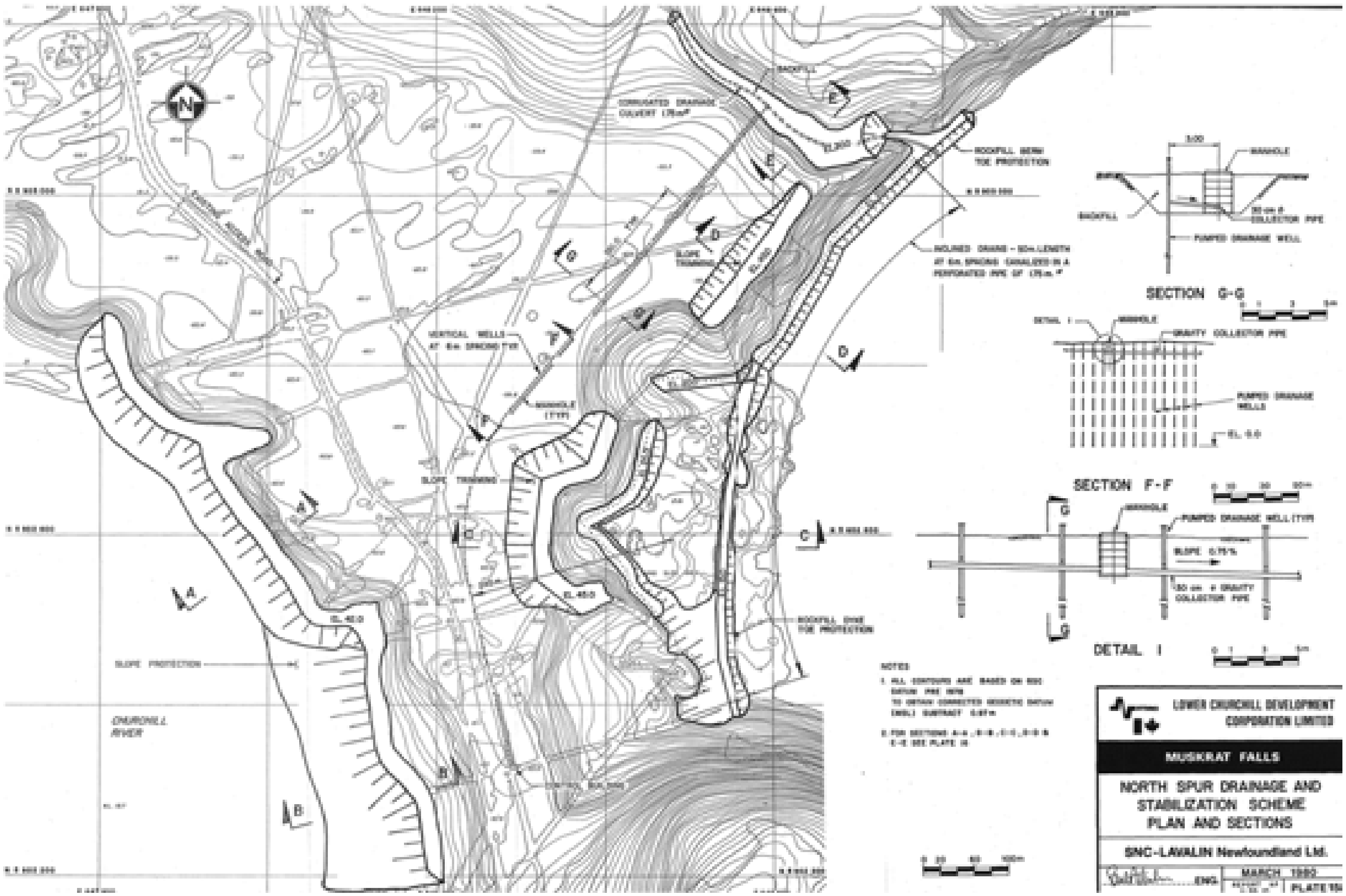
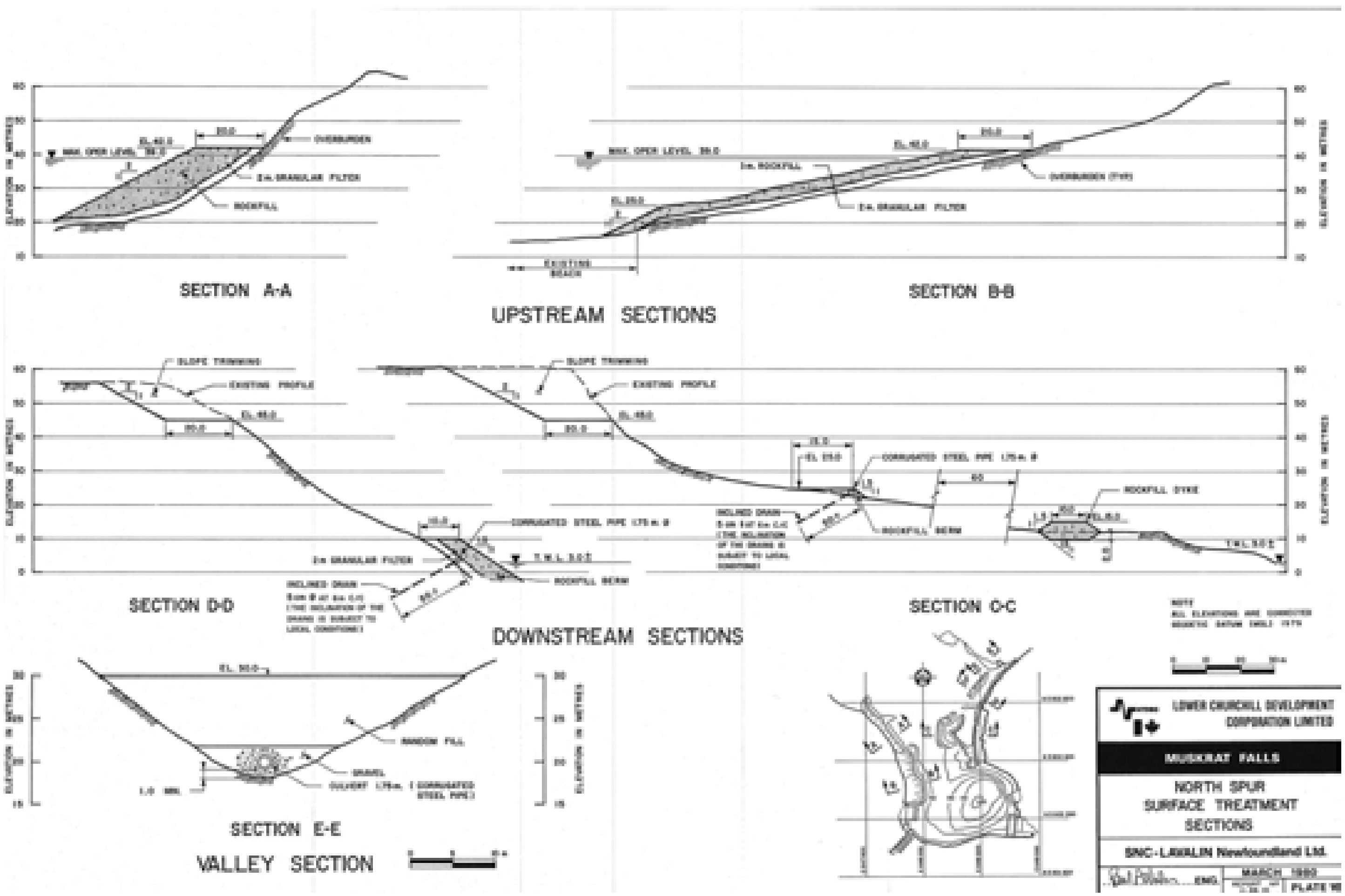


Figure 3-10 : 1979 Stabilization Works – Cross Sections



3.10 1981 FIELD WORKS

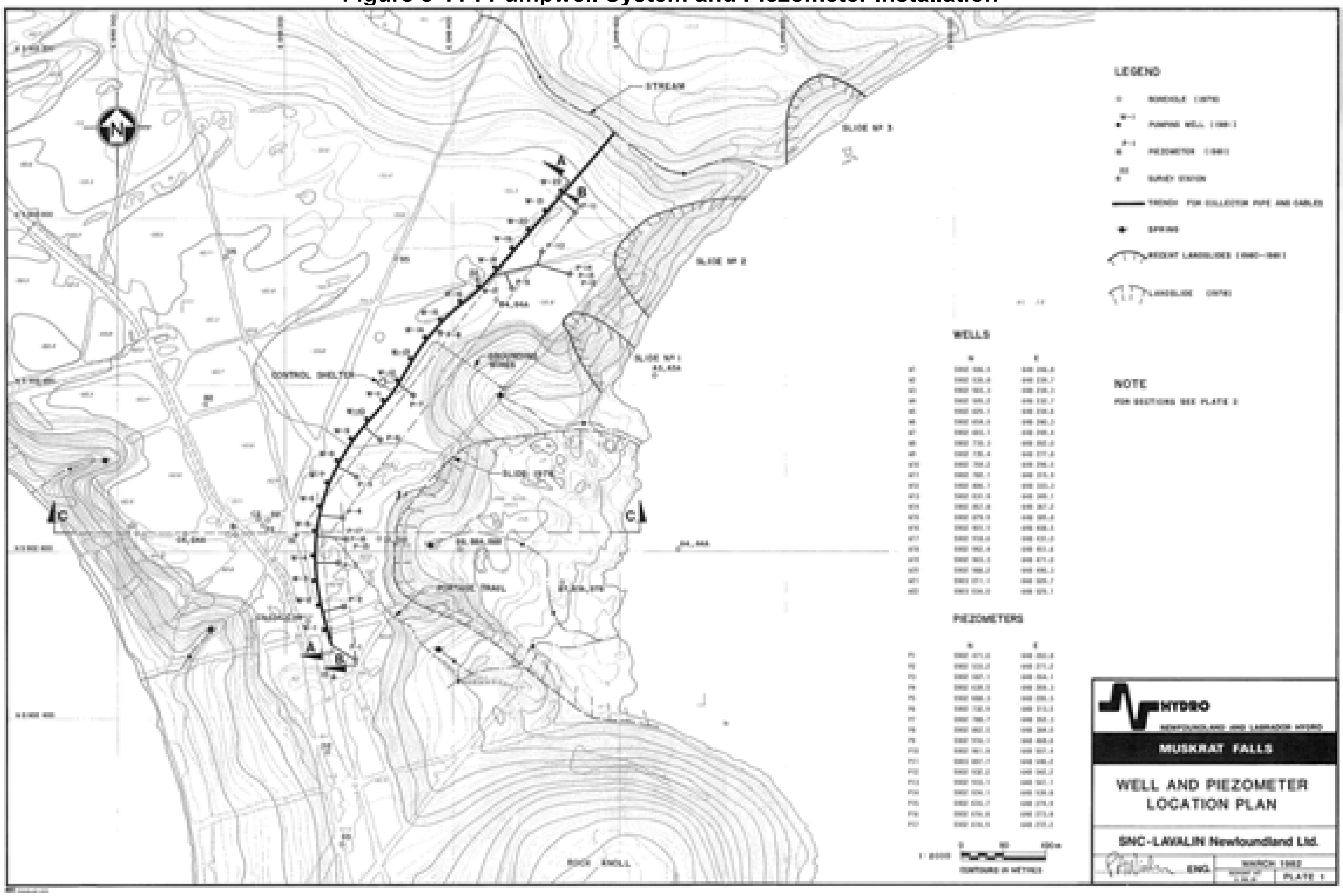
Engineering assessment prepared by SNC-Lavalin Newfoundland Ltd. in 1979, proposed to install a curtain of closely spaced pump wells in the downstream side of the crest of the Spur to control the groundwater level as part of the stabilization measures.

In 1981, the services of SNC-Lavalin Newfoundland were retained to carry out partial works in order to temporarily improve the stability of the North Spur.

A complete dewatering system was installed in 1981 as a temporary stabilization measures. The complete dewatering system was made up of several components which function collectively to lower the groundwater level within the North Spur. The dewatering system consisted of a row of 22 pump wells drilled into the spur at 30 m spacing adjacent to the top of the downstream slope of the Spur.

The natural groundwater level is lowered by pumping groundwater, intercepted by the row of wells, into a common collector pipe which discharges it, by gravity, to the three Kettle Lakes outlet flowing into the downstream pool of the Churchill River. Layout of the dewatering system installed in 1981, including the pump wells, piezometers and the control shelter is shown in Figure 3-11.

Figure 3-11 : Pumpwell System and Piezometer Installation



The wells were drilled with large diameter tri-cone 12 inch bits using “revert” as the drilling mud to support the hole. Geological logs were interpreted from soil samples recovered from the drilling fluid.

Geophysical logging by Mount Sopris continuous recording geophysical logging apparatus were performed in 20 drilled holes for wells and piezometers to interpret the soil stratigraphy, and the results were compared to geological logs.

Results obtained during construction and operation of the pumping system, including groundwater levels measured in the wells and piezometers, were used to better understand the stratigraphy and hydrogeology of the North Spur.

Details of construction works and engineering assessments are presented in the reports prepared by SNC-Lavalin Newfoundland Ltd [Refs. 16 and 17].

The main features of the dewatering system are as follows:

- Screen and riser pipes 15 cm diameter;
- Level control assembly, which components are the level detector electrodes and the level control panel;
- Motor control panel, located inside the control shelter, is the main control for the operation of the submersible pumps and level control assembly;
- Well head, which is the protective shelter of the well components and which provides to the submersible pump level control assembly and discharge pipes of each well;
- Collector pipe. Main conduit for discharging the water pumped from the 22 wells. This conduit is trenched between well W-1 and the outlet in the area of the Kettle Lakes at the northeast area of the Spur;
- Piezometers. A total of 17 vibrating wire piezometers were installed downstream of the pump well line at an average elevation of 0 m. The piezometer cables were connected with a piezometer readout unit and the monitoring subsystem in the control shelter;

- Telemetric system, which transmits alarms between the Muskrat Falls site and the Hydro Happy Valley office by way of VHF radio link allowing the monitoring of the equipment located on site by an operator in Happy Valley. Switching and running time of the pumps and water levels information is transmitted in real time and recorded;
- Control shelter, housing the equipment used to operate and monitor the dewatering system. The control shelter is located near wells W-11 and W-12;
- Power source. Electric power for the operation of the dewatering system was assured at the beginning of the dewatering works by a diesel generator. A transmission line from the existing Newfoundland and Labrador grid was later connected to the unit.

Figure 3-12 presents the as built pump well system.

Figure 3-12 : As Built Pump Wells

Wells Elevations As Built

WELL NUMBER	CONCRETE SLAB, ELEV.	LENGTH OF RISER PIPE ABOVE THE SLAB	RISER PIPE (PVC) ELEV.	CORRIGATED STEEL PIPE ELEV.	BOTTOM OF WELL ELEV.
W-1	57.83	1.96	59.79	59.84	-3.85
W-2	57.81	1.85	59.66	59.82	-9.77
W-3	57.79	1.88	59.67	59.79	-10.20
W-4	57.77	1.90	59.67	59.78	-9.51
W-5	57.70	1.85	59.55	59.71	-2.89
W-6	57.68	1.85	59.53	59.68	-0.07
W-7	57.66	1.85	59.51	59.67	-2.69
W-8	57.62	1.84	59.46	59.63	-1.54
W-9	57.59	1.89	59.48	59.60	-3.11
W-10	57.56	1.84	59.40	59.57	-0.16
W-11	57.53	1.82	59.35	59.53	+2.43
W-12	57.46	1.83	59.29	59.45	-0.77
W-13	57.36	1.91	59.27	59.36	-0.96
W-14	57.23	1.78	59.01	59.24	+2.63
W-15	57.05	1.86	58.91	59.07	-0.75
W-16	56.91	1.85	58.76	58.92	-1.23
W-17	56.60	1.86	58.46	58.61	-1.69
W-18	55.99	1.88	57.87	57.99	+2.57
W-19	55.12	1.89	57.01	57.12	-2.45
W-20	54.22	1.88	56.01	56.23	-5.28
W-21	52.13	1.86	53.99	54.73	-2.50
W-22	50.41	1.85	52.26	52.42	-7.52

The dewatering system installed in 1981 included 17 vibrating wire piezometers. The piezometers were installed on the downstream side of the pump well line on the crest of the Spur to monitor the efficiency of the pump well system in lowering the water table.

As part of the 1981 field works, 3 pumping tests were carried out in the intermediate aquifer during the pump well system installation in 1981. The tests were conducted in wells W-3, W-10 and W-17 to evaluate the aquifer parameters. Water levels were monitored in the surroundings wells and piezometers during the pumping and recovery periods.

3.11 1996 FIELD WORKS

A rehabilitation program was carried out on the dewatering system in summer 1996. The major works included:

- Acquisition and review of data;
- Retrieving, checking and cleaning of pumps;
- Video inspection of wells before flushing;
- Cleaning of wells by flushing;
- Video inspection of wells after flushing;
- Reinstalling pumps and functional testing.

The dewatering system began operations in November 1981 and has continued essentially uninterrupted since then. In 1984, the power line to Muskrat Falls was struck by lightning and the resulting spike destroyed all of the vibrating wire piezometers. As a result, the piezometric measurements terminated in December 1983. Subsequent records covered pump functions only, namely pumping duration and the number of pump cycle initiations per day. Inspections were carried out by Hydro staff in 1994 and 1995, at which time pumps were retrieved, cleaned and reinstalled.

During the assessment it was noted that well W-4 has the highest yield and was responsible for more than 80% of the total discharge of the system.

During the field work it was found that there was no pump in well W-1. It was also found that well W-2 had been decommissioned in 1995.

During the periods of cleaning and flushing, the water level in the wells was closely monitored.

Chemical analyses were carried out in water samples obtained from some wells, the collector and from the Churchill River.

An assessment of the groundwater regime in the stratified drift unit and its response to pumping operations was performed.

The assessment stated that the most significant conclusion from the standpoint of spur stabilization is related to the lowering of the water table as a result of the operation of the dewatering system.

It was recommended to reinstall new piezometers to replace those destroyed in 1984 and that consideration should be given to supplementary instrumentation to measure seepage flows at critical locations of the North Spur.

3.12 1997 FIELD INVESTIGATIONS

An instrumentation installation program was undertaken in 1997. A new set of standpipe piezometers were installed to monitor the ground water level within the North Spur and 2 V-notch weirs were installed in the downstream area to measure seepage/runoff from the downstream slope.

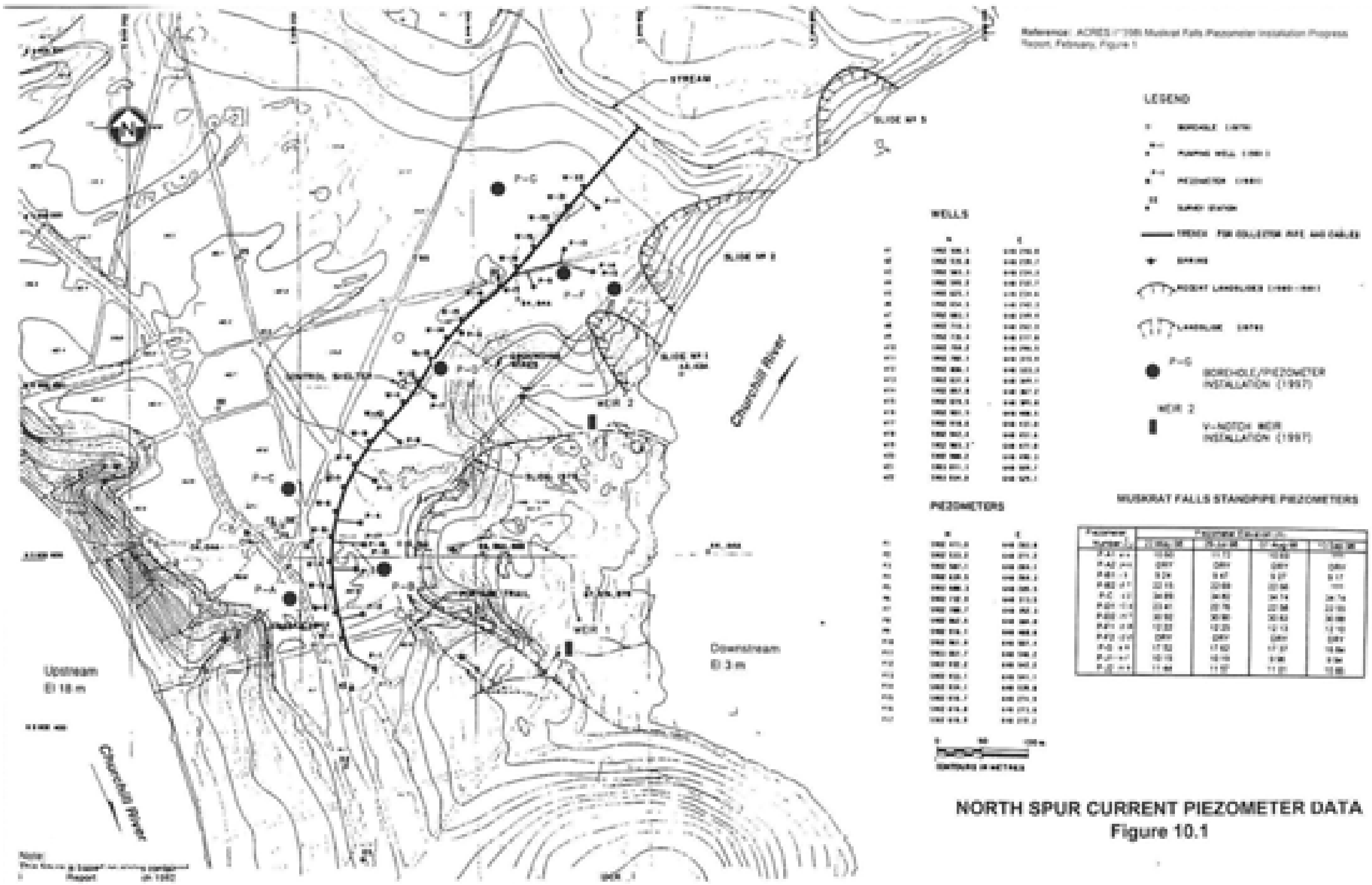
From 12 standpipe piezometers installed in 7 different boreholes, 4 were installed on the upstream side and 8 on the downstream side of the pump well system to assess the performance of the system. All piezometers were installed within the stratified drift, except for piezometers P-J1, PF-1 and P-G on the north side of the dewatering system which were installed in the lower clay layer. Tip elevations of the piezometers are listed in Table 3-5 and layout of the instrument installation is shown in Figure 3-13.

Laboratory testing of silty clay and sand samples retrieved from the boreholes was also performed. Testing consisted of soil index properties tests and grain size analysis.

Table 3-5 : 1997 Piezometers – Tip Elevations

Piezometer	Tip Elevation (m)
P-A1	6.55
P-A2	24.35
P-B1	1.30
P-B2	12.75
P-C	15.25
P-D1	12.63
P-D2	25.74
P-F1	0.82
P-F2	12.01
P-G	4.00
P-J1	-6.14
P-J2	10.61

Figure 3-13 : 1997 Piezometers and Weirs Installation



3.13 1998 FIELD INVESTIGATIONS

Two boreholes were drilled on the downstream side of the Spur during 1998 field investigation campaign as part of the final feasibility study carried out by SNC-AGRA [Refs. 20 and 21]. These boreholes were drilled to provide more information for the potential rockfill toe berm foundation design. Boreholes are listed in Table 3-6 and the locations are shown in Figure 3-14.

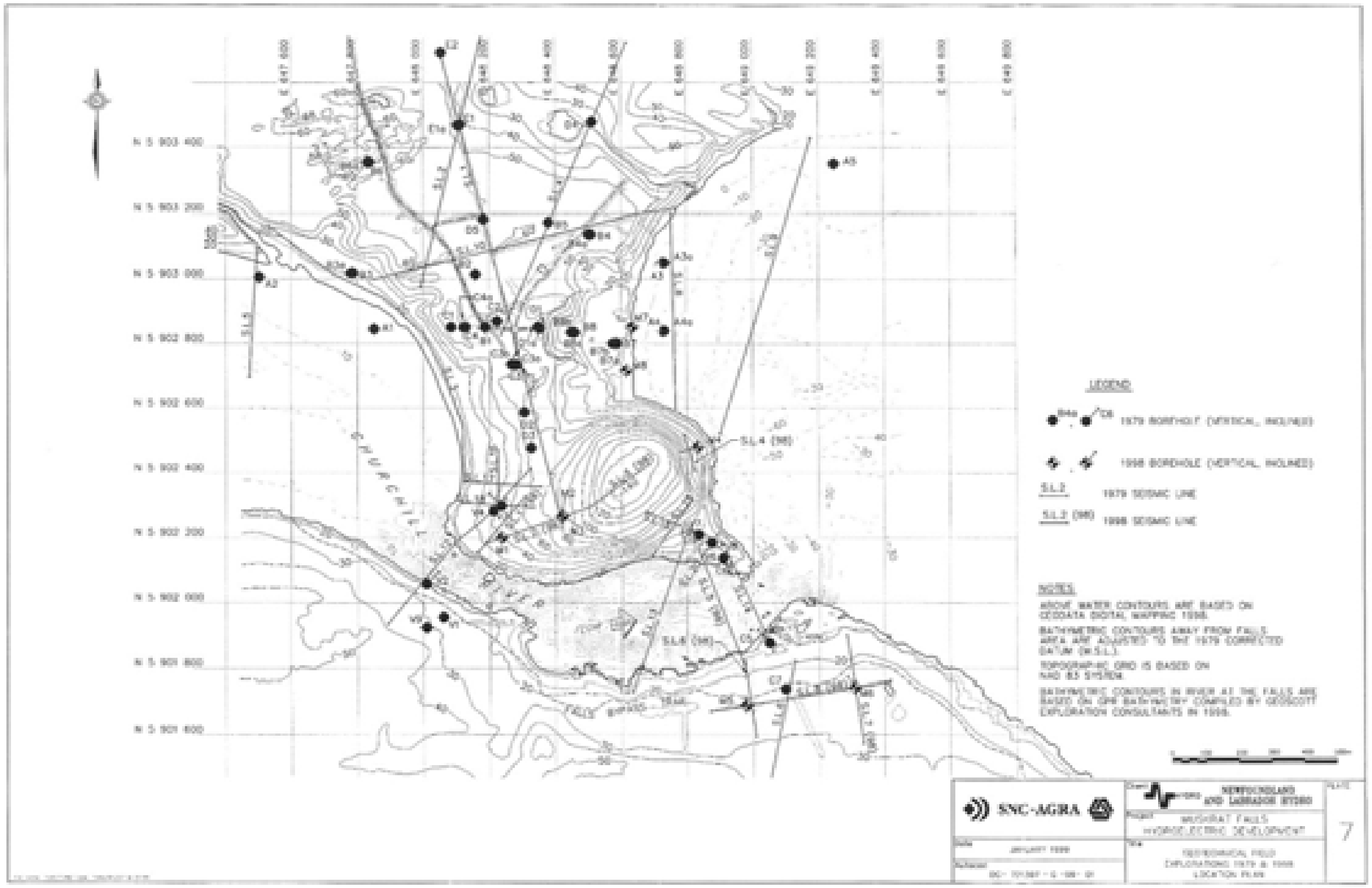
Table 3-6 : 1998 Field Investigations – Boreholes

Borehole No.	Elevation (m)	
	Ground surface	End of Hole
M7	2.57	-28.26
M8	5.64	-25.3

Test pitting was performed during the investigations in potential construction materials borrow areas. Details of the test pitting is presented in more details in Section 6.

Laboratory tests were conducted on samples retrieved from boreholes and test pits.

Figure 3-14 : 1979 and 1998 Investigations



3.14 1998 ENGINEERING STUDY

A Final Engineering Feasibility Study of the Muskrat Falls Hydroelectric Development was carried out during 1998 [Refs. 20 and 21]. As part of this study, reassessment of the stratigraphy and hydrogeological conditions of the North Spur was performed. During this study, it was concluded that in order to increase the stability of the Spur sufficiently to provide adequate stability for long-term operating conditions after reservoir impounding, a number of stabilization measures would be necessary.

Previous engineering studies had considered a number of different approaches to stabilization. Information gained from installation and operation of the interim dewatering system showed that control of the groundwater in the North Spur would be the most cost-effective and reliable approach to stabilization.

The stabilization measures included lowering the groundwater table in the downstream area of the spur by the installation of additional wells, construction of a drainage trench in the south part of the Spur near the rock knoll and construction of a partial till blanket in the south part of the upstream area; provision of downstream erosion protection and downstream stabilizing fill; local top-cutting of the spur; provision for erosion protection and stabilizing berm in the upstream area and improvement of the drainage in the Kettle Lakes area.

Stabilization works would also include installation of some relief wells in the lower aquifer.

It was also recommended that in order to assess the effectiveness of the stabilization measures, instrumentation and monitoring of the area during construction, reservoir impounding and operation, should be included as part of the planed works.

Layout and cross-sections of the proposed 1998 stabilization works are presented in Figure 3-15 and Figure 3-16.

Figure 3-15 : 1998 Stabilization Works – Layout

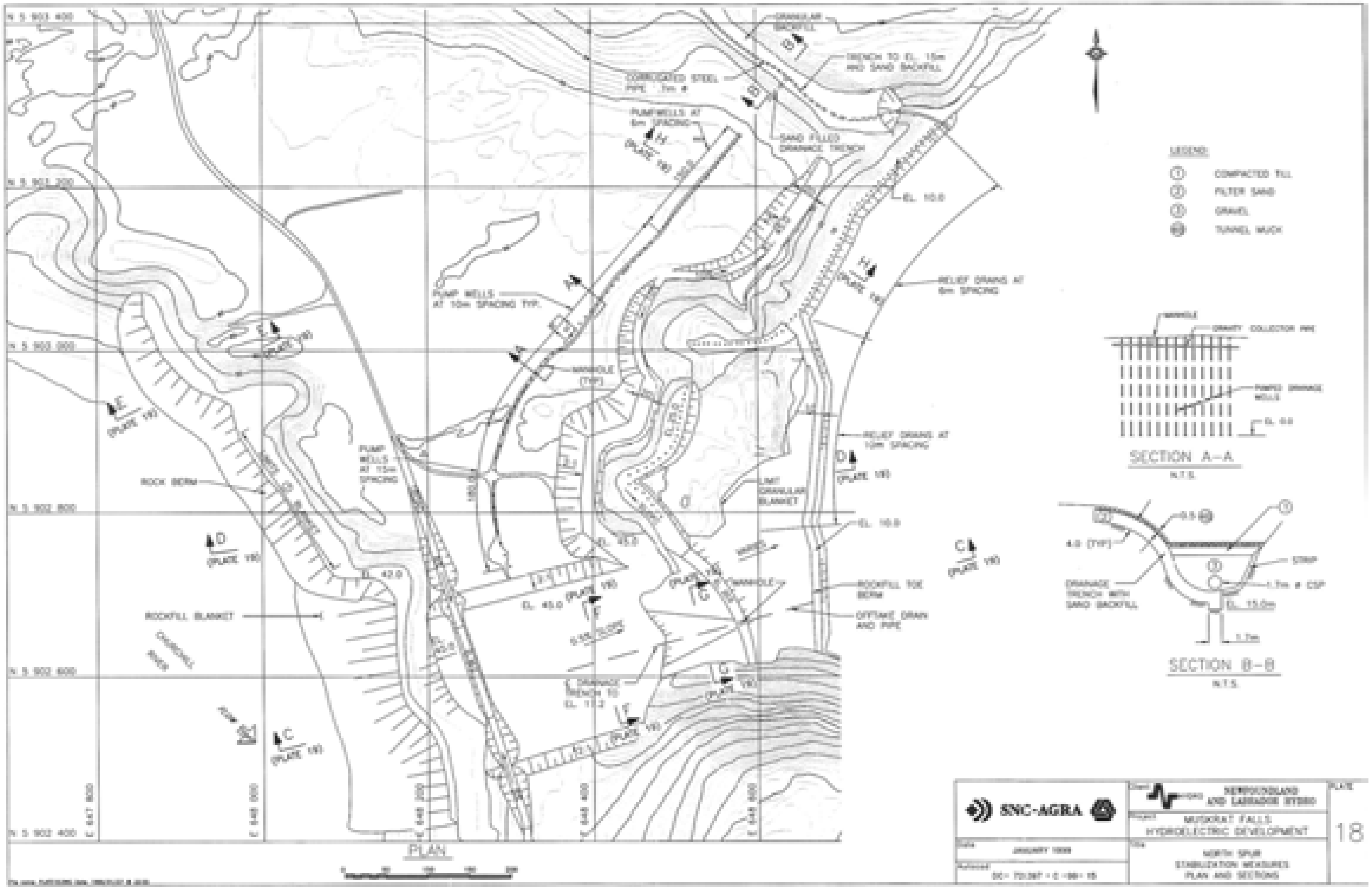
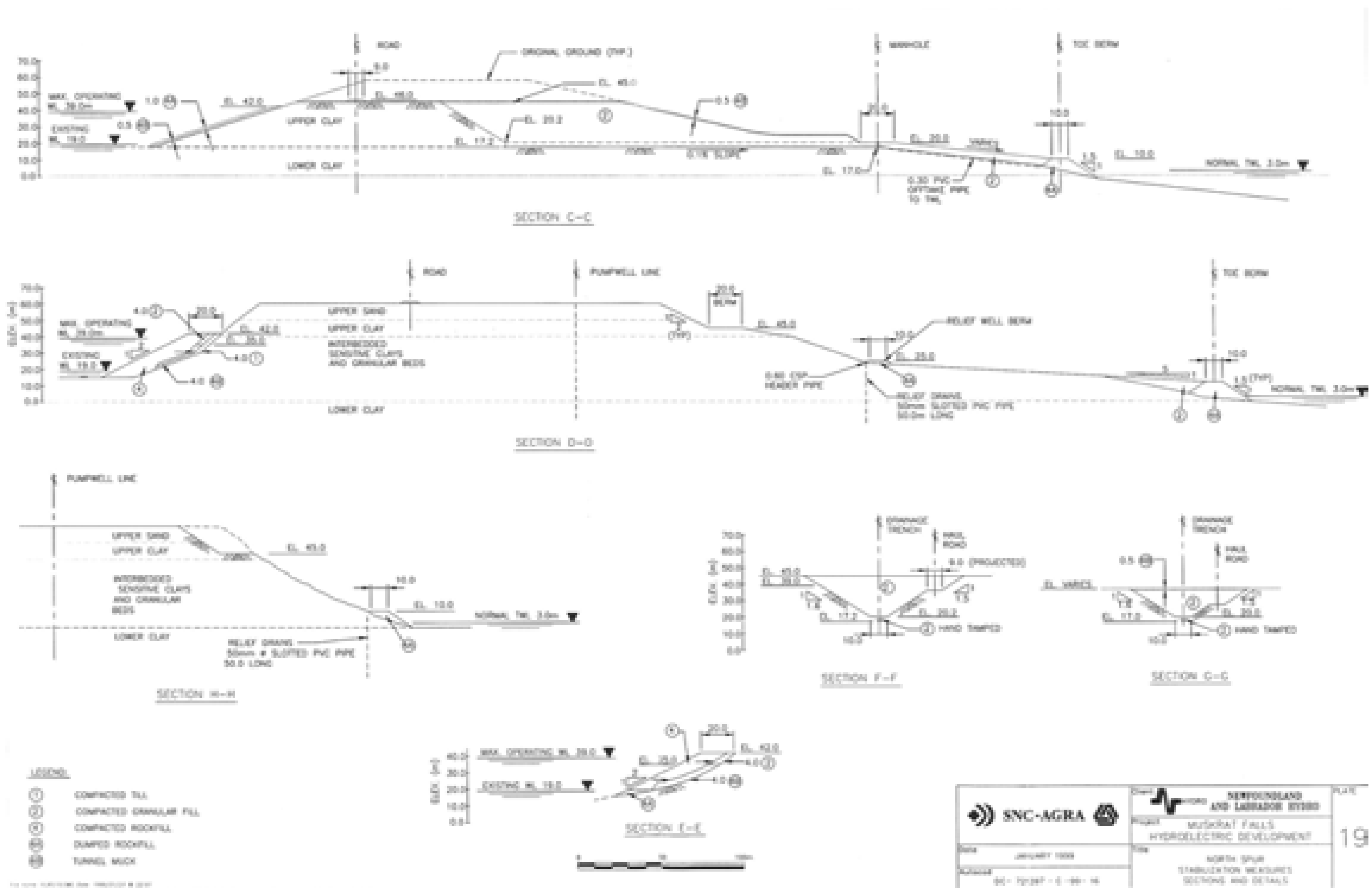


Figure 3-16: 1998 Stabilization Works – Cross Sections



3.15 2007 FIELD INVESTIGATIONS

From September to November 2007 an assessment of the existing dewatering system was carried out to determine the suitability of the system with a view to a life extension of 10 years.

The review included a site visit and inspection of the system to determine the present physical condition and operational characteristics. During the visit it was found that well W-22 was pulled out due to malfunctioning.

During the assessment work, the system was shut down and water level variations in wells and piezometers were recorded during and after the shut down period. Water discharge rate was measured and water quality was assessed at the collector pipe outlet.

Assessing of the performance of the dewatering system was made by analysis of: historical data, groundwater levels before, during and after the field works, meteorological and hydraulic data and historical pump operation.

As result of this assessment it was recommended to undertake a rehabilitation of the entire system including replacement of decommissioned wells, installation of new piezometers and wells, testing of all electrical components and review of the data monitoring and transmission system. General recommendations to assist with the ongoing dewatering operations were also done.

3.16 2009 FIELD WORK

During summer 2009, cleaning and inspection of the 22 wells of the existing dewatering system was performed and 8 standpipe piezometers were installed, as recommended following the 2007 assessment This was undertaken in order to extend the life of the system and ensure continued operation for a further 10 years.

Details of piezometer installation including water table level readings at that time are shown in Figure 3-17 and piezometer locations in Figure 3-18.

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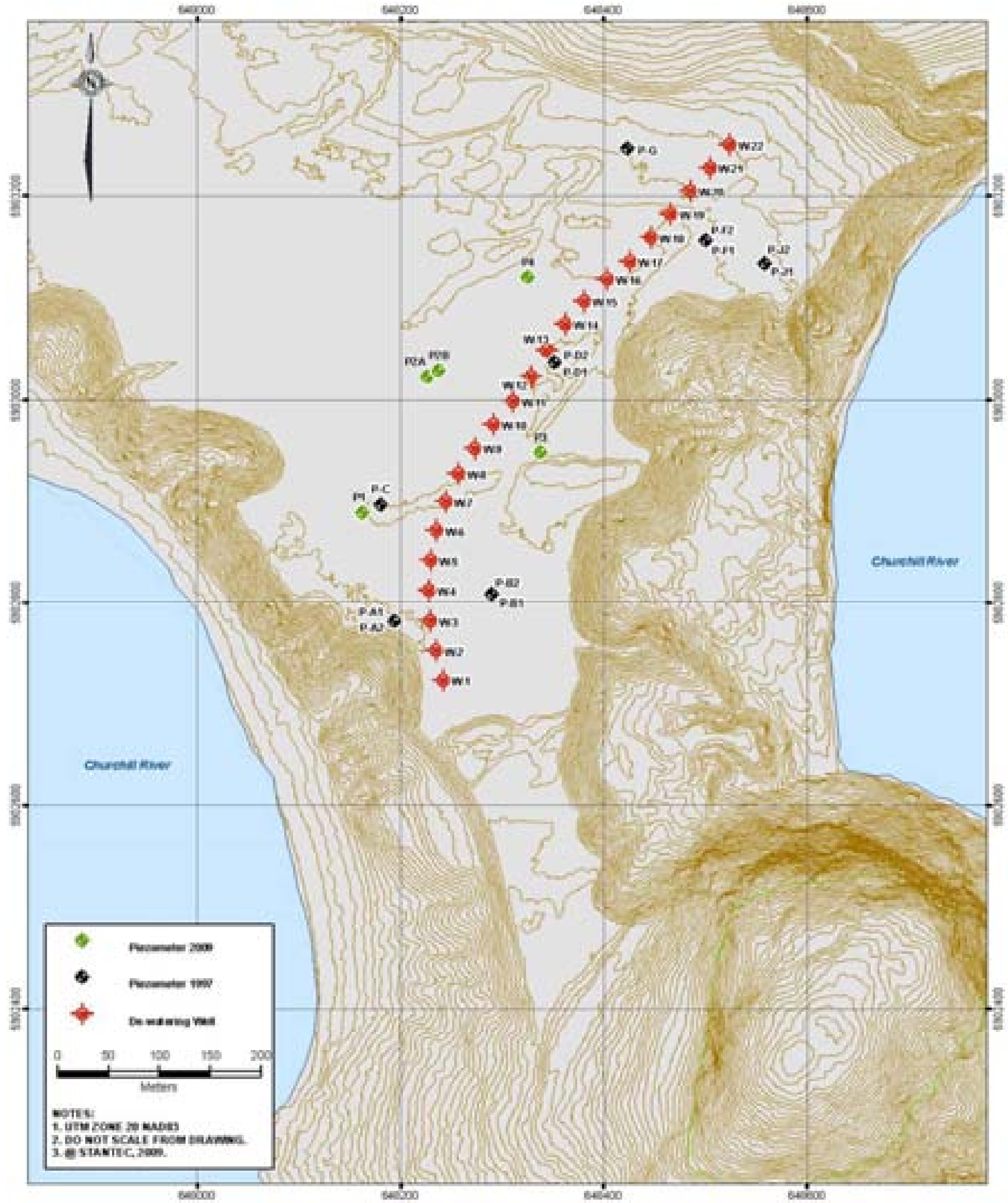
Figure 3-17 : 2009 Piezometer Installation

Piezometer Number	Northing	Easting	Ground Surface Elevation (m)	End of Hole (m)	Elevation of Top of Protective Casing (m)	Bottom of Piezometer Tip (m)	Top of Monitoring Zone (m)	Bottom of Monitoring Zone (m)	Water Level Sept. 9/09
2009 P1A	5902903.1	648228.9	(61.01)	42.7 (18.31)	(61.84)	25.02 (35.99)	21.92 (39.09)	24.97 (36.04)	15.89 (45.95)
2009 P1B						42.58 (18.43)	39.53 (21.59)	42.52 (18.49)	26.08 (35.76)
2009 P2A	5903029.9	648290.9	(59.39)	35.37 (24.02)	(60.33)	33.53 (25.86)	30.43 (28.96)	33.48 (25.91)	17.86 (42.47)
2009 P2B	5903032.8	648296	(59.45)	50.50 (0.95)	(60.27)	47.95 (11.50)	44.85 (14.60)	47.90 (11.55)	24.63 (35.64)
2009 P3A	5902950.1	648369.8	(58.39)	40.93 (17.46)	(59.21)	23.17 (35.22)	20.07 (38.32)	23.12 (35.27)	14.88 (44.33)
2009 P3B						40.63 (17.76)	37.58 (20.81)	40.48 (17.91)	23.96 (35.25)
2009 P4A	5903119.9	648378.9	(54.26)	46.0 (8.26)	(55.02)	29.11 (25.15)	25.40 (28.86)	29.06 (25.20)	N/A
2009 P4B						44.07 (10.19)	40.97 (13.29)	44.02 (10.24)	N/A

Figure 3-18 : 2009 Piezometer Installation – Layout



During the work performed in the wells, flow rate testing was conducted prior to the start of the well inspection and after completion of the program. A downhole camera inspection was completed in all wells prior to and after cleaning of the wells. Following the initial camera inspection, pumps, hardware and infrastructure were inspected and all findings documented and registered. Well cleaning operations followed and post cleaning downhole camera inspection performed. Once these operations were completed, pump, riser sections and associated wiring were returned into the well and reconnected.

After the rehabilitation work, it was concluded that this work was going to aid in ensuring the continued operation of the system over the following 10 years.

Other recommendations presented in the report included replacement of some of the well hardware, creation of a maintenance record register, electrical repairs to the system, assessment of the location and state of the sensors, installation of flow monitoring devices, installation of 3 to 4 new wells and manual recording of water levels.

Selected recommendations from the 2007 field works were the basis for the 2009 piezometer installation program, which included the drilling of 5 boreholes and the installation of 8 standpipe piezometers. Soil samples were collected from the boreholes and grain size analyzes completed on samples. Falling head permeability tests were also performed inside 2 boreholes.

Recommendations with respect to the piezometers installed in 1997 and 2009 included the installation of a data acquisition system and automatic data transmission for all piezometers.

3.17 2011 FIELD WORK

In April 2011, a total of 8 water level loggers were installed in the 2009 standpipe piezometers. The reading frequency was set to every 12 hours which would enable a data base of readings to be assembled prior to the first stages of river diversion. An automatic recording rain gauge was also installed on the shelter roof to measure the precipitation; a daily reading frequency was established for the rain gauge.

Details of the installation of the data loggers and groundwater level are presented in Table 3-7.

Table 3-7 : 2011 Data Logger Installations

Piezometer	Elevation (m)			Groundwater Table Elevation (m)	
	Tip	Ground Surface	Sensor Tip	Datalogger	Manual
P1A	35.99	61.01	36.00	45.42	45.41
P1B	18.43	61.01	24.81	35.36	35.0
P2A	25.86	59.39	27.89	34.92	35.1
P2B	11.5	59.45	17.83	25.29	25.24
P3A	35.22	58.39	35.22	39.02	38.74
P3B	17.76	58.39	22.18	27.84	27.24
P4A	25.15	54.26	25.15	29.77	29.56
P4B	10.19	54.26	12.87	19.55	19.76

3.18 2013 FIELD INVESTIGATIONS

The 2013 investigation campaign was planned to fill the gaps in the existing information from the previous investigations and to gather required parameters to finalize the design of the stabilization works.

The field works were completed in summer 2013 and the results are presented in the report prepared by AMEC [Ref. 25].

The 2013 investigation works included 6 conventional boreholes, 5 sonic boreholes, 27 Cone Penetration Tests (CPT), in-situ tests (SPT and VST), and laboratory testing. Ten piezometers were installed during the 2013 investigation campaign.

The bored and sonic holes and CPTs performed are listed in Table 3-8 and Table 3-9, respectively, and the locations are shown in Figure 3-19.

As part of the field works in the North Spur, the natural frequency of the soil material in the North Spur was determined using Tromino device [Ref. 30].

Table 3-8 : 2013 Field Works – Boreholes

Borehole No.	Type	Elevation (m)			
		Ground surface	End of Hole	Piezometer tip	Groundwater Table
NS-1-13	Conventional	55.3	-13.7	-	-
NS-1B-13	Conventional	55.6	10.5	11.48	Dry
NS-1C-13	Conventional	55.6	30	30.18	37.59
NS-4-13	Conventional	58.6	17.3	17.76 26.59	Dry Dry
NS-9-13	Conventional	16.3	-2.7	-	-
NS-11-13	Conventional	20.5	-12.6	-	-
NS-2-13	Sonic	61.4	-53.6	-53.2 4.45	34.93 Dry
NS-3-13	Sonic	59.1	-0.6	-	-
NS-3B-13	Sonic	59.1	4.8	5.21 19.76	31.48? 21.30?
NS-5-13	Sonic	59.1	-9.8	-	-
NS-6-13	Sonic	58.9	-8.3	15.91 26.26	16.91? 38.74
NS-9V-13	FVST	16.3	-0.8	-	-
NS-13-13	FVST	18.1	1.1	-	-

Table 3-9 : 2013 Field Works – CPT Testing

CPT No.	Ground Elevation (m)	Depth (m)	In-Situ Test		
			Seismic cone test	Dissipation Test	
				Tip level (m)	Piezometric Level (m)
1A-13	20.6	6.7	-	-	-
1B-13	20.8	7.5	-	13.51	15.55
2A-13	33.6	36.7	-	2.20 -2.93 -3.00	14.44 9.10 8.83
2B-13	29.8	32.0	-	-1.26 -2.16 0.76	8.94 8.34 11.37
3-13	21.0	30.0	-	-	-

CPT No.	Ground Elevation (m)	Depth (m)	In-Situ Test		
			Seismic cone test	Dissipation Test	
				Tip level (m)	Piezometric Level (m)
4-13	19.2	30.0	-	-	-
5-13	58.6	70.0	-	-	-
6-13	16.1	30.0	-	7.20 2.04	7.83 7.88
7-13	5.3	20.0	-	-	-
8-13	16.0	40.0	-	-	-
9-13	54.6	65.0	x	19.52 13.10	21.56 14.12
10A-13	47.9	38.5	-	18.34 15.51	27.62 18.57
10C-13	49.1	48.9	-	7.07 0.34	15.94 8.91
11-13	61.5	70.0	x	31.88 27.08	37.82 36.16
12-13	58.7	70.0	-	15.17	37.51
13-13	21.0	40.0	-	10.97 -4.63	22.49 17.81
14-13	20.0	30.0	-	-0.17	25.32
15-13	58.7	48.5	-	31.90 19.25 10.62	34.76 34.24 35.20
16-13	61.8	65.0	-	36.69 32.95 11.23	38.83 33.56 34.69
17-13	56.9	52.5	-	37.59 29.06 25.36	38.00 39.46 39.03
18-13	20.1	30.0	-	-	-
19-13	20.8	30.0	-	-	-
20-13	21.4	12.0	-	-	-
21-13	19.9	10.0	-	-	-
22-13	19.3	13.0	-	-	-
23-13	8.5	25.0	-	5.21	5.98
24-13	2.8	21.0	-	-	-

Figure 3-19 : 2013 Field Works

3.19 2013 ENGINEERING

The 2013 final design included a review of the different solutions presented in previous engineering studies and a review and re-interpretation of all the available data using the latest analysis techniques. The result of this evaluation has led to the final design of the engineering works for the stabilization of the North Spur. Section 4 presents all the geotechnical and hydrogeological data gathered during the investigation campaigns and also presents the North Spur geotechnical and hydrogeological model used in the final design.

Section 5 outlines the optimized layout of remedial measures for stabilization of the North Spur during construction and impoundments and under long-term operating conditions.

During the process of the final engineering, several technical reviews were performed on the design of the stabilization works. Outcomes of these reviews are also discussed in Section 5.

4 GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS

4.1 INTRODUCTION

Geological and hydrogeological conditions of the North Spur are reviewed in this section. Stratigraphy of the North Spur, material properties and groundwater regime inside the spur are discussed in detail. Discussion on causes of previous slides and assessment of the current stability of the slopes is also presented in this Section.

Final assessment of the stratigraphy of the North Spur was obtained from the interpretation of the available data gathered from the field investigations and the associated engineering studies presented in the previous sections.

Most of the current knowledge of the stratigraphy of the North Spur and the soil characteristics were derived from the outcomes of the extensive investigation campaigns carried out in 1979 and 2013. Complimentary information related to the stratigraphic and hydrogeological conditions was also obtained from the previous campaigns and field work. The hydrogeological information obtained from the pumping tests, piezometer readings, in-situ permeability tests and CPT dissipation tests allowed assessment of the hydrogeological conditions on the North Spur.

A 3D hydrogeological model of the North Spur was prepared and different hydrogeological conditions were simulated. Findings and relevant conclusions of these works are discussed in this section and are presented in detail in the report "Three Dimensional (3D) Hydrogeological Study for the North Spur", [Ref. 31].

4.2 STRATIGRAPHY

The stratigraphy of the overburden layers, from ground surface to bedrock level, was interpreted based on available data from geotechnical investigation campaigns. Continuous logs obtained from CPTs and sonic drillings during 2013 investigations along with conventional boreholes drilled during various investigations and test pitting operations, provided a good source to study and to identified the stratified nature of the soil.

The available information was analyzed and interpreted and a stratigraphic and hydrogeological model was prepared which included physical, mechanical and hydraulic characteristics of the in-situ materials.

Four distinct sedimentary units have been identified in and underlying the Spur.

- Upper Sand, generally from the crest of the Spur down to about elevation 50 to 45 m;
- Stratified Drift, including 2 major deposits of silty sand and silty clay materials, generally from elevation 50 m to 45 m to elevation 15 m and 5 m;
- Lower Marine Clay, generally from elevation 15 m and 5 m to elevation -70 m;
- Lower Aquifer, consisting of glacial sand, gravel and boulder infill of the preglacial valley to bedrock, generally below elevation -70 m.

As part of the current engineering works, a three-dimensional stratigraphic model of the North Spur was developed using commercial software Catia-V.5 (Dassault Systems).

A north-south cross-section of the North Spur and different schematic stratigraphic cross-sections of the identified sedimentary units interpreted from several CPT tests within the North Spur are presented in Figure 4-1 to Figure 4-5. Figure 4-6 to Figure 4-10 present a plan view and different sections through the North Spur, with major stratigraphic units and location of boreholes, wells and piezometers shown in each section. Piezometric levels recorded at boreholes and piezometers readings are shown in the figures. The nature and the physical and mechanical properties of each soil unit are discussed in the following sections.

Correlations between in-situ tests and physical properties and CPT tests were performed and correlations between them were used to prepare different cross-sections showing the physical and mechanical properties of the soils in each different stratigraphic unit.

Figure 4-1 : North Spur Simplified Stratigraphy – North-South Cross-Section

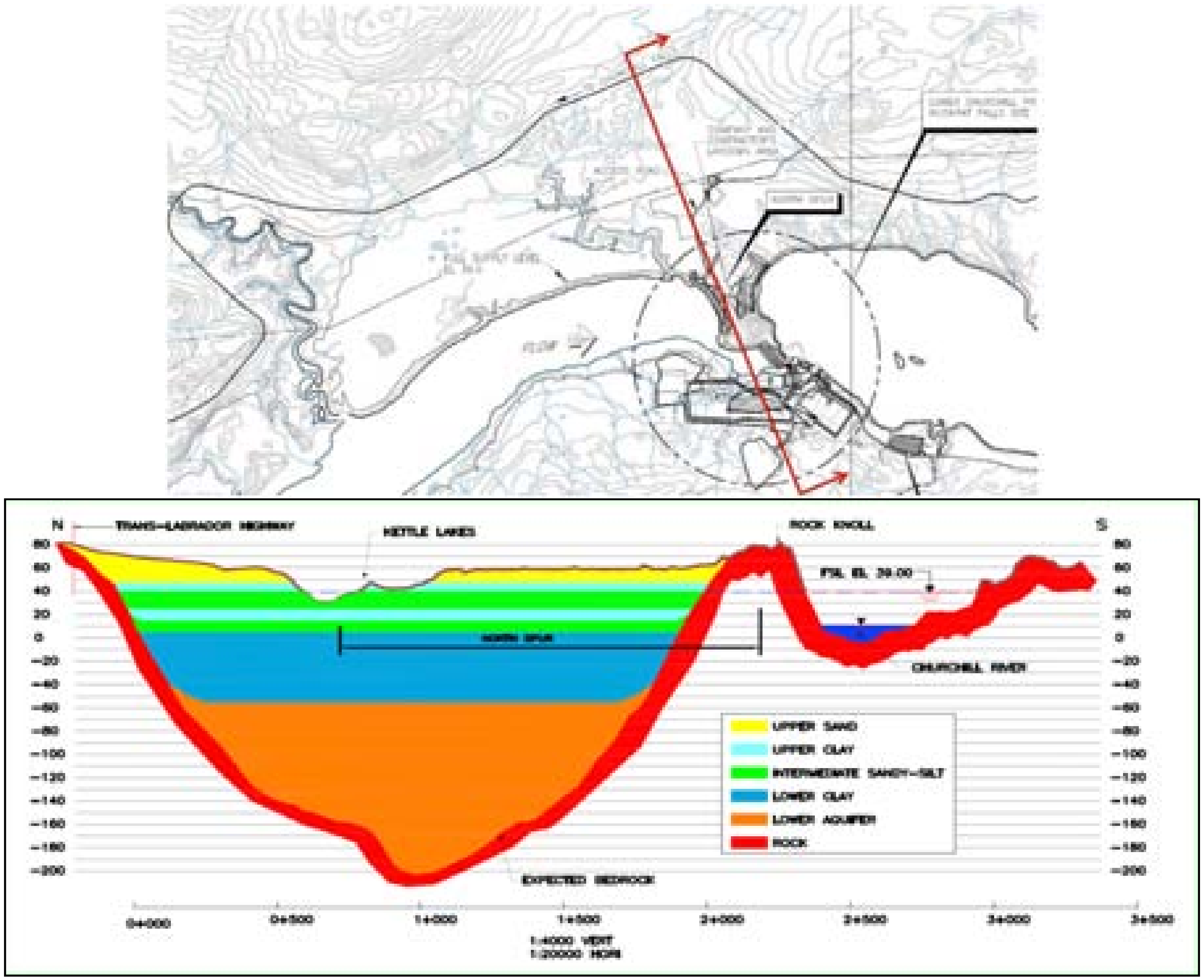


Figure 4-2 : CPT Locations

SNC·LAVALIN	NORTH SPUR STABILIZATION WORKS – DESIGN REPORT	Revision		
	Nalcor Doc. No. MFA-SN-CD-2800-GT-RP-0004-01	B1	Date	Page
	SLI Doc. No. 505573-3281-4GER-0001	PB	30-Jan-2016	66

Figure 4-3 : Schematic Stratigraphy – Cross Section – CPTs 22, 15, 12 and 17

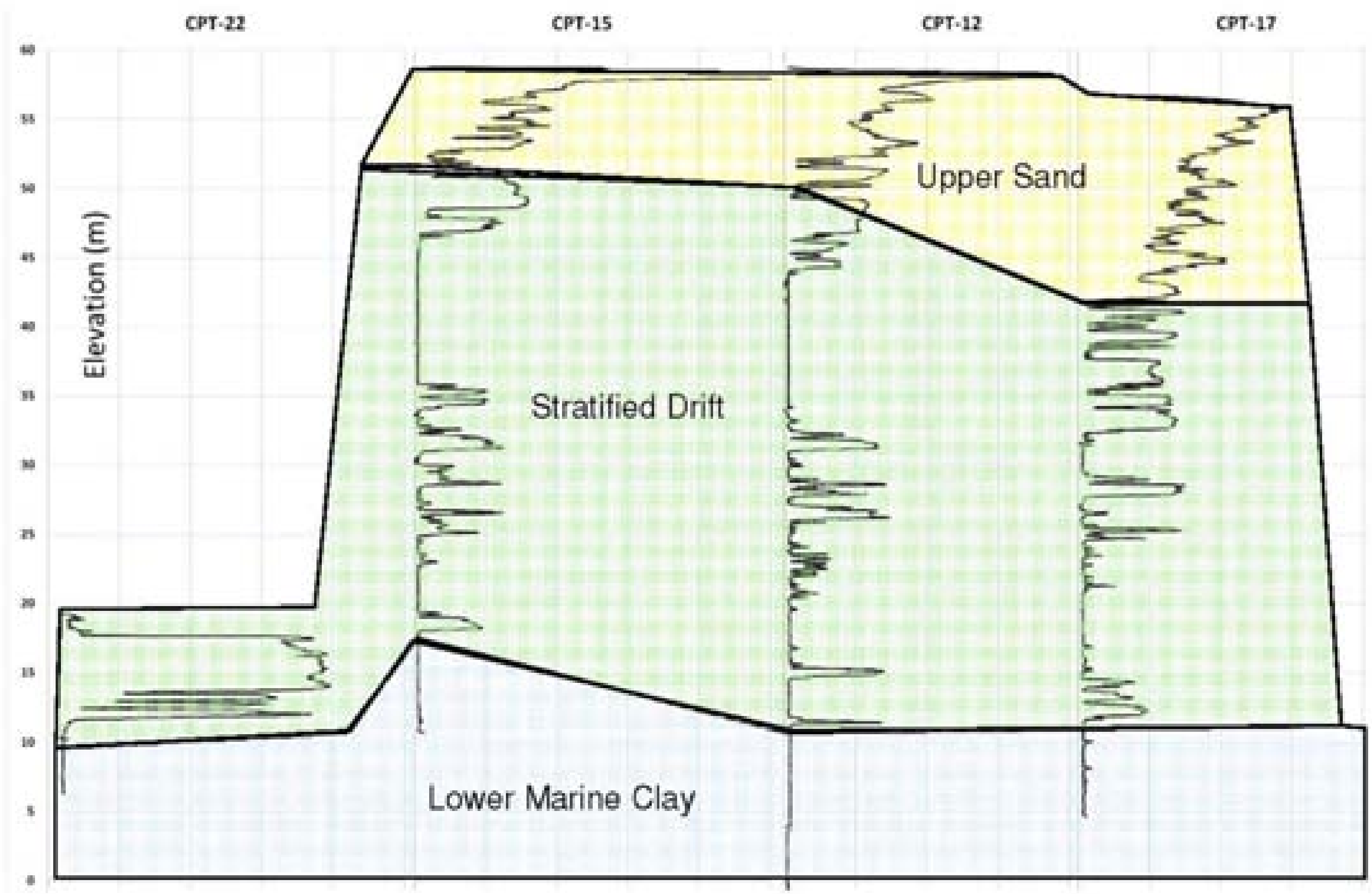
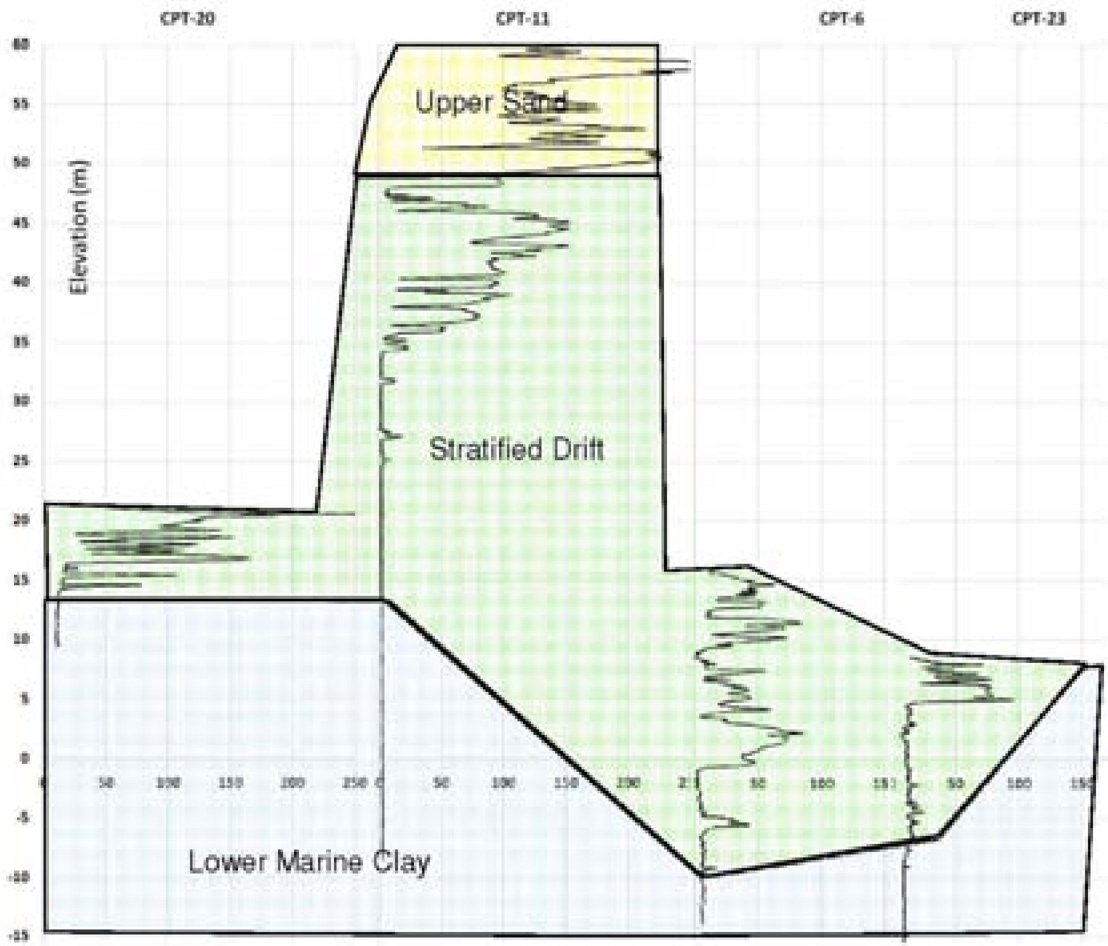


Figure 4-4 : Schematic Stratigraphy – Cross Section – CPTs 20, 11, 6 and 23



CIMFP Exhibit P-00448

Figure 4-5 : Schematic Stratigraphy – Cross Section – CPTs 9, 10A, 10C and 24

Figure 4-6 : Geotechnical Investigations – Plan View

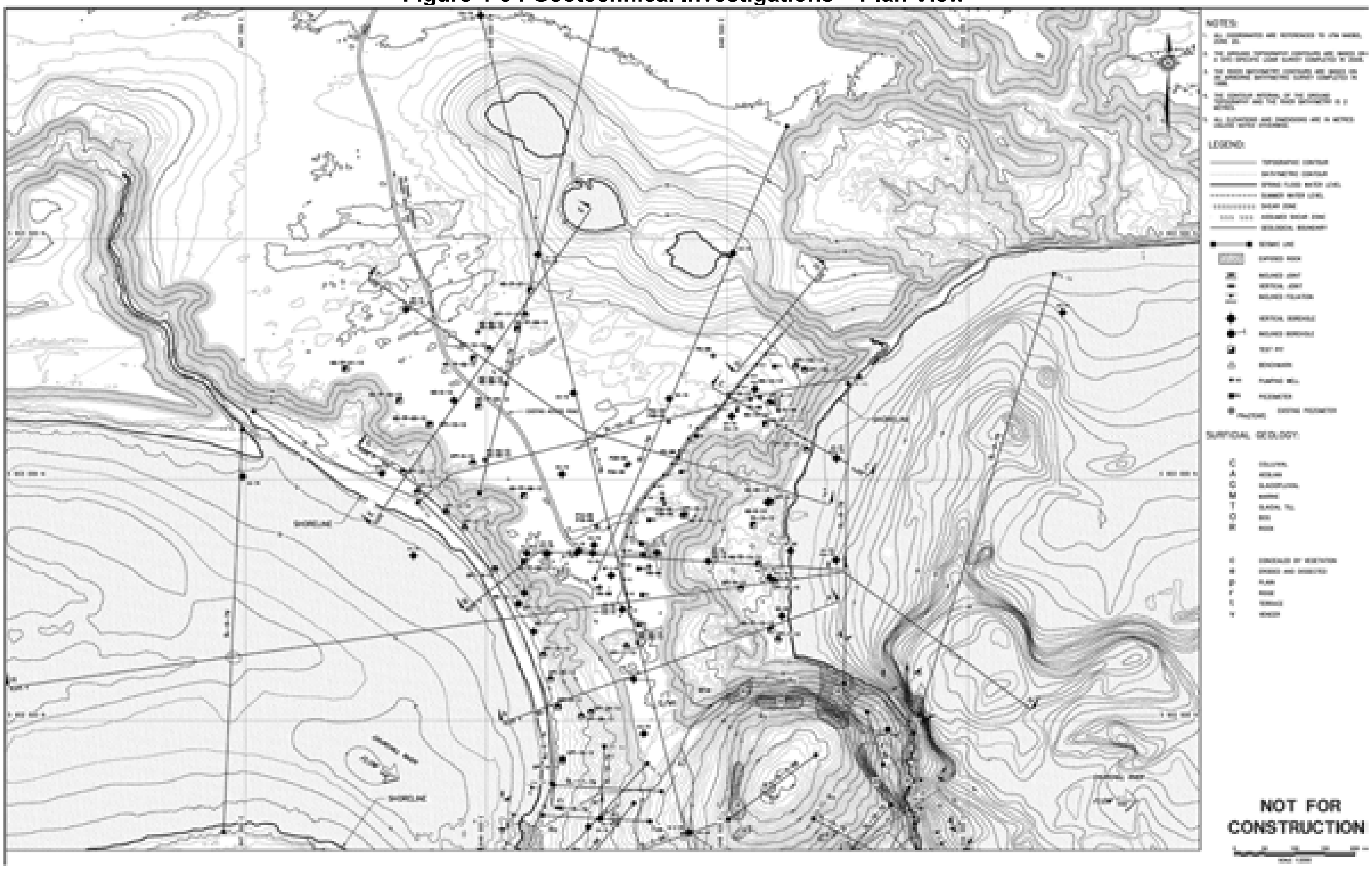


Figure 4-7 : Cross Sections A-A and B-B

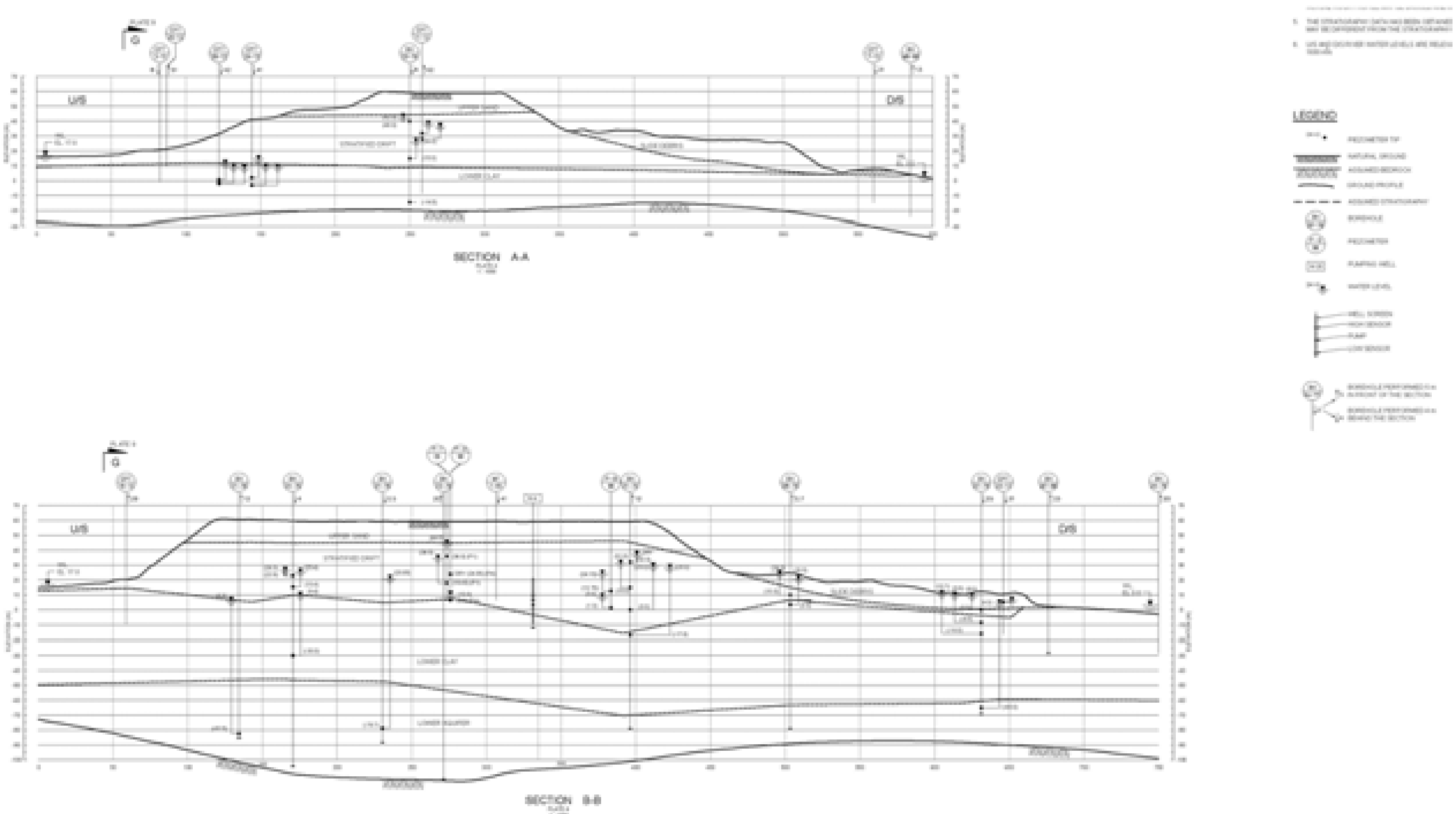
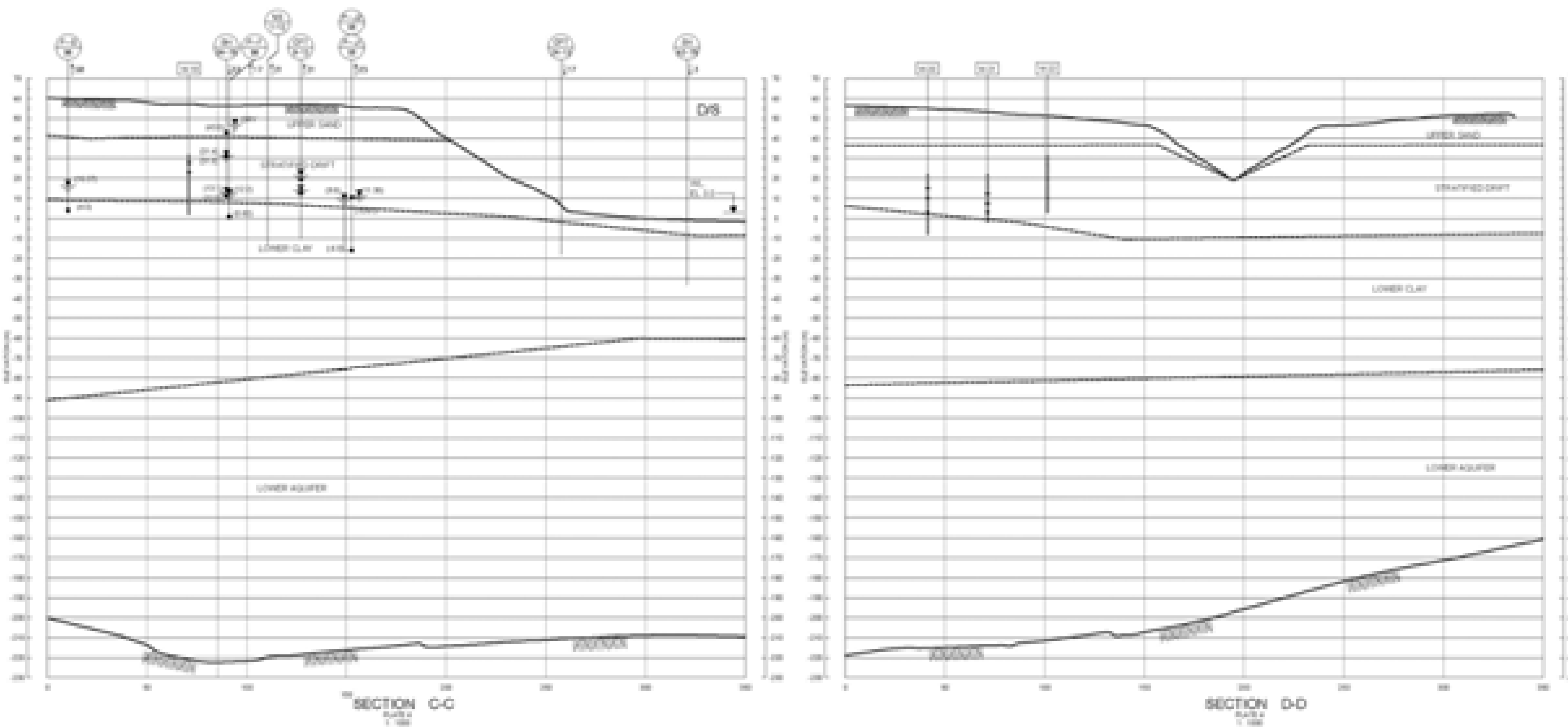
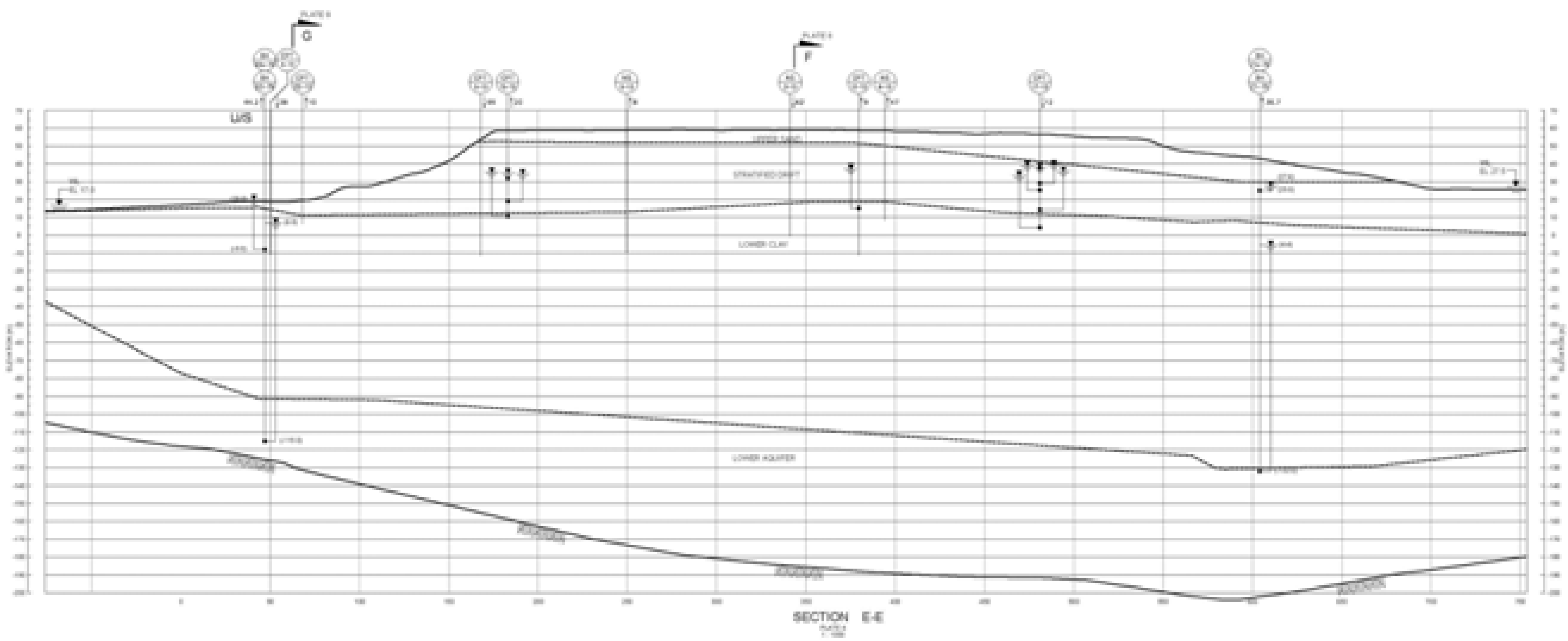


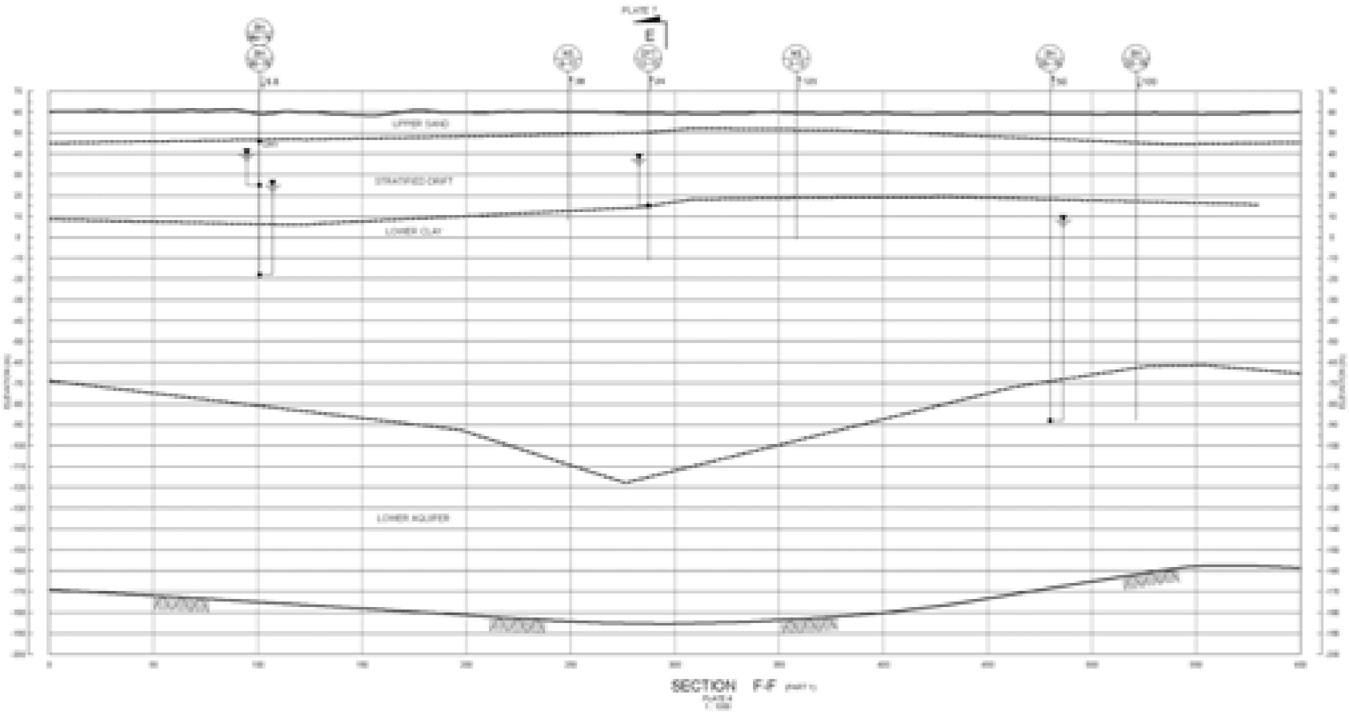
Figure 4-8 : Cross Sections C-C, D-D and E-E





G
F
U/S
SECTION E-E

Figure 4-9 : Cross Section F-F



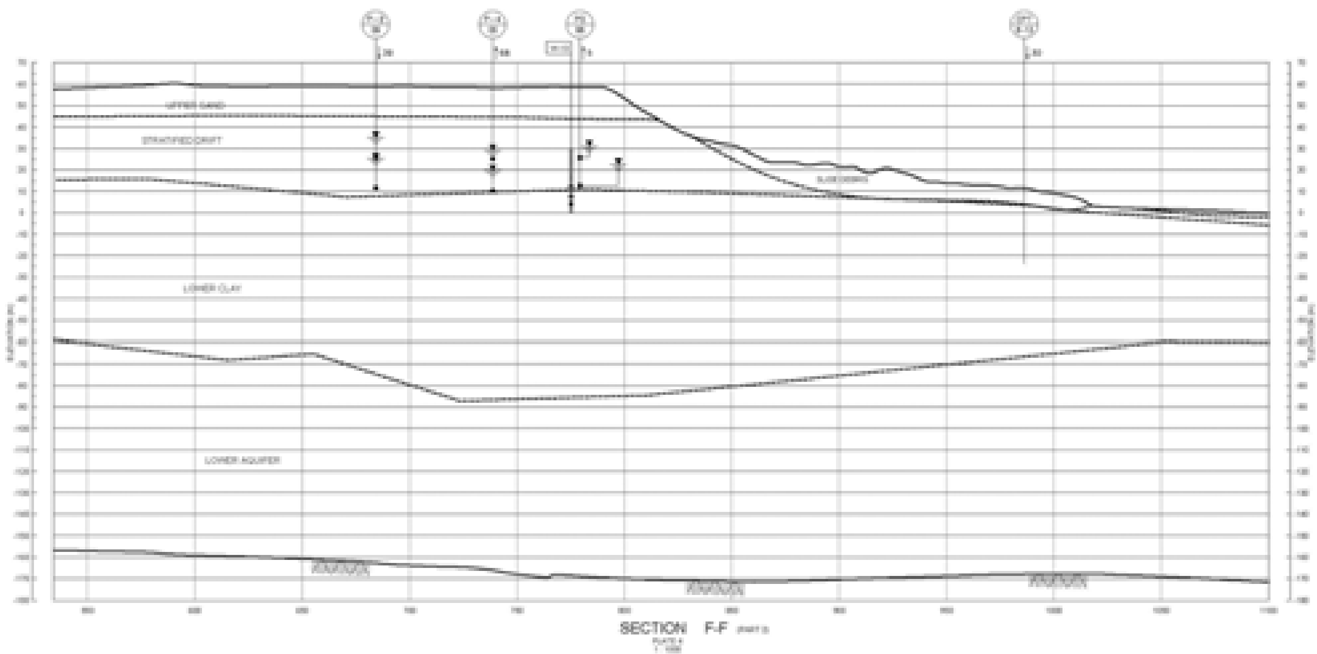
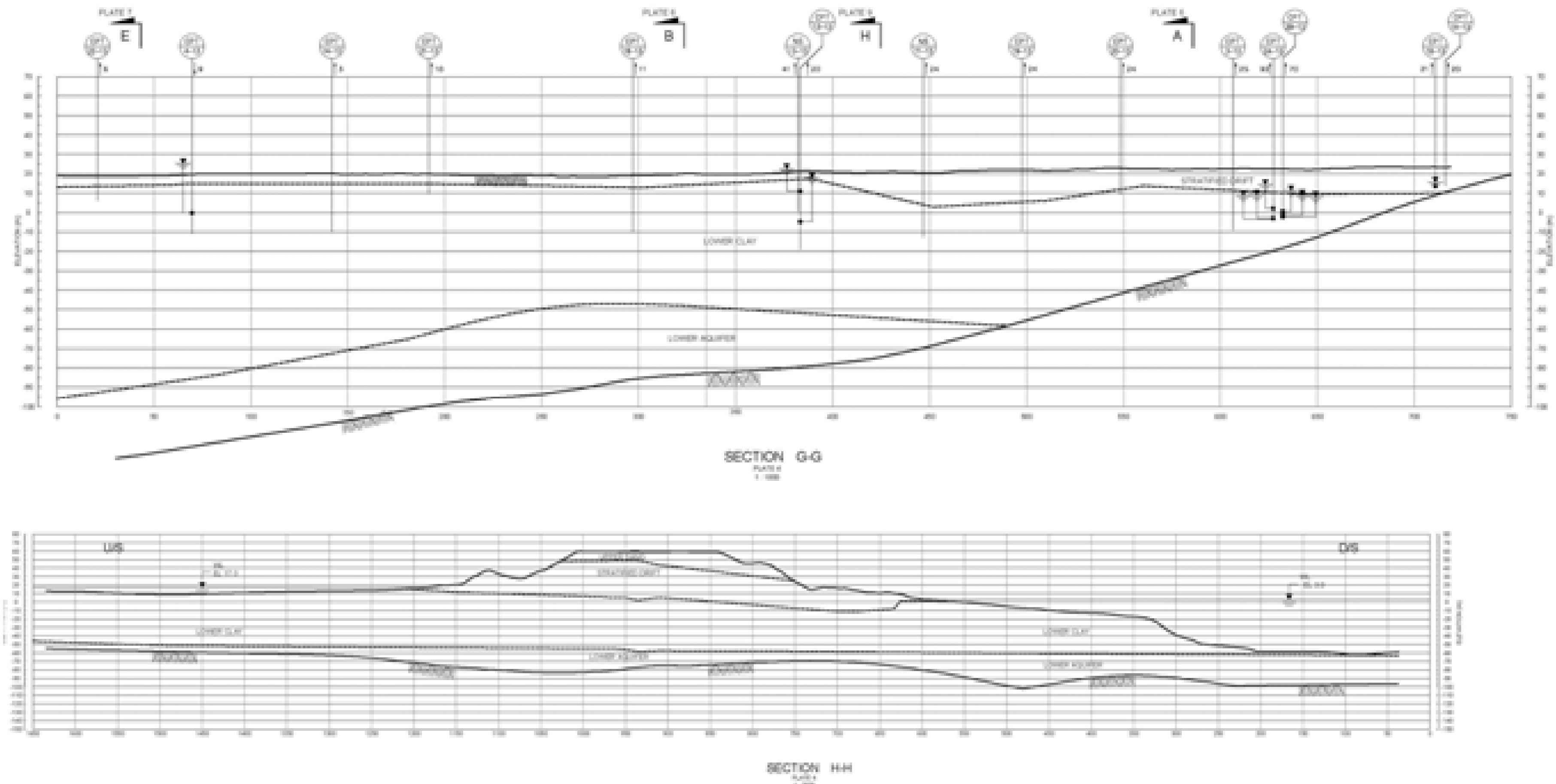

SECTION F-F

Figure 4-10 : Cross Sections G-G and H-H



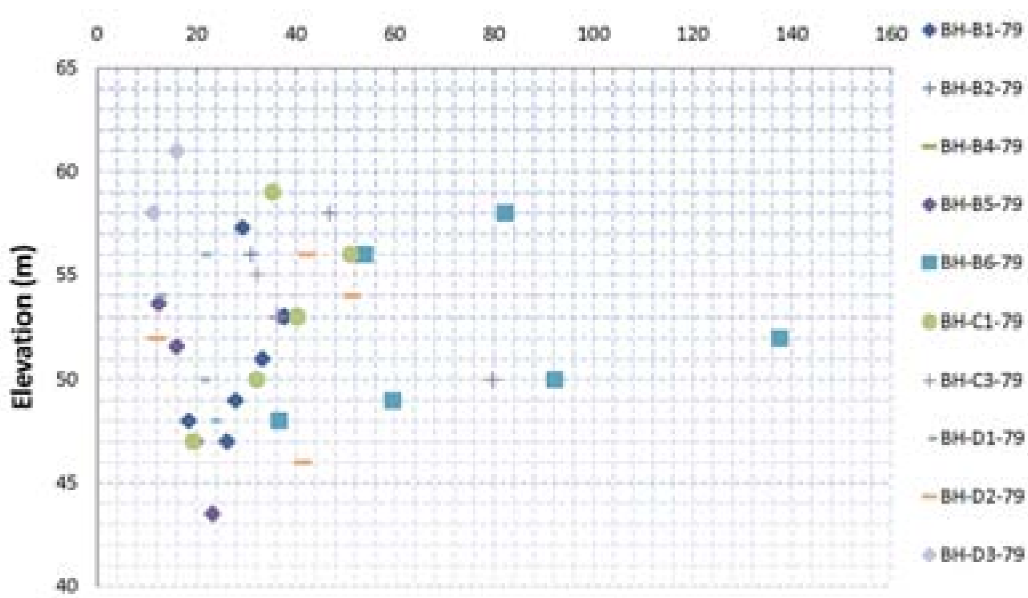
4.2.1 UPPER SAND UNIT

The upper sand unit covers the surface of the North Spur generally from elevation 60 to 50 to 45 m. This unit mainly consists of a grey, brown fine to medium sand with low fines content.

This layer is mostly dry and well drained with a perched water table above the underlying clay or silty clay layer.

According with the results of the Standard Penetration Test (SPT) performed in this unit during the 1979 investigation campaign, the compacity of this sand layer can be qualified as a compact to very dense. SPT values in the upper sand are shown in Figure 4-11.

Figure 4-11 : Upper Sand – SPT Values



The thickness of the layer is about 4 m in the south end of the Spur (BH-D3-79) which increases to 10 to 15 m for most part of the surface of the Spur.

Generally, grain size analysis on the samples recovered from this layer resulted in a range of fine content (percent passing sieve # 200 or 0.075 mm) from 1 to 9 percent.

The coefficient of uniformity (Cu) varies from 1.7 to 5.5 with an average of 2.6, which indicates a generally uniform material. The grain size curves, including the limit envelope of the construction material Zone 2F are presented in Figure 4-12 for the investigation performed in 1979, Figure 4-13 to Figure 4-16 for the investigations performed in 2013.

One set of direct shear test was performed on a remoulded sample obtained during the 2013 investigation campaign at elevation 47 m to 45 m. The tests resulted in peak and critical state effective friction angles of 35 º and 34º, respectively, with no cohesion. Values of unit weight of 19.2 kN/m^3 were calculated during the tests.

The layer is mostly dry and well drained except for a perched water table above the underlying silty clay layer.

Figure 4-12 : Upper Sand Layer – Grain Size Analyses – 1979 Investigations

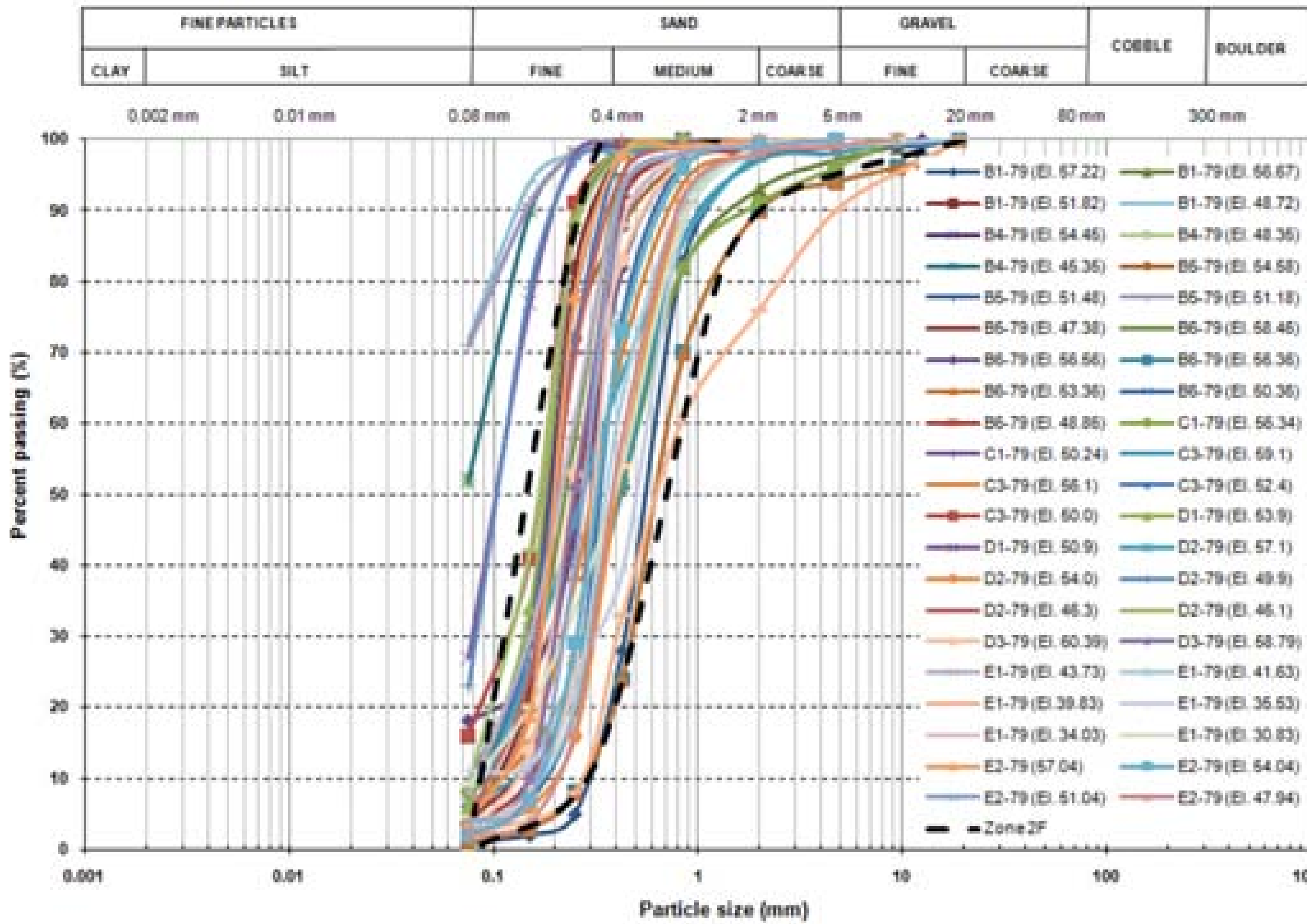


Figure 4-13 : Upper Sand Layer – Grain Size Analyses – 2013 Investigations

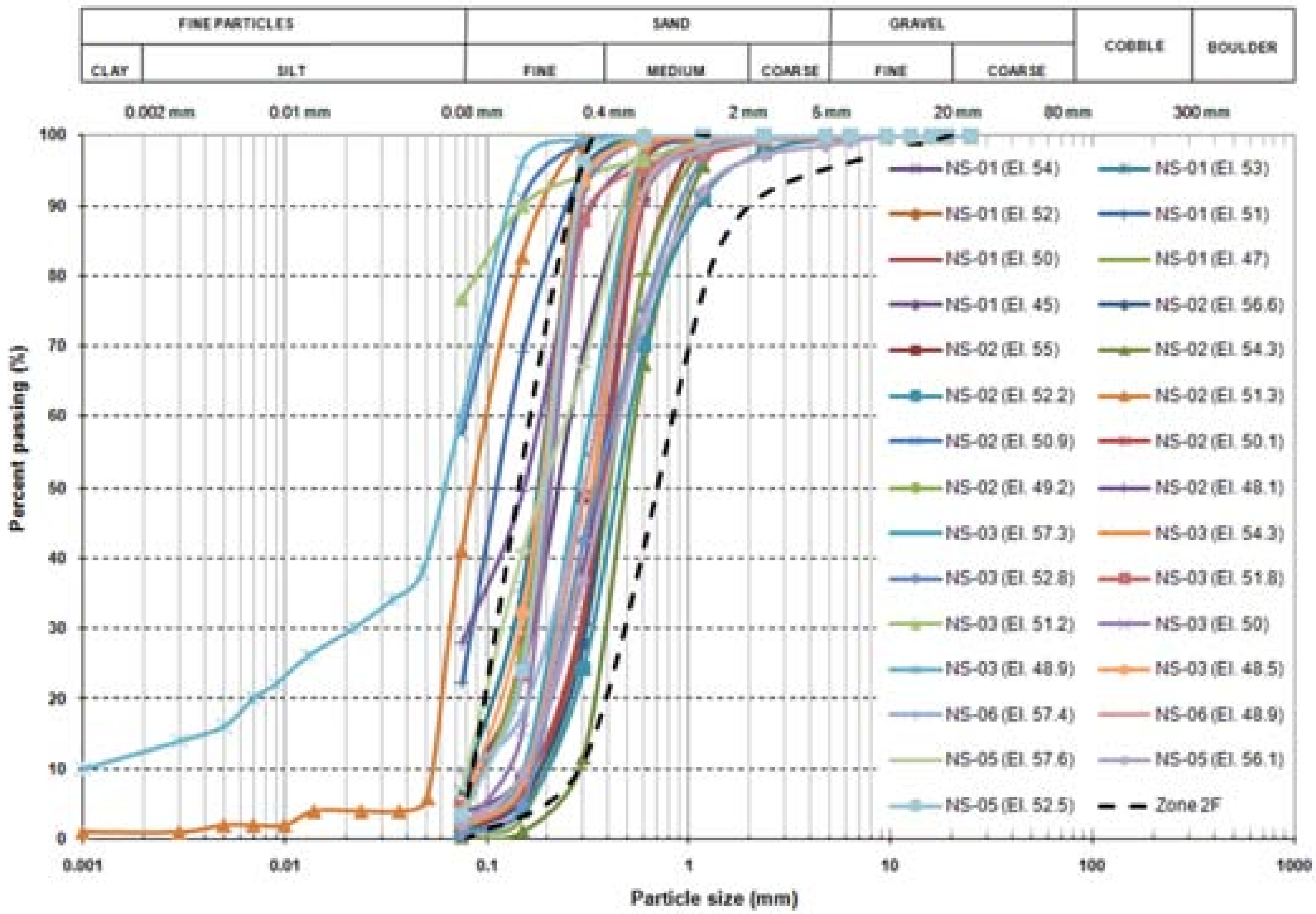


Figure 4-14 : Upper Sand Layer – Grain Size Analyses – 2013 Investigations – NW Cut-off Wall Area

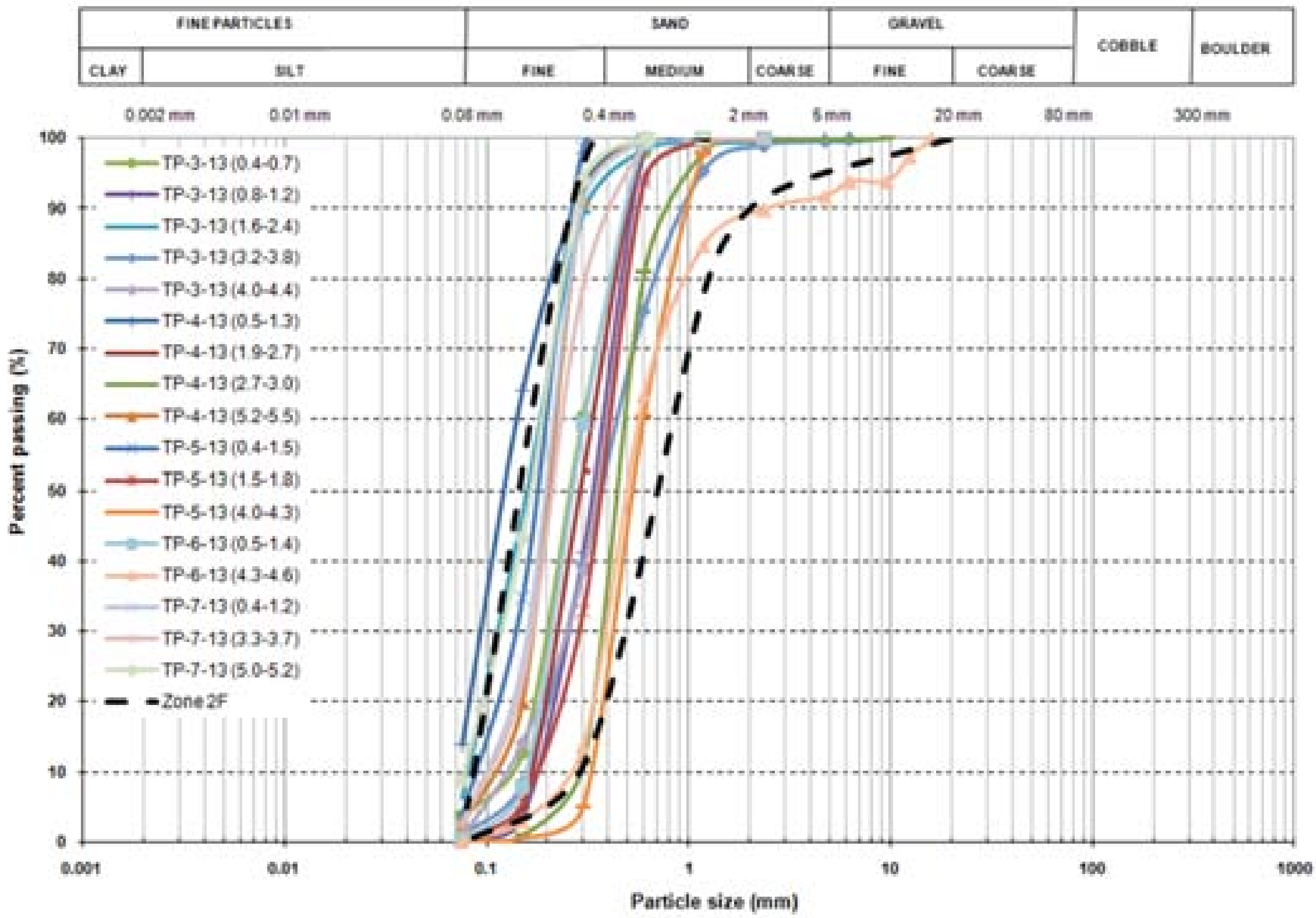


Figure 4-15 : Upper Sand Layer – Grain Size Analyses – 2013 Investigations – Upstream Slope Area

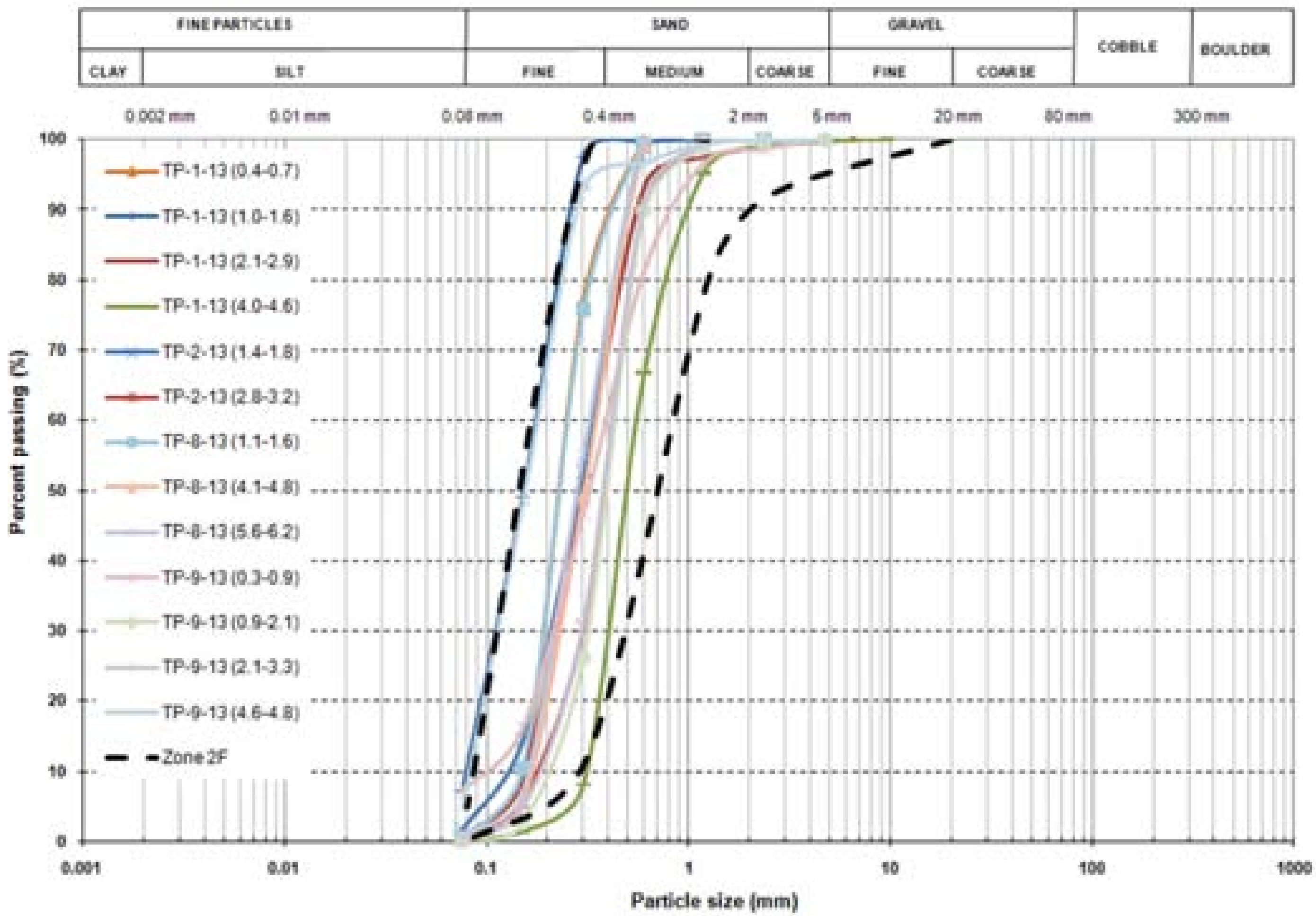
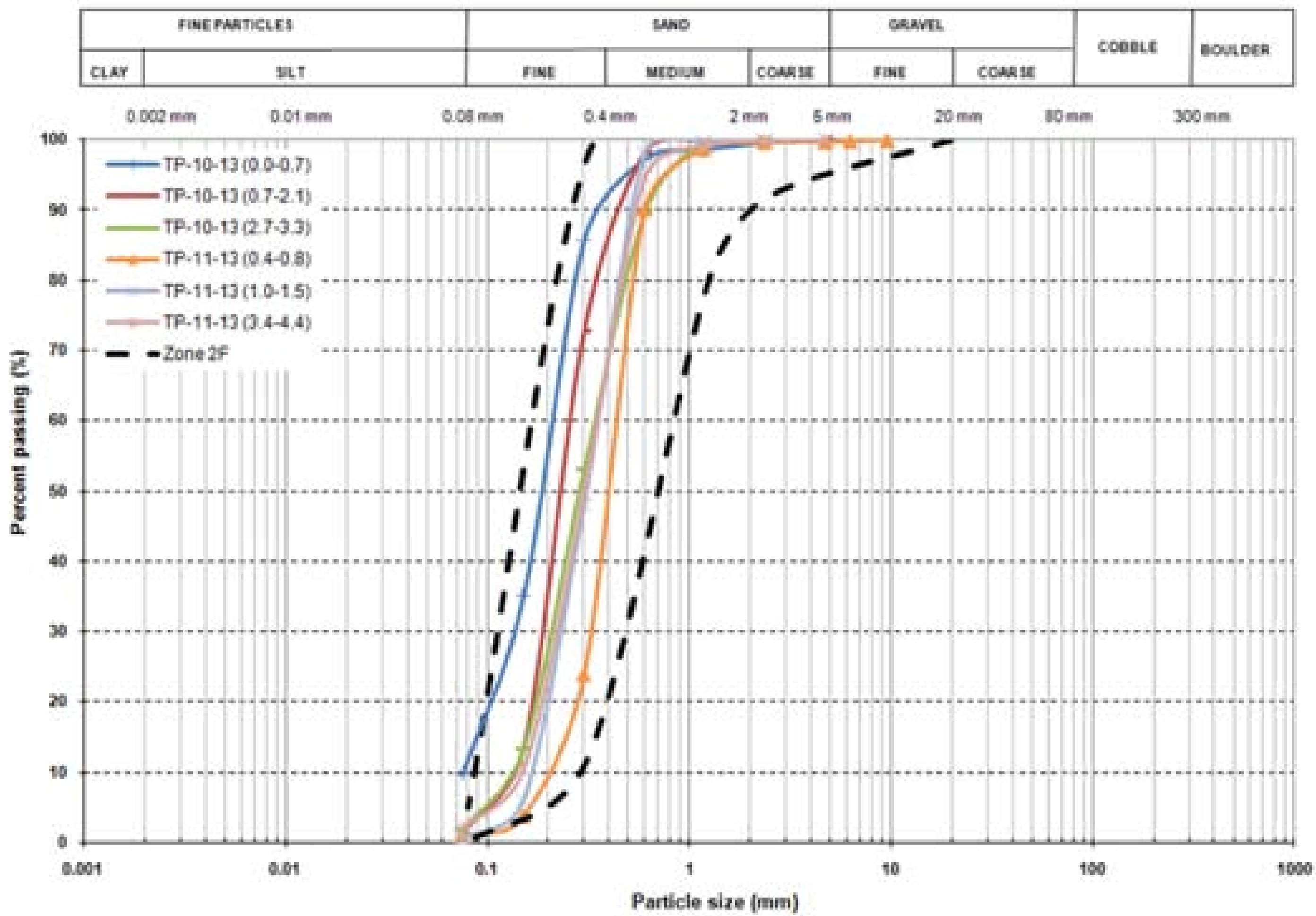


Figure 4-16 : Upper Sand Layer – Grain Size Analyses – 2013 Investigations – Downstream Slope Area



4.2.2 STRATIFIE D DRIFT

A complex unit formed of different interbedded type of soils varying between a silty clay to a silty sand soil have been identified underlying the upper sand layer. The stratified drift is, in fact, a heterogeneous mix of clays, silts and sands with sub-horizontal layering due to the marine and estuarine deposition.

This unit was observed approximately from elevation 50 m to 45 m to 15 m to 5 m. This unit consists of alternating layers of silty clay of low to medium plasticity which is referred to as the "upper silty clay", and silty sand or sandy silt and which is called "intermediate silty sand/sandy silt". Details of the layers referred to are presented in the following sections.

4.2.2.1 Upper Silty Clay Layer

Several in-situ and laboratory tests were performed in this layer. In the report prepared by Lower Churchill Consultants in 1976 a summary of properties of sensitive clays from various sources was presented. In 1978, Acres presented data obtained from work carried out in the laboratories of Acres in 1965, 1976 and 1977, the Norwegian Geotechnical Institute in 1966 and Queen's University Kingston, Ontario in 1972.

For the observed values of sensitivity, the NGI report gives values greater than 100. A sensitivity value of 112 was obtained from cone tests performed in an untreated block sample retrieved during the 1965 campaign.

Figure 4-17, Figure 4-18 and Figure 4-19 present the clay properties reported in 1976 and 1978.

Figure 4-17 : Clay Properties – 1976 Report

PROPERTIES OF MARINE CLAYS

Property	Norway	Ottawa	Muskrat
Moisture content, percent	34	40 - 60	36
Liquid limit, percent	28	30 - 50	32
Plastic limit, percent	19	20 - 25	22
Plasticity index, percent	9	10 - 25	10
Particles smaller than 2 micron, percent	45	57	55
Activity	0.2	0.2 - 0.5	0.2
Salt content gm/l	<2	<1	2
Undrained strength Su, tonnes/m^2	1	4 - 10	50
Sensitivity	>100	20 - 500	100
Preconsolidation pressure tonnes/m^2	1	21	50
c', tonnes/m^2	0.3	2	1.5
ϕ', degrees	30	25 - 30	30

Figure 4-18 : Clay Properties – 1976 Report

EFFECTIVE STRENGTH PARAMETERS
OF MUSKRAT FALLS MARINE CLAY

	Type of Test	Peak ø' (deg)	Peak c' (psi)	Residual ø' (deg)	Residual c' (psi)
Acres Canadian Bechtel (1965)	Triaxial	35	0	-	-
Norwegian Geotechnical Institute (1966)	Triaxial	31	2.6	31	-
	Shear Box	-	-	27	-
Acres Consulting Services (1976)	Triaxial	27	4.1	27	2.2
	Shear Box	26	1.1	24	0.5

Figure 4-19 : Clay Properties – 1978 Report

GEOTECHNICAL PROPERTIES
OF MUSKRAT FALLS CLAY

Property	
Sensitivity	100 – 400
Moisture content, percent	22 – 38
Liquid limit, percent	21 – 44
Plastic limit, percent	14 – 24
Liquidity index	0.3 – 2.7
Plasticity index	3.0 – 22.0
Particles smaller than 2 micron, percent	55
pH	9
Salinity, percent	1.3
Mineralogy (clay fraction)	
Hydrous mica, percent	60
Iron chlorite, percent	10 – 20
Montmorillonite type, percent	2.5
Quartz and Feldspar, percent	10 – 20
Activity	0.13
Cation concentration in pore water	
Calcium, ppm	4.5
Sodium, ppm	146.0
Iron, ppm	0.7
Potassium, ppm	8.5
Magnesium, ppm	2.2
Manganese, ppm	0.05
Undrained shear strength, psf	700 – 2,500
Tensil strength, psf	60 – 160
Preconsolidation pressure, tons/ft^2	2.7 – 6.0

As stated previously, the 1979 and 2013 investigation campaigns allowed reassessment of the properties for this stratigraphic unit.

A low to medium plastic, sensitive, firm to very stiff silty clay to clayey silt material was observed within the stratified drift.

This upper silty clay represents all the various silty clay layers present between the upper sand unit and the lower marine clay unit. The general inclination of these strata is horizontal to sub-horizontal.

This deposit is generally stratified with silty sand/sandy silt layers, the frequency of occurrence and thicknesses of these layers vary from one location to another. Grain size analyses were performed during the 1979 investigation from samples retrieved from boreholes. The grain size distribution curves are presented in Figure 4-20.

Atterberg limits tests were performed in samples retrieved from the boreholes. Main values are presented in Table 4-1.

The Liquidity Index values vary between 0.6 and 2.8, with an average value of 1.3.

The in-situ intact undrained shear strength S_u obtained by vane shear tests ranged from 35 to 135 kPa which indicates that the consistency of the clay material can be qualified of firm to very stiff. The in-situ remoulded undrained shear strength resulted in a range of S_{ur} values from 2 to 60 kPa.

From 43 in-situ vane shear tests conducted in this layer, a range of sensitivity from 1 to 36 with an average value of 10 was obtained which indicates the class of sensitivity varies from low to quick clay.

A series of Direct Shear Tests carried out on samples recovered from this layer during the 1979 investigations, indicated effective stress shear strength parameters of ϕ'=30° to 32° and c'=0 kPa under large strains condition.

Four direct shear tests and 3 triaxial tests were performed on silty clay/clayey silt samples between elevations 10 and -13 m, during the 2013 investigations.

An evaluation and validation of the laboratory tests performed during the 2013 investigation campaign were performed by the LCP Team and by external consultant (Prof. Serge Leroueil, Université Laval). As result of this evaluation, the average shear strength parameters of ϕ'=31° and c'=6 kPa were retained from the test results.

Eight consolidation tests were performed in 1979 on samples recovered between elevations 45 m and 5 m. The consolidation tests carried out on samples taken at a relatively shallow depth (Elevation 44 m to 33 m) showed a preconsolidation pressure σ'_p of approximately 200 kPa to 270 kPa.

Values of overconsolidation ratio (OCR) close to unity were obtained from the interpretation of CPT tests performed from the top of the spur with this value increasing for CPT tests performed in slopes.

A falling head permeability test was conducted in this layer during the 2009 piezometer installation work. A hydraulic conductivity equal to 2.2×10^{-8} m/s was determined for the silty clay material inside the piezometer P2B at about elevation 13.0 m.

During 2013 investigation works, slug permeability tests were performed inside boreholes NS-3B-13, NS-4-13 and NS-6-13 in the stratified drift. Hydraulic conductivity values varied between 3.1×10^{-8} m/s and 8.1×10^{-9} m/s.

From CPT dissipation tests performed during CPT testing values of the hydraulic conductivity of the unit were assessed.

A representative value of the hydraulic conductivity of 5×10^{-8} m/s was retained for this layer.

Hydraulic conductivity obtained from slug tests and CPT dissipation tests are presented for the entire stratified drift unit in Figure 4-21.

Figure 4-20 : Stratified Drift Unit, Upper Silty Clay Layer – Grain Size Analyses – 1979 Investigations

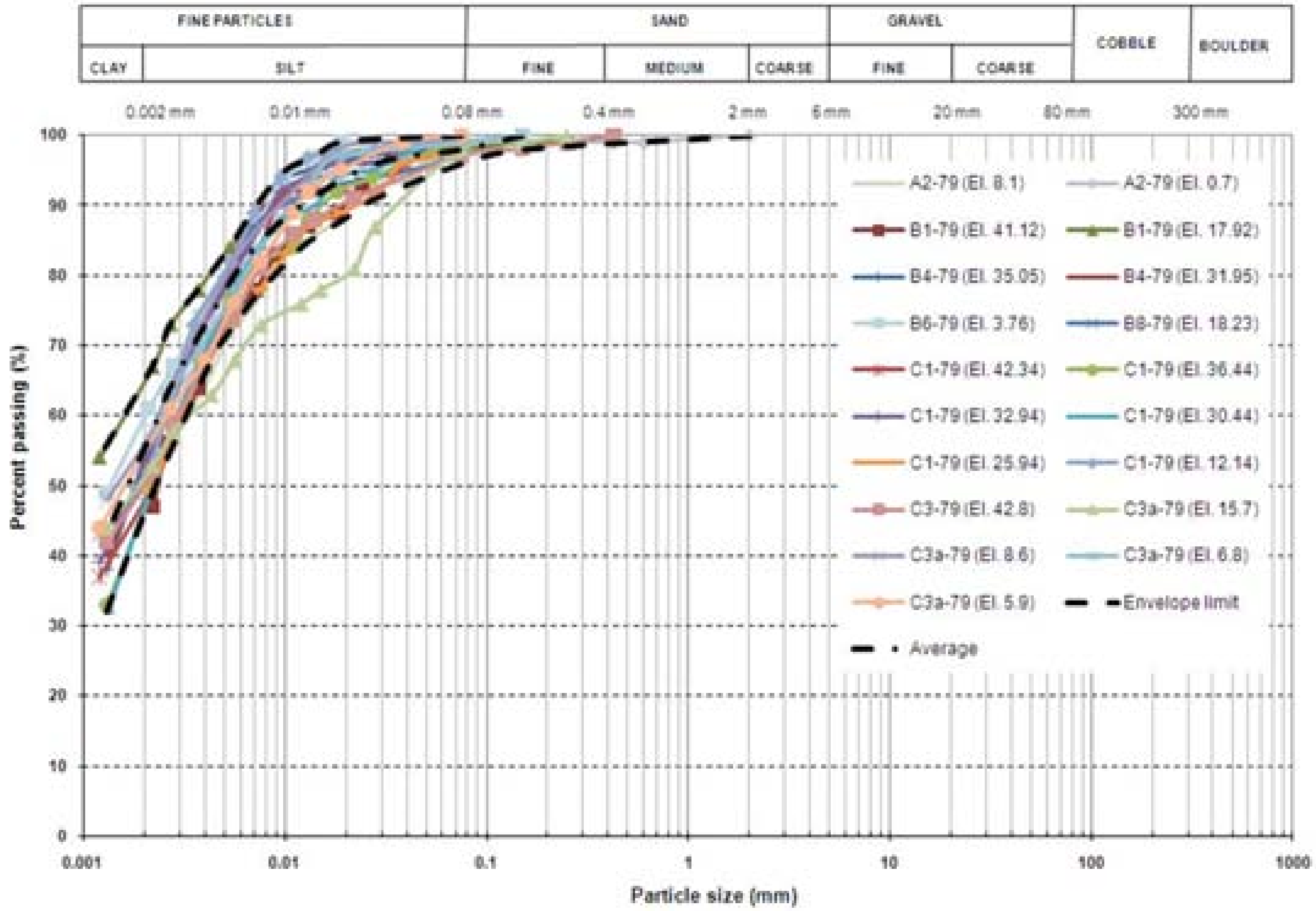
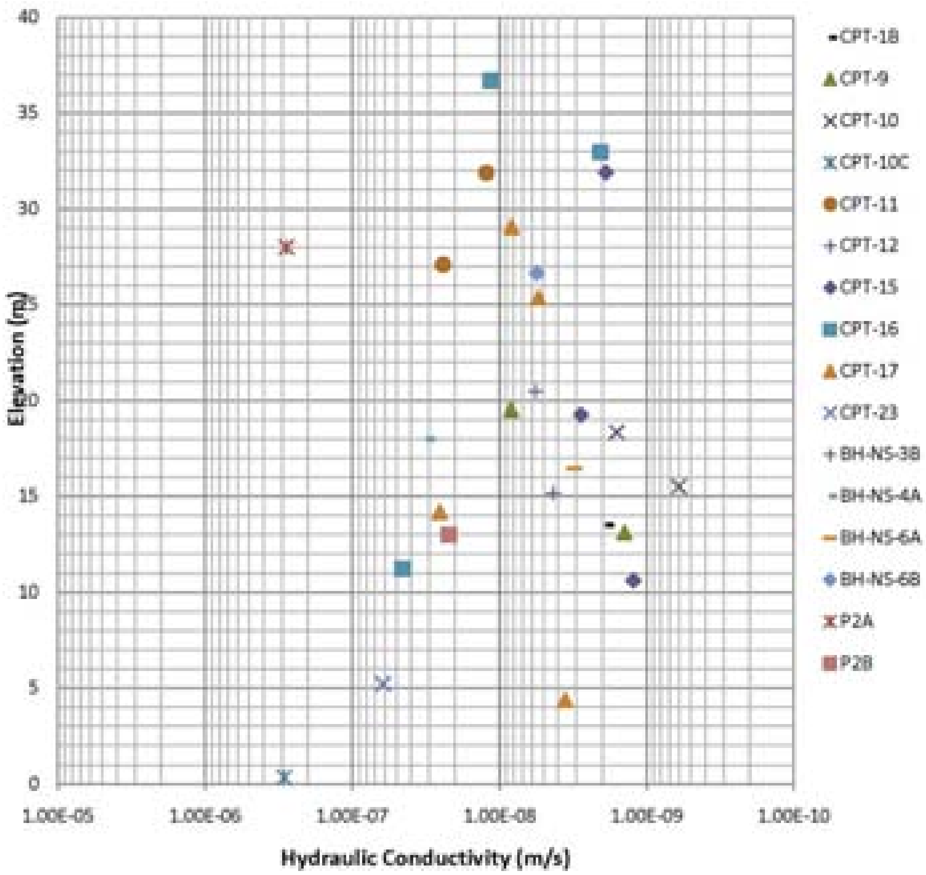


Figure 4-21 : Stratified Drift – Hydraulic Conductivity



A summary of the physical and mechanical properties of the upper silty clay layer is presented in Table 4-1.

Table 4-1 : Upper Silty Clay Layer – Physical and Mechanical Properties

Property	General Range	Average	Number of tests
Percent finer than 2 microns	35 – 45	–	19
Water content, w %	17 – 43	31	199
Liquid limit, LL %	17 – 43	30	168
Plastic limit, PL %	13 – 32	19	168
Plasticity Index, PI %	2 – 22	11	168
Liquidity Index, LI	0.6 – 2.8	1.3	168
Intact Undrained shear strength, S_u kPa	35 – 135	–	–
Remoulded Undrained shear strength, S_{ur} kPa	60 – 2	–	–
Sensitivity, in-situ, S_t	1 – 36	10	43
Large strain friction angle, ϕ'_{cv} °	30 – 32	–	–
Effective cohesion, c', kPa	0 – 10	–	–
Unit weight, γ kN/m^3	18.4 – 19.7	–	11
Initial void ratio, e_0	0.93 – 1.06	–	–
Compression index, c_c	0.32 – 0.5	–	–
Recompression index, c_r	0.03 – 0.06	–	–
Hydraulic Conductivity, k, m/s	10^{-7} – 10^{-9}	–	–
Salt content, g/l	0.8 – 1.5	–	–

4.2.2.2 Intermediate Silty Sand/Sandy Silt Layer

As part of the stratified drift layers of silty sand/sandy silt materials were identified during the investigation work.

This silty sand/sandy silt material layer includes all the major silty sand/sandy silt layers present between the upper sand unit and the lower marine clay unit. The number, elevations and thicknesses of the various layers vary from one point to another.

The results of sieve analyses on samples recovered from the intermediate silty sand/sandy silt layers indicated an average of 27% fines content. The coefficient of uniformity for this layer is generally between 3 and 4. Grain size distribution curves for samples obtained during the 1979 and 2013 investigations are presented in Figure 4-22 and Figure 4-23 respectively.

Figure 4-22 : Stratified Drift Unit, Intermediate Silty Sand/Sandy Silt Layer – Grain Size Analyses – 1979 Investigations

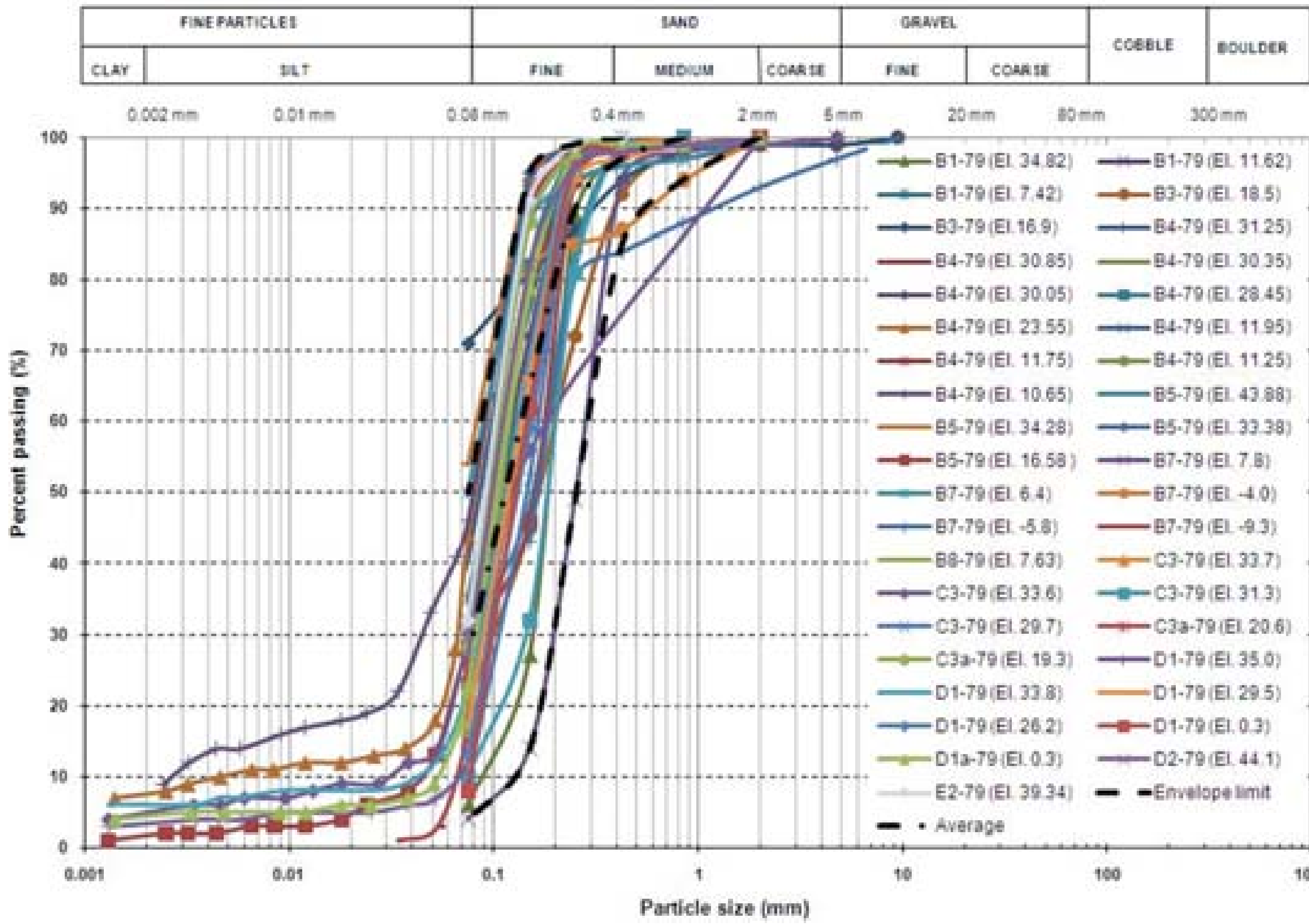
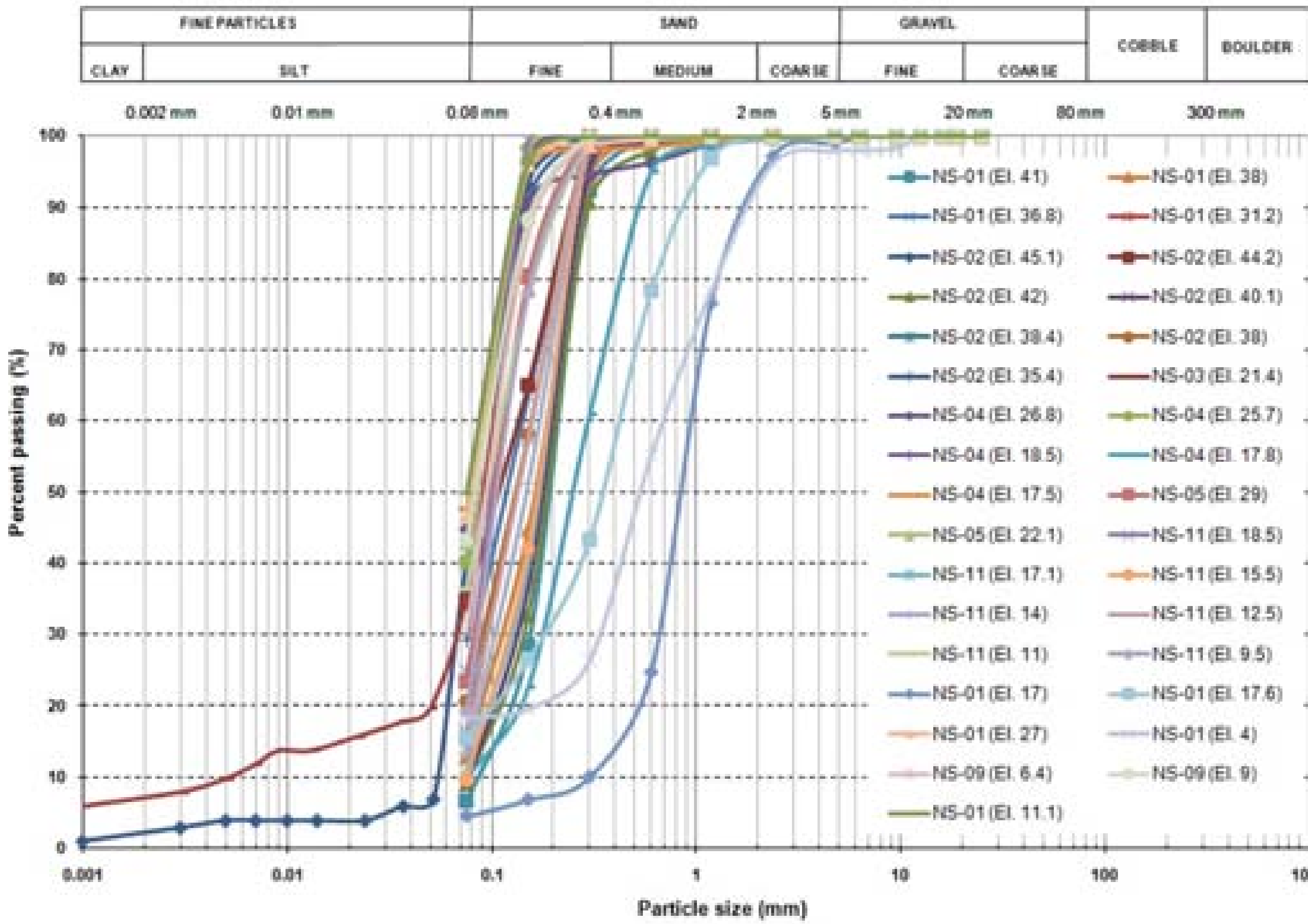


Figure 4-23 : Stratified Drift Unit, Intermediate Silty Sand/Sandy Silt Layer – Grain Size Analyses – 2013 Investigations



The standard penetration tests carried out in this layer resulted in N_{SPT} values generally higher than 50 which indicate a very dense condition.

Three consolidated undrained triaxial tests were conducted on samples from intermediate layers, during the 1979 investigation works, which resulted in an average effective friction angle of 35° to 37° and effective cohesion of 0 kPa under large strains condition. In 2013, 2 direct shear tests were completed on silty sand and sandy silt samples retrieved from borehole NS-1-13, between elevations 28 m and 38 m, which resulted in average values of ϕ'=35° and c'=0 kPa.

One falling head permeability test was conducted in this layer during the 2009 piezometer installation works. Coefficient of permeability equal to 2.8×10^{-7} m/s resulted for this material (piezometer P-2A at elevation 28.0 m). One laboratory permeability test was conducted on a sample from this layer (borehole B7 at elevation -9 m) during the 1979 investigations which resulted in a coefficient of permeability of about 10^{-6} m/s.

Presence of silty clay or clayey silt strata interbedded within the intermediate layer affects its hydrogeological behaviour, as shown in Figure 4-21, which results in a range of permeability from 10^{-7} to 10^{-9} m/s with an average of 10^{-8} m/s.

A representative value of the hydraulic conductivity of 1×10^{-8} m/s was retained for this layer.

Main physical and mechanical properties of the intermediate silty sand layer are presented in Table 4-2.

Table 4-2 : Intermediate Silty Sand/Sandy Silt Layer – Physical and Mechanical Properties

Property Gener	al Range	Average
Fine contents	55 – 5	27
Unit weight, γ kN/m^3	18.4 – 19.7	–
Large strain friction angle, ϕ'_{cv} °	35 – 37	36
Effective cohesion, c', kPa	0	–
Hydraulic Conductivity, k, m/s	10^{-7} – 10^{-9}	–

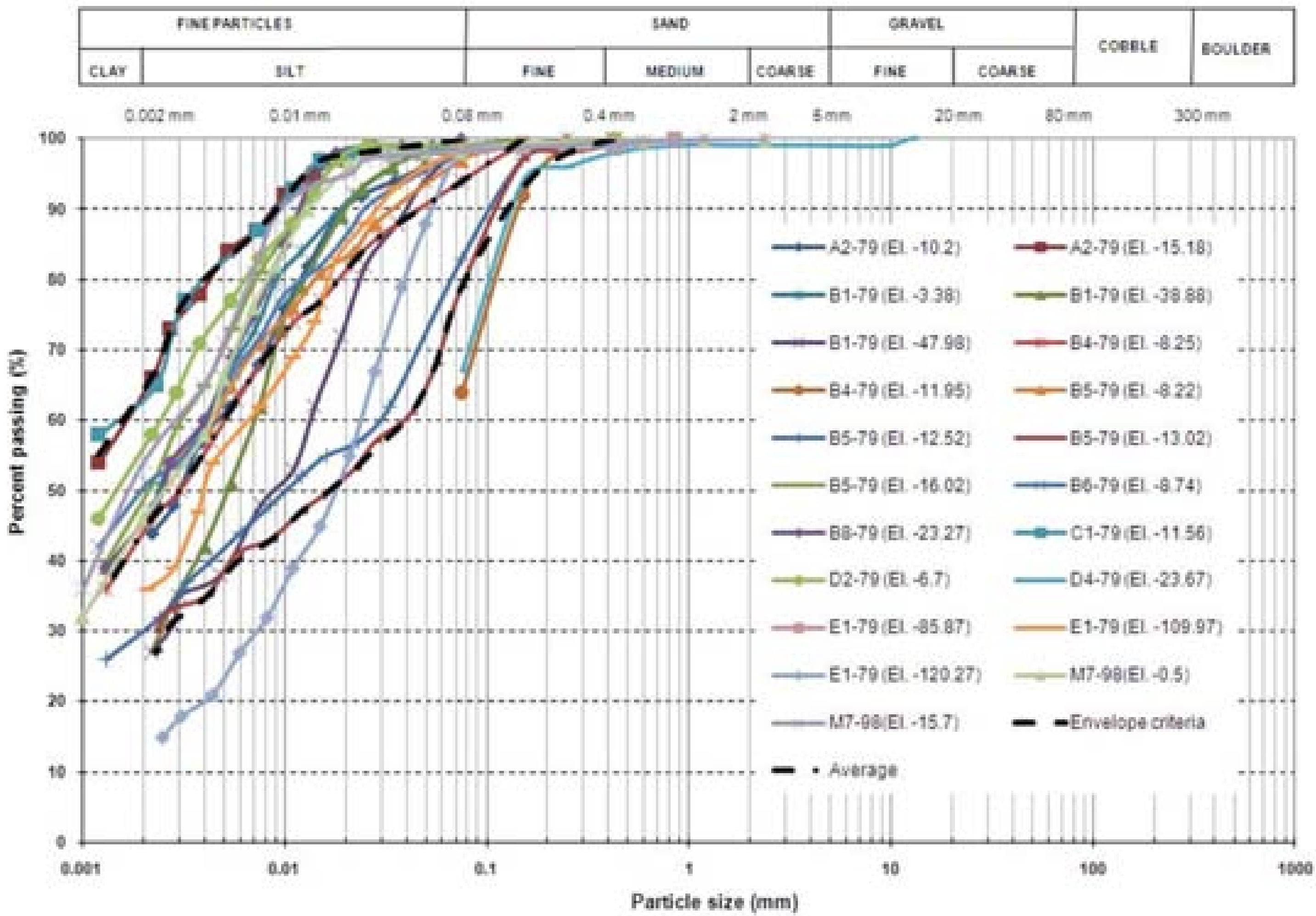
4.2.3 LOWER MARINE CLAY UNIT

The lower marine clay unit was identified below the stratified drift unit, generally below elevation 15 m to 5 m and above the lower aquifer unit at approximately elevation -70 m. This layer consists of clay of low to medium plasticity which exhibits lower values of liquidity index than the upper clay layer.

Upper boundary of this layer was defined from the outcomes of the performed CPT tests in 2013.

Grain size distribution curves for samples obtained during the 1979 investigation works are presented in Figure 4-24.

Figure 4-24 : Lower Marine Clay Unit – Grain Size Analyses – 1979 Investigations



Atterberg limits were performed on samples retrieved from the boreholes. Main values are presented in Table 4-3. Liquidity Index values vary between 0.1 and 2 with an average value of 0.6.

The consistency of clay can be qualified as stiff to very stiff, measured with in-situ undrained shear strength values varying between 53 and 200 kPa.

The in-situ remoulded undrained shear strength presented values varying between 8 and 96 kPa.

Sensitivity values obtained for this unit vary between 2 and 11, with an average of 4, indicating that the class of sensitivity of the clay can be classified as low to extra sensitive.

Four direct shear tests were conducted in 2013 on samples representatives of this unit. Measured values of ϕ' varied between 26 and 28º with no cohesion, c' = 0 kPa.

Consolidated undrained triaxial tests were performed and average values of ϕ' = 33º and c' = 6 kPa were obtained.

One dimensional consolidation tests were performed in specimens of the lower clay unit. Due to possible disturbance of the samples, the values of the results were considered to be not reliable.

Values of OCR varying between 2 and 5 were obtained from the interpretation of the CPT results.

Salt content profile prepared during the 1979 investigations indicated values varying between 8 and 22 g/l. This measured salt content indicates a marine depositional environment.

A summary of the physical and mechanical properties of the lower clay layer is presented in Table 4-3.

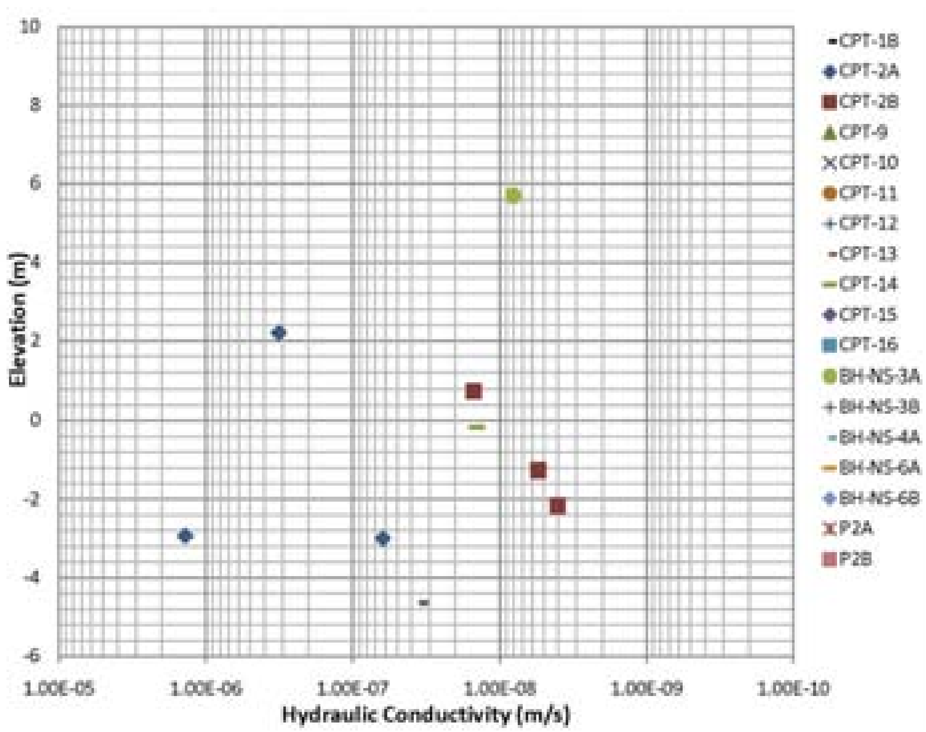
Hydraulic conductivity values of the lower marine clay unit, obtained from the slug tests and CPT dissipation tests are presented in

Figure 4-25 - A representative value of the hydraulic conductivity of 1×10^{-8} m/s was retained for this layer.

Table 4-3 : Lower Marine Clay Unit – Physical and Mechanical Properties

Property	General Range	Average	Number of tests
Percent finer than 2 microns	15 – 35		
Water content, w %	17 – 45	29	201
Liquid limit, LL %	22 – 48	37	123
Plastic limit, PL %	13 – 27	21	123
Plasticity Index, PI %	7 – 25	16	123
Liquidity Index, LI	0.1 – 2	0.6	123
Intact Undrained shear strength, S_u, kPa	53 – 200	_	_
Remoulded Undrained shear strength, S_{ur}, kPa	8 – 96	_	_
Sensitivity in-situ, s_t	2 – 11	4	35
Large strain friction angle, ϕ'_{cv} °	33	_	_
Effective cohesion, c', kPa	6	_	_
Salt content, g/l	8 – 22	_	8
Unit weight, γ , kN/m^3	19.2 – 19.5	_	3
Hydraulic Conductivity, k, m/s	10^{-7} – 10^{-9}	_	_

Figure 4-25 : Lower Marine Clay – Hydraulic Conductivity



4.2.4 LOWER AQUIFER UNIT

The lower aquifer is generally observed from elevation -70 m to bedrock level and consists generally of sand and gravel with some cobbles and boulders. This layer is located below the lower clay layer and above the bedrock. The lower aquifer was not encountered in the boreholes BH-D2-79 and BH-D3-79 close to the rock knoll where bedrock is located at shallow depths and it was encountered in borehole NS-2-13.

In borehole BH-D3-79 the lower clay overlays directly the bedrock while in borehole BH-D2-79 a layer of glacial till was encountered overlying the bedrock.

Grain size distribution curves from samples taken from boreholes drilled in this layer are presented in Figure 4-26 and Figure 4-27.

A representative value of the hydraulic conductivity of $1.6x10^{-4}$ m/s was retained for this layer from the obtained values coming from the pumping tests performed in 1979.

Figure 4-26 : Lower Aquifer Unit – Grain Size Analyses – 1979 Investigations

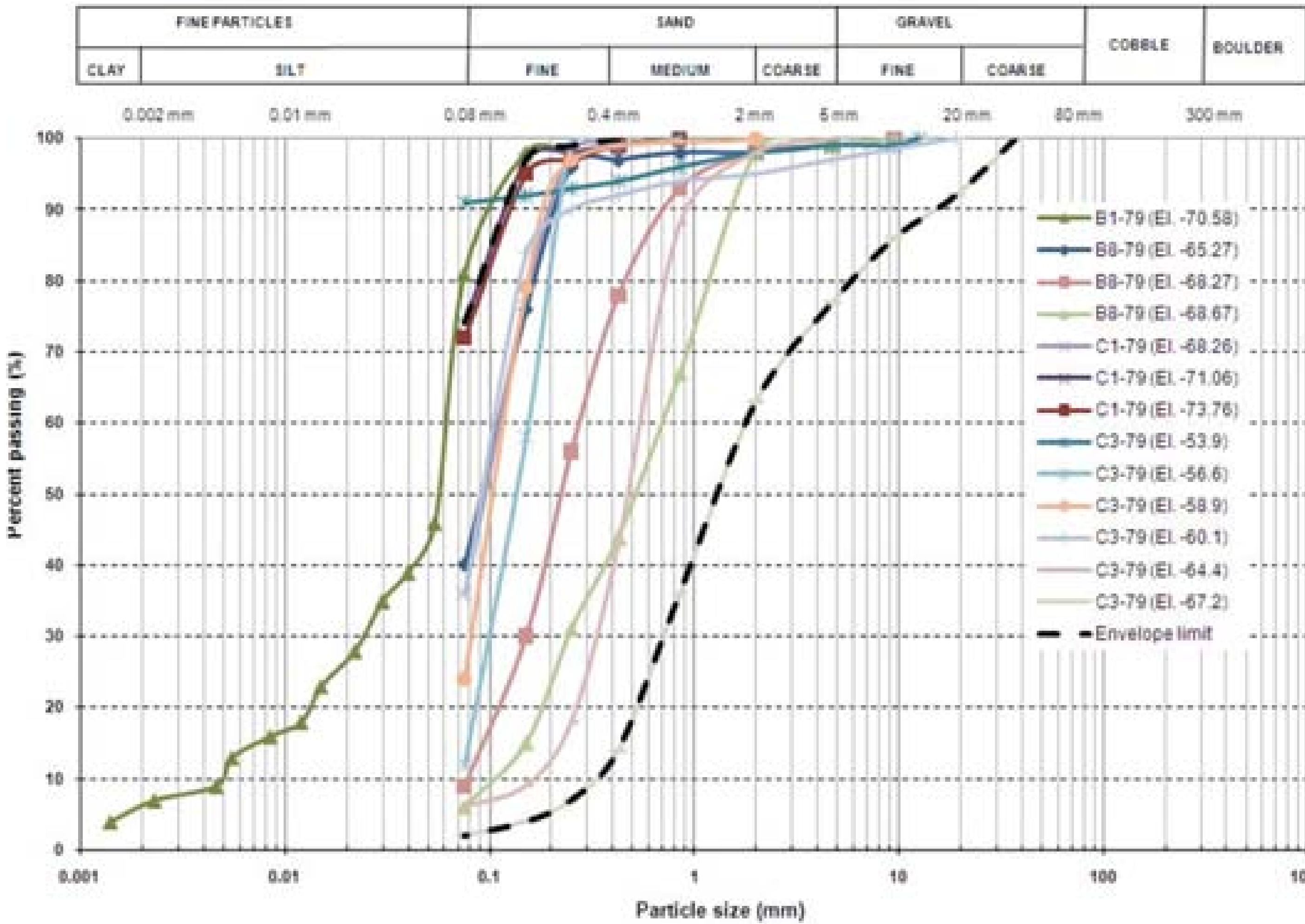
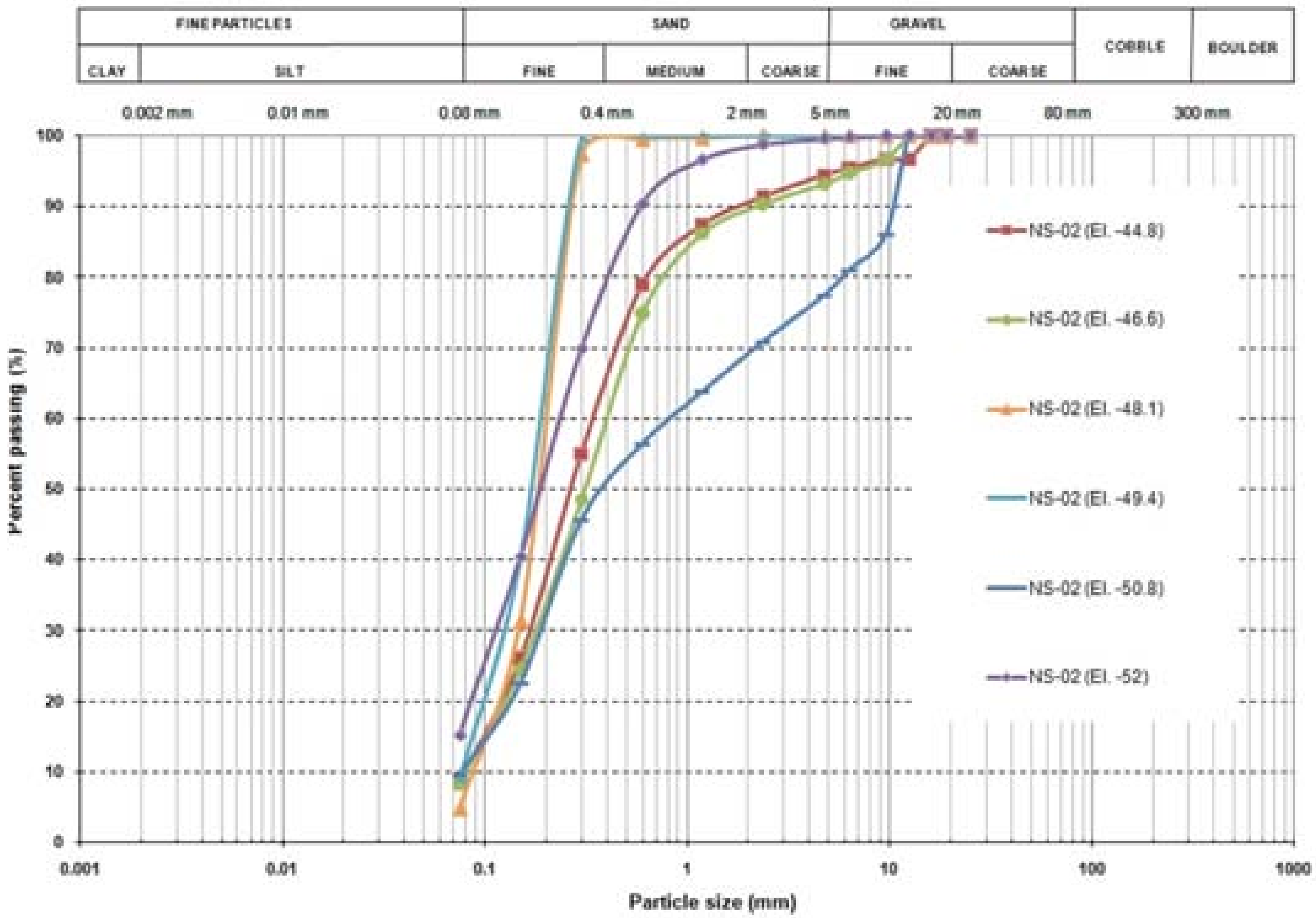


Figure 4-27 : Lower Aquifer Unit – Grain Size Analyses – 2013 Investigations



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4.2.5 BEDROCK FORMATION

The bedrock has been drilled and sampled in 7 boreholes. The type of bedrock is generally granite gneiss with pegmatite intrusions. The Rock Quality Designation (RQD) values in boreholes D2-79 and D3-79, close to the rock knoll, vary generally between 55 and 89 indicating the quality of the rock as of fair to good, except for one value (17) indicating the quality of the rock as very poor at the interface with the overburden inside borehole D3-79.

4.3 HYDROGEOLOGY

4.3.1 INTRODUCTION

Interpretation of the different stratigraphic and hydrogeological data permitted identification of 3 different aquifers in the North Spur.

In the surficial upper sand layer unit overlying the stratified drift, a perched aquifer exists below the ground surface. A second aquifer, labelled as “intermediate aquifer”, was identified inside the stratified drift unit. Finally, overlying the bedrock and limited in the upper boundary by the lower marine clay unit, the “lower aquifer” was identified during the investigations.

Existing conditions of these aquifers and perched water tables are discussed in this section.

Hydrogeological assessments of the aquifers in the North Spur are based on the following datasets collected from 1979 to present:

- Piezometers installed in the 1979 boreholes which were in use from fall 1979 to March 1981;
- Pumping tests conducted in the lower aquifer in 1979;
- Pumpwell installations and pumping tests conducted in the intermediate aquifer in 1981;
- Piezometers installed in 1981 which were in use until 1984;
- Piezometer readings during 1994 and 1995 by Labrador Hydro;
- Pumpwell system assessment and rehabilitation in 1996;
- Piezometers installed in 1997;
- Pumpwell system assessments and rehabilitation in 2007 and 2009;
- Piezometers installed in 2009;
- Installation of dataloggers inside some piezometers in 2011;

- CPT dissipation tests and piezometer installations in 2013;
- Rehabilitation of the pumpwell system in 2013.

Details of the 3 different hydrogeological units are presented in the following sections.

4.3.2 PERCHED WATER LEVEL IN THE UPPER SAND LAYER

Characterization of this aquifer was made basically by interpretation of the data coming from the investigations performed in 1979 and 2013.

A perched water table was observed in the upper sand layer in some boreholes (BH-B2-79, BH-C3b-79 and BH-E2a-79). This water table is recharged by precipitation and water infiltration from top of the Spur. Due to high permeability of this layer the ground water easily drains through the upstream, downstream and kettle lakes slopes at elevations 40 to 45 m. Piezometers installed in the upper sand layer in boreholes BH-B4-79, BH-B5-79, BH-B6a-79 and BH-D3-79, show the layer is dry at elevations 43, 42, 46 and 55 m, respectively, which can be attributed to the free drainage of this layer.

The upper sand layer is underlain by the stratified drift. The connection between the pore water in the upper sand and the intermediate silty sand layer may result in precipitation infiltration into the stratified drift.

Using the available grain size distribution and empirical relationships, a hydraulic conductivity of this unit of about 1×10^{-4} m/s was estimated.

4.3.3 INTERMEDIATE AQUIFER

Characterization of this unit was made by the interpretation of the data obtained during the 1979, 1997, 2009 and 2013 piezometer installation works; 1981 pumpwell system and piezometers installation works; 1996, 2007 and 2009 pumpwell system assessments; 2013 piezocone dissipation tests and through the piezometer readings from 1979 to present.

The intermediate aquifer is observed in the stratified drift, approximately between elevations 50 m to 45 m and 15 m to 5 m throughout the Spur.

Controlling the seepage and lowering the pore water pressure in this aquifer is extremely important for the stability of the Spur.

Layers of fine to medium sand with variable silt content provide conductors for groundwater originating from north-west of the Spur, from the kettle lakes region in the north and for water infiltrating from precipitation on the surface of the Spur.

The two main intermediate sandy silt/silty sand layers within the stratified drift are locally separated by a thick (10 m to 20 m) layer of clayey material (upper clay unit). The upper intermediate sandy silt/silty sand layer which is laid between two upper clay layers is drained near the upstream and downstream slopes.

The lower intermediate sandy silt/silty sand layer is observed generally between elevations 20 m to 5 m and is limited by upper clay and lower clay layers. This layer is connected to the River in the upstream side and partially connected to the river in the downstream side and recharged from the north side of the Spur.

In 1979, the maximum piezometric level in the intermediate aquifer was observed in boreholes BH-B6-79 (39.6 m) in the north-west side of the Spur.

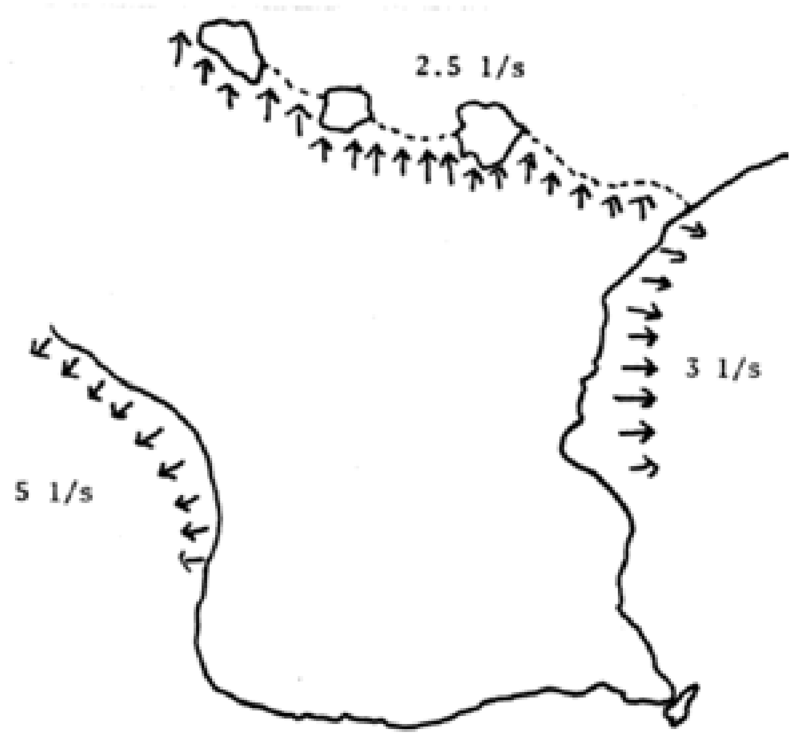
Before installation of the dewatering system, the maximum water level measured in this aquifer in the south side of the Spur, was recorded in boreholes BH-C3a-79 (27.5 m) and BH-C4-79 (26.3 m) on the upstream side, and in borehole BH-D1a-79 (31.2 m) and piezometers P3-81 (30.8 m) and P16-81 (29.5 m) in the downstream side of the Spur.

At the boundaries, the water level in this aquifer is controlled by the upstream river water level (17 m), downstream river water level (3 m), and by the water level in the kettle lakes and outlet system (i.e. 32 m in the upper lake, 25 m in the lower lake and 3 m at the outlet to the downstream river level).

The piezometric levels in this aquifer indicate that it is recharged from the North side of the Spur and discharges through the upstream and downstream slopes and through the three kettle lakes and their outlet to the north-east of the Spur.

A water budget and seepage evaluation carried out in 1979 estimated the total outflow from this aquifer to be 10.5 l/s including; 5 l/s from upstream slopes, 3 l/s from downstream slopes and 2.5 l/s towards the kettle lakes (Figure 4-28).

Figure 4-28 : Intermediate Aquifer – Seepage Flow Rate Estimation



Several springs have been observed on the downstream and upstream slopes of the Spur. Most of them are located at the centre of the old landslide back scarps around elevations 25 m to 35 m and higher. These springs are mainly caused by discharge from the intermediate aquifer towards the upstream and downstream slopes.

As part of the 1981 field work, 3 pumping tests were carried out in the intermediate aquifer during the pump well system installation. The tests were conducted in wells W-3, W-10 and W-17 to evaluate the aquifer parameters. Water levels were monitored in the surroundings wells and piezometers during the pumping and recovery periods.

The results obtained from the analysis performed of the representative data are presented in Figure 4-29.

Figure 4-29 : Pumping Tests

Pumping Test Well No.	Observation Well and Piezometer No.	Pumping Test Transmissivity (T, m^2/day)	Recovery Test Transmissivity (T^1, m^2/day)	Storage Coefficient (S)	Ratio (S/S')
W-3	W-2	21.96	21.37	1.4×10^{-4}	5.4
	W-3	19.80	23.26	-	-
	W-4	26.00	29.1	4×10^{-4}	5
	W-5	16.07	22.7	2.1×10^{-4}	1.4
	W-6	32.09	23.3	2.6×10^{-4}	2.4
	P-3	33.5	41.2	6.7×10^{-4}	-
	P-4	24.7	18.8	7.3×10^{-5}	1.75
	P-5	20.4	17.0	1.6×10^{-4}	1.30
	P-15	26.0	24.7	1.8×10^{-4}	1.0
	P-16	38.0	54.9	9.6×10^{-4}	-
	P-17	39.5	61.8	9.2×10^{-4}	-
W-10	W-9	0.54	0.21	5.5×10^{-5}	-
	W-10	0.35	0.10	-	-
W-17	W-17	0.075	0.030	-	-
	W-17A	-	-	-	-

Average hydraulic conductivity values derived from the transmissivity of the aquifer were established in the range of 10^{-5} m/s over the area of wells W-1 to W-7 and of the order of 10^{-7} to 10^{-8} m/s over the areas of wells W-8 to W-22.

Boundaries were defined by comparison of pumping and recovery test data in conjunction with the interpreted stratigraphy. Two distinct negative boundaries were identified, one between wells W-1 and W-2 in the south, and the other one between wells W-6 and W-7.

As a result, it was confirmed that the general groundwater flow over the North Spur is from northwest to southeast, as it was previously established in the 1979 studies.

Total seepage in the downstream area of the North Spur was evaluated from the transmissivity values obtained from the pumping tests. The total flow was estimated to be 5 to 6 l/s. Flow rate in the ravine north of well W-22 was measured to be 150 l/s.

Water samples were taken from 10 wells after adjustment of the pumping rate in November in order to perform water quality tests.

Partial results of the chemical analysis performed from water samples taken in different wells are presented in Table 4-4.

Table 4-4 : Chemical Analysis

Sample From Well No.	Alkalinity (ppm)	Hardness (ppm)	Iron (ppm)	Manganese (ppm)	Sand Content (ppm)
W-2	148	30	2.8	0.09	NIL
W-3	214	55	0.13	0.10	NIL
W-5	402	44	0.05	0.02	NIL
W-7	496	18	0.33	0.02	NIL
W-9	130	25	4.2	0.08	NIL
W-11	336	17	0.49	0.02	NIL
W-13	43	25	4.5	0.12	NIL
W-15	96	30	151	1.5	NIL
W-17	85	21	0.71	0.02	NIL
W-19	192	20	150	1.8	NIL

A total of 17 vibrating wire piezometers were installed during the construction of the pumpwell dewatering system in 1981 and were functioning until 1984. Recorded water levels between these dates are presented in Table 4-5.

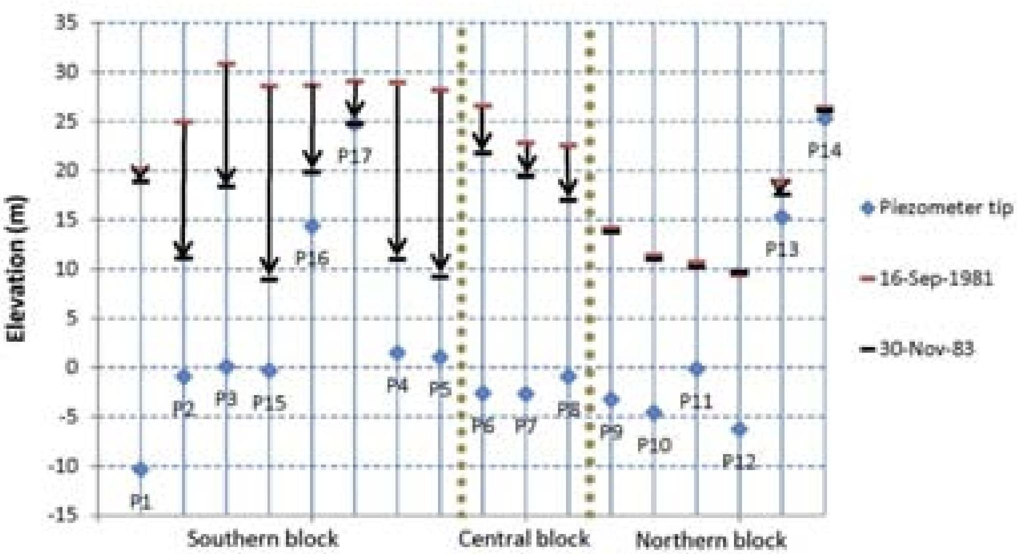
Table 4-5 : 1981 Piezometers – Water Level

Piezometer	Tip Elevation (m)	Date					
		16-09-1981	30-09-1981	07-10-1981	25-11-1981	30-10-1982	30-11-1983
P1	-10.35	20.1	19.9	20.08	19.49	19.8	18.9
P2	-0.93	24.95	24.7	24.87	22.25	12.5	11.1
P3	0.07	30.84	30.62	30.68	25.72	19.65	18.4
P4	1.47	28.91	28.89	28.91	19.19	11[2]	–
P5	1.03	28.17	28.04	27.97	20.64	11.3	9.2
P6	-2.64	26.57	26.32	26.51	25.03	22.65	21.8
P7	-2.7	22.8	22.56	22.7	22.76	20	19.45
P8	-0.94	22.56	22.77	22.48	22.57	17.65	17
P9	-3.26	14.15	13.89	14.07	14.09	13.5	13.75
P10	-4.61	11.39	11.17	11.37	11.23	11.25	11
P11	-0.15	10.69	10.49	10.69	10.58	10.5	10.2
P12	-6.27	9.4	9.12	9.28	8.94	9.65	9.7
P13	15.27	18.93	18.73	18.89	18.33	17.6	17.55
P14	25.3	26.4	26.4	26.42	26.38	26.3	26.1
P15	-0.34	28.59	28.38	28.44	19.18	9.45	8.9
P16	14.37	28.65	29.51	29.48	25.67	20.55	19.85
P17	24.75	29.03	28.91	28.91	26.00	21.8	20.95

Figure 4-30 presents the water levels recorded in the piezometers P1 to P17 before and after 27 months of pumping operation. From the system assessment performed in 1996 and for discussion purposes, the wells were arbitrary divided into 3 groups, based on the original water level before operation of the dewatering system; the southern, central and northern blocks.

The southern block is defined in the area between the rock knoll and the well No. 8, the central block between wells No. 9 and No. 16 and the northern block from well No. 17 to well No. 22.

Figure 4-30 : Water Level Before and After Pumping Operations



Several piezometers were installed in the stratified drift unit in 1997, 2009 and 2013. As part of the water level monitoring program and in order to prepare a database of the hydrogeological regime of this aquifer before the start of the stabilization works, 8 water level loggers were installed inside the existing standpipe piezometers.

Piezometric contours of the intermediate aquifer are shown in Figure 4-31 and recorded piezometric levels before and after installation of the dewatering system are shown in Table 4-6.

Figure 4-31 : Intermediate Aquifer – Piezometric Contours

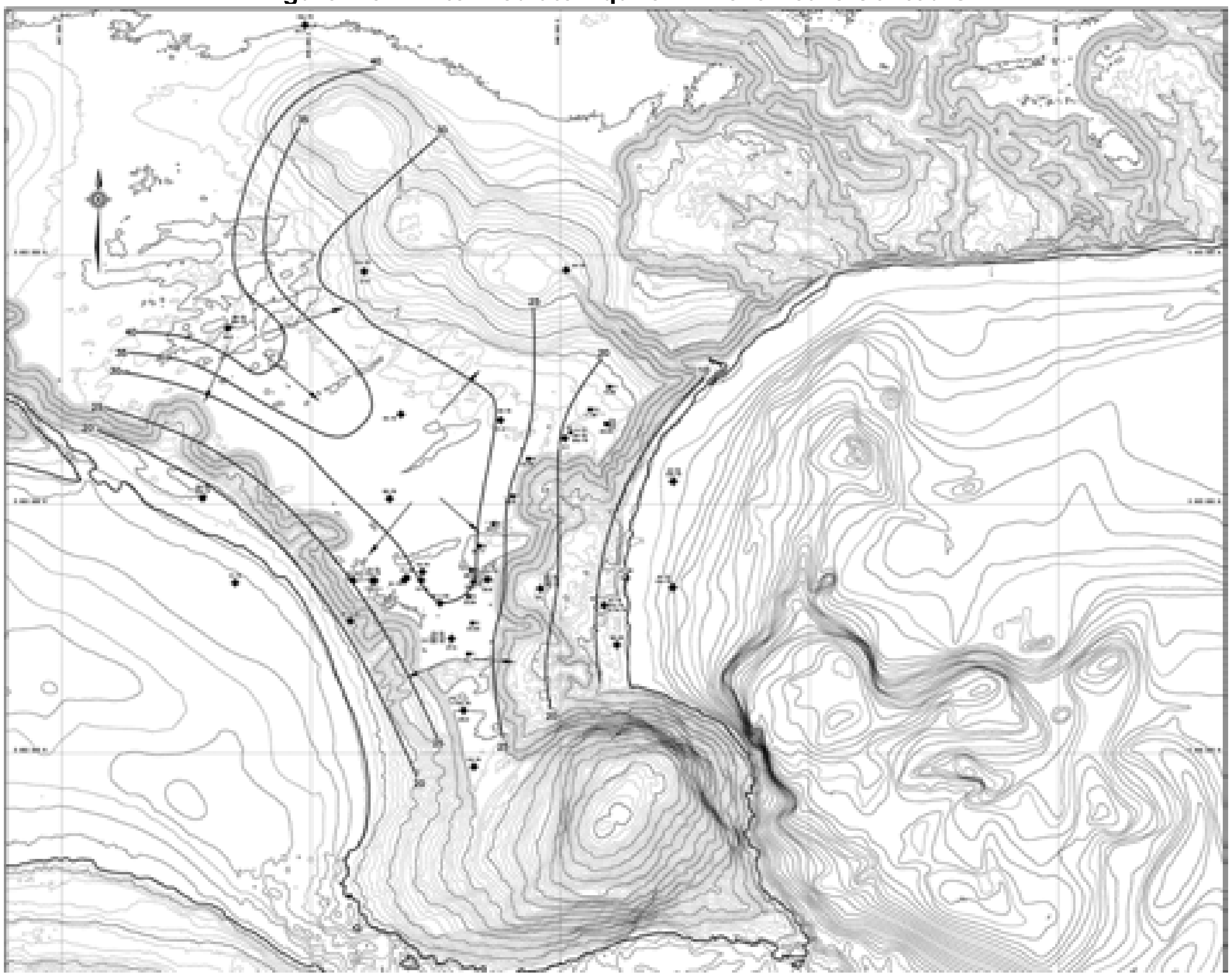


Table 4-6 : Piezometric Levels Before and After Dewatering System Installation

	Location with respect to pump well line	piezometer label	Tip elevation (m)	Stratigraphic Unit	Piezometric Level (m)					
					Before wells installation	1983	1998	2009	2012	Drawdown
Southern Block	D/S	P2-81	-0.93	Intermediate sand	24.95	11.10	–	–	–	13.85
		P3-81	0.07	Intermediate sand	30.84	18.40	–	–	–	12.44
		P4-81	1.47	Intermediate sand	28.91	11.00	–	–	–	17.91
		P5-81	1.03	Intermediate sand	28.17	9.20	–	–	–	18.97
		P15-81	-0.34	Intermediate sand	28.59	8.90	–	–	–	19.69
		BH D1-79	-2	Intermediate sand	28.57	–	–	–	–	19.97
		P-B1-97	1.3	Intermediate sand	–	–	9.24	7.93	8.6	
	U/S	BH C3-79	4.5	Intermediate sand	27.80	–	–	–	–	17.8
		P-A1-97	6.55	Intermediate sand	–	–	10.60	9.42	10.0	
	D/S	P16-81	14.37	Upper clay	28.65	19.85	–	–	–	8.8
		P17-81	24.75	Upper clay	29.03	20.95	–	–	–	8.08
		P-B2-97	12.75	Upper clay	–	–	22.15	23.71	24.15	N/D
	U/S	P-1B-09	18.43	Upper clay	–	–	34.80	35.00	35.30	N/D
		PA2-97	24.35	Upper clay	–	–	dry	dry	dry	N/D
Central Block	D/S	P-D1-97	12.63	Intermediate sand	–	–	23.41	23.38	22.6	N/D
	U/S	BH B5-79	18	Intermediate sand	29.50	–	–	–	–	9.8
		P-4B-09	10.19	Intermediate sand	–	–	–	21.32	19.7	
		P-2B-09	11.5	Intermediate sand	–	–	–	26.26	25.2	N/D
	D/S	P-D2-97	25.74	Upper clay	–	–	30.92	31.13	31.22	N/D
		P-3B-09	17.76	Upper clay	–	–	–	27.3	27.3	N/D
Northern Block	D/S	BH B4-79	11	Intermediate sand	13.1	–	–	–	–	N/D
		P-J2-97	10.6	Intermediate sand	–	–	11.44		11.38	N/D
		P-F2-97	12.01	Intermediate sand	–	–	dry	dry	dry	N/D
	U/S	BH B5-79	18	Upper clay	29.5	–	–	–	–	N/D
	D/S	P13-81	15.27	Upper clay	18.93	–	–	–	–	N/D
		P14-81	25.3	Upper clay	26.40	26.1	–	–	–	0.3

Data of the recorded water level in each piezometer from their installation until the current days in the 3 different blocks are presented in Figure 4-32, Figure 4-33 and Figure 4-34.

Figure 4-32 : Piezometric Levels – Southern Block

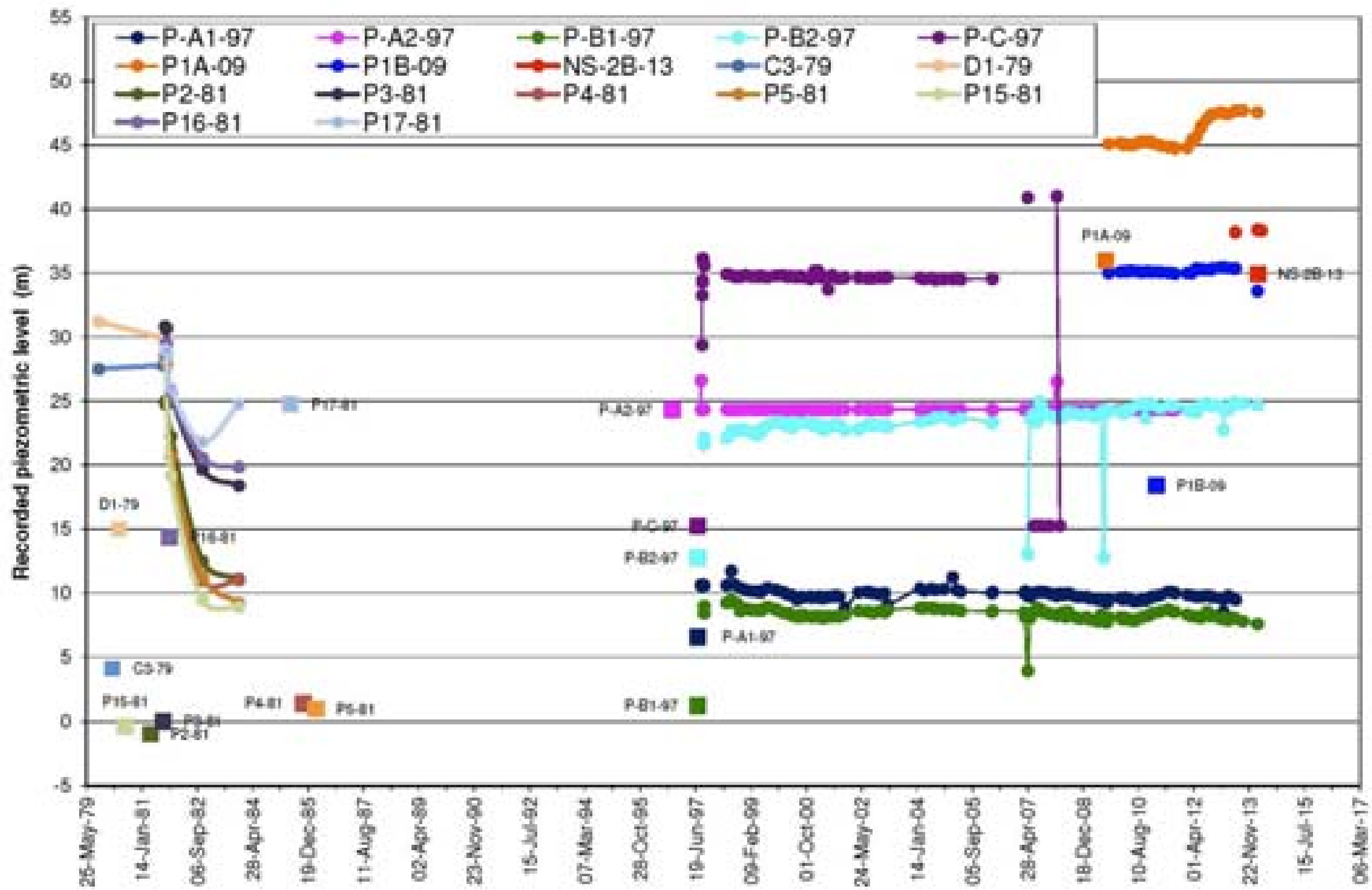


Figure 4-33 : Piezometric Levels – Central Block

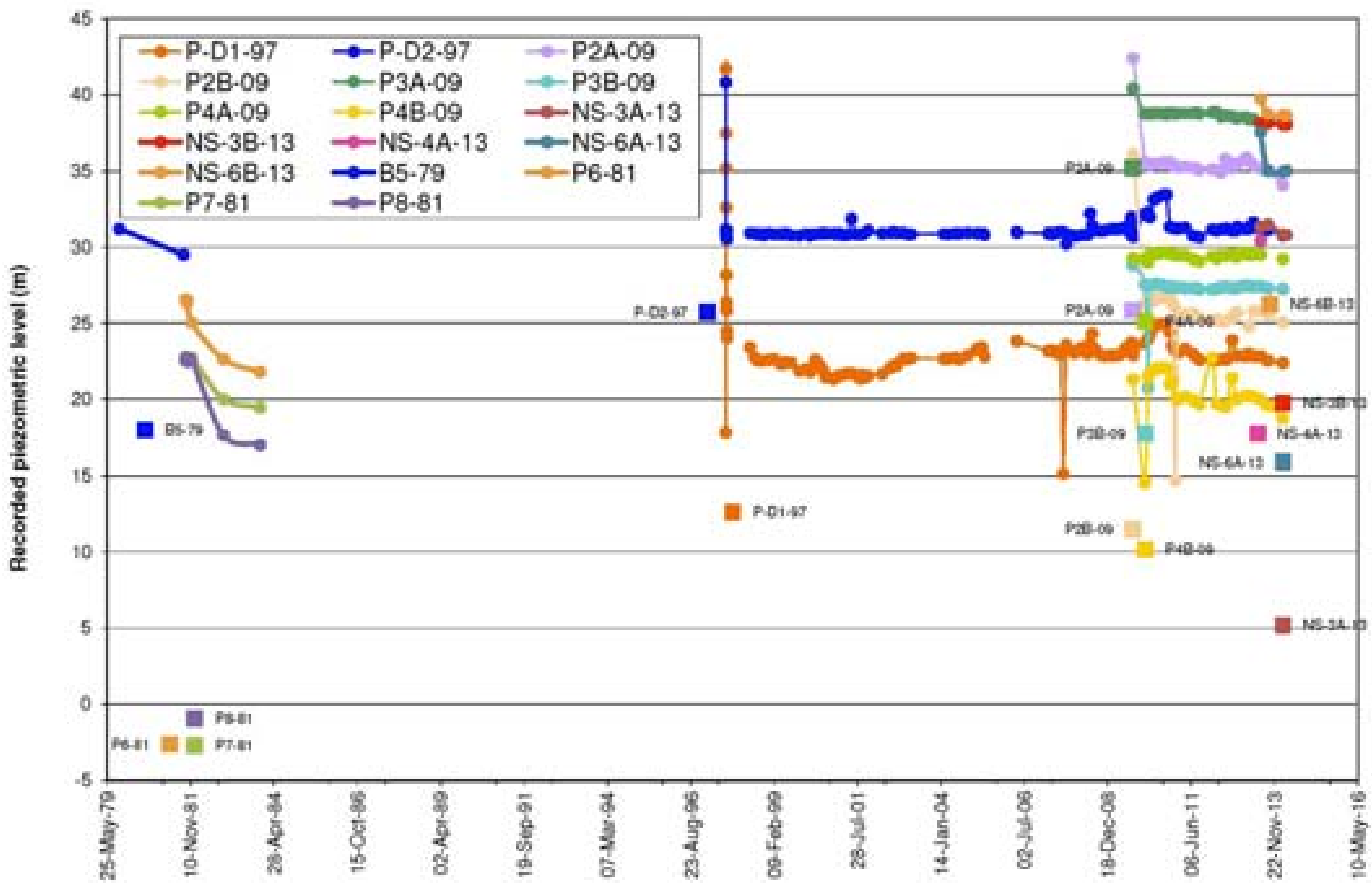
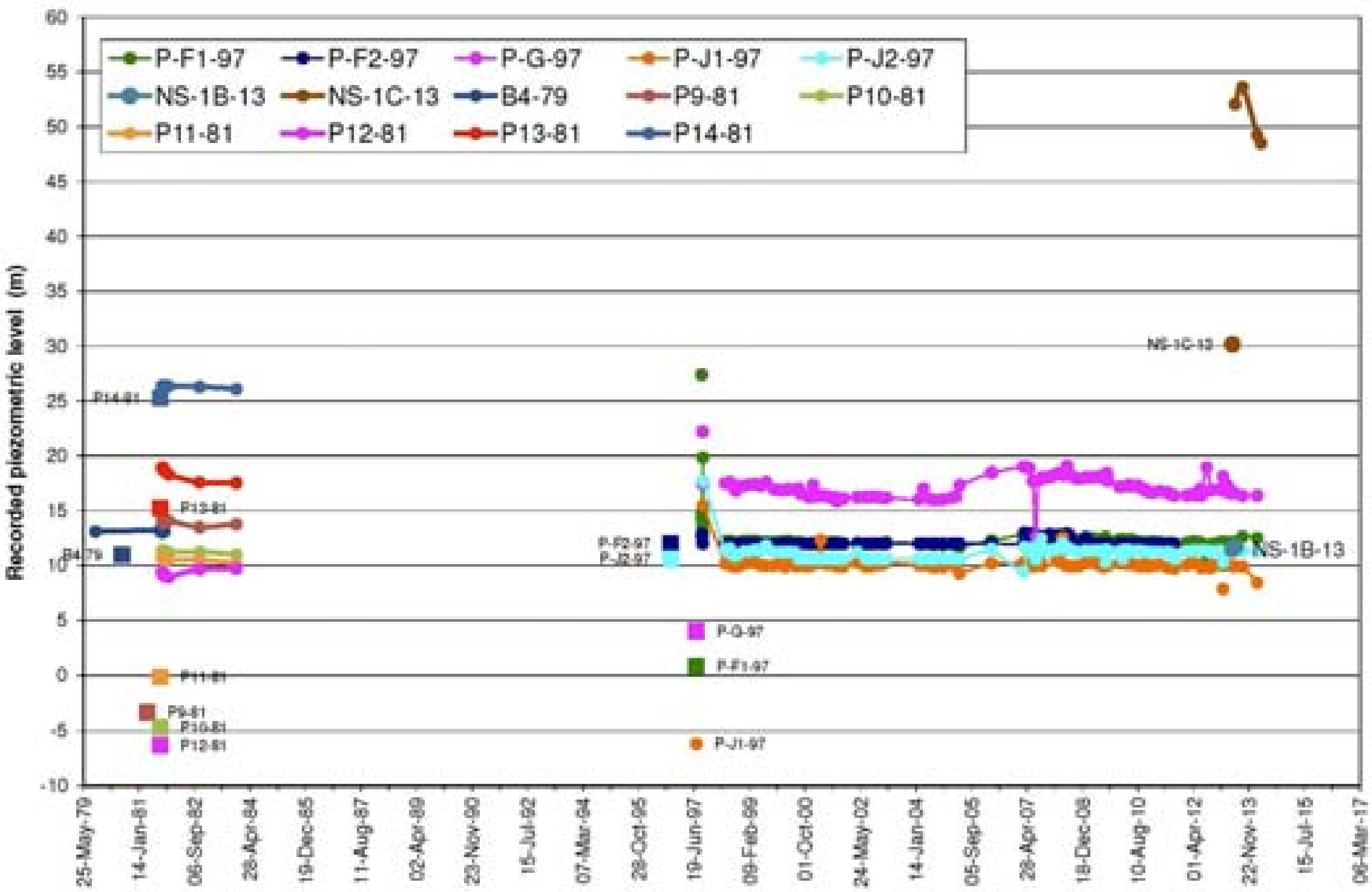


Figure 4-34 : Piezometric Levels – Northern Block



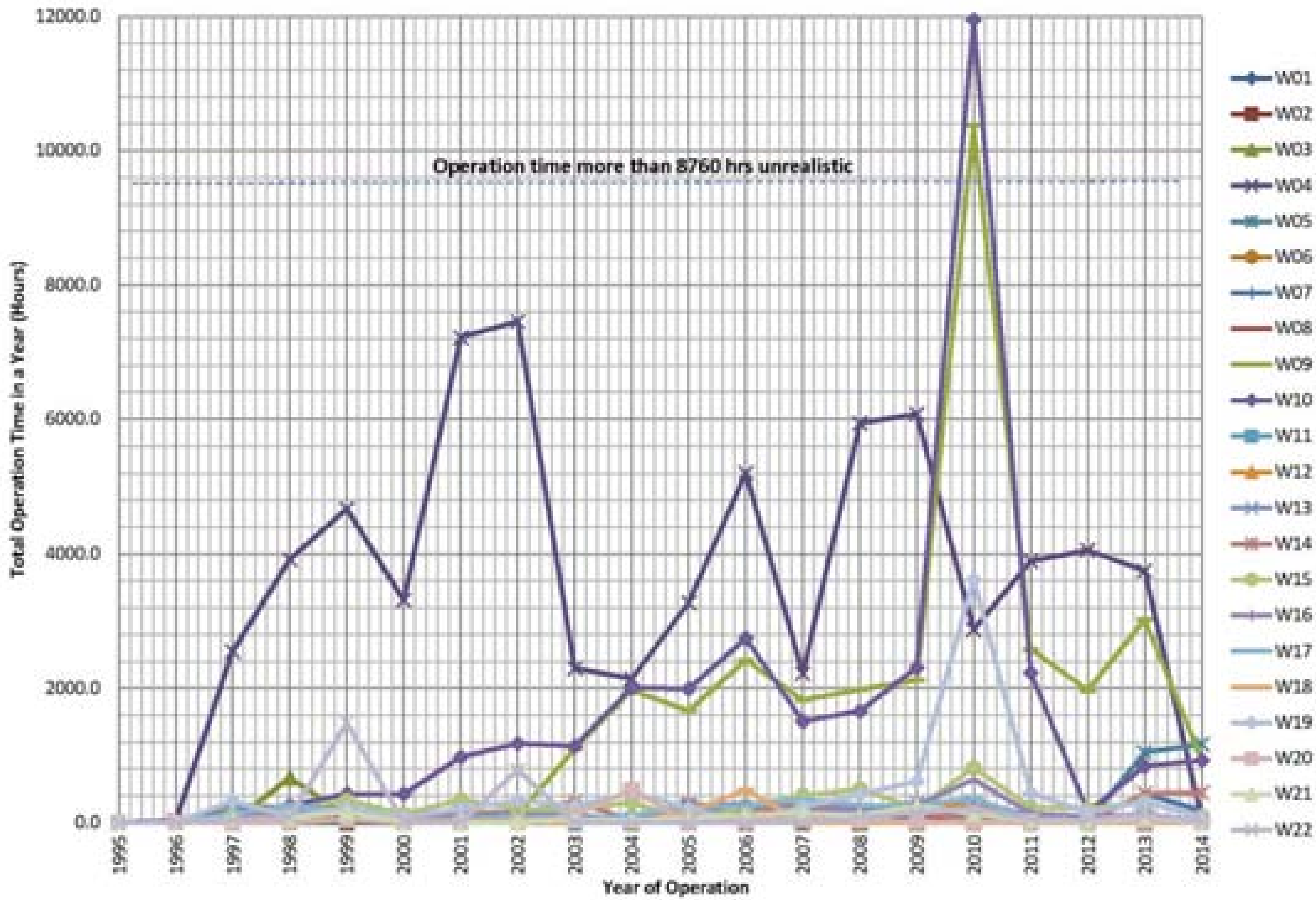
During operation of the pumpwell system, several repair and maintenance operations were performed. Details of the inventory of the system after last repair work are presented in Table 4-7.

Table 4-7 : Inventory of the Pumpwell System

Well No.	Elevations (m) (as per 2009 assessment)					Elevations (m) (after 2013 repair works)	
	Screen		Sensors		Top of pump	Sensors	
	Top	Bottom	High	Low		High	Low
1	32	1.5	-	-	No pump	-	-
2	23	-10	-	-	No pump	-	-
3	20	-11	11.65	6.65	3.21	10	6.65
4	21	-9	9.96	6.81	3.31	9.96	6.81
5	25	-2	13.8	8.72	4.1	9.96	6.81
6	25	7	13.81	8.86	3.88	9.96	6.81
7	28	-3	15.74	10.66	4.16	9.9	6.9
8	14	-2	14.09	9.01	4.24	9.95	6.8
9	28	-2	19.83	14.75	10.68	16.83	11.75
10	30.5	-1	13.05	7.97	3.65	10	6.9
11	35	2.6	22.5	17.5	13.14	15	10.9
12	30	5	12.71	7.73	3.97	10	6.9
13	30	0	12.46	7.48	4.13	12.46	7.48
14	35	5	12.06	6.96	0.41	10	6.8
15	30	4	13.01	8.03	3.87	13.01	8.03
16	30	0	14.23	9.2	3.72	11.3	7.2
17	28	0	17.05	11.97	5.85	13.5	8.5
18	32	2	27.9	22.87	No pump	27.9	22.87
19	33	-2	12.61	7.63	2.88	12.61	7.63
20	22	-8	15.16	10.08	2.9	15.16	10.08
21	22	-2	12.43	7.4	3.21	12.43	7.4
22	27	-8	12.42	7.47	3.01	12.42	7.47

Current pump wells data monitoring and transfer system records on and off sequences of the pumps and transfers data to NL Hydro offices. Figure 4-35 compares the operation times for the wells since their installation.

Figure 4-35 : Summary of Pump Wells Operation



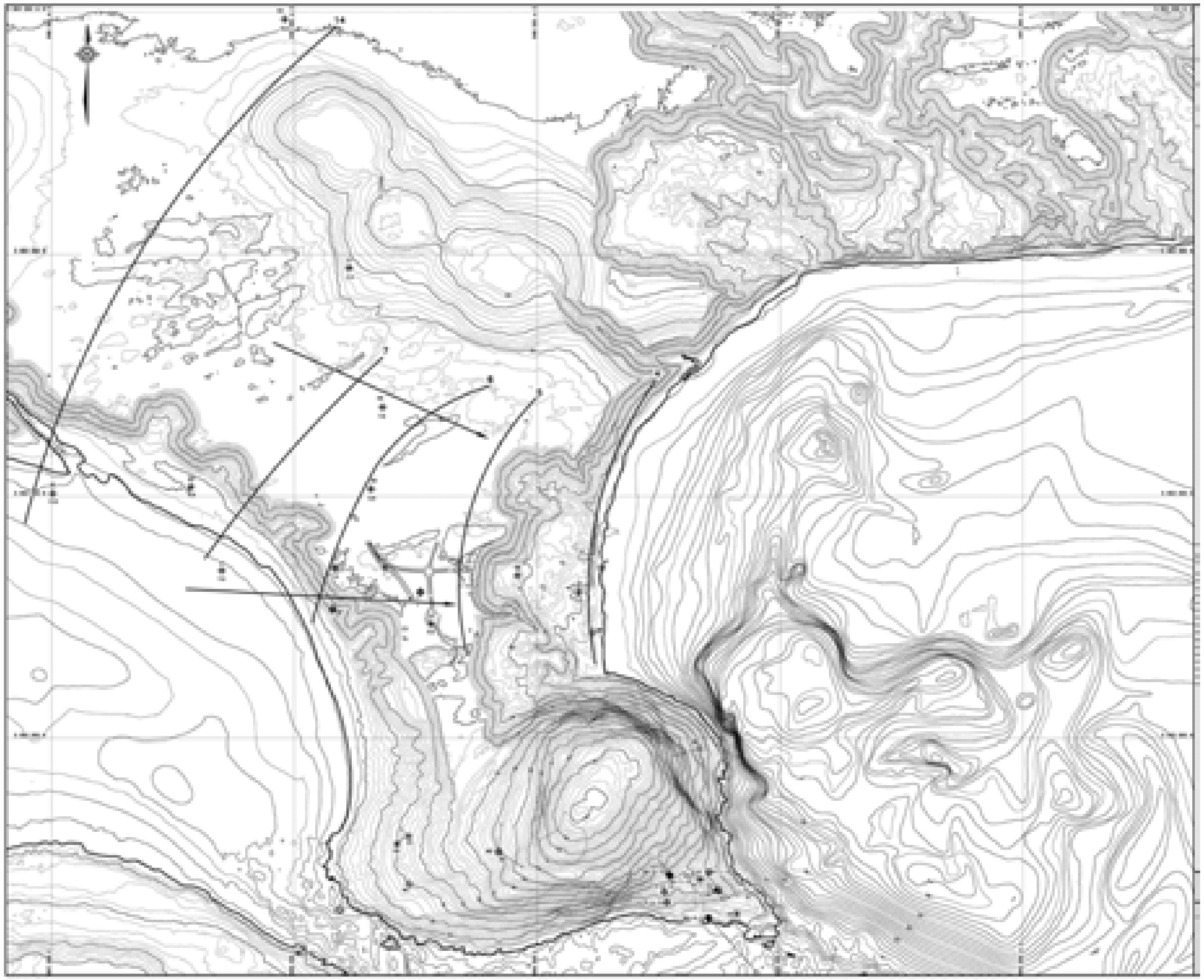
4.3.4 LOWER MARINE CLAY LAYER

A downward ground water flow, from intermediate aquifer to lower aquifer, was identified in the lower clay layer. Piezometers installed in this layer (C4-2-79, B7-2-79, B7-3-79, B6-1-79, P-G-98, P-J1-98, P-F1-98) indicate a downward hydraulic gradient of about 0.35 exists in this layer. The piezometers installed in the lower clay layer in the downstream side in the area of the south and central blocks of the dewatering system have responded to lowering the water level in the intermediate aquifer.

4.3.5 LOWER AQUIFER

A lower aquifer was identified in the lower granular layer (generally below elevation -70 m). The average thickness of this aquifer is about 44 m in the vicinity of the North Spur. The south limit of this aquifer can be inferred between boreholes NS-2-13 and BH-D2-79. The piezometric levels in this aquifer were measured during 1979 investigation to change from 15 m and 13.5 m in boreholes BH-E2-79 and BH-A2-79 on the upstream side, to 4.3 m in borehole BH-B7-79 on the downstream side of the North Spur. The piezometric contours in the lower aquifer are estimated as shown in Figure 4-36.

Figure 4-36 : Lower Aquifer – Piezometric Contours



A maximum hydraulic gradient of about 0.01 exists in the lower aquifer from upstream to downstream side of the Spur. In 1979 a flow rate of 18 l/s, across an effective width of 1 km of the aquifer in the vicinity of the North Spur, was estimated.

The river bathymetry shows that a deep depression, with a minimum elevation of about - 60 m, exists in the bay immediately downstream of the Spur. Considering the top elevation of the lower aquifer, it is expected that the aquifer is connected to the downstream side of the river in this area.

Figure 4-10 (Section H-H) presents a cross section of the North Spur, with extrapolated stratigraphy, which shows this connection in the downstream side.

From October 20th to 30th 1979, a discharge from the Churchill Falls power plant caused an increase in the River water level. The maximum increase in water level was 2.8 m and 1.88 m at the upstream and downstream sides of the Spur, respectively. Comparing the trend of responses in the piezometers, it was inferred that they are consistent with the rise in the River level in the downstream side of the Spur, which can be attributed to the abovementioned connection between the downstream side of the river and the lower aquifer.

During the 1979 pump test in the lower aquifer, the piezometric levels in the intermediate aquifer did not change, which confirms there is no connection between the two aquifers and the lower marine clay layer acts as an aquitard. Aquifer coefficients obtained from pumping tests in 1979 are presented in Table 4-8.

Table 4-8 : Lower Aquifer – Coefficients

Piezometer or well	Aquifer thickness (m)	Transmissivity (m^2/s)	Storage coefficient	Hydraulic conductivity (cm/s)
A 1 (D)	51	9.31×10^{-3}	4.38×10^{-3}	1.8×10^{-2}
(R)	51	6.41×10^{-3}	5.16×10^{-3}	1.2×10^{-2}
A 2 (R)	56	7.19×10^{-3}	4.83×10^{-3}	1.3×10^{-2}
B 3 (R)	39	5.79×10^{-3}	1.05×10^{-3}	1.5×10^{-2}
B 7 (D)	32	9.63×10^{-3}	4.08×10^{-3}	3.0×10^{-2}
(R)	32	8.50×10^{-3}	2.57×10^{-3}	2.7×10^{-2}
C 1 (D)	36	2.72×10^{-3}	6.06×10^{-4}	7.5×10^{-3}
(R)	36	2.84×10^{-3}	9.68×10^{-4}	7.9×10^{-3}
C 3 (D)	17	6.41×10^{-3}	2.7×10^{-4}	3.7×10^{-2}
(R)	17	5.69×10^{-3}	2.76×10^{-4}	3.3×10^{-2}
D 5 (R)	93	6.59×10^{-3}	4.47×10^{-3}	7.1×10^{-3}
E 1 (D)	76	6.86×10^{-3}	1.29×10^{-3}	9.0×10^{-3}
(R)	76	6.29×10^{-3}	1.52×10^{-3}	8.3×10^{-3}
E 2 (D)	36	8.54×10^{-3}	2.24×10^{-3}	2.4×10^{-2}
F 2 (D)	40	6.66×10^{-3}	-	1.7×10^{-2}
F 2 (R)	40	6.66×10^{-3}	-	1.7×10^{-2}
P 1 (D)	32	3.91×10^{-3}	3.56×10^{-3}	1.2×10^{-2}
(R)	32	2.91×10^{-3}	1.93×10^{-3}	9.1×10^{-3}
P 2 (R)	42	2.17×10^{-3}	1.95×10^{-3}	5.2×10^{-3}
Distance drawdown	44 (average)	6.06×10^{-3}	-	1.6×10^{-2}

D = Drawdown

R = Recovery

In order to provide information to create a database of the hydrogeological behaviour of this unit, a microdiver permitting recording of the water levels was installed inside the borehole NS-2-13 in 2014.

4.3.6 3D HYDROGEOLOGICAL MODEL

As previously noted a 3D hydrogeological study of the North Spur was performed as part of the specialized studies.

The Lower and Intermediate Aquifers (LA and IA) were modeled for different hydrogeological scenarios. Assessment of their responses during reservoir impoundments was carried out. Two separate models were developed for these aquifers since the lower marine clay unit acts as a boundary region between them.

Main conclusions of this study are presented hereafter.

Regarding the LA, the study concluded that after reservoir impoundments to elevation 25 m (first impoundment) and to elevation 39 m (final impoundment) the piezometric level in the lower aquifer will rise to elevations 5.4 and 6.9 m, respectively, which is lower than the design elevation of the outlet pipe of the relief wells.

Sensitivity analyses were performed on the IA regarding the anchor length of the cement bentonite cut-off wall in the lower clay deposit. The results showed that the penetration of the barrier in the lower clay has a negligible impact on the piezometric levels in the intermediate aquifer.

Variation of the levels in the three Kettle Lakes was analyzed. The study showed that the stabilization works will impact these levels by raising them about 5 m in the upper lake and 4 m in the lower one for the first impoundment and rising by 7 m in the upper lake and 5 m in the lower one during final impoundment. Continuing the pumpwell operation after impoundment will have an insignificant impact on these areas.

4.4 SLOPES STABILITY

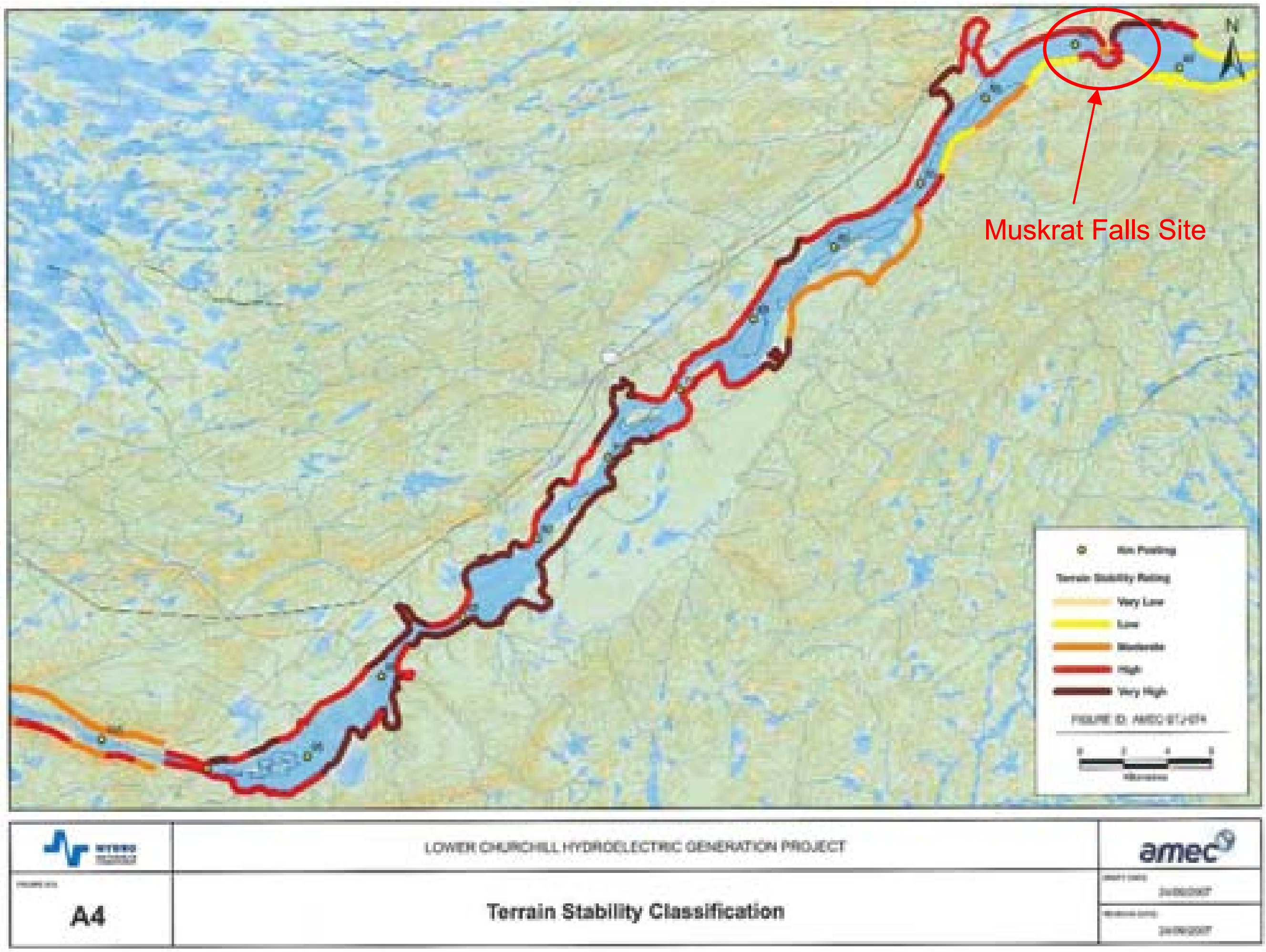
4.4.1 INTRODUCTION

The Muskrat Falls site is situated within the Churchill River Lowlands, which are characterized by a deep accumulation of estuarine and fluvial sediments resting on glacial deposits, glaciofluvial drift or bedrock and overlain by an accumulation of sands of littoral, deltaic or near shore origin. The estuarine sediments were believed to be deposited in brackish, non-turbulent water and, as a result, often consist of silts and sensitive clay which are susceptible to retrogressive landslides. Numerous evidences of slope instabilities can be found in the form of landslide scars along the Churchill River shoreline.

In June 2008, AMEC [Ref. 32] carried out a bank stability study as part of the project Environmental Impact Assessment (EIA). One of the goals of this study was to identify the potential effects of the reservoir on the new shorelines and riverbanks. A rating and classification system of the terrain stability was developed for the reservoir banks and a Terrain Stability Map (TSM) was produced. The purpose of the TSM was to delineate areas where the project may be affected by slope failures and where the project may affect slope stability.

A listing of known failure sites, including location, failure type and the soil type of the Muskrat Falls reservoir and the area downstream of the Muskrat Falls was prepared. Findings from this study for both areas were that the majority of the reservoir downstream of Muskrat Falls presents a terrain stability rating of low and the majority of the proposed Muskrat Falls reservoir presents a terrain stability rating of high to very high. Results of the classification are presented in Figure 4-37.

Figure 4-37 : Terrain Stability Map



In 2010, AMEC [Ref. 33] conducted an extensive bank stability assessment of the Muskrat Falls Reservoir which included air photo/LiDAR imagery interpretation, field reconnaissance, field investigations and laboratory testing. The study led to the identification of the locations and estimate of the size of all historic and contemporary landslides along the lower Churchill River. The study also identified areas showing the greatest potential for future instability that could result in landslides and provided an estimate of the potential size of each. Earth flow and probable earth flow scars identified in the vicinity of Muskrat Falls are presented in Figure 4-38.

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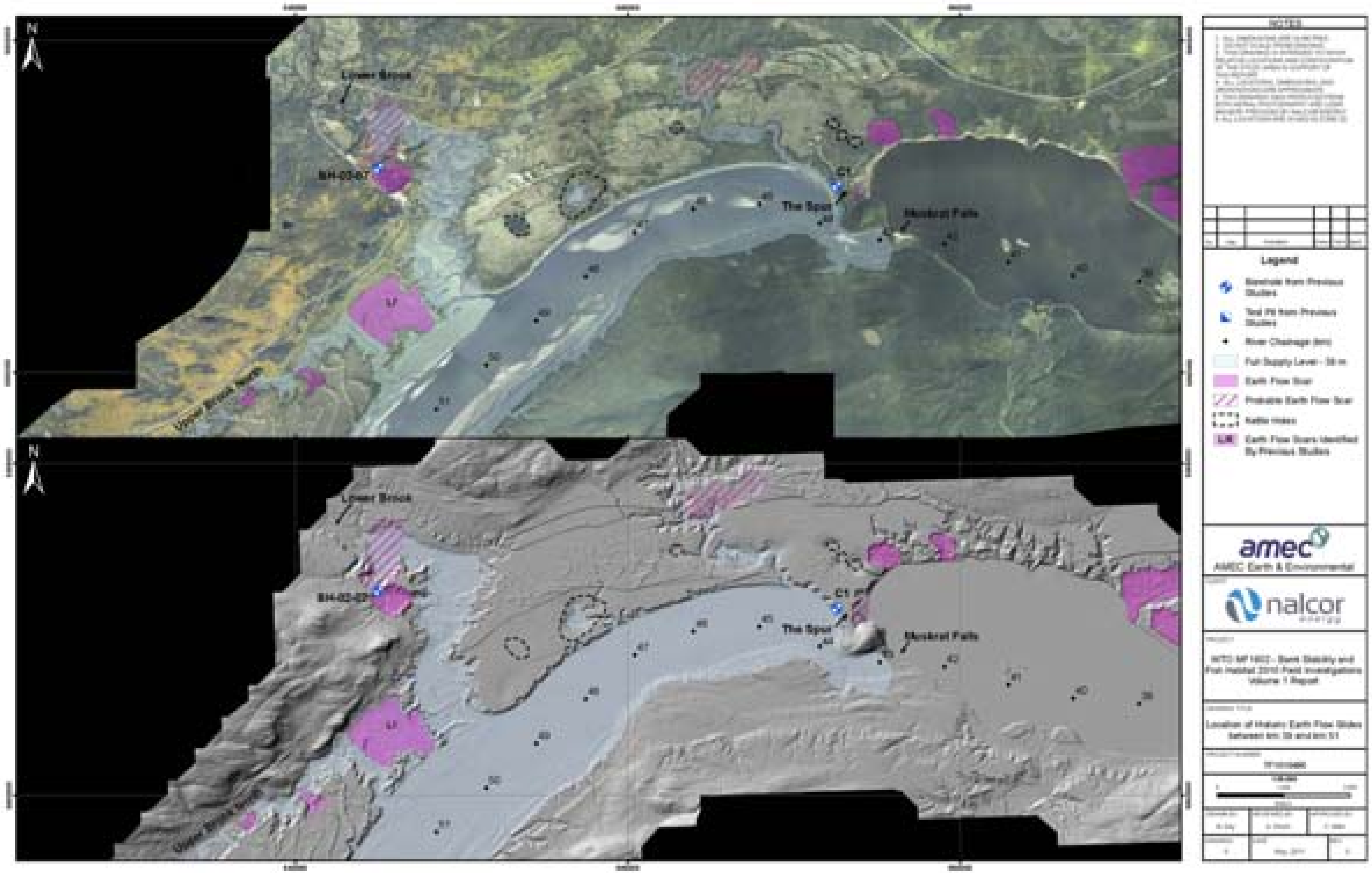
Figure 4-38 : Bank Stability Assessment



4.4.2 NORTH SPUR – PREVIOUS LANDSLIDES

Natural conditions, including hydrogeological regime and mechanical properties of soils have an important role on slopes stability. The main causes of slope instabilities on the Spur for the existing conditions and the different potential slope failure modes are presented and discussed in the following sections.

4.4.2.1 Cause of Slope Instabilities

4.4.2.1.1 Toe erosion due to water and ice level fluctuations

Toe erosion occurs as the river continuously scours the toe of the slope and because of the ice level fluctuations on the downstream area during winter. Downstream water level may rise up to 20 m during winter because of frazil ice accumulation (ice dam) at the downstream side of the Muskrat Falls.

The effect of the toe erosion can be observed in Figure 4-39, which compares the downstream view of the North Spur immediately after the 1978 landslide and in July 2011.

4.4.2.1.2 High pore-water pressures within the stratified drift

The pore water pressure within the stratified drift is a function of seepage from the north-west of the Spur towards the upstream, downstream slopes and the kettle lakes outlet. Any increase in the pore water pressure will decrease the effective strength of the soil and adversely affect the stability of the slopes.

4.4.2.1.3 Surface erosion due to run off water

Rainfall and subsequent runoffs can cause surface erosion. Rainfall can also increase the pore water pressure and destabilize the slopes.

Figure 4-39 : 1978 Downstream Slide – View in 1978 and 2011

4.4.2.2 Potential Slope Failure Modes

4.4.2.2.1 Shallow slab slide

The most common type of landslide is a shallow slab slide with a failure surface parallel to the slope. This mode of failure is triggered by toe erosion at the shoreline. Because of fast rate of erosion, these slopes are usually free of vegetation. This type of slide does not involve a large volume of soil and is observed in the northern area of the downstream side of the Spur. Shallow slab slides can be observed along the upstream shorelines of the river.

4.4.2.2.2 Block sliding

This mode of slide is triggered by toe erosion but the extent of it is generally larger in volume than shallow slab slides and is probably a function of high piezometric levels.

4.4.2.2.3 Flow slides

This mode of slide is also initiated by toe erosion and high pore pressures, but the retrogressive nature and rapid transformation into a mass flow are the consequence of the sensitive materials with low remoulded strengths. In a flow slide, multiple failure surfaces occur and change continuously as the flow proceeds. At the end of the process, the slide may involve a considerable mass of soil. Flow slides on the North Spur are caused by a combination of high pore-water pressures, toe erosion and presence of sensitive layers in the stratified drift. Inclination of these weak layers can also play an important role.

4.4.2.2.4 Progressive failure slides

Experience shows that spreads in sensitive clays generally start from a disturbance such as erosion or a small landslide at the toe of the slope and that progressive failure develops along a quasi-horizontal shear surface or shear zone and that finally the soil above the shear surface dislocates into horsts and grabens. If there is erosion or a small landslide at the toe of the slope, the shear stress will locally increase and possibly locally reach the undrained peak shear strength of the clay. If this happens, progressive failure develops upwards. Then the soil above the shear surface, which is on a layer of more or less remoulded soil will dislocate in horsts and grabens.

Progressive failure may also develop as a result of loading or piling in the slope or beyond the crest of the slope. If loading or piling generates a shear stress that locally exceeds the undrained shear strength, failure may progress downward towards the toe of the slope and generate a global failure. Whatever the progression direction of failure, to have progressive failure, it is necessary for the shear stress to locally reach the undrained shear strength of the clay.

4.4.2.3 Slides in the North Spur

Three major landslides can be observed on the downstream side of the Spur (No. 1, 2, and 3 in Figure 1-1), with No. 2 being the most recent one that occurred in 1978. On the upstream side 4 landslide scarps are labelled as No. 4, 5, 6 and 7 in Figure 1-1.

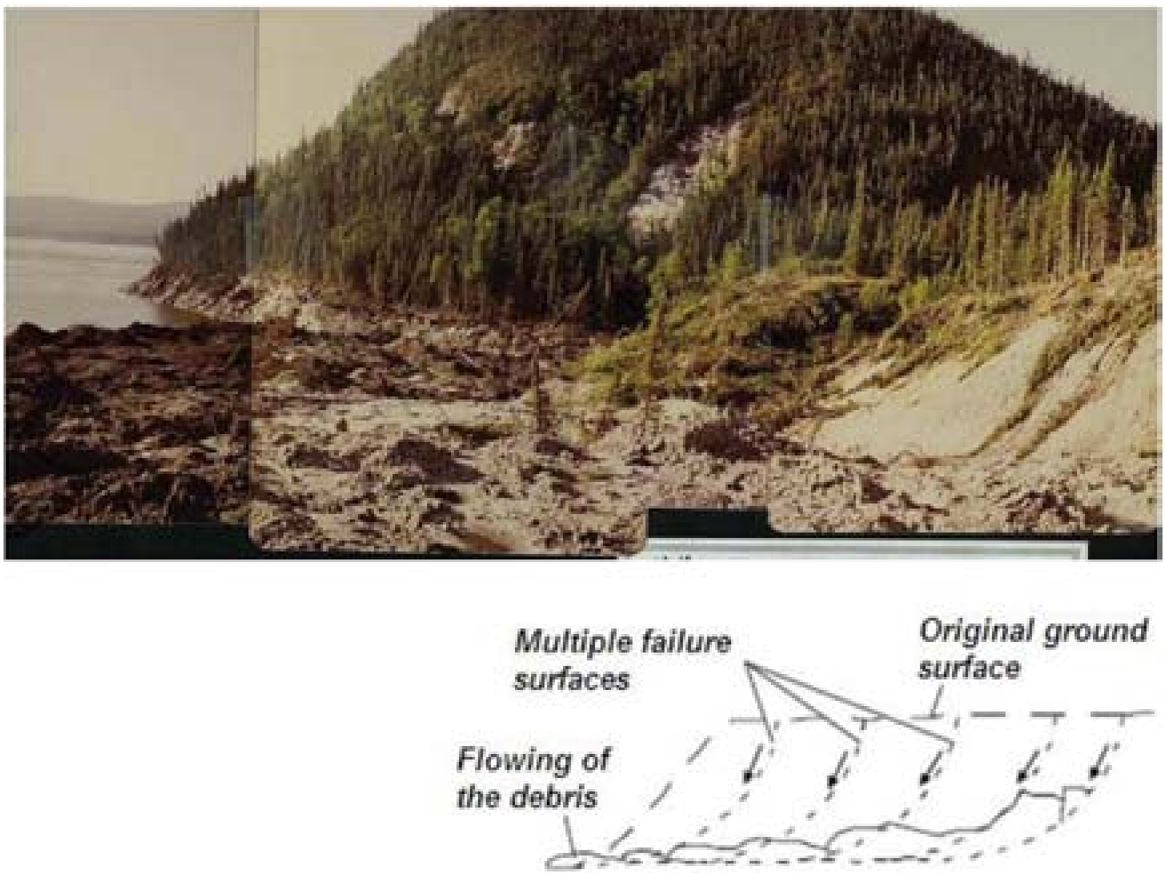
A common feature of the previous slides on the North Spur is that very little slide debris remained in the slide bowl which is an indication of the sensitive nature of the soils involved in the slide.

The latest large deep seated landslide occurred in November 1978 on the downstream side of the North Spur which removed about 1 million m^3 of soil. The maximum distance of the retrogression from the original slope was less than 200 m. The slide involved a block movement within the stratified drift, followed by retrogressive flow slides. Figure 4-40 shows an aerial view of the slide area. The slide reduced the top width of the Spur to about 80 m at elevation 60 m.

Figure 4-40 : Downstream Slide – 1978 Aerial Picture

Three landslides on the downstream side (including the slide that occurred in 1978) are all of retrogressive behaviour similar to flowslides. Figure 4-41 compares a lateral view of the 1978 slide with a schematic retrogressive flowslide. Debris flow and multiple failure surfaces can be observed from the 1978 slide picture. Based on the existing information and general configurations, the four smaller slides on the upstream side of the North Spur are of flowslide type.

Figure 4-41 : 1978 Landslide and Retrogressive Flowslide Scheme



4.4.3 EXISTING SLOPES – STABILITY

The existing slides on the upstream and downstream sides of the North Spur occurred in the stratified drift layers. Scars of three large block landslides on the downstream side and four smaller flowslides on the upstream side can be observed (Figures 1-1 and 2-1)

The steepest existing slopes in the stratified drift layer, on the upstream and downstream sides, are in the range of 1.4H:1V (about 35°) which is an indication of relatively high undisturbed shear strength in this layer.

The existing conditions of the Spur, confirm that degradation of the Spur is an ongoing process that needs remedial work to preserve the Spur integrity.

4.4.3.1 Upstream Slope

In the area of the stabilization works from the rock knoll to the north (about 200 m) the slopes of the Spur are of about 4.5H:1V up to elevation 50 and above this elevation of about 2H:1V.

Following this section and over approximately 100 m, the slopes are gentler below elevation 30 m (7H:1V) and of about 2H:1V above this elevation.

Over the next 250 m, the upstream slope presents the steepest part with a slope of about 1.4H:1V up to the crest of the Spur.

Gentler slopes exist northern of the previous section with inclinations varying between 7H:1V to 4.5H:1V up to elevation 30 and generally of about 2H:1V in the upper part.

4.4.3.2 Downstream Slope

The downstream slope for the first 400 m from the rock knoll is affected by 3 large landslides. Generally the slopes below elevation 30 m are gentle (typically varying between 5 to 7H:1V) and the soil stratigraphy consists of slide debris over the natural soil. The slopes above elevation 30 m are about 1.5H:1V.

The steep intact slope on the downstream side is located between the kettle lakes outlet and the slide No. 3 (Figure 1-1). This slope extends for about 200 m and is sloped at 1.5H:1V from elevation 4 m to elevation 54 m. The slope does not have a full or complete vegetation cover because of ongoing surface sloughing and shallow slab slides.

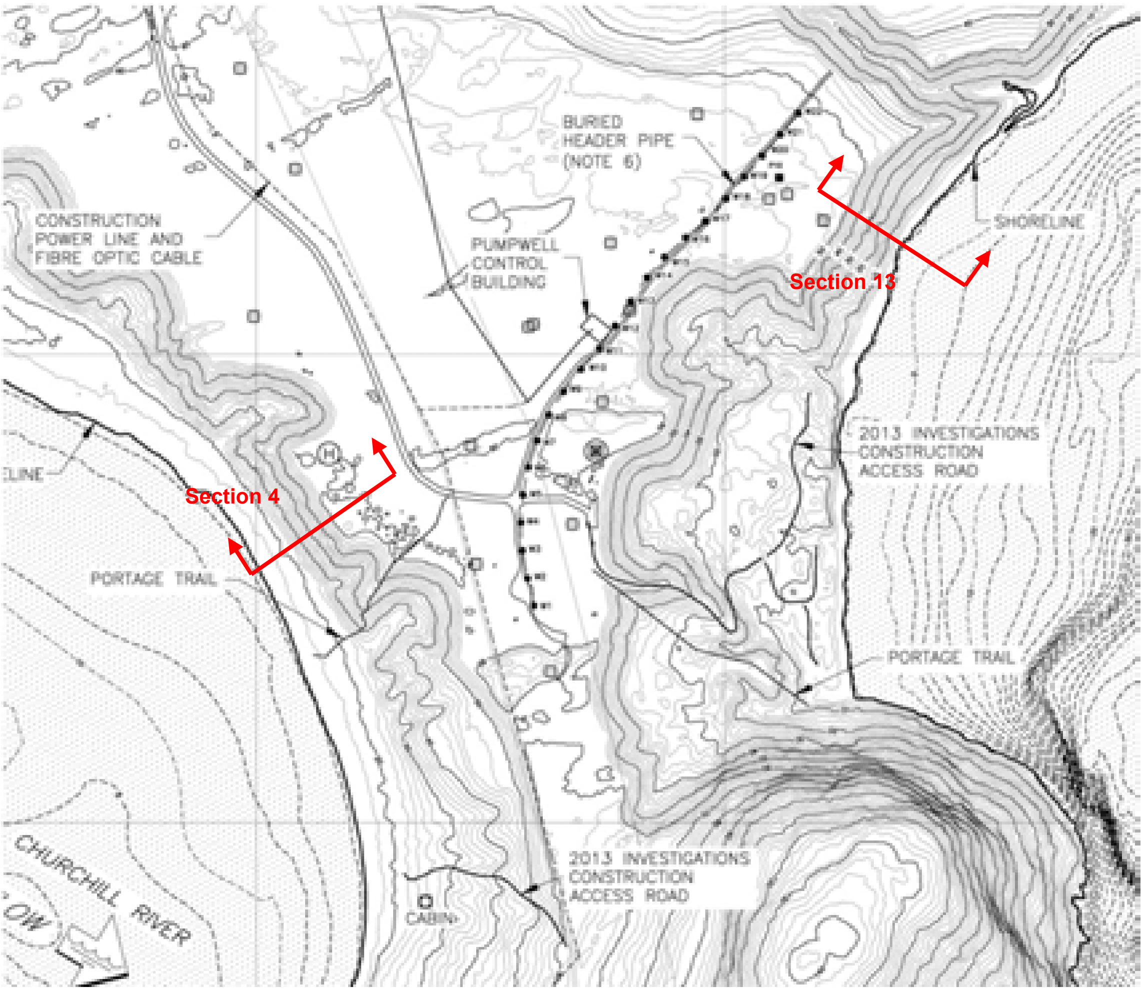
4.4.4 SLOPE STABILITY ANALYSES

The procedure for the slope stability analyses performed involved definition of the slope geometry, hydrogeological conditions and evaluation of mechanical properties of the different soil layers constituting the Spur.

Several representative sections of various slopes within the North Spur, were considered for slope stability analysis. The soil stratigraphy and material properties were defined based on the available and interpreted data, as discussed in previous sections.

One section representing the worst case of the existing conditions was retained for each side of the Spur. Location of the retained sections 4 and 13 is shown in Figure 4-42.

Figure 4-42 : Slope Stability Analysis – Retained Sections



Slope stability analyses were performed in 2 dimensions (2D) by limit equilibrium method using Slope/W version 8 software (GEOSLOPE 2012). The critical failure surfaces and minimum factors of safety based on Morgenstern-Price method are presented and discussed for the different retained slopes.

Effective stress parameters were used for slope stability analyses. Material properties are summarized in Table 4-9.

Table 4-9 : North Spur – Material Properties

Material	Unit Weight (kN/m^3)		Effective Friction Angle, ϕ' (°)	Effective cohesion, c' (kPa)
	Wet	Saturated		
Upper Sand	19	-	35	0
Stratified Drift	18	19	31	6
Lower Clay	-	19.3	31	6
Lower Aquifer	-	19.5	35	0

Results of the stability analyses performed for both sections are presented in Figure 4-43 and Figure 4-44. These figures show the safety map of the range of critical slip surfaces with factors of safety less than 1.5. Current stability conditions of the retained slopes are stable with minimum factors of safety around unity.

The factors of safety of about 1.0 obtained in the natural slopes upstream and downstream of the slopes confirms the validity of the mechanical properties used for the calculations.

Figure 4-43 : Stability Analysis – Upstream, Section 4

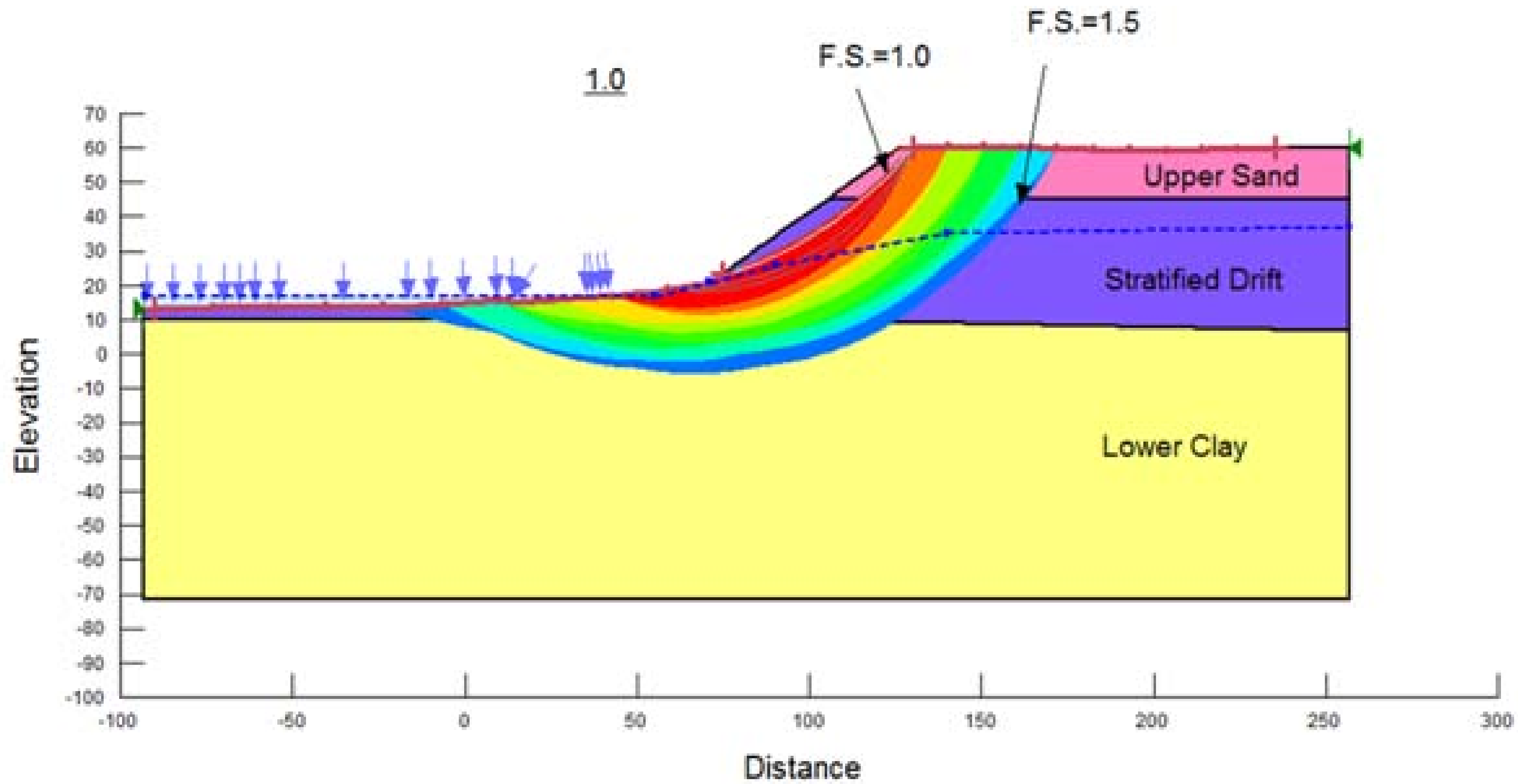
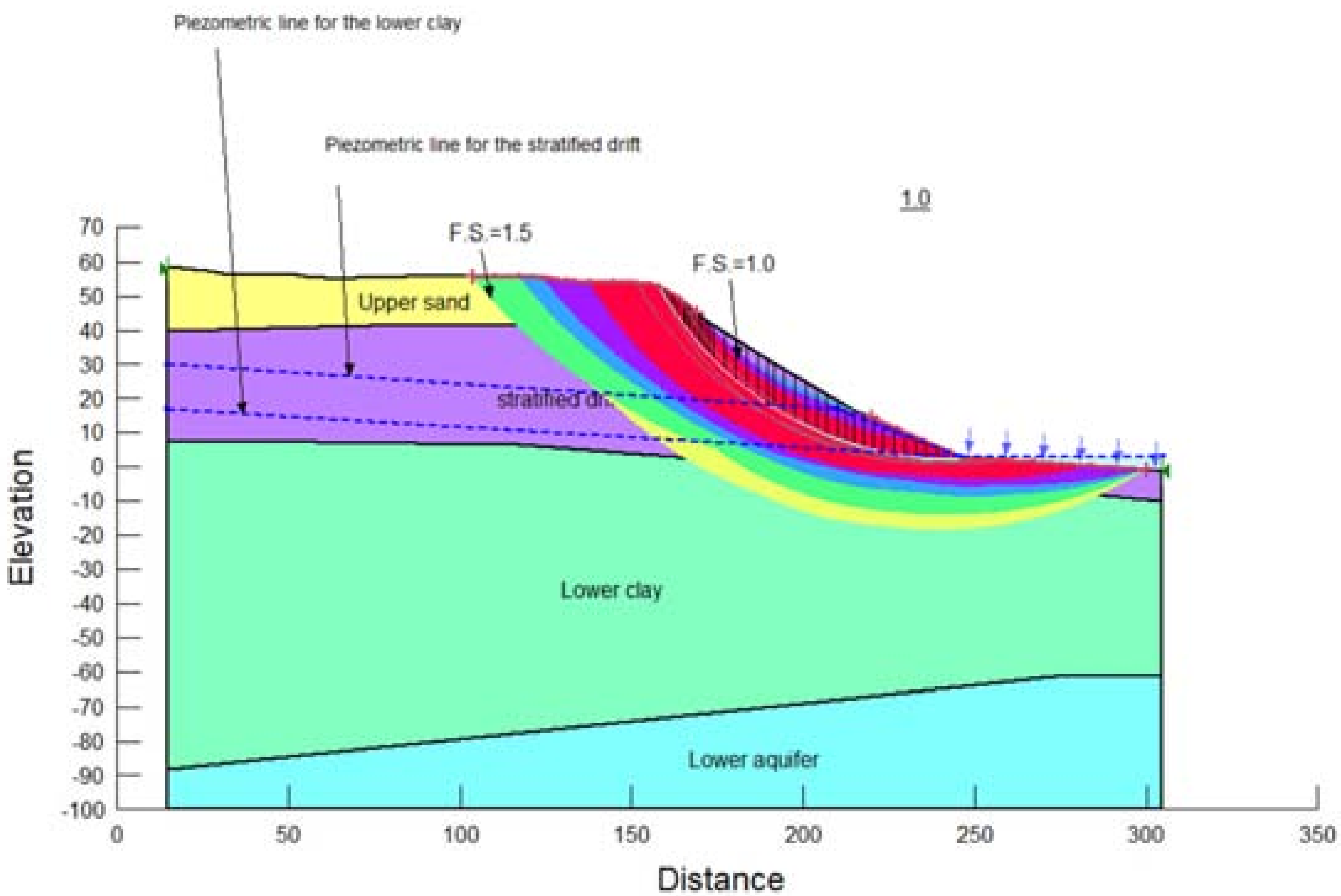


Figure 4-44 : Stability Analysis – Downstream, Section 13



5 DESIGN OF STABILIZATION MEASURES

5.1 INTRODUCTION

The design of the stabilization works of the North Spur follow state-of-the-art methods and comply with appropriate standards and guidelines. The guidelines prepared by the Canadian Dam Association (CDA) were used as a basis for the design and constitute the primary framework. In the context of the Lower Churchill project, the North Spur is treated as a dam for the selection of design criteria.

The 2007 CDA Dam Safety Guidelines [Ref. 34] presents several principles for the assessment of dam safety. These are equally applicable to the design stage of dam construction.

The final engineering of the stabilization works of the North Spur was undertaken after final assessment of the geological and hydrogeological conditions of the Spur and refinement of the findings of previous engineering studies, including considerations of new technologies available.

The final design calls for a control of the groundwater in the North Spur; erosion protection in both sides, upstream and upstream of the Spur and local unloading of the upper part of the Spur.

Groundwater control on the North Spur will be assured by:

- Construction of a cement bentonite cut-off wall and till blanket barrier in the upstream area;
- Construction of a cement bentonite cut-off wall in the northwest area of the North Spur;
- Construction of finger drains and inverted drains in the downstream area;
- Improvement of drainage of the Kettle Lakes;
- Installation of relief wells in the lower aquifer.

It should be noted that the existing dewatering system has to be maintained in operation during the construction work. Discussions on future operations and potential decommissioning of this system are presented in more detail later in this report.

Construction of berms and rock embankments to protect the upstream and downstream slopes against erosion and local excavation of the upper layers to unload the slopes have been planned as part of the stabilization works.

Details of the different stabilization works are presented in more detail in the following sections.

5.2 UPSTREAM SLOPE

Works on the upstream side of the North Spur will have the double purpose of stabilizing the natural slopes to prevent landslides and to create a more impervious barrier against infiltration from the reservoir.

The barrier consists of two different structures. The lower part includes a cement-bentonite cut-off wall built through the sandy silt/silty sandy foundation from elevation 20.5 m down to at least 2.0 m into the lower clay layer. Upper limit of the clay layer has been mainly assessed from the results of the 2013 investigation campaign. Final key elevation of the cut-off wall must be confirmed on site by the geotechnical Engineer based on visual inspections of the excavated materials. The upper part of the barrier is completed with an inclined till blanket up to elevation 42.4 m.

It is anticipated that the depth of the upstream cut-off wall will vary between 15.5 and 10.5 m where anchored in overburden and between 10.5 and 0 m where in contact with rock. Approximate length and area of the cut-off wall were assessed to be 800 m and 8,000 m^2 respectively.

The upstream cut-off wall will be constructed from a temporary granular platform at approximately elevation 21.0 m. The elevation of the working platform was defined at 1.5 m above the 1:20 year river water level in summer and fall, which corresponds to a design flood of 3,500 m^3/s and thus, a water level of 18.5 m.

The construction of the upstream cut-off wall will not be executed during the spring flood, for which the river discharge is estimated to be 6,000 m^3/s with river water level of 21.5 m, before diversion, or 21.7 m during the first stage of diversion, before headpond. The cut-off wall and the embankment have to be built up to at least elevation 27.0 m before impoundment for the winter headpond.

The construction of the cement-bentonite cut-off wall is described in more detail in Section 6.

Beyond the junction between the upstream and the North-west (NW) cut-off walls, the upstream slope protection is extended over a distance of about 180 m to prevent any damage to the cut-off walls in the event of landslides in the reservoir rim in that area. The works consist of grading the natural slopes and backfilling using the in-situ soil and building a rockfill protection berm.

The various upstream sections are quite similar, except variation of slope inclinations to adapt the embankment geometry to the natural topography and to differentiate cases with excavation or backfilling operations in the upper part of the sections. The slope of the embankment is 2.5H:1V below elevation 29.0 m, and from this elevation to elevation 43.0 m, the slope of the fill varies between 3H:1V to 4.5H:1V. Above the crest, at elevation 43.0 m, the slope is 2H:1V either for the cut and for the fill cross-sections.

The base of the till blanket (Zone 1 material) is linked to the top of the cut-off wall by means of a trench excavated to elevation 18.0 m, ensuring a minimum seepage path of 5 m along the contact of the till with the cut-off wall. The upper contact between the cut-off wall and the till blanket is built in Zone 1C material to assure full bonding between these materials. The top of the till blanket is at least 3 m higher than the FSL, preventing reservoir water overtopping the impervious blanket. The thickness of the till blanket is at least equal to one third of the hydraulic head at the FSL. The hydraulic head during operation is calculated assuming a minimum piezometric level of 30 m below the till blanket. The till blanket is built against the graded overburden foundation.

The till blanket is protected on the external side with successive layers of granular and rock materials, including a granular transition (Zone 2C material) at contact with till, a rockfill transition (Zone 3C material) and finally an external rockfill shell (Zone 3D material).

The total weight of the fill materials must counteract the potential uplift pressures beneath the till blanket created by water trapped behind (downstream) the till blanket during construction, before reservoir is filled to FSL. For this temporary condition, a factor of safety of 1.1 against uplift was considered adequate assuming a piezometric level of 36 m behind the till blanket.

The toe of the rockfill embankment, at contact with foundation, is reinforced by means of a small berm, 1.8 m wide x 1.8 m high at elevation 20.0 m, built with rockfill (Zone 3D above water level and Zone 3E below water level) .

A riprap protects the rockfill embankment against the action of waves and ice. In the lower part of the slope, the riprap protection (Zone 4 class 2 material) extends from elevation 23 to 27 m, for the period when the river water level will be raised to elevation 24 (open water) to 25 m (winter ice cover) to form a temporary headpond during diversion. During permanent operation at FSL (el. 39.0 m), the upstream slope is protected against waves and ice with a riprap layer (Zone 4 class 1 material) from the elevation 35.8 m to the crest of the embankment at elevation 43.0 m. A rockfill layer (Zone 3C) is placed above elevation 43.0 m up to elevation 47.0 m to complete the protection of the upstream shore in the event of the PMF.

The crest of the embankment is 7.5 m wide at elevation 43.0 m. This crest will serve as permanent access road and the surface layer is composed of crushed stone (maintenance grade No 3 material). A longitudinal ditch along the downstream side of the crest will drain the surface runoff from the top of the North Spur through culverts installed below the crest.

The stabilization works in the north part of the upstream side, beyond the cut-off walls remain the same as presented previously in this section except that no previsions are made for seepage infiltration from the reservoir. The till blanket is replaced by fill granular material (Zone 2G material) product of the overburden excavations.

Figure 5-1 and Figure 5-2 show typical fill and cut cross-sections of the stabilization works.

Figure 5-1 : Upstream Typical Cross-Section – Cut

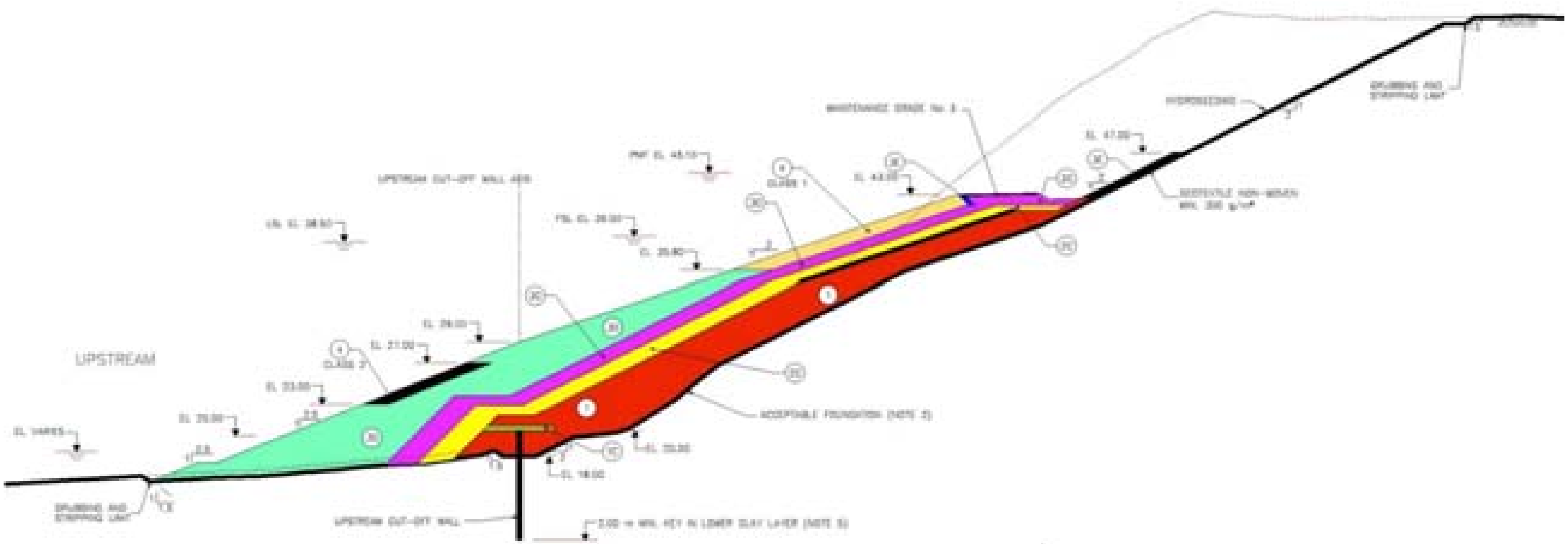
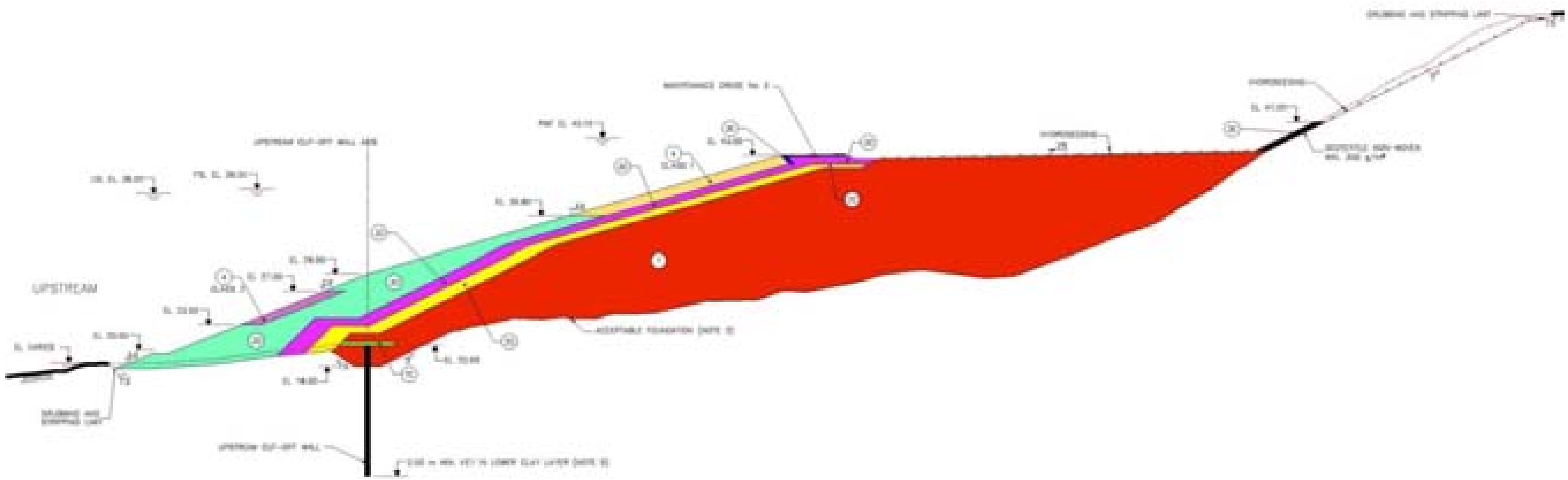


Figure 5-2 : Upstream Typical Cross-Section – Fill



5.3 NORTH-WEST CUT- OFF WALL

The main objective of the North-west (NW) cut-off wall is to control and minimize underground seepage coming from the North-west side of the North Spur, and prevent unfavourable effects on the stability of the downstream slopes.

The North-west (NW) cut-off wall is built using a temporary granular platform at approximately elevation 50.5 m, developed at the bottom of a 10 m deep trench excavated in overburden from the surface of the natural ground at an average elevation of 60 m. The trench slopes are 2H:1V and the width of the bottom is 20 m to accommodate the construction equipment.

The NW cut-off wall is built from the elevation 50.0 m down to at least 2.0 m in the lower clay layer. Upper limit of the clay layer in the area of the NW cut-off wall was assessed from the results of the 2013 investigations. Final key elevation of the cut-off wall must be confirmed on site by the Geotechnical Engineer based on visual inspections of the excavated materials. The expected depth of the cut-off wall is about 40 m. Approximate length and area of the cut-off wall were assessed to 600 m and 19,000 m^2 respectively.

The construction aspects of the NW cement-bentonite cut-off wall are described in more details in Section 6.

5.4 DOWN STREAM SLOPE

The two main objectives of downstream stabilization works are:

- To develop an underground internal drainage in order to control and minimize the piezometric levels and to drain the seepage water.
- To obtain appropriate safety factors of the slopes against sliding after the completion of the stabilization works. Currently, the natural slopes in the upper

part and in the northern area present a factor of safety against sliding of around 1.

On the downstream side, stabilization works below elevation 8.5 m and 1.0 m above water level consist of a layer of riprap (Zone 4 class 3 material) placed over successive layers of rockfill transition (Zone 3C material), drain materials (zone 3A material) and granular filter (zone 2A material). Below this elevation the embankment is made of dumped materials of zone 3E and 2E. The toe of the embankment, at contact with the foundation, is reinforced by means of a berm 1.3 m wide. The external slope of the embankment is 3H:1V below elevation 8.5 m

In order to facilitate the start of the construction work, a zone 3E material berm of > 6.0 m width is built up to 0.50 m above the water level at the time of construction works. This berm will also act as a protection for the toe embankment.

Two main areas (south and north) have been considered for the stabilization works above elevation 8.5 m. Over a distance of about 400 m starting from the rock knoll, the stabilization works (south area) from the elevation 8.5 m to elevation 35.0 m consist of a layer of granular filter (zone 2A material) overlaid by a layer of draining material (zone 3A material) and 2 layers of rockfill (zone 3C and 3D materials) preventing surface erosion. The filter material is placed either on graded in-situ overburden or on graded backfill, built with clean sand (zone 2F material) recuperated from the required excavations.

Between elevations 8.5 and 35.0 m the thickness of the filter material (zone 2A material) is 900 mm and the thickness of the draining and rockfill layers are 450, 450 and 900 mm respectively (Zones 3A, 3C and 3D materials). Between elevations 35.0 and 40.0 m, a 3E rockfill material, 600 mm wide, is placed against the graded slope.

In order to avoid migration of fine particles of the foundation, a geotextile is placed between the acceptable foundation and the placed rockfill.

Internal drainage below the current downstream slopes is designed to collect and evacuate the seepage occurring in the bottom of the main valleys previously formed by the landslides and filled with remoulded soils. Internal drainage of the downstream slope is designed with finger drains located mainly in the center of the natural depressions formed following the old landslides. These finger drains consist of selected crushed stone, draining material (zone 3A material) wrapped in a granular filter (zone 2A material), all embedded in a semi-pervious sand (zone 2F). Seepage water is evacuated through the riprap placed along the river shore.

The slope of the embankment is 7H:1V between elevations 8.5 m and 25.0 m and from elevation 25.0 m to elevation 35.0 m, the slope of the embankment varies from 2.5H:1V to 7H:1V to take advantage of the actual topography and minimize the volume of excavation and backfill works. Above elevation 35.0 m, the natural ground is regraded at 2H:1V slopes and protected with placed rockfill (zone 3E material) up to elevation 40.0 m.

Berms are developed at nominal elevations 8.5 m, 14 m and 25 m. The lowest berm at elevation 8.5 m will serve as permanent access road along the river shore. Berm at elevation 14 m will serve as an access road to instrumentation installed at that level for their protection in the event of the PMF. Berm at elevations 25 m gives access for inspection. A permanent access road is developed on the overall downstream slope to link the top of the North Spur at elevation 60 m to the shore line road at elevation 8.5 m.

For the north part of the downstream shore, from about stations (STA) 0+420 to 0+740 (drawing # MFA-SN-CD-2800-CV-PL-0009-01), stabilization works consist of grading the natural ground, above elevation 8.5 m, to a slope of 2.0 to 2.5H:1V and protecting it with a layer of placed rockfill (zone 3E material) up to elevation 14.0 m ensuring the protection of the overburden slope in the event of the PMF. At the toe of the 2.0 to 2.5H:1V slope, a ditch along the longitudinal berm (or road) at elevation 8.5 m will collect and evacuate seepage exiting from the slope.

Figure 5-3, Figure 5-4 and Figure 5-5 show typical fill and cut cross-sections of the stabilization works.

Figure 5-3 : Downstream Typical Cross-Section – Cut Above Elevation 25 m

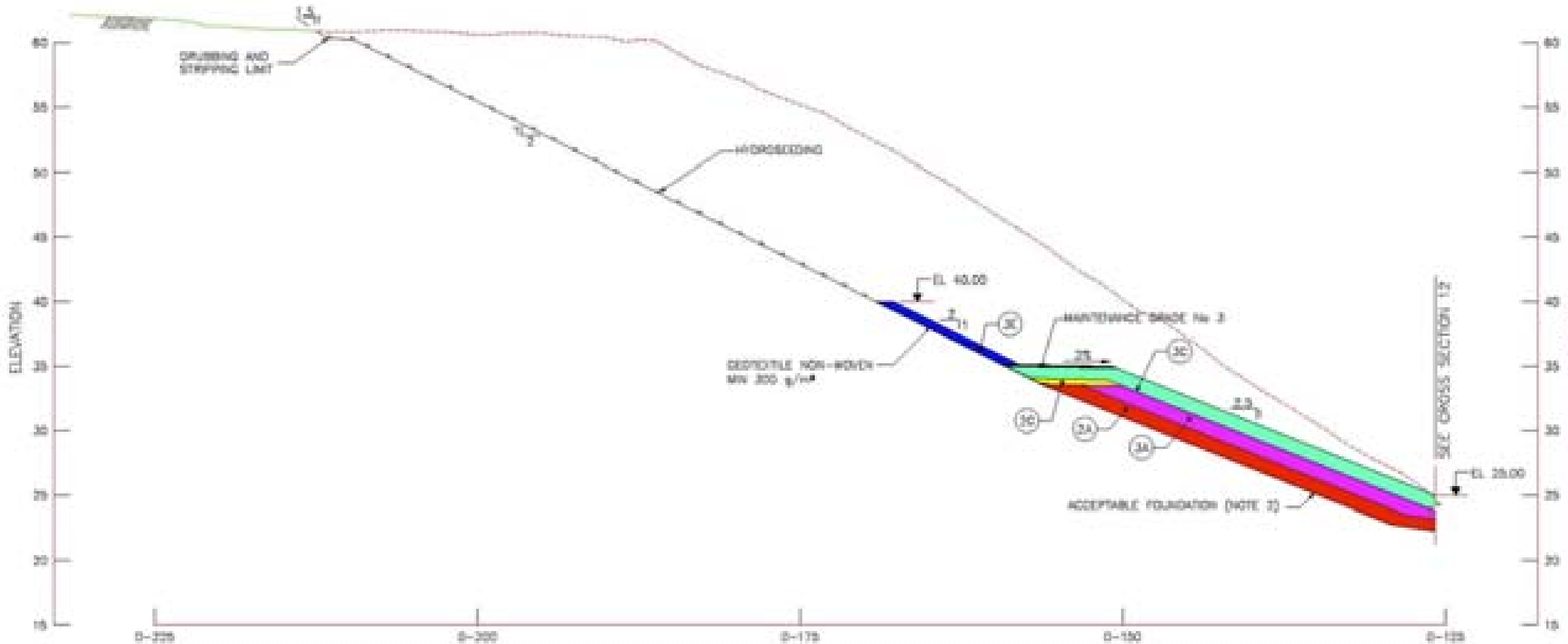


Figure 5-4 : Downstream Typical Cross-Section – Fill Above Elevation 25 m

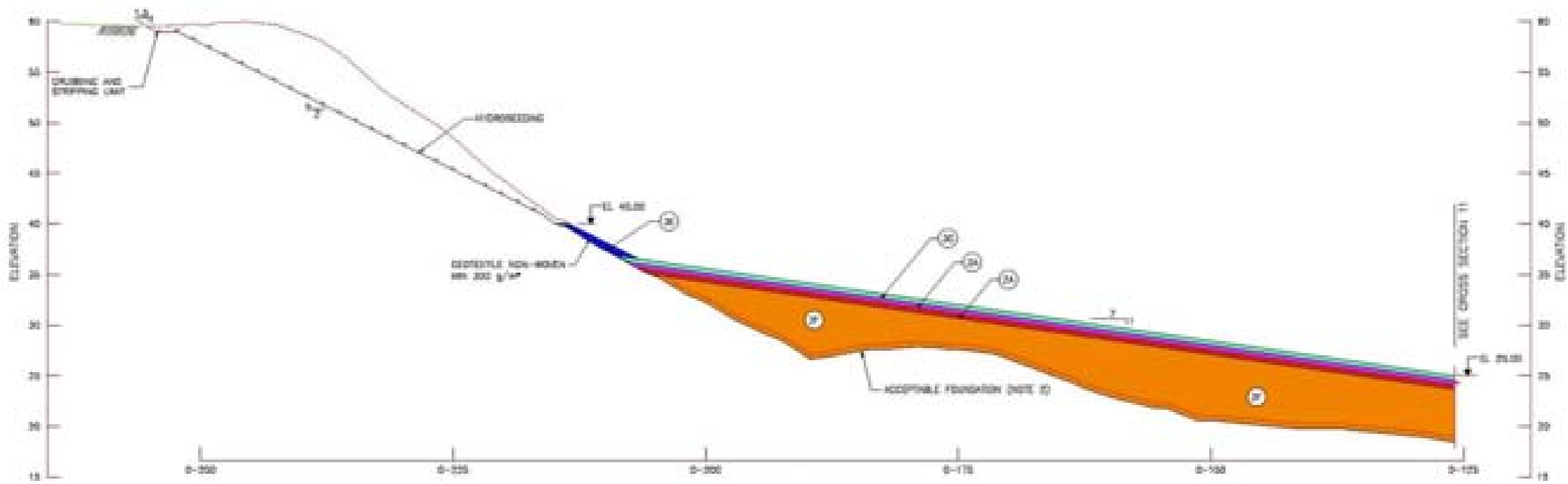
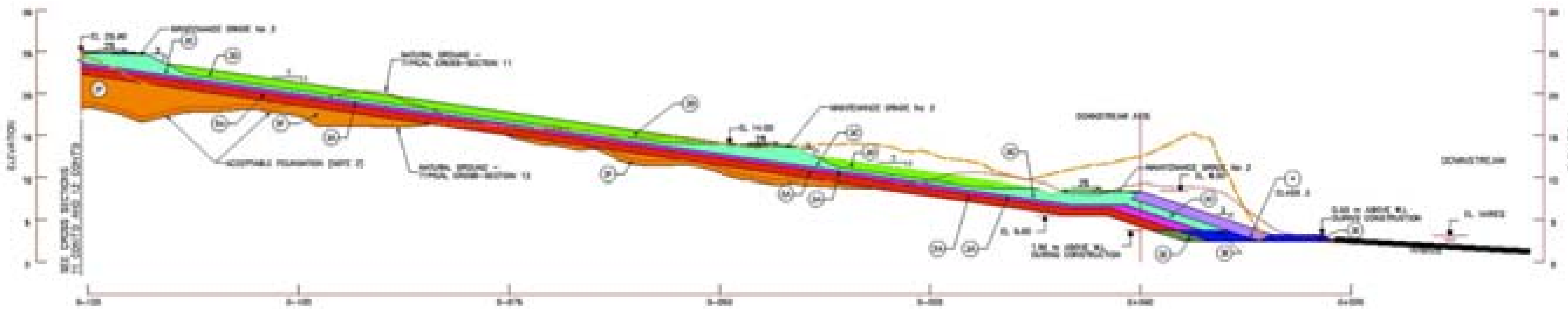


Figure 5-5 : Downstream Typical Cross-Section – Cut and Fill Below Elevation 25 m



5.5 KETTLE LAKES AREA

The purpose of the construction works in this area is to stabilize the natural slopes against instability and to facilitate the continuous flow of water in the kettle lakes outlet.

The natural slopes of the valley, in the downstream side of the lower kettle lake up to the river bank, are stabilized with an embankment built at the bottom of the valley, from the river shore. A 3.0 m wide channel of zone 3C material will be built to protect against erosion and to facilitate the construction. The embankment is constructed over a geotextile placed against the acceptable foundation after grubbing and stripping operations. The thickness of the protection against erosion in the slopes of the channel is defined regarding the longitudinal slopes of the discharge channel. The lateral slopes of the discharge channel will be regraded at 2H:1V or gentler. A stilling basin is designed at the exit of the channel to mitigate potential erosion of the materials.

5.6 RELIEF WELLS

During the fieldwork performed and as per the data collected in recent years, variations in the water level inside the lower aquifer (build-up pressures) were observed following variations of the river water levels upstream and downstream of Muskrat Falls.

To be prepared for the possibility of high water pressures in the lower aquifer following the reservoir impoundment, provisions have been made for the installation of relief wells at the toe of the stabilization works in the downstream shore of the North Spur, should they be required.

A total of 10 relief wells would be installed in the lower aquifer with a 5.0 m minimal length inside the pervious layer. Each relief well would be connected to a collector pipe, which discharges in the river in the downstream area.

The necessity and final arrangement of the relief wells will be determined after the initial impoundment of the reservoir up to elevation 25.0 m during diversion, and analysis of the piezometric levels obtained from the piezometers installed in the lower aquifer.

5.7 EXISTIN G DEWATERING SYSTEM

The stabilization works of the North Spur are designed with the assumption that the installed pumping system will no longer be required.

The cut-off walls and the till blanket will reduce seepage from the reservoir and from the NW sources of water. The downstream slope is designed to be stable without the active pumping system. Therefore no upgrading of the pumping system was planned at the engineering stage. However, the pumping system will be refurbished so that it can be kept in operation during the construction period and for a time to be defined after reservoir impoundment at FSL. The decision to continue the operation of the pumping system will be taken only after a detailed monitoring study of the North Spur after stabilization works completed and the reservoir is in operation.

5.8 EXCAVATIONS

Prior to the excavation works to reach the acceptable foundation, grubbing and stripping works, which consist in the removal of organic matter and contaminated materials covering the overburden, are required.

In the areas where only grubbing and stripping are required and after these works, only the overburden not complying with the criteria of acceptable foundation shall be excavated.

The excavation limits shall be established on site based on the limits shown on the drawings and the current conditions found after clearing operations and during excavation works.

Temporary excavation slopes shown on the drawings are considered stable in normal conditions; however they are established only for measurement purposes. In particular conditions, such as water infiltration, it may be necessary to excavate gentler slopes or to protect them with embankment material.

Excavation procedures shall be planned to take into account of conditions that may potentially affect stability, such as thaw of the ground in spring, water table fluctuations and the sensitive nature of some of the soils. Progressive excavations, such as those for the finger drains, are likely to experience water flow from the advancing face.

No bedrock excavations are anticipated, if necessary, bedrock treatment shall be performed as explained in the next section.

5.9 FOUNDATION PREPARATION

5.9.1 OVERBURDEN FOUNDATION

Overburden foundation preparation works include clearing, grubbing and stripping and excavation of all organic matter, erodible, disturbed and soft materials until reaching the acceptable foundation overall the embankment footprint and the compaction of the overburden surface, when possible. Compaction works shall be required only if the nature of soil allows for this type of operations. Surface of the acceptable foundation shall be graded, uniform, without ruts and potholes.

Compaction of the foundation shall be as directed by the Engineer depending on the site conditions.

All acceptable foundation shall be approved by the Engineer before placement of any embankment material.

5.9.2 BEDROCK FOUNDATION

Foundation preparation on bedrock comprises excavations of rock to eliminate unsuitable foundation conditions, dental excavation, scaling and cleaning of the rock surface. Type and extent of the foundation treatment shall be defined by the Engineer on site.

The main objectives of the foundation preparation of the bedrock are to assure the watertightness of the contact with the embankment, to avoid migration of till materials and to obtain a uniform profile to allow an effective compaction of the embankment materials over the foundation and an acceptable stress distribution in the embankments. In the south side of the North Spur, the embankment will be founded on bedrock at the abutment with the rock knoll, in both upstream and downstream areas. Bedrock preparation operations in the downstream area are less restrictive, as no till material is placed against the bedrock.

On the upstream side, rock surface treatment starts with excavation of overburden down to solid rock on the overall footprint of the embankments. At the contact of the till zone of the impervious blanket, rock treatment includes:

- Cleaning of the rock surface with a light excavator equipped with a bucket without teeth;
- Correction of geometry of the rock surface, if required, in order to eliminate deep depression, excessive inclination, overhang and divergent planes. This correction can be achieved by rock excavation, concrete placement, or by a combination of both;
- Cleaning of shear zones, faults and open joints;
- Placement of concrete or mortar in open joints, shear joints or faults and application of liquid mortar on fissured bedrock just before placement of till;
- Cleaning with water/air jets of the rock surface before placement of concrete, mortar or till.

Even though the hydraulic heads are small and it is anticipated that the condition of the bedrock will be good, provision has been made for grouting of the bedrock contact. The necessity of this will be determined on site after inspection of the exposed bedrock foundation.

In the downstream area only cleaning of the bedrock surface using a backhoe with a smooth (without teeth) bucket of 0.5 m³ capacity is required.

All acceptable foundation shall be approved by the Engineer before placement of any embankment material.

5.10 EMBANKMENT MATERIALS

5.10.1 GENERAL

The embankment materials to be used are presented in the following sections. More detail on the requirements concerning procurement, production, placement and compaction of the embankment materials are presented in the Technical Specifications.

Different types of embankment materials are required for the North Spur stabilization works. These materials are classified as till material, fill material, filter material, granular material and rock material.

Materials classified by zones are as follows:

- Zone 1: Compacted till, max. 300 mm;
- Zone 1C: Compacted till, max. 80 mm;
- Zone 2A: Selected/Processed compacted sand and gravel, max. 80 mm;
- Zone 2C: Selected/Processed compacted coarse granular, max. 300 mm;
- Zone 2E: Selected/Processed placed coarse granular, max. 300 mm;
- Zones 2F and 2G: Natural selected, compacted fine to medium sand, max. 20 mm;

- Zone 3A: Processed, compacted crushed stone max. 80 mm;
- Zone 3C: Selected/Processed compacted rockfill, max. 450 mm;
- Zone 3D: Random compacted rockfill, max. 900 mm;
- Zone 3E: Selected/Processed placed rockfill, max. 450 mm;
- Zone 4 class 1: Riprap, max. 750 mm (D_{50}= 600 mm);
- Zone 4 class 2: Riprap, max. 650 mm (D_{50}= 550 mm);
- Zone 4 class 3: Riprap, max. 600 mm (D_{50}= 500 mm);
- Rockfill Type 1: Selected placed rockfill 100-250 mm (D_{50}=150 mm);
- Rockfill Type 2: Selected placed rockfill 200-400 mm (D_{50}=300 mm);
- Selected Granular “B”: Crushed stone and/or granular, compacted, maximum 50 mm;
- Maintenance Grade No 3: Crushed stone and/or granular, compacted, maximum 25 mm.

5.10.2 TILL MATERIAL, ZONES 1 AND 1C

Impervious embankment material for the upstream impervious blanket is composed of till material (Zones 1 and 1C). Till material shall comply with the gradation requirements presented in Table 5-1 and Table 5-2.

In order to ensure imperviousness, the fine content (passing sieve 0.080 mm) shall be at least 15% (25 % for zone 1C material). To prevent pore water pressure increase during embankment construction, the maximum fine content shall be limited to 60%.

In order to avoid potential problems of internal instability and to minimize segregation during placing of the material in layers of 450 mm thickness (150 mm for zone 1C material), the maximum particle size shall be 300 mm (80 mm for zone 1C material).

The particle size distribution shall be well-graded without absence or excess of any fraction whatsoever within the limits of the specified gradation.

The maximum moisture content of the zone 1 material shall be between -1% and +1.5% of the optimum moisture content as determined by ASTM D698 or ASTM D1557 standards.

The maximum moisture content of the zone 1C material shall be between the optimum moisture content and 2% above of the optimum moisture content as determined by ASTM D698 standard.

Table 5-1 Zone 1 Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
300	100
80	80 – 100
20	65 – 100
5	50 – 95
1.25	37 – 85
0.315	26 – 75
0.080	15 – 60

Table 5-2 Zone 1C Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
80	100
20	80 – 100
5	63 – 95
1.25	48 – 85
0.315	35 – 75
0.080	25 – 60

5.10.3 FILL MATERIAL, ZONES 2F AND 2G

Zones 2F and 2G materials are fine granular materials recuperated from the required overburden excavations and used for upstream and downstream backfilling.

5.10.3.1 ZONE 2F MATERIAL

Zone 2F material is used in the downstream backfills as a semi-pervious material to favour seepage evacuation and to avoid migration of fines particles of the foundation. It is generally used to fill all depressions to achieve the specified cross-sections and it is also used as part of the finger drains. This material is compatible with the in-situ materials.

Zone 2F material is a uniform fine to medium sand with a maximum fines content of 5% and a maximum particle size of 20 mm. Gradation requirements of the 2F material are presented in Table 5-3.

Table 5-3 Zone 2F Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
20	100
2.5	92 – 100
1.25	80 – 100
0.315	0 – 100
0.080	0 – 5

5.10.3.2 ZONE 2G MATERIAL

Zone 2G material is used in the upstream backfilling to fill the existing depressions and to achieve the required cross-sections. This material is compatible with the in-situ soils.

Zone 2G material is a uniform fine to medium sand with a maximum fines content of 20% and a maximum particle size of 20 mm.

Gradation requirements of the 2G material are presented in Table 5-4.

Table 5-4 Zone 2G Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
20	100
2.5	92 – 100
1.25	80 – 100
0.315	0 – 100
0.080	0 – 20

5.10.4 FILTER MATERIAL, ZONE 2A

Gradation of the Zone 2A material was established according to gradation distribution of the foundation materials and the Zone 2F material in accordance with the filter criteria defined by Terzaghi and refined by Sherard et al. (1989) and Kenney et al. (1985 and 1986) [Ref. 35]. Three series of criteria are complementary and for each parameter, the most severe value was used.

The Terzaghi filter criteria are presented in Table 5-5.

Table 5-5 : Terzaghi Filter Criteria

Filter Criteria
$\frac{D_{15\max}\text{filter}}{D_{85\min}\text{base}} \leq 5$
Permeability Criteria
$\frac{D_{15\min}\text{filter}}{D_{15\max}\text{base}} \geq 5$

Where:

$D_{15\max}\text{filter}$ is the diameter corresponding to the 15% percentage passing of particles, obtained from the coarse limit of the filter material's gradation curve.

$D_{85\min}\text{base}$ is the diameter corresponding to the 85% percentage passing of particles, obtained from the fine limit of the foundation or Zone 2F material's gradation curve.

$D_{15\min}\text{filter}$ is the diameter corresponding to the 15% percentage passing of particles, obtained from the fine limit of the filter material's gradation curve.

$D_{15\max}\text{base}$ is the diameter corresponding to the 15% percentage passing of particles, obtained from the fine limit of the foundation or Zone 2F material's gradation curve.

SHERARD ET AL. CRITERIA

Filter design has evolved over the years but the criteria as published by Sherard and Dunnigan (1989) are still widely applied and are appropriate.

For particle retention, the Sherard et al. criteria are presented in Table 5-6

The fine contents of the foundation soils are measured on the gradation curve regraded to particles smaller than 5 mm.

Table 5-6 : Filter Design – Particle Retention

Base Soil Description	
Fine Contents (A) (< 0.08 mm)	**Filter Criteria**
A < 15%	$D_{15\max}\text{filter} \leq 4\ to\ 5 \times D_{85\min}\text{base}$
15 % < A < 39 %	$D_{15\max}\text{filter} \leq \left(\frac{40-A}{40-15}\right)[(4 \times D_{85\min}\text{base}) - 0.7\ mm] + 0.7$
40 % < A < 85 %	$D_{15\max}\text{filter} \leq 0.7\ mm$
A > 85%	$D_{15\max}\text{filter} \leq 9 \times D_{85\min}\text{base}$

KENNEY ET LAU CRITERIA

Gradation of the filter material shall be well graded, without lack or excess of any fraction, to avoid internal instability of the filter.

Criteria is verified as,

H > F

Where,

H = F4D – FD, percentage of particles passing the equivalent diameter to 4D minus the percentage of particles passing the diameter D;

F = percentage of particles passing the diameter D and

D = Chosen diameter in the gradation curve for criteria verification.

COMPLEMENTARY CRITERIA

Others criteria are also used to assure the internal stability of the filters.

In order to avoid segregation of the materials, the maximum size of the filter is established at 80 mm.

The fine contents of the filter shall be less than 5% to assure self healing of the cracks in the filter zone.

The gradation curve of the filter material shall be relatively parallel to the gradation curve of the protected material.

Zone 2A filter material is a sand and gravel mixture used to avoid migration of fines particles of the zone 2F material and to allow rapid drawdown of the piezometric level and drainage of the seepage downstream of the North Spur.

Zone 2A filter material shall comply with the gradation requirements presented in Table 5-7.

Table 5-7 Zone 2A Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
80	100
20	70 – 100
10	57 – 100
5	43 – 95
2.5	33 – 81
1.25	23 – 67
0.315	4 – 30
0.080	0 – 5

5.10.5 COARSE GRANULAR MATERIAL, ZONES 2C AND 2E

The Zone 2C and 2E materials are constituted of a mixture of sand, gravel and cobbles and are used in the upstream slope of the spur as a transition between the till and the rockfill materials and in the downstream slope as transition between the filter and the rockfill materials. The fine content is less than 5% and the maximum particle size is 300 mm.

Zone 2E material is a placed material below water.

Gradation requirements of the 2C and 2E materials are presented in Table 5-8.

Table 5-8 Zone 2C and 2E Materials – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
300	100
80	60 – 100
20	38 – 85
5	23 – 70
1.25	11 – 50
0.315	0 – 20
0.080	0 – 5

5.10.6 ROCK MATERIALS

5.10.6.1 DRAINAGE MATERIAL, ZONE 3A

The material is used in the downstream area as a transition between the filter and the rockfill materials and in the construction of the finger drains in order to assure sufficient drainage of the filter materials. The 3A material is a crushed stone with a maximum particle size of 80 mm and with a maximum fine content of 5%.

Gradation requirements of the 3A material are presented in Table 5-9.

Table 5-9 Zone 3A Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
80	100
40	65 – 100
20	30 – 75
10	12 – 50
5	0 – 30
1.25	0 – 15
0.315	0 – 10
0.080	0 – 5

5.10.6.2 ROCKFILL MATERIAL, ZONES 3C, 3E AND 3D

Material of zones 3C and 3D are random rockfill materials used in embankment construction in the upstream and downstream areas of the spur. Zone 3C material is used as transition between coarser materials of zone 3D and zone 4 materials and finer materials of zone 2C and 3A materials and as erosion protection in the upper part of the downstream area of the North Spur.

Zone 3C and 3D materials act as bedding for riprap material (zone 4 class 1 and class 3 and zone 4 class 2 respectively).

The 3C and 3E zone materials shall be have a maximum size particle of 450 mm. 3E material is a dumped material under water and a placed material above water where it is used for erosion protection.

The maximum size of the random rockfill 3D material is equal to the thickness of the layer (900 mm). The zone 3D material is used as erosion protection in the lower part of the downstream area and as an external shell in the upstream area of the North Spur.

5.10.6.3 RIPRAP MATERIAL, ZONE 4

The rip rap material is used as a protection of the upstream and downstream slopes of the spur against the effect of ice and waves.

Riprap nominal size (D), defined as $D = (a \times b \times c)^{1/3}$, where a, b and c are length, width and thickness respectively, is presented in Table 5-10.

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Table 5-10 Zone 4 Material – Nominal Size (mm)

Nominal Size (D)	Zone 4 Class 1	Zone 4 Class 2	Zone 4 Class 3
Minimum, D_{min}	450	400	400
Median, D_{50}	600	550	500
Maximum, D_{max}	750	650	600

Sizing calculation of the riprap material and thickness of the protection is presented in Section 5.16.

5.10.6.4 ROCKFILL TYPE 1 AND 2 MATERIALS

Rockfill type 1 and 2 materials are used as erosion protection in the Kettle Lakes discharge channel.

Characteristics of the rockfill protection are presented in Table 5-11.

Table 5-11 : Rockfill Type 1 and 2 – Characteristics

Channel Slope %	Rockfill				
	Type	Dimension mm			
		D_{min}	D_{max}	D_{50}	Thickness
3	1	100	250	150	300
7	2	200	400	300	600

5.10.6.5 SELECTED GRANULAR "B" AND MAINTENANCE GRADE No 3 MATERIALS

Selected granular "B" and maintenance grade No 3 materials are used as granular base and grade surfacing materials respectively in the permanent access roads and they can be either crushed stone or crushed granular material.

Maintenance grade No 3 material is also used as a grade surfacing material in all the platforms and permanent roads in the upstream and downstream slopes of the North Spur.

Selected granular "B" material has a maximum particle size of 50 mm and shall comply with the gradation requirements presented in Table 5-12.

Table 5-12 Selected Granular "B" Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
50	100
25	50 – 100
5	20 – 55
1.25	10 – 35
0.315	5 – 20
0.080	2 – 8 (from Rockfill Source)
	2 – 6 (from Borrow Area)

Maintenance grade No 3 material has a maximum particle size of 25 mm and shall comply with the gradation requirements presented in Table 5-13.

Table 5-13 Maintenance Grade No 3 Material – Gradation Requirements

Sieve Size (mm)	% Passing (weight)
25	100
20	90 – 100
10	55 – 80
5	35 – 60
1.25	15 – 35
0.315	9 – 20
0.080	6 – 10

5.10.7 GEOTEXTILE

In order to avoid particle migration, a non-woven geotextile is used in the slopes in the areas where rockfill embankment is to be placed against the acceptable foundation and in the instrumentation trench.

Physical properties of the specified geotextiles 300 g/m^2 and 530 g/m^2 are presented in Table 5-14 and Table 5-15.

Table 5-14: Physical Properties, Geotextile 300 g/m^2

Physical Property	Value
Thickness	2.0 mm
Tensile strength CD (ONGC 148.1 No.7.3)	1,050 N
Tensile strength MD (ONGC 148.1 No.7.3)	1,050 N
Elongation CD (ONGC 148.1 No.7.3)	45 – 105 %
Elongation MD (ONGC 148.1 No.7.3)	45 – 105 %
Tear strength CD (ONGC 4.2 No.12.2)	460 N
Tear strength MD (ONGC 4.2 No.12.2)	460 N
Bursting strength (ONGC 4.2 No.11.1)	2,910 kPa
Permeability (ONGC 148.1 No.4)	0.19 cm/s
Apparent opening size (ONGC 148.1 No.10)	145 µm

Table 5-15: Physical Properties, Geotextile 530g/m^2

Physical Property	Value
Thickness	3.5 mm
Tensile strength CD (ONGC 148.1 No.7.3)	1,450 N
Tensile strength MD (ONGC 148.1 No.7.3)	1,450 N
Elongation CD (ONGC 148.1 No.7.3)	70 – 110 %
Elongation MD (ONGC 148.1 No.7.3)	70 – 110 %
Tear strength CD (ONGC 4.2 No.12.2)	600 N
Tear strength MD (ONGC 4.2 No.12.2)	600 N
Bursting strength (ONGC 4.2 No.11.1)	3,500 kPa
Permeability (ONGC 148.1 No.4)	0.19 cm/s
Apparent opening size (ONGC 148.1 No.10)	40 – 110 µm

5.10.8 COMPATIBILITY OF MATERIALS

The Terzaghi filter criteria described previously is used to assess the compatibility of different materials. Compliance with these criteria assures the stability of the materials in their boundaries, avoiding migration of fine particles from base material to coarser embankment layers.

5.11 STABILITY

The objective of the stability analyses is to evaluate the Factor of Safety (FS) of the slopes of the Spur during and after the stabilization works under different loading conditions and to compare these results with the minimum established values.

Results are presented for selected upstream and downstream cross-sections.

5.11.1 CALCULATION METHODS AND HYPOTHESIS

Two dimension analyses have been performed with the SLOPE/W 2012 Version 8.12 (GeoStudio 2012) software and the FS have been obtained with the Morgenstern – Price method. Pore water pressure conditions have been modeled with either piezometric levels or R_u factor. A pseudo static approach was used to calculate the stability of the slopes for dynamic conditions.

5.11.2 LOADING CONDITIONS

The loading conditions studied and the required FS against sliding are presented in Table 5-16. These factors are in accordance with the Canadian Dam Association (CDA) guidelines and the design criteria established for the project.

Table 5-16: Stability Analyses – Loading Conditions and Factors of Safety

Loading Conditions		Factor of Safety	
Water retention structures		**U/S Slope**	**D/S Slope**
1	End of construction	1.3	1.3
2	Partial pool	1.3	N/A
3	Steady state at FSL	1.5	1.5
4	FSL with seismic loading	1.15	1.15
5	Rapid drawdown	1.3	N/A
6	Rapid drawdown with seismic loading	1.1	N/A
Non water retaining structures			
7	Temporary excavated slopes during construction	1.1	
8	Permanent excavated slopes	1.5	
9	Permanent slopes with seismic loading	1.15	
N/A: Not applicable, U/S: upstream, D/S: downstream			

5.11.2.1 END OF CONSTRUCTION

To take into account the induced pore pressures during construction, a conservative pore pressure ratio (R_u) of 0.3 in the till embankment and in the clayey foundation materials was used to perform stability analyses.

5.11.2.2 PARTIAL POOL

As previously described in this report, a first impoundment will take place after the river closure. At that time the reservoir level will rise to elevation 25 m. Stability analyses were performed on the upstream slope for this loading case which has been called “Partial Pool”.

5.11.2.3 STEADY STATE AT FSL

For operating conditions, the FSL was considered. The groundwater level in the upstream and downstream slopes is considered hydrostatic and equal to the level of the reservoir in the upstream side and to the water level on the downstream side during operations.

5.11.2.4 FSL WITH SEISMIC LOADING

The stability analyses in seismic conditions were performed using a pseudo static approach, this approach assumes that the foundation and embankment materials are not susceptible to vibration. A dynamic analysis, which is described in more detail in Section 5.12.2 of this report, was performed for the foundation materials.

As per the seismic hazard analysis performed in 2014 [Ref. 2], at the probability of 1/10,000, the expected PGA from natural earthquakes for the reference site condition (NEHRP B/C) at Muskrat Falls is 0.06g. A k seismic coefficient of 0.06 was retained for the analyses.

5.11.2.5 RAPID DRAWDOWN

Rapid drawdown has a negative effect on the stability of the upstream slope of embankment structures since the drawdown of the groundwater table inside the embankment and in the foundation is usually much slower than the reservoir drawdown which could induce higher hydraulic gradients in the embankment materials.

In the rockfill and granular embankments at the upstream slope, the velocity of the drawdown groundwater is considered equal to that of the reservoir due to the high permeability of the materials. Dissipation of the hydraulic pressure generated inside the till and foundation materials are slower compared with the rockfill and granular materials.

For the North Spur, stability analyses were performed for a rapid drawdown of the water level between the PMF reservoir level and the FSL. The rise of the reservoir from the FSL to the PMF level will last about 10 to 15 days and the drawdown of the reservoir back to FSL will last about 25 days after reaching the PMF level. This relatively short period of time should not create a hydrostatic condition in the till and foundation materials; however the analyses were performed for this conservative scenario and confirmed that this was the case.

5.11.2.6 RAPID DRAWDOWN WITH SEISMIC LOADING

Pseudo static method with the same seismic coefficient obtained from the seismic hazard assessment was used to perform the stability analyses during a rapid drawdown event.

5.11.2.7 TEMPORARY EXCAVATED SLOPES DURING CONSTRUCTION

Stability analyses were performed to take into account the different stages of construction activities, which included temporary excavation works. Actual construction methods presented by the Contractor shall comply with these conditions.

5.11.2.8 PERMANENT EXCAVATED SLOPES WITHOUT AND WITH SEISMIC LOADING

The stability analyses of the permanent excavated slopes, those which are above the embankment works on the upstream and downstream sides were performed together with the analyses of the upstream and downstream slopes.

Stability analyses were also performed for the permanent slopes of the Northwest cut-off trench.

5.11.3 MATERIAL PROPERTIES

The geotechnical properties of the materials considered in the stability analyses are presented in Table 5-17. Native soils properties have been obtained from the interpretation of the geotechnical and hydrogeological conditions of the North Spur and embankment material properties have been defined based on values obtained from previous projects and from literature.

Table 5-17: Material Properties

Native Soils					
Material		Unit weight (kN/m³)		Effective cohesion c' (kPa)	Effective friction angle ϕ' (º)
Zone	Description	Wet γ_h	Saturated γ_{sat}		
N/A	Upper Sand	19.0	-	0	35
N/A	Silty Clay	18.0	19.0	6	32
N/A	Silty Sand	18.0	19.0	0	32
N/A	Lower Clay	-	19.3	6	32
N/A	Slide Debris	19.0	20.0	0	20
N/A	Lower Aquifer	-	19.5	0	35
Embankment Materials					
1/1C	Compacted Till	22.0	22.7	0	37
2A	Compacted Granular	20.5	22.0	0	36
2C	Compacted Coarse Granular	20.5	22.0	0	36
2E	Placed, Coarse Granular	18.5	20.4	0	32
2F/2G	Compacted Fine to Medium Granular	18.5	20.4	0	33
3A	Compacted Stone	20.5	22.0	0	40
3C/3D	Compacted Rockfill	19.5	21.7	0	45
3E	Placed Rockfill	19.0	21.0	0	42
4	Riprap all classes	17.5	20.0	0	42

5.11.4 STABILITY ANALYSES

Stability analyses have been performed for different sections in the upstream and downstream areas of the North Spur. Results of these analyses are presented in the following sections.

5.11.4.1 UPSTREAM SLOPE

Stability analyses were performed for all loading conditions in the most critical cross-section (Typical Cross-Section 4, Drawing MFA-SN-CD-2810-CV-SE-0001-02). Results of the analyses are presented in Table 5-18 and Figure 5-6 to Figure 5-12.

Table 5-18 : Upstream Slope – Stability Analyses

Loading Conditions		Factor of Safety	
		Required	Calculated
1	Temporary excavated slopes during construction	1.1	1.1
2	End of construction	1.3	1.6
3	Partial pool	1.3	1.6
4	Steady state at FSL	1.5	1.8
5	Steady State at FSL with seismic loading	1.15	1.4
6	Rapid drawdown	1.3	1.3
7	Rapid drawdown with seismic loading	1.1	1.4
8	Permanent excavated slopes	1.5	> 1.8
9	Permanent slopes with seismic loading	1.15	> 1.4

Figure 5-6 : Upstream – Stability Analysis, Temporary Excavated Slopes

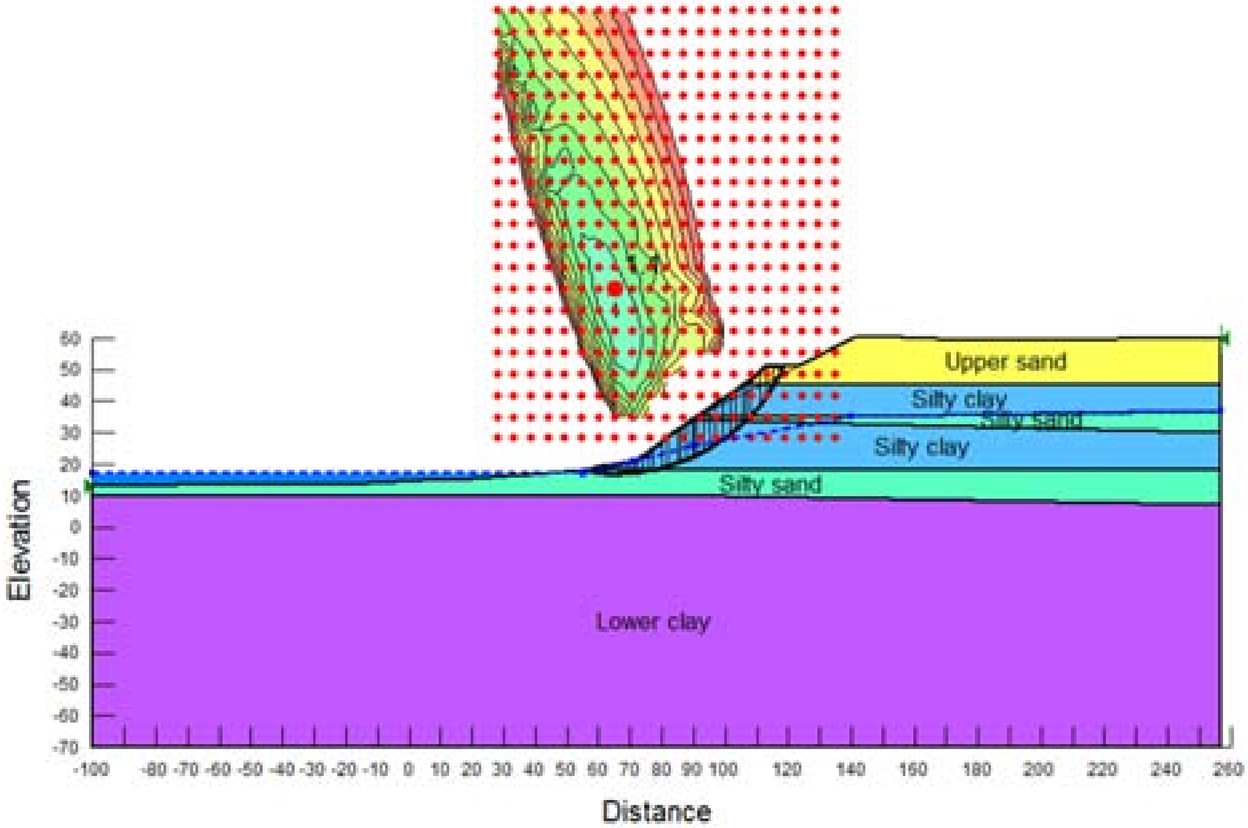

Figure 5-7 : Upstream – Stability Analysis, End of Construction

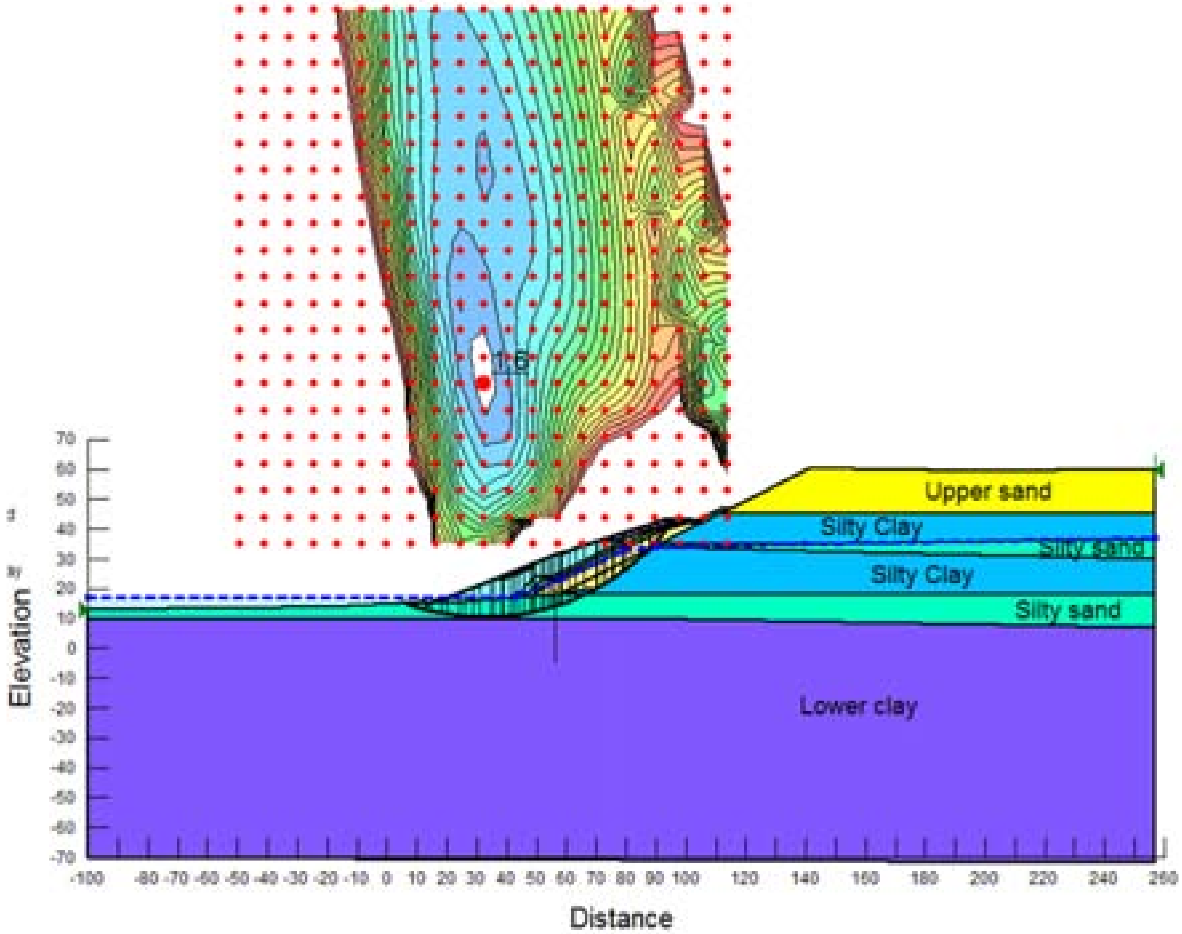

Figure 5-8 : Upstream – Stability Analysis, Partial Pool

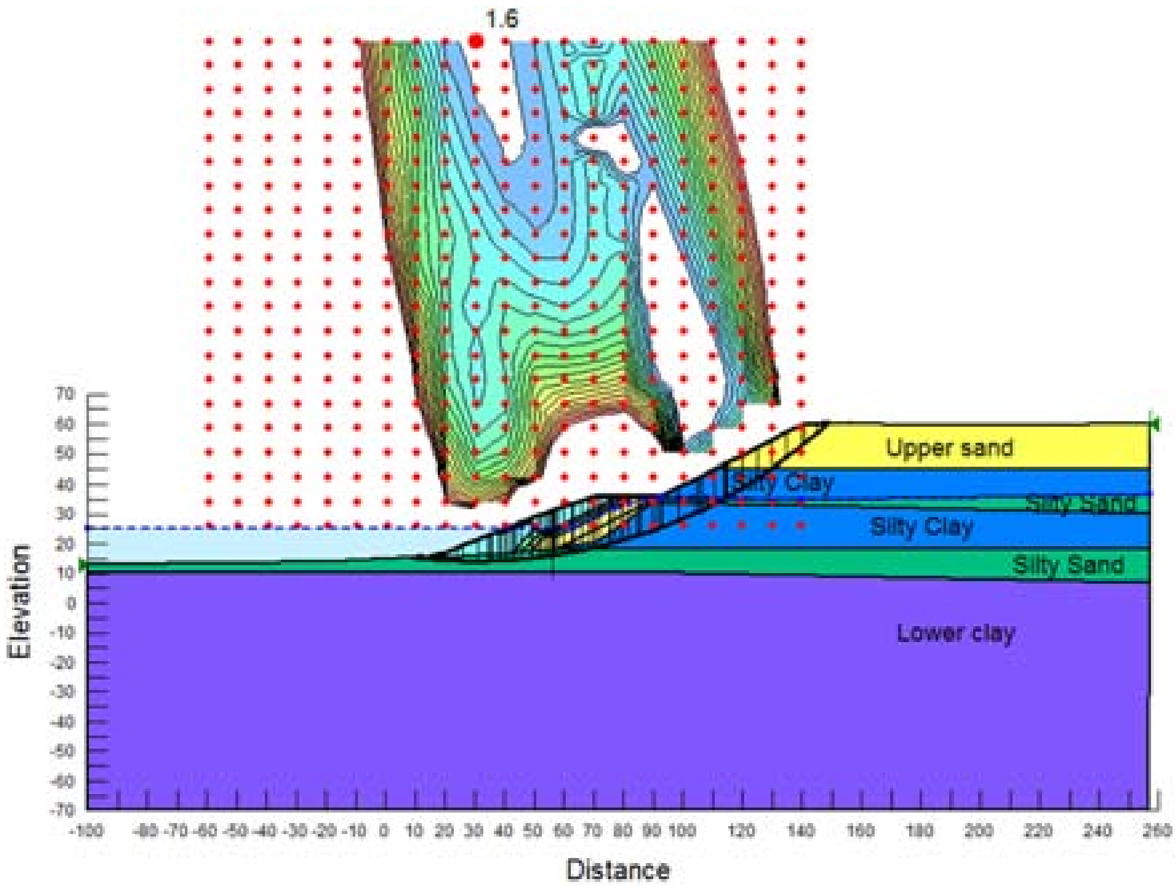

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Figure 5-9 : Upstream – Stability Analysis, Steady State at FSL

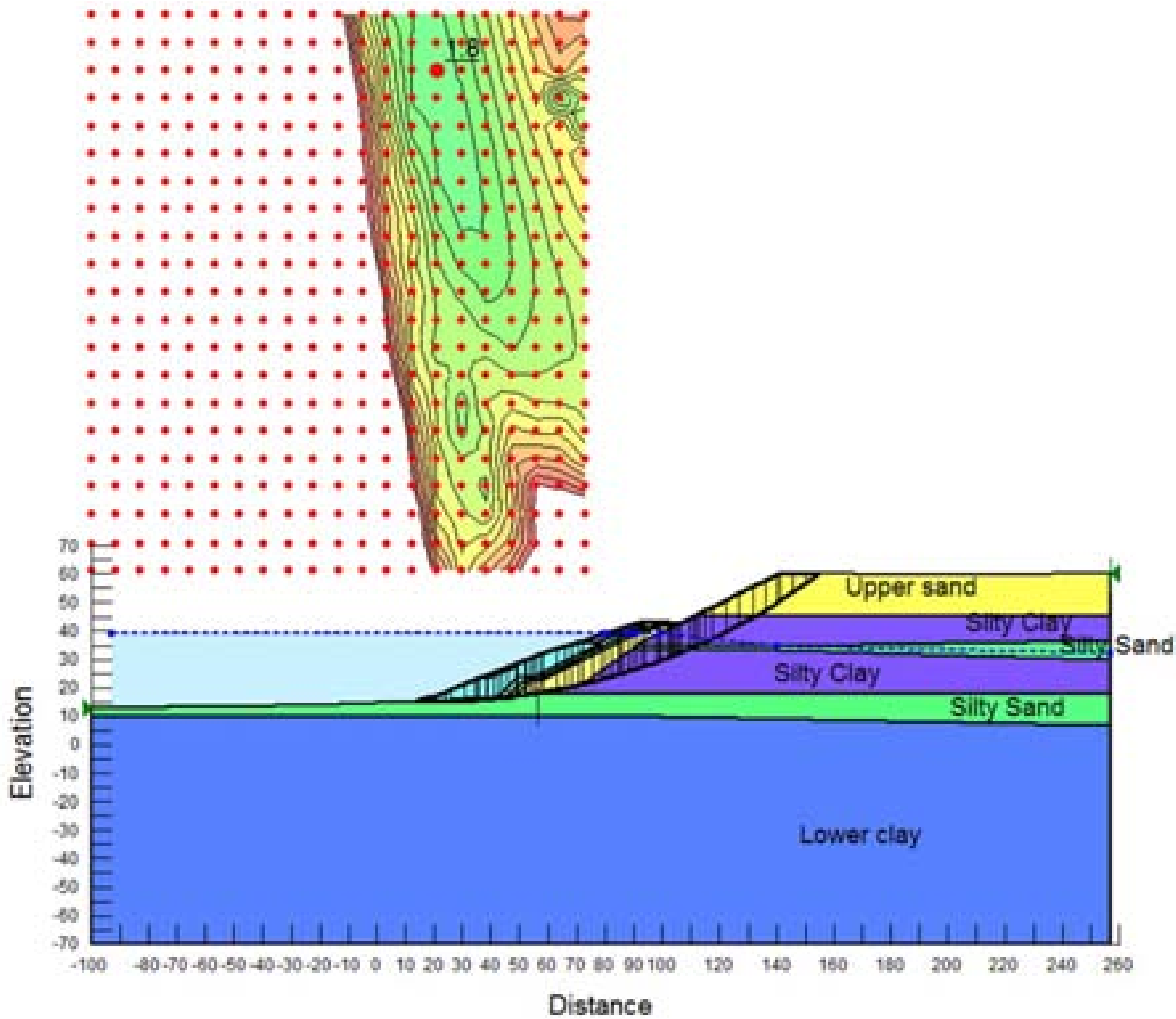


Figure 5-10 : Upstream – Stability Analysis, Steady State at FSL with Seismic Loading

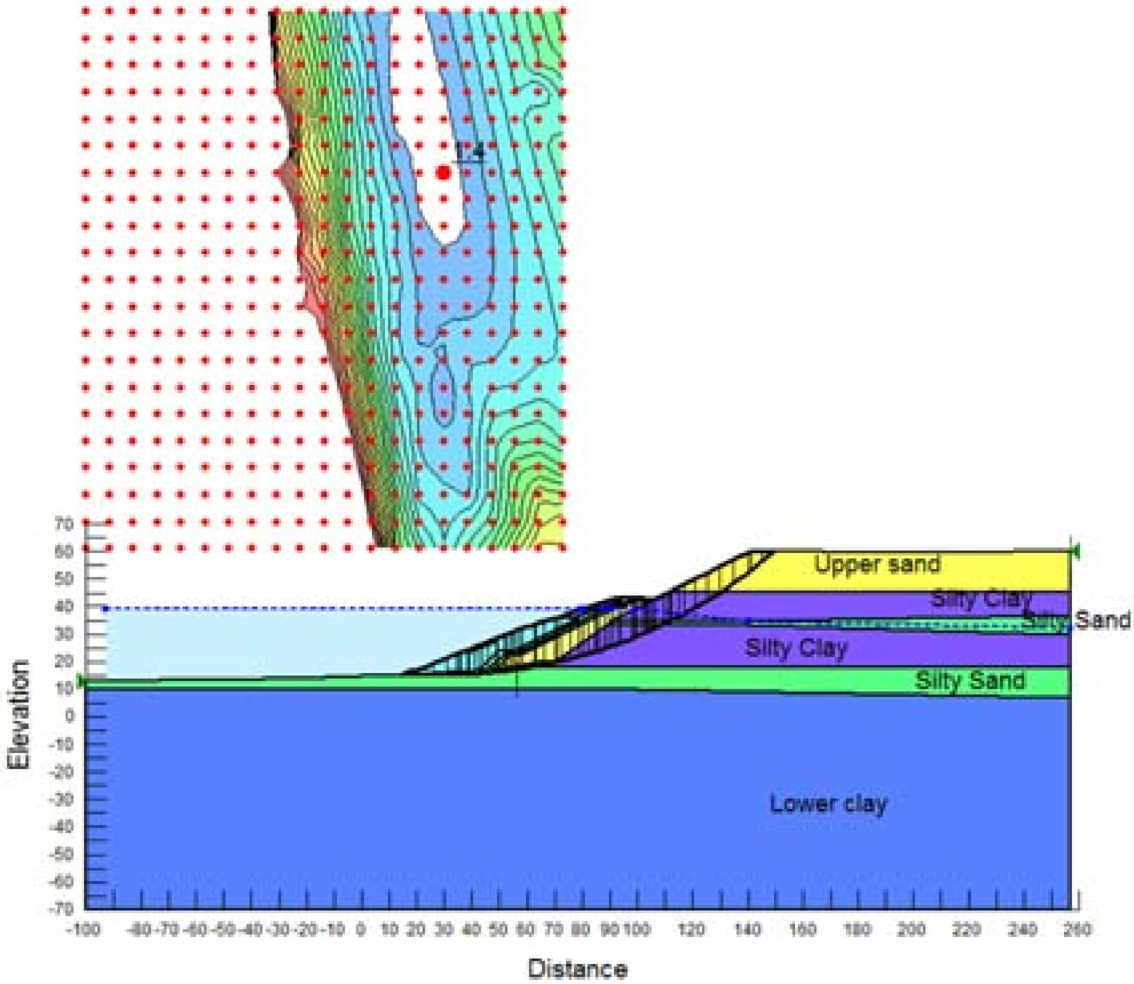


Figure 5-11 : Upstream – Stability Analysis, Rapid Drawdown

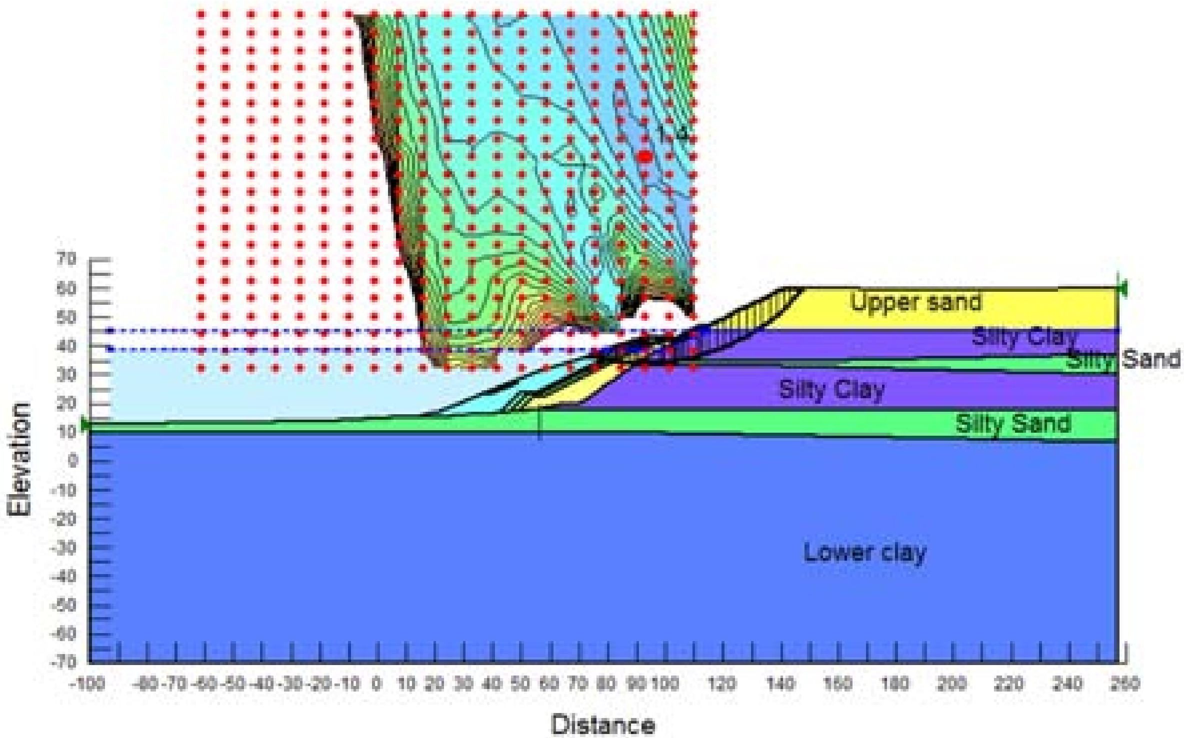
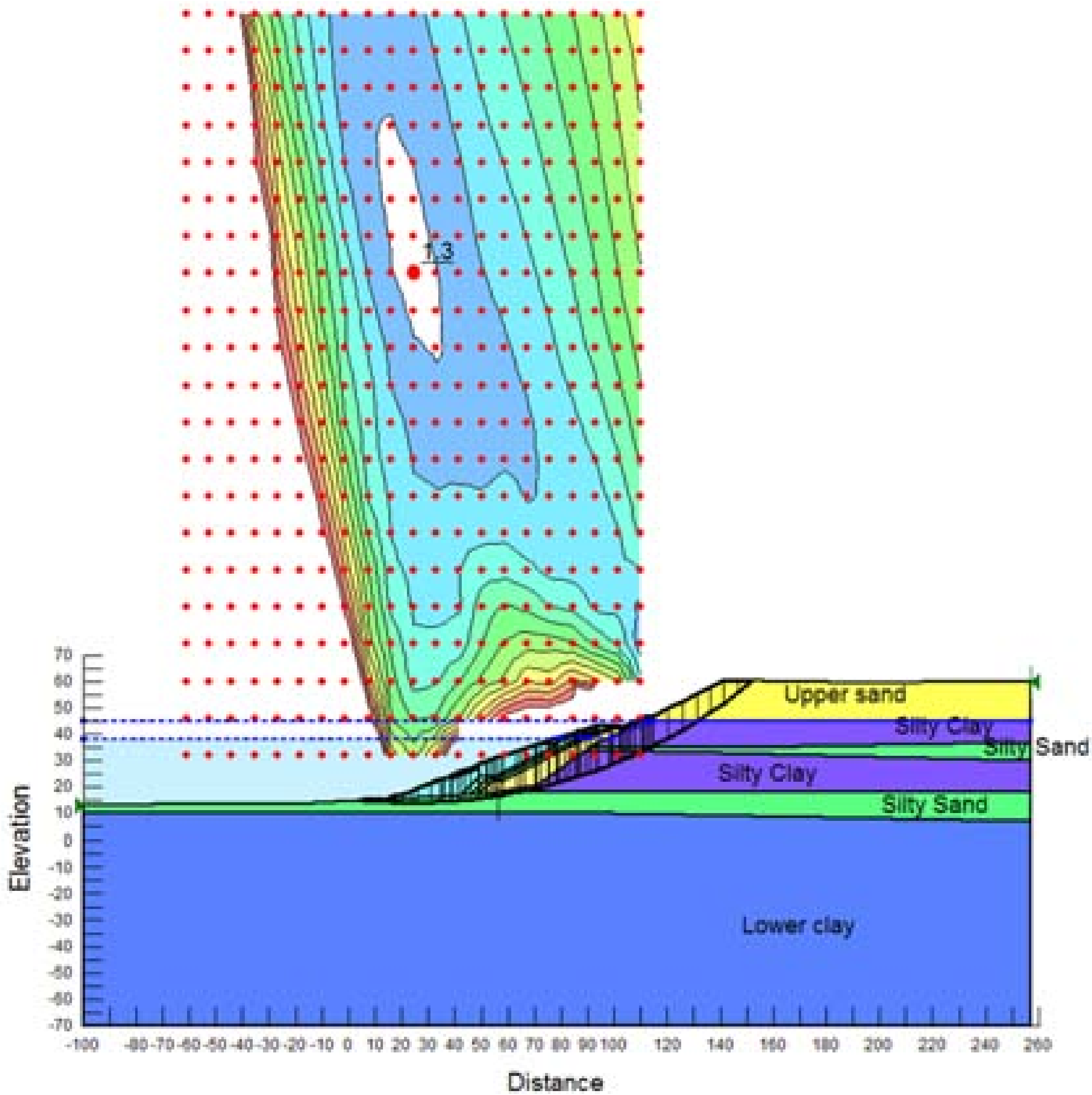


Figure 5-12 : Upstream – Stability Analysis, Rapid Drawdown with Seismic Loading



5.11.4.2 DOWNSTREAM SLOPE

Stability analyses were performed for all loading conditions in the most critical cross-section (Typical Cross-Section 13, Drawing MFA-SN-CD-2820-CV-SE-0001-03). Results of the analyses are presented in Table 5-19 and Figure 5-13 and Figure 5-14.

Table 5-19 : Downstream Slope – Stability Analyses

Loading Conditions		Factor of Safety	
		Required	Calculated
1	Steady state at FSL	1.5	1.5
2	FSL with seismic loading	1.15	1.3

Figure 5-13 : Downstream – Stability Analysis, Steady State at FSL

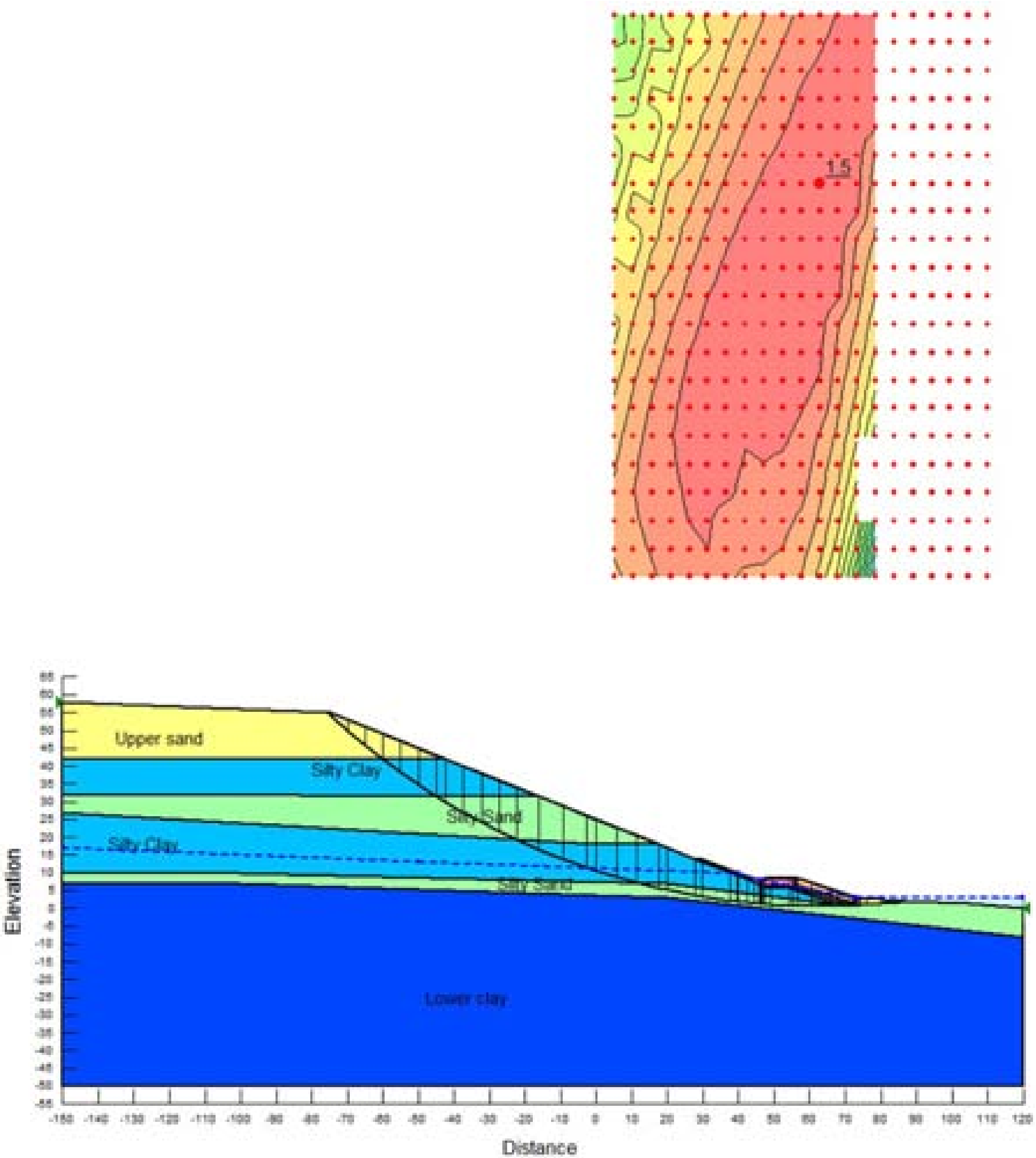
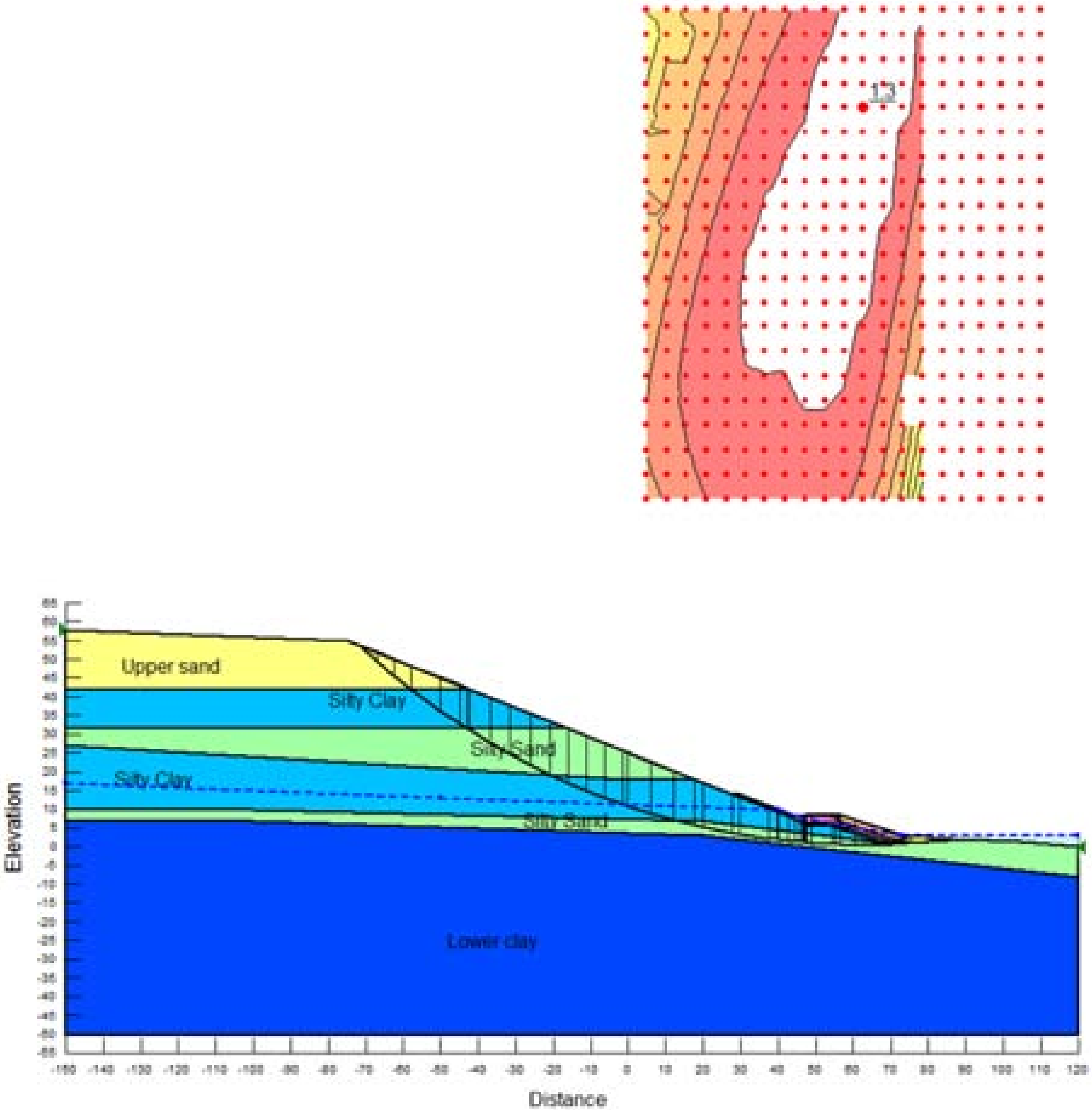


Figure 5-14 : Downstream – Stability Analysis, Steady State at FSL with Seismic Loading



5.11.4.3 RESULTS

The results of the stability analyses performed showed that the calculated FS are equal to or greater than the required values for all the loading conditions.

5.12 LIQUEFACTION POTENTIAL AND CYCLIC SOFTENING ASSESSMENT

5.12.1 PRELI MINARY ASSESSMENT

A preliminary assessment of liquefaction potential in the saturated cohesionless soils and cyclic softening in the clays and silts within the North Spur is presented in this section.

The method presented by Youd et al. (2001) [Ref. 36] was used for this assessment. This method compares a stress ratio, the Cyclic Stress Ratio (CSR) as a measure of seismic demand on a soil layer, against a resistance ratio, the Cyclic Resistance Ratio (CRR) as a measure of soil resistance against liquefaction.

According to Youd et al. (2001), the CRR may be evaluated using CPT, SPT, Becker penetration test (BPT) or shear wave velocity (V_s) measurements.

In this section available SPT values from the 1979 and 1998 investigation campaigns and CPT results from the 2013 investigation campaign were used to evaluate the soil liquefaction potential and cyclic softening.

As a conclusion of the earthquake hazard analysis carried out in 2014 [Ref.2], at the probability of 1/10,000, the expected peak ground acceleration (PGA) from natural earthquakes for the reference site condition (NEHRP B/C) at Muskrat Falls was defined approximately at 0.06g.

Cyclic Stress Ratio (CSR) is evaluated using seismic acceleration and in-situ stress conditions:

CSR = $\frac{\tau_{av}}{\sigma'_{v0}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right)r_d$

Where;

a_{max} : peak ground acceleration,

g : acceleration of gravity,

σ_{v0} , σ'_{v0} : total and effective vertical overburden stresses,

r_d : stress reduction coefficient:

$$r_d = \frac{1.0 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5}}{1.0 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.00121z^2}$$

Cyclic Resistance Ratio for magnitude 7.5 earthquakes ($CRR_{7.5}$) is evaluated using the following equation:

$$CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10.(N_1)_{60}+45]^2} - \frac{1}{200}$$

This equation is valid for values of $(N_1)_{60}$ <30. For $(N_1)_{60}$ ≥30 the clean granular material is considered too dense and non-liquefiable. $(N_1)_{60}$ is the SPT blow count normalized to an overburden pressure of approximately 100 kPa ($C_N = (P_a/\sigma'_{vo})^{0.5}$, C_N ≤ 1.7) and a hammer energy ratio or hammer efficiency of 60%.

The equation is developed for clean sand condition. To account for the mitigative effect of fine content the following equation is used:

$(N_1)_{60cs} = \alpha + \beta.\ (N_1)_{60}$

where $(N_1)_{60cs}$ is the equivalent clean sand SPT value, and coefficients α and β are related to fine content (FC) and determined from the following relationships:

If FC≤5% $\begin{cases} \alpha = 0 \\ \beta = 1.0 \end{cases}$, If 5%<FC<35% $\begin{cases} \alpha = \exp[1.76 - \left(\frac{190}{FC^2}\right)] \\ \beta = [0.99 + \left(\frac{FC^{1.5}}{1000}\right)] \end{cases}$,

If FC≥35% $\begin{cases} \alpha = 5.0 \\ \beta = 1.2 \end{cases}$,

Youd et al. (2001) simplified method has not been verified with case history data for depths greater than 15 m. However the method has been used in the absence of better tool to evaluate the liquefaction resistance.

Factor of safety is defined as:

F.S. = $\frac{CRR_{7.5}}{CSR}$

A summary of factors of safety for liquefaction, at the location of 12 boreholes scattered in the North Spur area is presented in Figure 5-15.

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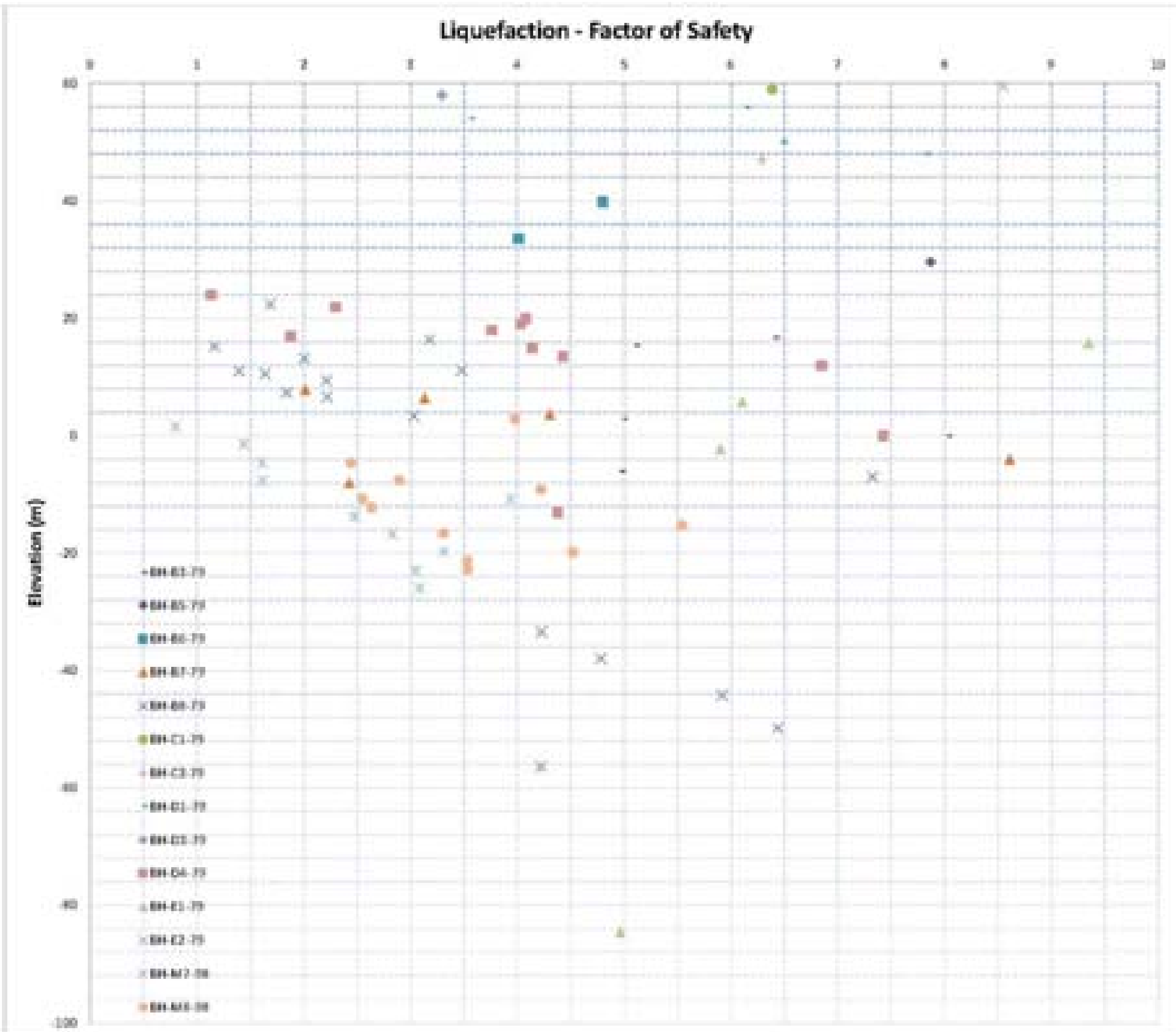
Figure 5-15 : Liquefaction – Factors of Safety

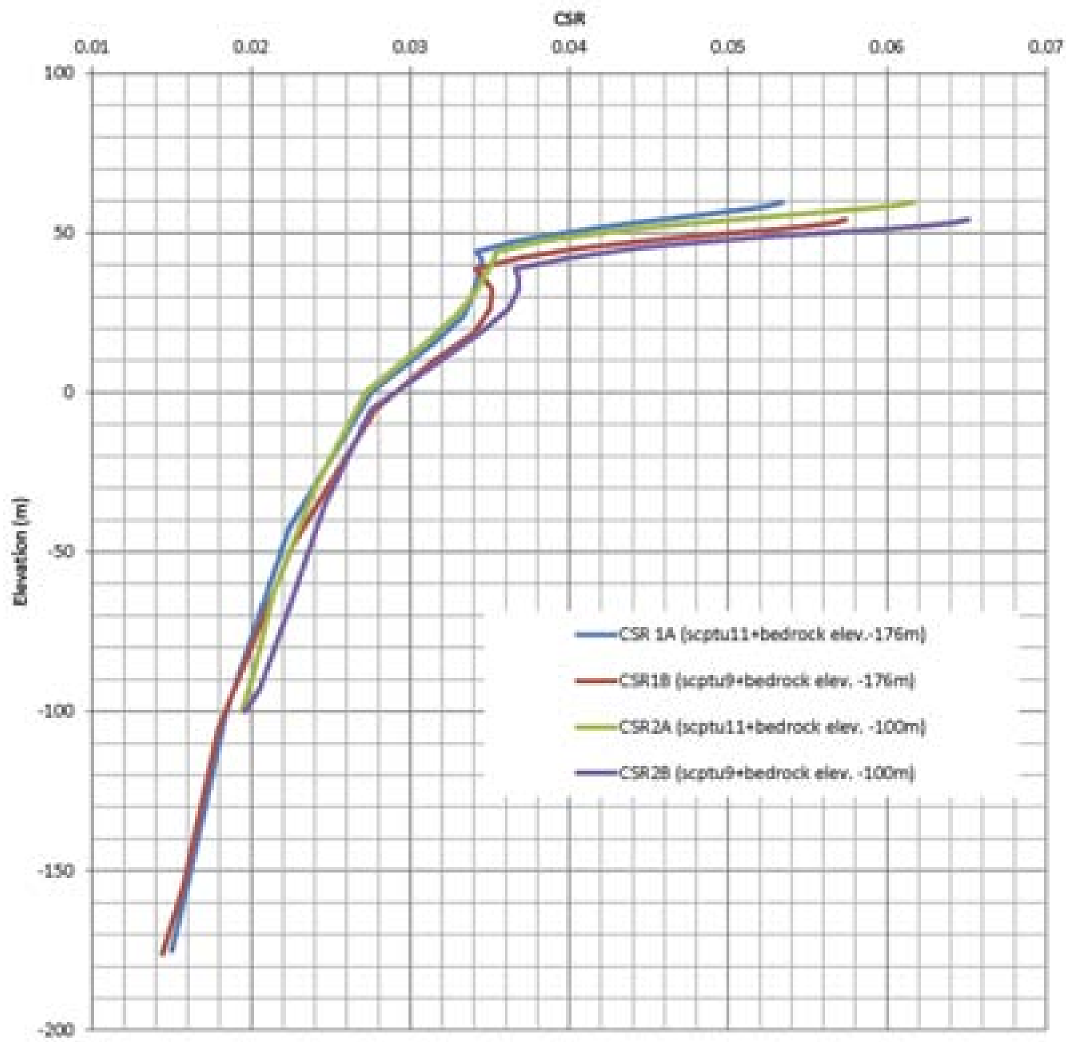


Except for one value in borehole BH-M7-98, the calculated factor of safety is greater than 1.

A second preliminary assessment was performed with the measured shear wave velocities in SCPTU9 and SCPTU11. This analysis was performed before issuing of the updated earthquake hazard analysis. Thus, the retained expected peak ground acceleration for a NEHRP A site was 0.09g and 0.11g for a NEHRP D site.

Four different soil profiles where prepared coming for the 2 SCPTU profiles with the bedrock at 2 different elevations (-100 and -176 m), CSR 1A, CSR1B, CSR2A and CSR2B. Twenty time histories were used to calculate the Cyclic Stress Ratio (CSR) using ProShake software. Results of the calculated CSRs are shown in Figure 5-16.

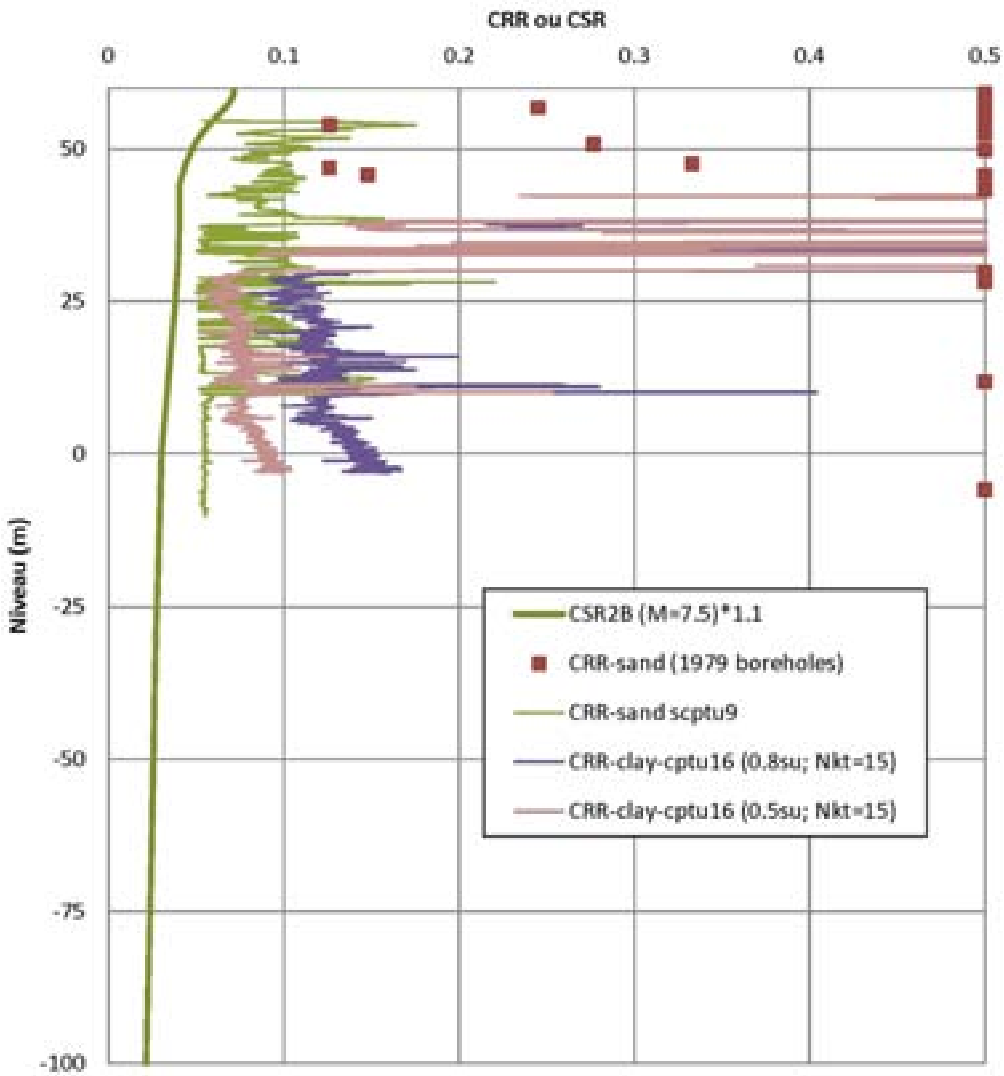
Figure 5-16 : CSR 1A, 1B, 2A and 2B Profiles



CSR 2B profile was considered as the most unfavorable case and was retained for comparison against CRR values.

In order to evaluate the liquefaction potential of cohesionless soils, CRR profiles were prepared from SPT values obtained during 1979 investigations and from CPT9. Cyclic softening of the clayey and silty soils were evaluated empirically estimating CRR based on the undrained shear strength profile obtained from CPT16 and factored by 0.8 and 0.5. Results of the CRR evaluation for liquefaction and cyclic softening against the retained CSR profile (CSR2B) is shown in Figure 5-17.

Figure 5-17 : CRR and CSR Profiles



The results show that the safety factor against the potential liquefaction of cohesionless soils and cyclic softening of clays and silts is adequate.

5.12.2 DYNA MIC STUDY

Among the recommendations presented by the independent technical reviewers of the North Spur stabilization works was a recommendation that additional assessment of the liquefaction potential of granular soils and cycling softening of cohesive soils should be performed. To comply with these recommendations, a workshop which included internal and external resources specialized in these topics was convened in December 2013. Professors Izzat Idriss (Eng., M.Sc., Ph.D., Professor Emeritus of Civil Engineering University of California) and Serge Leroueil (Eng., M.Sc., Ph.D., Professor of Civil Engineering Laval University) participated in this workshop.

During the workshop, the results of the preliminary assessment were presented and the recommendations of the external reviewers were analyzed. The workshop was oriented to provide a “first phase” response to the reviewer’s recommendations in order to determine if, in fact, the situation warrants more laboratory and analytical work.

Professor Leroueil pointed out in his report that the slope stability analyses performed presented a satisfactory factor of safety; the salinity profile changes in depth accordingly with the properties of the deposit; the grain size analyses performed showed that there is not clean silt material in the stratigraphy and there is no measured plasticity index lower than 5 to 7%.

In his presentation at the end of the workshop, Professor Idriss concluded that if the stabilization measures of the North Spur are built as currently designed, they will have a satisfactory performance during future earthquakes.

It was also concluded by Professor Idriss that an update of the earthquake hazard analysis, including deaggregation of the PSHA results, should be performed by Dr.

Atkinson. With this update, calculations in the potential for triggering liquefaction in the sand layers and the potential for cyclic softening in the Upper Sensitive Clay Unit should be redone. In addition, a dynamic nonlinear analysis should be conducted to assess the pattern of deformations that may be induced by the postulated earthquake ground motions.

At the end of the workshop, it was concluded that the planned stabilization measures will counteract the increase of the driving forces and/or the reduction of the resistance through liquefaction due to pore pressure increase of cyclic softening of the soils due to the dynamic loadings created during potential earthquakes.

Conclusions from the workshop stated that there was agreement between the external and internal experts that there was no issue with the preliminary assessment of the liquefaction potential and cyclic softening of the soils constituting the North Spur performed by the design team , but recommended an update of the Seismic Hazard Study as well as the performance of an additional dynamic nonlinear analysis to give further confidence in the findings.

Following the conclusions and recommendations of this workshop, a Dynamic Study [Ref. 37] was carried out to finalize the assessment of the potential liquefaction of granular soils and cyclic softening of cohesionless soils.

Different methods were used during the assessment, including empirical methods, one dimensional (1D), two dimensional (2D) equivalent-linear methods and two dimensional (2D) non-linear methods.

Representative sections of the upstream and downstream areas of the North Spur, regarding geometry of the stabilization works and physical, mechanical and hydrogeological properties of the soils were retained to carry out this dynamic study.

Seismic parameters used in the assessment were obtained from the updated Earthquake Hazard Analysis and seismic scenarios were selected based on deaggregation of the hazard (Dr. Atkinson, 2014). Representative input motions for the Muskrat Falls site for a 1/10 000 annual probability were selected and treated based on the updated Uniform Hazard Spectrum (UHS).

The selected sections were subjected to the selected input motions using 1D and 2D dynamic response modelling. The results were compared to the resistance of the different soil layers calculated and obtained from the available geotechnical and geological data.

Finally, the resistance to liquefaction of the granular soils and to cyclic softening of the cohesionless soils was evaluated with the methods proposed by Idriss and Boulanger (2008).

Results of these analyses indicated no liquefaction potential of the granular materials, nor cyclic softening potential of cohesionless soils.

In conclusion, based on the findings of this Dynamic Study, the North Spur integrity is not expected to be affected by the occurrence of the design seismic event.

5.13 PROGRESSIVE FAILURE

In the process of assessing the stability of the North Spur before and after stabilization works stability analyses were performed using the Limit Equilibrium Method (LEM) in the appropriate locations.

There is no approved and accepted method to estimate in advance a safety factor before a progressive failure landslide occurs. The cases presented in the literature are always related with a landslide that has already occurred and so all cases presented are examined through a back calculation analysis

For the cases where the failure of the slope progresses from uphill to downhill the LEM type of analysis is not applicable.

The Lower Churchill River valley shows numerous landslides scarps which suggest they are of a flowslide type.

To address the potential for occurrence of progressive failure landslides (both downward and upward) a specialized study was performed [Ref. 38]

Three type of analysis including seepage analysis, LEM stability analysis and stress distribution analysis assuming an elastic plastic behaviour were performed.

The possibility of occurrence of uphill or downhill progressive landslide was studied and the results show that with the mitigation measures taken, the stability of the North Spur regarding such events is adequate and that a progressive failure landslide will not occur. Details of the analysis and results can be found in the referenced document.

5.14 DEFORMATIONS AND SETTLEMENTS

No lateral deformation and no significant settlement is expected at the top of the North Spur during construction and during and after reservoir impoundment. In general the water table is expected to lower in the North Spur. Lowering the water table will increase the effective vertical stress in the lower layers of soils and this condition could generate some consolidation in cohesive soil layers. However the settlement is expected to be very small and with no consequences.

Minor settlement, with no significant effect on the structures is expected to occur in the upstream and downstream slopes beneath the embankments, due to the weight of the backfill built over the natural soil constituted of medium dense silty sand layers and normally consolidated silty clay deposits.

5.15 FREEBOARD

Freeboard represents the height between the crest of a retaining structure and the maximum operational level of the reservoir. The purpose of the freeboard is to prevent overtopping of the crest of the structure by waves and, in cold regions, guarantee a minimum cover of till core or blanket to limit effects of freeze and thaw cycles. In the case of the North Spur structure, no risk of overtopping exists because the top of the Spur is about 15 m higher than the maximum water level. However, freeboard calculations were undertaken to determine the maximum height required for protection of the slopes both during the initial impoundment and following final impoundment.

The freeboard is the sum of the reservoir setup and the wave run-up on the slope of a structure or embankment due to the wind effect. Calculation takes into account the wind velocity on a given return period blowing in the same direction over the surface of the reservoir at the maximum water level of operation. The run-up is calculated as a function of the design wave height and the inclination and roughness of the slope material.

Calculation of the freeboard has been the subject of a separate study and the results are presented in a separate technical document [Ref. 39]. Different approaches have been used to evaluate the wave characteristics. The values of the hydraulic freeboards calculated for the upstream and downstream slopes of the North Spur, for the construction period and during operation are summarized in Table 5-20.

Table 5-20: Upstream and Downstream Freeboard

During construction					
		Maximum water level	Wind recurrence	Calculated freeboard	Minimum el. of protection
Upstream	Headpond[1]	24.0	1:20y	2.5	26.5
Downstream	Normal	6.0	1:20y	1.7	7.7
During operation					
Upstream	FSL[2]	39.0	1:1,000y	3.9	42.9
	PMF[3]	45.1	1:2y	1.8	46.9
Downstream	Normal	3.9	1:1,000y	2.4	6.3
	PMF	12.5	1:2y	0.9	13.4

[1] Headpond corresponds to the temporary level of reservoir during diversion, set-up to prevent formation of frazil
[2] FSL: Full Supply Level
[3] PMF: Probable Maximum Flood

Based on the calculated values of freeboard, the riprap protection of the upstream embankment has been set at elevation 43.0 m for the FSL and the highest protection level of the embankment has been set at elevation 47.0 m for the PMF.

The top of the till blanket is set at elevation 42.4 m which would be for a short period of time during a PMF event (2 days). There would be negligible seepage through the granular and rockfill material placed and this would have no effect the North Spur.

For the downstream shore, the level of the lowest berm, which also takes into account the protection against ice effects, has been set at elevation 8.5 m.

It should be noted that during construction of the stabilization works at the North Spur, and before diversion of the Churchill River through the spillway channel, the downstream water level of the river may rise during winter up to elevation potentially as high as 20.0 m due to an ice jam which occurs every winter the river downstream of the lower falls, with the possible consequence of flooding the lower part of the stabilisation works under construction. This ice jam will not occur following initial impoundment to 25 m.

At the end of each construction season before diversion of the Churchill River, the downstream stabilization works which have been started will have to reach the minimum elevation 10 m. Ice observation made during winter 2011-2012 indicate that the ice in the jam accumulated away from the shore of the river and that only a normal ice cover was observed at the contact with the shore line. Variation of ice level during winter could cause damage at the shore line. Ice sheet of up to 1 m thick was observed at shore line in May-June 2012. If construction is not completed before winter, there is potential for some damage to occur at the shoreline and to any work not completed, and remedial works may be required. Any such damage during the construction phase would not have any long term impact on stability.

5.16 SLOPE PROTECTION

Slopes of the North Spur are protected against the erosive action of waves and ice with a riprap material (zone 4, various classes). Calculations of riprap size and thickness due to wind generated waves are covered in a separate study and the results are presented in the relevant technical report [Ref. 40].

The riprap was first designed according to the usually accepted standards to resist wind generated waves and considerations for ice conditions were complementary.

The U.S. Army Corps of Engineer and the Société d'Énergie de la Baie James methods were used to design the riprap protections against wind generated waves.

A separate study was carried out with regards to riprap design based on ice conditions [Ref. 41].

The values of the required riprap size and thickness for the various slopes are summarized in Table 5-21.

A literature review was carried out to identify the state-of-the-art for the design of structures to resist the potential damaging effects of ice. No rigorous procedure has been established and much is based on empirical approaches and past experience.

As result of this review, the following considerations were made.

Relatively gentle slopes (1V:3H) are preferred for limiting the damage due to ice run-up.

Well graded riprap, placed to create a smooth surface with no protruding stones will better resist plucking action.

Stones sizes resisting plucking should be such that the thickness of the riprap layer normally twice the D_{50} size or 2.5 times D_{min} is at least equivalent to the anticipated thermal ice cover thickness.

The riprap dimensions selected for construction exceed the minimum required dimensions.

The upstream slope is protected against wave action between elevations 23.0 m and 27.0 m for the temporary headpond and between elevations 35.8 m and 43.0 m during normal operation (FSL).

Riprap also extends below the wave attack level to prevent its destabilization by erosion of its toe.

An additional slope protection, consisting of a placed rockfill, is provided above elevation 43.0 m up to elevation 47.0 m to prevent damage during the PMF event.

Along the downstream shore, riprap will be installed to protect the embankment against wave and ice action below elevation 8.5 m. The toe of the downstream slope is protected against damage resulting from scour with a rockfill embankment.

Table 5-21: Riprap Size and Thickness

Shore and elevation	Slope	Riprap size (mm)						Riprap Thickness (m)	
		Required			Specified			Required	Specified
		D_{min}	D_{50}	D_{max}	D_{min}	D_{50}	D_{max}		
During construction									
Upstream 23.0 m 27.0 m	2.0H:1V	370	500	630	400	550	650	0.8	0.9
During operation									
Upstream 35.8 m to 43.0 m	3H:1V	440	600	750	450	600	750	1.0	1.2
	3.5H:1V	420	560	700				0.9	
	4H:1V	400	540	680				0.9	
Downstream below 8.5 m	3H:1V	350	470	600	400	500	600	1.2	1.2

The design of the riprap bedding layer is based on filter criteria as presented below:

$$\frac{D_{15}\,(riprap)}{D_{85}\,(bedding)} \leq 4$$

$$D_{85}\,(bedding) \geq 50 \text{ mm}$$

$$\frac{D_{85}\,(bedding)}{D_{max}\,(riprap)} > 0.2$$

The upper portion of the downstream area up to elevation 40 m in the south area of the North Spur, up to elevation 14.0 m in the north part of the North Spur and the exit of the Kettle Lakes channel will be protected against erosion with rockfill materials.

All excavated slopes (upstream, downstream and Kettle Lake channel) not covered above the elevations previously mentioned and the slope of the northwest trench will be protected against erosion by hydroseeding.

5.17 INSTRUMENTATION AND MONITORING

5.17.1 GENERAL

Typical retaining structure instrumentation and monitoring system is designed to allow monitoring the structure behaviour during construction, reservoir impoundment and operation. Verifying different parameters evolution and validate the design of the structure is the main purpose of the instrumentation and monitoring system.

It should be noted that the instrumentation is not to be used as a replacement for regular visual inspections, but in combination with visual inspections and a surveillance program, so as to continually evaluate the performance of the retaining structures. Data obtained from inspections and instrumentation will have to be documented, compiled and analyzed in a timely manner for any indications of unusual structure performance, as part of a surveillance program.

Retaining embankment structures, like dams and dikes, are typically instrumented to measure pore pressure, seepage and deformations in the embankment and the foundation.

Instrumentation designated for the North Spur includes:

- Standpipe and vibrating wire piezometers;
- Inclinometers;
- Flow weirs.

Most instruments allow remote parameter monitoring by transducers and cables which are connected to the main data acquisition system located in the Pumpwell Control Building in the crest of the North Spur. The data acquisition system will be then remotely connected to the Nalcor server via the existing optical fiber converter.

Location of the instruments shown on the drawings is indicative of the required locations which have been established in order to meet the objectives of each instrument. Due to actual conditions, relocation of the instruments could be necessary in order to avoid construction issues or to ensure obtaining the most relevant information.

Individual surge protection has to be installed for each instrument to address lightning and shielding issues.

The Contractor is responsible for the supply of all instruments and materials, the installation of the instruments and the construction of related appurtenant structures.

5.17.2 PIEZOMET ERS

The North Spur has already been instrumented with many piezometers in the past to measure pore pressures. Many of these instruments are still functioning and have been upgraded during the years.

The proposed design incorporates installation of new piezometers to extend the pore pressure measurements in the North Spur, increasing the piezometers network in the various layers of soil deposits (intermediate stratified drift, the lower clay layer and the deep granular aquifer). Piezometers will also be installed in the downstream side of both upstream and north-west cut-off walls.

All new piezometers are standpipe type, equipped with a vibrating wire piezometer and transducer allowing automatic data acquisition and recording. In the existing standpipe piezometers, vibrating wire piezometers are to be installed, cabled and connected to the data acquisition system.

Vibrating wire piezometers are specified as model 4500S or 4500B as manufactured by Geokon Inc. or equivalent, main characteristics are as follows:

- Measure range 0-350 kPa;
- ± 0.1 percent full scale accuracy;
- 0.025 percent full scale resolution.

Standpipe piezometers are of type CP1 from Roctest or equivalent.

5.17.3 INCLINOMETERS

Stability of the slopes will be monitored by inclinometers. Three of these will be installed in the downstream area to allow measurements inside the remoulded colluvium soils. Three others will be installed in the crest of the Spur, 2 in the downstream area and 1 on the upstream side.

The inclinometers are specified as model 6300 vibrating wire in-place inclinometer as manufactured by Geokon Inc. or equivalent. The inclinometer consists of a string of vibrating wire tilt sensors mounted on lengths of stainless steel tubing which are linked together by universal joints.

Main characteristics of the inclinometers are as follows:

- Standard range ±10º;
- ± 0.1 percent full scale accuracy;
- 0.05 mm/m (±10 arc second) resolution.

5.17.4 FLOW WEIRS

In the downstream side of dam or dike sites, seepage is usually measured with weirs. At the North Spur the layout of the stabilization works does not allow to direct or concentrate the seepage at one or some particular points on the downstream side to permit the measurements. No surface runoff is expected because the surface will be covered with rockfill and underground seepage will be discharged through the finger drains, evacuated through the riprap and diffused across the rockfill at the submerged toe of the slope. Measurement of seepage is not possible under these conditions.

Flow measurement will be performed at the exit of the Kettle Lakes area. An automatic flowmeter will be installed inside an aluminized corrugated steel pipe

1 200 mm diameter downstream of the kettle lake. The flowmeter is specified as model Stingray 2.0 as manufactured by Greyline or equivalent.

Flow coming from the lower aquifer will be measured, through a V-Notch weir installed in each collector pipe outlet of the relief wells and recorded with a vibrating wire weir monitor device.

The vibrating wire weir monitor specified is model 4675 as manufactured by Geokon Inc. or equivalent. The water level monitoring system uses a vibrating wire force transducer to provide a highly stable and sensitive means of monitoring water levels. The cylinder and force transducer are contained within a slotted PVC pipe housing.

5.17.5 CABLE S TRENCH

All instruments are connected via cable to the data acquisition system and these cables are routed in trench excavated in the overburden or built in the embankments. Details of the trench are shown on the technical drawings. The trench has been designed to protect the cables against embankment placement and compaction, all cables are covered with a zone 2F material. The zone 2F material for the cables installed in the upstream and downstream areas of the Spur is separated from the embankment by successive layers of compatible materials.

5.18 3D STRATIGRAPHIC MODEL

As part of the final design, a 3D CATIA model of the North Spur has been developed. This model helped to confirm some elements of the design, to build a stratigraphic and hydrogeological 3D model, to obtain plan views and cross-sections of the North Spur and to prepare the bill of quantities of the works. However, CATIA functionality presents limitations and the model cannot represent all the design elements. Quantities included in the bill of quantities have been calculated with this model and verified by other methods.

The natural ground contours in the area of the project were based on a site specific LiDAR survey completed in 2006 and this served as a source file for the model. Ground level adjustments were made with survey of boreholes and geophysical surveys calibrated with boreholes were used to determine the bedrock surface. The stratigraphic model of the North Spur was then calibrated with the information from the investigations. The model was first developed with the available information in 2011 and then updated with the new data coming from the 2013 field investigation.

5.19 CURRENT ENGINEERING TECHNICAL REVIEWS

Two Advisory Board meetings were held in April and October 2013 which included review of the North Spur area and the related stabilization works to be performed [Refs. 26 and 27]. As part of their conclusions, the Board was supportive of the effort and attention being placed on the various issues associated with the design, supported the passive approach for the operation but recommended a long-term contingency provision to be made to allow future pumping from the existing wells, should it be required. The Board also agreed with recommendations to perform additional specialized studies (Dynamic and Hydrogeological).

In September 2013 an independent review of the design of the North Spur stabilization works was undertaken by the Independent Engineer (represented by MWH Americas Inc.) [Ref.28]. As part of this review, a site visit was held in September 24 to 26. The independent reviewer commented that “the stabilization works have been designed in accordance with currently accepted geotechnical design practices and effectively stabilizes the North Spur when the reservoir is impounded”.

The Independent Engineer reviewed various aspects of the geotechnical designs and planned works and concluded that all the works have been carried out to a high standard.

The group concluded among others, that the planned North Spur remediation measures were appropriate to stabilize the slopes, arrest natural mass wasting and to control seepage and piezometric pressures after impoundment of the reservoir.

It was also mentioned that the planned long term monitoring program is an important component of the works which will ensure safe operation of the reservoir and detect on a timely basis an anomalous behaviour that may affect safe operations.

Another independent review of the North Spur stabilization works was performed by Hatch Ltd in September 2013 (Cold Eye Review of Design and Technical Specifications, North Spur Stabilization Works) [Ref. 29]. The review team stated that the design approach is considered to meet the general requirements for the satisfactory and long term stability of the Spur and concluded that the basis of the design is in general robust and all the main elements for the most part have been considered. Aligned with the other external reviewers, additional specialized studies were recommended and also the presence of designer personnel on site and concurred with the planned Observational Method to be implemented during the stabilization works.

6 CONSTRUCTION

6.1 CONSTRUCTION STAGES

The overall works schedule was been defined by Nalcor. Wherever technical constraints imposed a specific schedule or construction stage, these were established by the Engineer.

All embankment works below elevation 28.0 m on the upstream side and below elevation 10.0 m in the downstream area, stabilization works in the area of the Kettle Lakes area and both cut-off walls shall be finished by the time of the winter impoundment of the reservoir.

Instrument installation shall also be finished before winter impoundment, with the exception of the installation of the vibrating wire piezometers, inclinometers, related cables and automatic data acquisition systems

Construction of the relief wells in the lower aquifer, if required, will be done after the first or final reservoir impoundment.

6.2 CLEARING, GRUBBING AND STRIPPING

Clearing operation requirements are presented in the Technical Specifications and they also must be performed in accordance with the requirements of Exhibit 6 – Environmental and Regulatory Compliance Requirements of the Contract.

Clearing limits in the area of the North Spur are shown on the drawings. In the quarries and borrow areas no clearing limits are shown, clearing shall be limited to the required exploitation area. Clearing operations in these areas shall be rigorously controlled.

In the North Spur area, grubbing and stripping are considered a separated operation of excavations when the excavated materials shall be reused in the embankments.

If excavations are needed north of the area of the northwest cut-off wall to produce embankment materials which cannot be obtained from the planned excavations, the approval of the Engineer must be obtained prior to excavation and related clearing.

6.3 DEWATERING AND WATER MANAGEMENT

Dewatering and water management are considered critical during the construction works. A complex hydrogeological regime is present in the North Spur. All dewatering and water management shall be planned in conjunction with each of the construction operations (clearing, grubbing and stripping, excavations, embankment construction, etc.).

The objective of the dewatering and water management operations is to permit the execution of all works related to excavation, foundation preparation, embankment and other construction operation in dry conditions, to assure the stability of excavated slopes and to prevent erosion materials during the work period.

The works include the dewatering of all Contractor work areas; all streams in the area of the structures in order to perform the excavations, foundation preparation and embankment construction in dry conditions; all areas inside the footprint of the structures; till borrow areas in order to control the moisture content of the exploited materials to comply with the requirement of the technical specifications and granular borrow areas, if exploitation below the water table is required.

Contractor is responsible for design, supply, operation, maintenance, relocation and removal of the dewatering and water management systems.

Ponding is not allowed in the work areas. Surface runoff water, seepage infiltration and artesian water flow shall be intercepted and evacuated to the exterior of the work areas. Contractor shall build and maintain trenches and sumps as necessary. In the areas where the artesian flow cannot be stopped, drains shall be built to capture and evacuate the water outside the work area. Once the embankment has reached an elevation higher than the water pressure, the pump can be stopped and the drain shall be grouted.

As much as possible, the trenches shall be built outside of excavation zones in order to capture and collect the water before entering the excavations.

The dewatering and water management system shall be capable of lowering and keeping the water table below the foundation elevation to allow the work in dry conditions and to permit the embankment placement over an adequate dried foundation.

The dewatering and water management system removal shall be done as embankment works progress or at the end of the structure construction, and must always be done in a proper manner to allow placement and construction of the embankments in dry conditions.

6.4 EXCAVATIONS

The limits of the excavations in the footprint of the structures are established on site as per the lines, elevations and profiles specified on the drawings, observed conditions on site after clearing and as works advance.

During excavation work above elevation 50.0 m, the materials coming from grubbing and stripping operations shall be separated from the excavated materials and disposed of in the spoil disposal areas and excavated materials complying with the technical requirements shall be reused (Zone 2F and 2G materials).

In excavations to be performed below elevation 50.0 m, grubbing and stripping and excavation activities can be done simultaneously.

Provision must be made during the overburden excavations above elevation 50.0 m to avoid loss of material due to inappropriate grubbing and stripping operations.

This division of the excavations has been adopted based on assumption made on the nature of the material which will be reusable and that which will not to be reusable in the embankment works.

Special provision was also made for the excavation of the finger drains, where the conditions are expected to be more difficult than normal excavation.

In all cases, the Contractor shall assure that the slope excavations are stable. Slopes shown in the drawings are considered stable in normal conditions and they were established for measurement purposes only. Gentler slopes might be necessary in particular conditions such the presence of water table.

6.5 FOUNDATION PREPARATION

6.5.1 OVERBURDEN FOUNDATION

Inside the structures footprint, all ditches and trenches shall be backfilled as required in the specification. Depending on site conditions, some areas of the acceptable foundation might be compacted; provision has been made to allow for these works.

All grubbed and stripped acceptable foundation that cannot be backfilled before winter time, shall be protected against freezing as per the requirement of the specifications.

No disturbance of acceptable foundation shall be allowed.

6.5.2 BEDROCK FOUNDATION

Foundation preparation requirements for bedrock shall be established on site by the Engineer depending on site conditions. The foundation is to be inspected by the Engineer and foundation preparation work required will be transmitted to the Contractor.

6.6 FOUNDATION APPROVAL

All foundations, either in overburden or rock shall be approved by the Engineer before placement of any embankment. Foundation approvals for overburden and rock shall be made separately. The Contractor shall inform the Engineer, in a timely manner, the foundations for which the Contractor intends to request approval.

Any approved foundation that is not backfilled, but has signs of disturbance shall be discarded and new approval shall be requested.

6.7 EMBANKMENT CONSTRUCTION

6.7.1 MATERIAL SOURCES

The field investigations performed allowed assessment of the quality and quantities of the different construction materials needed for the stabilization works.

Detail on the selection criteria for the sources of material, the methodology and assessment of the quantities (proven and potential), definitions of the different variables and the exploration works performed in each source are presented in more detail in the engineering report “Construction Materials – North Bank, Borrow Areas and Quarries” [Ref. 42].

Embankment material characteristics, details and results of the analyses and tests performed for each designated source are presented in this document.

The borrow areas and rockfill sources designated for the embankment works of the North Spur stabilization works are presented in Table 6-1

Table 6-1: Material Sources

Material Source	
Till Material	T-4B
Filter and Granular Materials	GR-2, GR-3
Fill Material	Required Excavations
Rock Material	Q-1, Q-6

Borrow area T-4B, which is located about 7.0 km from the North Spur and 1.5 km from the Trans Labrador Highway (TLH), is designated as the source of till material. Access to this deposit will require the construction of an access road from the TLH.

Borrow areas GR-2 and GR-3 are designated as sources of filter and granular materials. Borrow area GR-2 is located about 12 km west of the North Spur and 0.4 km from the TLH. Borrow Area GR-3 is located about 10.5 km west of the North Spur and 0.1 km from the TLH. Construction of access roads will be necessary between these deposits and the TLH.

Construction of the Northwest cut-off wall and stabilization works in the upstream and downstream areas require the excavation of sandy materials in the upper part of the North Spur. The excavated material complying with the requirements of Zone 2F and Zone 2G material will be used in the embankment works. Priority in using the Zone 2F material will be accorded to the downstream area. More silty materials (Zone 2G material) will be used in the upstream area.

The required rock materials for the North Spur stabilization works will be sourced from the north bank quarries. Quarries Q-1, Q-2, Q-3, Q-6 and Q-7 were identified during the investigations on the north bank. Quarries Q-1 and Q-6 were designated as sources of rock materials for the stabilization works. Q-1 is about 3.0 km away from the North Spur and 0.4 km from the TLH. Q-6 is about 4.5 km away from the North Spur and 0.1 km from the TLH. Construction of access roads will be necessary between the quarries and the TLH.

Other sources may be approved for exploitation if testing shows compliance of the material to the specifications.

During works, crossing of the TLH has to be planned and prepared in accordance with the Department of Transportation and Works requirements.

6.7.2 MATERIAL BALANCE

Each source of materials has been designated with respect to the nature of the materials (till, fill and granular, rock material) and the proven and potential volumes.

Proven and potential volumes of each source of materials have been evaluated by interpretation of the investigation available data, topography of the source and environmental constraints. The results are presented in a separate report on construction materials [Ref. 42].

Table 6-2 shows the construction material balance comparing available quantities from the sources (proven and potential) and required quantities for the construction works.

Table 6-2: Construction Materials – Balance

Source	Proven Quantity (m^3)	Potential Quantity (m^3)	Required Quantity (m^3)	Balance	
				Required vs. Proven (m^3)	Required vs. Potential (m^3)
TILL MATERIAL					
T-4B	520,000	1,245,000	265,000	255,000	980,000
FILL AND GRANULAR MATERIALS					
GR-2	209,000	361,000	191,000	30,000	187,000
GR-3	12,000	17,000			
Required Excavations	260,000	-	260,000	Volume excavations can be adjusted if required	
ROCK MATERIAL					
Q-1	-	> 1,000,000	551,000	49,000	> 449,000
Q-6	600,000	> 600,000			> 58,000

6.7.3 SOU RCES EXPLOITATION

6.7.3.1 GENERAL

Till and granular borrow areas will be exploited at a minimum distance of 35 m from permanent roads and 70 m from quarries. Exploitation of borrow areas or quarries shall be done at more than 100 m from a water course, unless a special authorization is approved. Depth of borrow area exploitation is limited by groundwater level. The preparation and exploitation of the borrow areas shall be carried out in stages. The areas of the borrow area to be exploited are first to be surveyed by Contractor and then approved by Engineer.

6.7.3.2 TILL BORROW AREAS EXPLOITATION

As a general rule, when the natural moisture content of the till is above 2% of the optimal value obtained during compaction tests, measures are required in order to obtain a suitable till material.

In the borrow areas, these measures consist mainly in draining the surface runoff far from the exploitation faces, excavating the borrow area in vertical faces of at least 3.0 m high to allow acceptable drainage and to obtain a more homogeneous till material.

It is also recommended to prepare wide work areas to allow selection of dry material while drainage is being carried out in other areas of the deposit. The production of the till in the borrow area shall be adapted to the weather conditions at the time of the exploitation and material placement. Thus, till with low moisture content shall be exploited and placed in wet periods and till with high moisture content shall be exploited and placed in dry periods.

For internal drainage of till deposit wet areas, deep trenches should be excavated prior starting of the exploitation of the borrow pit. This measure also helps the drainage of these areas during snow melting in the spring season prior to exploitation.

It is essential during exploitation of the borrow area that all measures are necessary measures are undertaken to avoid the increase of the natural moisture content of the till.

Generally, all cobbles larger than 300 mm and all boulders are discarded directly in the borrow area. If such unsuitable material is hauled to the site construction, it is to be removed from the till prior to the placement and compaction of the embankment.

6.7.3.3 GRANULAR BORROW AREAS EXPLOITATION

In general, granular borrow areas are to be exploited using optimized heights and inclination of faces in order to obtain the best possible mix of the different layers of the exploited material. Usually, surface drainage of this type of borrow area is not a problem.

The granular borrow areas are to be exploited systematically and rationally in order to extract all the available material. Thus, the material and the embankment characteristics are to be considered while choosing the exploitation areas inside the borrow pit.

6.7.3.4 REQUIRED EXCAVATIONS EXPLOITATION

The construction of the northwest cut-off wall calls for a trench excavation between the natural ground at the crest of the Spur and elevation 50 m. Based on the interpretation of the available geotechnical data of this area, it was concluded that the material from these excavations will be reusable in the embankments. This also applied to material from the regrading of the upstream and downstream slopes. , For the excavations in the area of the Kettle Lakes, where all the excavated materials will be reused, including that below elevation 50 m.

No double handling of the materials is expected. After excavation, the materials will be transported and placed directly in the required areas. Fine contents and natural moisture content will be assessed on site in order to convey the materials to the right place.

If necessary, more volume of Zone 2F and 2G materials will be obtained from an area north of the northwest trench, which has been identified as the source for additional material, should it be required.

6.7.3.5 ROCK MATERIAL SOURCES EXPLOITATION

6.7.3.5.1 QUARRY EXPLOITATION

The quarry will be exploited in a rational manner in order to prioritize the exploitation of the suitable sectors containing the quality rock material which complies with the requirements.

Geological characteristics of the source are to be taken into account while exploiting the quarry in order to obtain embankment materials complying with the technical requirements, especially those related with specified sizes.

6.7.4 EMBANKMENT PLACEMENT AND COMPACTION

The embankment placement sequences have been established in a logical order to comply with the specified requirements regarding the theoretical boundaries between zone materials. All different material zones shall be supported against each other before compaction. All necessary measures shall be taken by the Contractor to avoid segregation and concentration of uniform size particles.

At the outer part of the slopes the material is to be placed wider than required to permit compaction of all materials inside the design limits and then the surplus material is excavated back to the neat lines.

The specified equipment and compaction methods are those commonly used in the construction of retaining structures.

Placement and compaction requirements for each zone material are presented in Table 6-3.

Table 6-3: Embankment Materials – Placement and Compaction Requirements

Material Zone	Layer Maximum Thickness (mm)	Compaction Equipment	Number of passes	Minimum Compaction Degree
1	450 150	Padfoot Compactor Special Compactors	5 -	98% MDD as obtained in Standard Proctor test
1C	450 150	Padfoot Compactor Special Compactors	5 -	98% MDD as obtained in Standard Proctor test
2A	450 150	10 t Vibratory Roller Special Compactors	3 -	93% MDD as obtained in Vibratory Table test
2C	450	10 t Vibratory Roller	3	93% MDD as obtained in Vibratory Table test
2F	450	10 t Vibratory Roller	3	93% MDD as obtained in Standard Proctor test
2G	450	10 t Vibratory Roller	3	93% MDD as obtained in Standard Proctor test
3A	450	10 t Vibratory Roller	3	-
3C	450	10 t Vibratory Roller	3	-
3D	900	10 t Vibratory Roller	4	-
2E, 3E, 4, Rockfill Type 1 and 2	Specified on drawings	None – Only placed	-	-
Selected Granular “B”	Specified on drawings	10 t Vibratory Roller	3	-
Maint. Grade No 3	Specified on drawings	10 t Vibratory Roller	4	97% MDD as obtained in Modified Proctor test

(1) While using special compactors, layer thickness shall be 150 mm or less;
(2) MDD : Maximum Dry Density

6.7.4.1 ZONE 1 MATERIAL

The width of the zone 1 material is sufficient to allow the use of the vibratory compactors. During placement and compaction of till materials, free drainage shall be assured for the acceptable foundation on the slopes away from the structure and abutments to avoid moisture content build-up. The surface of the embankment shall be temporary crowned or sloped with transverse grades varying between 3 and 5% and with longitudinal grades steeper or equal to 2%.

As specified, the compactness of the material shall be at least equal to 98% of the maximum dry density as determined by the ASTM D698 Standard.

6.7.4.2 ZONE 2A, 2C AND 2E MATERIALS

Zones 2A and 2C materials shall be abundantly moistened before compaction.

The Zone 2E material shall be placed as per the minimum thickness shown in the drawings, as measured from the actual underlying foundation.

6.7.4.3 COMPACTED ROCKFILL – ZONES 3C AND 3D MATERIAL

Control on the maximum size of these materials is particularly important as this parameter has a strong influence on the ability to achieve the required compaction. The Contractor shall prepare a work method that eliminates the presence of oversized blocks. This work method shall include appropriate drilling and blasting patterns to obtain the required material, selection of the equipment and selection works at the source.

6.7.4.4 RIPRAP – ZONE 4 MATERIAL

The riprap shall be placed in accordance with the thickness shown on the drawings, measured perpendicular to the actual surface. These materials shall not be dumped directly on the surface to be covered; they shall be dumped over a horizontal surface and then placed.

The structure of this zone shall be as dense as possible, avoiding the incorporation of undersized particles. The material shall be pressed with the bucket of the backhoe in order to assure a secure interlock and stable position of the riprap blocks.

Compliance with the gradation requirements shall be assured by preparing the benchmark stones for each class of riprap and by choosing the appropriate method to select only conforming stones before they are transported to the embankment.

A more frequent control of the riprap placement work is recommended at the beginning of these works in order to familiarize all personnel with the size and placement requirements. In particular, this is to be done in the presence of new equipment operators, Contractor personnel and the Engineer.

6.7.4.5 INSTRUMENTATION TRENCH MATERIAL

Trenching instruments and cables involves material placement and compaction in confined spaces and close to fragile instruments. Placement and compaction methods shall be adapted to these conditions. Embankment material shall be placed in compliance with the grades and elevations shown on the drawings.

Special compactors shall be used inside the piezometer and inclinometer shelters and in the instrumentation trench. Zone 2F material placed around the cables shall be compacted with special compactors in 150 mm thickness layers. The minimum specified degree of compaction shall be obtained for this material.

6.7.5 TEMPORARY SLOPES

Acceptable temporary longitudinal slopes are specified in “Embankment Construction” Section of the Technical Specification. Placement and compaction of the construction slopes must be performed horizontally.

Steps shall be left between layers to avoid slopes steeper than 3H:1V. Before placement of material on existing new embankment with a slope steeper than 8H:1V, the in-place material shall be excavated over a thickness of at least 1 m until well compacted material is reached. All concentration of particles of same size shall be excavated and removed over the entire length of the temporary slopes before the placement of the new embankment.

Upstream and downstream protection zones (Zone 4 material) shall not lag the adjacent material by more than the specified height. This restriction is to allow the use of standard equipment to place the materials, and to ensure that there will be sufficient force in the bucket of the backhoe to achieve acceptable compaction in pressing the materials. This restriction is also required to avoid the need for correction of non compliant materials with equipment with an excessive reach. Upstream and downstream protection slopes shall be approved by the Engineer at every 3 m height of construction (measured vertically).

6.7.6 WINTER WORKS

In the winter season, it will be necessary to halt the construction of impervious embankment sections because good compaction cannot be obtained if the soil is frozen. Cold weather can make it more difficult to obtain a given compaction, even at temperatures above freezing.

Prior to the winter season, the construction surface of any partially completed embankments of Zone 1 material shall be graded to allow free drainage and then compacted with a smooth cylinder compactor. These surfaces shall be insulated from freezing with snow or with other approved insulation material. As a practical guide, a 1.5 to 2 m layer of granular material or a 2 to 3 m thickness of snow are required as protection against freezing.

At the restart of the works, after cold weather, the insulation material shall be excavated and removed to disposal or stockpile. The material may be stockpiled for reuse in the appropriate embankment zone materials if it complies with the specified requirements.

If the depth of freezing for Zone 1 material is greater than 450 mm, then it shall be excavated and disposed of before restarting of embankment construction in the spring season. If the thickness of frozen material is less than 450 mm, it may be left in place until completely thawed and then compacted, or it can be excavated as described above.

All frozen material of zones 2A, 2C, 2F, 2G and 3A shall be left to thaw completely before compaction of the embankment surfaces.

If the frozen thickness of the acceptable foundation is greater than 450 mm and the material is acceptable (gradation and compaction), then material shall be left until completely thawed, and then compacted as required.

6.7.7 QUALITY CONTROL TESTS

Visual inspection is a very important and essential quality control tool, especially in the case of embankment construction such as this in which the methodology of construction is specified. This is supplemented and supported by quality control tests as presented in the following sections of this report. Some items cannot be controlled by a test itself and shall therefore be subject to visual control during the different stages of placing and compaction.

Items to be controlled are:

- Thickness of the layer before compaction;
- General horizontality of the layers, especially at the contact with the foundation and the abutments;
- Maximum permitted size of the materials in the different zones of embankment;
- Number of passes, overlap, speed and frequency of the compaction equipment;
- Absence of segregation, especially at contact of different embankment zones;
- Size dimension of riprap blocks which are not subjected to gradation tests;
- Riprap placement in order to obtain a uniform surface over the entire thickness and without blocks in unstable condition;
- Temporary slopes such that the embankments material are placed and levelled so that the contact with the same material can be compacted without contamination of other materials;
- Compliance of the construction tolerances in the contact between zones and with respect to the theoretical lines.

Quality control tests shall be performed on the materials at the source of materials (borrow pits and quarries) and in the embankments during the entire construction period.

These tests are related to the quality of materials, their placement and compaction, all in accordance with the Technical Specification. For a proper functioning of the embankment works, the control tests shall be performed initially at the material source. However, final acceptation will be made material tested in the embankments. An acceptable material at the source can become unacceptable during transportation and placement at the embankment and must be tested to show that complies with the requirements of the Technical Specification.

The quality tests, whose frequency varies with each zone, are performed generally more often at the beginning of works. The frequency of testing may be reduced once the inspectors are more familiar with the properties of the different embankment zones. Frequency of testing may be amended based on the experience gained during the quality control works.

The minimum frequency for testing to be performed on the embankment materials is presented in Table 1 in Section 31 23 23 of the Technical Specification. In addition to these routine tests, additional tests may be required if there is questionable quality of material at the embankment or at the source.

Before starting of the quality control work, the laboratory shall calibrate all the equipment to be used during testing. This calibration shall be repeated regularly during the works, as required in the Technical Specification.

In special cases, the Engineer may require the excavation of an inspection trench to allow verification of the density and uniformity of the embankment, and to confirm the absence of material stratification.

6.7.7.1 SIEVE ANALYSIS

This type of test shall be performed on all the different material zones in the sources, in the required excavations and in the embankments themselves to ensure that the specified particle size limits for each zone are observed.

6.7.7.1.1 ZONES 1, 1C, 2A, 2C, 2E, 2F, 2G AND 3A

For these materials the percentage of fine contents (passing the 0.080 mm sieve) is an important criterion. For material zones 1, 1C, 2A, 2C and 2F, no tolerance on the specified percentage of material passing 0.080 mm is permitted. For Till material, sieve analysis is performed on the fraction passing the 80 mm sieve. Tests on the entire sample are also performed, but at a lower frequency.

A maximum of 10% of samples taken from each material zone may be outside the specified gradation envelope, provided that the materials represented by these samples are well distributed in the embankment construction areas, as approved by the Engineer.

6.7.7.1.2 ZONES 3C, 3D, 3E AND 4

For these materials, there is no routine test schedule requirement. Visual inspection is made to control the maximum size of particles, the uniform gradation and the concentration of particles of a particular size. For Zones 3C, 3D and 3E, the length and the thickness of a block shall not exceed the maximum specified size. The length of a block can be larger than the specified dimension if this oversize does not result in an increase of the maximum thickness of the layer before compaction. However, the thickness of the block shall be at least 1/3 of its length.

For Zone 4 material, the block sampling benchmarks of each type of riprap shall be prepared by the Contractor at each source being exploited and shall be approved by the Engineer. The sample blocks shall cover the range of the specified blocks and include samples of over and undersize blocks. The benchmark samples shall be approved by the Engineer before starting of the selection or treatment for the production of the embankment of Zone 4. These samples are to be used both by the Contractor and by the Engineer.

In the embankments, sieve analyses control test of Zone 4 shall be performed as requested by the Engineer. The frequency of testing shall be established by the Engineer and may be adjusted depending on the success of the Contractor to comply with the requirements.

Measurement of the blocks shall be performed in compliance with the requirements of the Technical Specification (Article 2.1.7, Section 31 23 23 – Embankment Construction). The nominal size of the stones (*D*) shall be determined using the following formula, where, *a*, *b* and *c* are the length, width and thickness of the stone, respectively:

$$D = (a \times b \times c)^{1/3}$$

These dimensions are associated to those of the internal edges of a parallelepiped box which may contain the stone.

6.7.7.2 IN PLACE DENSITY

The density of the materials in place is determined by a method appropriate to the dimensions and the nature of the particle material.

6.7.7.2.1 ZONES 1, 1C, 2F AND 2G

The in-situ density of the till and zones 2F and 2G materials is usually determined using nuclear equipment (nucleodensimeter). This equipment also allows the reading of the moisture content of the material. Daily calibration shall be performed to ensure reliable results.

The criterion adopted to define the minimum compaction degree is 98% of the maximum dry density for the till and 93% for the 2F and 2G materials. The average dry density of the compacted material, determined by any 10 consecutive tests shall be at least equal to 98% of the maximum dry density as determined by ASTM D698 for till and 93% for the 2F and 2G materials. No single test result shall be less than 95% for till and 91% for the 2F and 2G materials.

Test results with values of density lower than the required shall not be concentrated in an area, but spread all over the volume of the embankment. If the above requirements are not respected, additional compaction shall be required. An adjustment of the moisture content of the material may also be required.

6.7.7.2.2 ZONES 2A, 2C AND 3A

The in-situ density of the granular material of zones 2A and 2C and crushed stone 3A is normally determined by the volume of water method using a membrane.

The average dry density of the compacted material of zones 2A and 2C, determined by any 10 consecutive tests shall be at least equal to 93% of the maximum dry density as determined by ASTM D4253. No single test result shall be less than 91%.

6.7.7.2.3 ZONES 3C AND 3D

For these rockfill materials, there is no routine test is schedule requirement. The control is performed by verifying that placement and compaction are executed in compliance with the requirements of the Technical Specification.

6.7.7.2.4 MAINTENANCE GRADE No 3

For this material, the average dry density of the compacted material, determined by any 10 consecutive tests shall be at least equal to 97% of the maximum dry density as per ASTM D1557.

6.7.7.3 MOISTURE CONTENT

Control of moisture content is only performed in embankment of Zones 1 and 1C. All results shall comply with the requirement of the Technical Specification. During compaction, the till moisture content of Zone 1 material shall be between 1 % below and 1.5 % above of the optimum moisture content as determined by ASTM D698, or ASTM D1557 where appropriate, and between the optimum moisture content and 2% above of the optimum moisture content as determined by ASTM D698 for the 1C material.

6.7.8 RESULT S STATISTICAL PROCESS

Test results shall be presented in tables and summary charts. Sources and embankment test results shall be presented separately. Summary tables shall present the location, elevation and date of the tests, quantity, mean and extreme values, the standard deviation, the reference values and the deviation from the reference values. Complete detail on each type of test, standard followed, nature and origin of the sample and any other relevant comment shall also been submitted.

6.8 SLURRY CUT-OFF WALLS

6.8.1 DESIGN

The design of the stabilization works calls for a construction of 2 cement bentonite cut-off walls. One in the upstream area, which will assure the watertightness of the structure in the lower part of the stratified drift unit, extending from elevation 20.5 to approximately elevations 10 m and 5 m, and anchored in the lower clay unit. The second one, located in the northwest area of the North Spur and built inside the intermediate drift unit between elevations 50 m and 10 m, will stop the groundwater recharge from the northwest into this unit.

The current design includes the construction of the cement bentonite cut-off walls from a platform established at a specified elevation. The cut-off wall, which is anchored in the lower clay, has a minimum width of 0.60 m. The cut-off wall is a one phase method excavation, “cast-in-place wall”, excavated with the aid of a cementitious bentonite slurry referred to as grout, which sets in a few hours and serves directly as the waterproofing material. Work is carried out continuously. The fresh grout mixes with the grout bearing sediments in the excavation, without any discontinuity.

The mix retained for the preparation of the self hardening grout comprises 25 kg of cement and 6 kg of bentonite for 100 kg of water. The slurry shall have a minimum unconfined compression strength of 130 kPa at 7 days and of 200 kPa at 28 days measured on an unconsolidated specimen in a triaxial cell at a strain of 0.1% per minute. A plastic strain greater than 6% without cracking shall be obtained in a triaxial test at 90 days on a consolidated specimen under a confining pressure of 100 kPa at a strain rate of 0.1% per minute. A hydraulic conductivity equal or less than 1 x 10^{-6} cm/s shall be obtained at 90 days in the referred triaxial test.

Unconfined compression strength requirements are due to the requirement that the cut off wall shall be able to endure the deformations which are imposed upon it without cracking. The material is also required to be able to follow the deformations imposed on the soil by the overlying embankments. The top of the cut-off wall shall have a rapidly increasing strength after construction to avoid delays in the construction of the overlying embankments. Conservatively, the strength at 10 days should be greater than 150 kPa, which is related to the actual vertical stress applied to the cut-off wall. The cement bentonite grout shall set for at least 10 days before cleaning of the top and starting the construction of the overlying embankment.

Laboratory testing of the grout shall be performed prior to start of the works in order to demonstrate the compliance of the mix.

6.8.2 EXCAVATION

Execution of the slurry trench requires the preparation of a horizontal platform over the full length of the wall prior to start trench excavations. The elevation of the platform was established at least 1 m above the groundwater table in order to preserve the integrity of the wall excavation. The platform will ensure the quality of works to obtain a fully bond connection between the cut-off wall and the till blanket and will provide a working surface of adequate width.

In order to allow the pore pressure dissipation induced during compaction of the till embankment, a delay of 1 week is required between the preparation of the platform and the starting of the trench excavation. The goal of this measure is to avoid stability issues in the upper part of the trench, which has been observed in other projects.

The excavation of the trench will be performed with a hydraulic excavator with extended long stick capable of excavating to a depth of about 20 m or with a hydraulic clamshell mounted on a crawler crane. The latter could require the construction of concrete guide walls at the top of the trench in order to minimize and/or eliminate the risk of local wall instabilities which could cause widening of the trench and overconsumption of grout. However, the choice of whether guide walls will be used has been left to the Contractor and the Contractor's choice of construction methods.

The panel excavation shall be continuous and the final depth shall be reached before the commencement of the initial set of the plastic slurry. Panels shall be linked together by overlap into adjacent previously constructed panels to ensure the watertightness of the trench.

The length of the open trench shall be determined with due consideration for trench wall stability, particularly at the toe of slopes. This may impose the requirement for excavation in short lengths using a clamshell rather using a backhoe in some sectors.

The bottom of the trench shall be sounded at regular intervals by an appropriate method. These soundings shall be compared with the established reference depths before start of the excavation in order to verify the correct anchoring of the trench in the clay unit.

Where the trench finishes in overburden, the Contractor shall sample uncontaminated soils at the anchor depth of the trench for characterization by the Engineer, Where the trench reaches the bedrock, all sand, gravel, cobble and boulder, loosened excavation and settled materials overlying the bedrock shall be removed.

Using of additives may be allowed, subject to review and approval by the cement-bentonite specialist and the Engineer. Additives which may be considered to maintain the plasticity of the slurry for a longer period during the excavation operations.

At the end of the excavation of each section, the coordinates of the section at the top and at the bottom shall be sounded by an appropriate method and transverse and longitudinal overlap of the sections shall be confirmed.

6.8.3 CEMENT – BENTONITE

The equipment used by the Contractor shall include a mixing pond for the bentonite hydration, mixing and storage of bentonite slurry, a high energy mixer for slurry production and sumps, pumps, piping, valves, fittings and hose/piping.

First step of the slurry preparation will consist in mixing of water and bentonite for a minimum of 12 hours to allow full hydration of the slurry. Cement shall be added by pumping just prior to delivery of the slurry in the trench.

In order to maintain the integrity of the top of the trench, a 10 days period shall be allowed for the trench to set before starting with the embankment construction phase.

6.8.4 QUALITY CONTROL

During construction of the trench the following controls shall be performed:

- Verticality and alignment of the trench;
- Bottom trench cleaning at the end of the section excavation;
- Sounding of the bottom final profile of the trench and
- Overlapping between panels.

A complete hydration of the slurry mix shall be assured and controlled by measures of viscosity and density of the slurry in the mixing pond. Values larger than 30 seconds and 1,030 kg/m^3 respectively are required.

After the cement has been added to the fully hydrated cement-bentonite mix, samples are taken at the end of the exit conduit to the trench. The viscosity of the slurry shall be at least 55 seconds, the viscosity at least 1,180 kg/m^3 and the slurry bleeding shall not be greater than 4 % and its average no greater than 3%.

Sampling of the cement-bentonite slurry shall be also performed inside the trench in order to determine mud balance density, Marsh Funnel viscosity, bleeding and sand content. Substantial variation of the sand content sampled in the upper and lower part of the trench will indicate heterogeneity of the slurry and thus, modifications at the plant shall be required.

Unconfined compression strength tests at 7 and 28 and permeability tests at 90 days are planned on samples retrieved at the end of the exit conduit to the trench in order to confirm the compliance of the mix with the design requirements.

Additional testing may be required by the Engineer on samples retrieved at the top of the trench before placement of embankments.

6.9 RELIEF WELLS

Analysis of the existing data and the new information obtained from the existing and installed piezometers in the lower aquifer after the first and final impoundment will define the necessity of the construction of the relief wells system.

6.10 INSTRUMENTATION AND MONITORING

6.10.1 GENERAL

The North Spur is treated like a dam and is instrumented and monitored as such. An Operation Maintenance and Safety (OM&S) manual will be in place laying out the inspections and monitoring plan (type and frequency) including regular Dam Safety Review in accordance with the recommendations of the Canadian Dam Association.

6.10.2 INSTRUMENTS SUPPLY

Supply of instruments is done by the Contractor; no separate contract was awarded for this item.

6.10.3 INSTRUMENT INSTALLATION

The installation works shall be carried out by highly qualified and experienced personnel. The instrument supplier shall mobilize on site an experienced representative to assist the Contractor during delivery, testing of the instruments before and after installation and calibration and monitoring the installation as well as assuring training of a local technician.

Special care shall be taken during construction in order to avoid damage in the existing and new installations. Cables and conduits exposed on the embankments or ground shall be clearly labelled and identified.

Instructions regarding safe operation around the existing and new instruments shall be transmitted to the equipment operators in the works areas near the instruments.

Verifications of instrument operation shall be done before the instrument installation to allow repair, adjustments, calibration or replacement of the instrument, if required.

During and after installation of the instruments, and during backfilling operations, tests shall be performed to assure the continued proper operation of the instruments.

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6.10.4 CALIBRATION

All instruments shall be calibrated on site following delivery. All defective instrument or instruments not complying with the requirement of the technical specification shall be replaced or repaired to the satisfaction of the Engineer.

6.10.5 DATA COLLECTION

Data sheets for the instruments provide the recommended procedures to perform the instrument readings. Generally, the most critical stages during the life structure are during construction, the impoundment and the period between impoundment and the time when the seepage steady state conditions in the foundation and embankments are reached.

Instrument readings shall be performed regularly following an established frequency during these stages. In all cases, a validation of readings is first performed by the inspector on site by comparing the current reading against the previous one. This procedure will avoid registering erroneous readings and enable identifying, at an early stage, readings which could correspond to anomalous performance of the structure.

During the construction stage is important to follow variations of pore water pressure in the North Spur.

The reading frequency shall be increased if a significant deviation or difficult to interpret reading of the normal or anticipated behaviour of the structure is observed. In such a case, additional investigations, including installation of new instruments, could be necessary. Reading frequencies shall also be increased after any occurrence of unusual events, such as drawdown of the reservoir.

Table 6-4 shows minimum recommended reading frequencies for each instrument. Monitoring also includes visual inspections of the structures.

Table 6-4 : Instrument Readings – Minimum Frequencies

		Vibrating Wire Piezometer	Standpipe Piezometer	Weir
During Construction	During Works	1 / 2 weeks	1 / 2 weeks	–
	Time Break [2]	1 / season	1 / season	–
During Impoundment		1 / day	1 to 2 / day	2 / week
After impoundment [3]	1st Year	1 / week	1 / week	1 / week
	2nd Year	2 / month	2 / month	1 / week
	2nd to 5th Year	6 / year	1 / month	1 / month
	Subsequent Years	As per exploitation standards		

[1]: reading frequency is the minimum frequency required and applies to the structures, which behaviour is considered normal.
[2]: time break is the period dividing the construction seasons or between the end of construction and the reservoir impoundment.
[3]: In all cases, the reading frequencies cannot be less than the minimum frequencies established in the exploitation monitoring program.

6.10.6 DATA PROCESSING

The input readings are done through an automated system and recorded in a database. The data processing includes calculations performed to translate the readings into usable engineering values (pressure, elevation, etc.). All the readings shall be validated before being used in analyses of structure behaviour.

6.10.7 DATA PRESENTATION

In general, graphical presentation of the data is more suitable. Tables should also be presented as reference. Numerically tabulated data are not conducive to easily detecting trends, evaluating unanticipated behavior or making comparison with design values. Plots of the data are needed to provide visual comparisons between actual and predicted behavior, a visual means to detect data acquisition errors, to determine trends or cyclic effects, to compare behavior with other instruments, to predict future behavior and to determine instrumentation maintenance requirement needs. Plotting enables data to be compared readily with events that cause changes in the data, such as construction activities and environmental changes. Data shall be processed and presented by qualified instrumentation personnel.

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Results shall be plotted to present reading variations as function of different key variables, such as depth, time, temperature, reservoir level, etc. (positional plot, time history plot, etc.). Conditions before, during and after data collection shall also be recorded in standard forms to facilitate data documentation and storage. The result presentation shall include a report of the instrument behaviour and operation.

6.10.8 DATA ANALYSIS

Independent of the method used to present the results it is important that the data is analyzed in a timely manner by specialized engineers in order to identify any anomaly in the behaviour of the structure. In order to facilitate the interpretation, all instrument readings shall be performed at the same period of time.

If an anomaly is detected, which cannot be linked to a fault or malfunction of the instrument, this shall be analyzed in order to evaluate its potential impact on the security and integrity of the structure and to establish if further action, such as increased monitoring, additional instrumentation or remedial works are deemed necessary.

6.10.9 THRES HOLD LEVELS

The data thresholds for setting of alarms for readings will be established after impoundment following a detailed analysis of the data collected by the specialized personnel reviewing the data..

Threshold levels could also be established during the construction activities, with respect to potential behavior of instruments to be monitored during this period.

6.10.10 VISUAL INSPECTIONS

Scheduled regular visual inspections of all areas of the structures shall be performed during construction periods and during operation for warning of potential problems.

An inspection program shall be implemented starting at the beginning of the impoundment in accordance with the frequencies and procedures of the owner and as recommended by the Engineer. Visual inspections shall be performed prior to impoundment to establish the reference stage for the project.

Regular inspections performed by the site personnel will provide a record of monitoring of the structures. These inspections will include the following:

- Crest, slopes and toe of downstream area during the raising of the reservoir water level. Any crack, resurgence, erosion or scour area, slide or other identified anomalies shall be promptly identified and reported to the responsible of integrity and maintenance of the structures. Channeled flows in the downstream and in the exit of the Kettle Lakes areas shall be regularly measured during and after impoundment. Any sudden increase of flow shall be reported to the responsible area;
- An inspection of the upstream slope shall be performed after a drawdown of the reservoir and after a period of high and continuous winds. This inspection shall be focused in occurrence of cracks, slides, scour, erosion and any other damage.

A maintenance program shall also be established to cover the following aspects:

- Regular maintenance of slopes, crest and berms;
- Maintenance of discharge ditches;
- Relief wells;
- Instruments, especially data acquisition points and weirs;
- Roads.

Suggested minimum frequencies of visual inspections are presented in Table 6-5.

Table 6-5 : Visual Inspections – Minimum Frequencies

During Construction			During Impoundment			After Impoundment							
						1st Year			2nd Year			3rd to 5th Year	
A	B	C	A	B	C A		B C		A	B	C A C		
2/season	–	–	2/day	2/day	1/day	1/2 days	1/2 days	1/month	1/week	1/week	4/year	1/week	2/year

A: pedestrian inspection [3]
B: inspection by vehicle [3]
C: inspection by helicopter

[1]: In all cases, the reading frequencies cannot be less than the minimum frequencies established in the exploitation monitoring program.
[2]: Table shows minimum frequencies in normal conditions. Frequencies shall be increased under unexpected behaviors, flood occurrence.
[3]: Pedestrian and in vehicle inspections shall be performed at regular and uniform distributed intervals.

7 APPURTENANT STRUCTURES

7.1 PERMANENT ROAD

The permanent road to the North Spur is defined from the Trans-Labrador Highway to the crest of the Spur and from there to the upstream and downstream areas of the Spur. The first section of the road shall be designed by the Contractor following the alignment of the existing road.

Borrow materials for the construction of the roads will come from the excavations. Final grade of the road consists of Maintenance Grade No. 3 material as defined in the Technical Specifications. The final grading of the road shall be undertaken at the end of works to avoid damage due to heavy traffic during the construction activities

Final design of this section of the road shall be submitted to the Engineer for approval. The maximum slope of the road was established at 8%. The required final width of the permanent road is 7.5 m.

Other design parameters are:

- Minimum horizontal radii: 125 m;
- Minimum K values for vertical curves: crest, 10 and SAG, 15.

7.2 LAYDOWN AREAS

An approximate area of 40,000 m^2 was identified by the Company to be used by the Contractor as its laydown area. All temporary buildings, trailers, containers shall be installed inside this area.

An area of about 5,000 m^2 was designated in the Contract as the Company Area to be used for Company site facilities and shall be built and maintained by the Contractor.

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Appendix A – Drawings

MFA-SN-CD-2800-CV-GA-0002-01 - MUSKRAT FALLS - GENERAL ARRANGEMENT OF WORKS - PLAN

MFA-SN-CD-2800-GT-PL-0012-01 - MUSKRAT FALLS - NORTH SPUR STABILIZATION WORKS – EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION - PLAN

MFA-SN-CD-2800-CV-PL-0009-01 - MUSKRAT FALLS - NORTH SPUR STABILIZATION WORKS – PLAN

MFA-SN-CD-2810-CV-SE-0001-02 - MUSKRAT FALLS - NORTH SPUR STABILIZATION WORKS – UPSTREAM AREA - TYPICAL CROSS SECTIONS – SHEET 2 OF 4

MFA-SN-CD-2820-CV-SE-0001-03 - MUSKRAT FALLS - NORTH SPUR STABILIZATION WORKS - DOWNSTREAM AREA – TYPICAL CROSS-SECTIONS - SHEET 3 OF 3

NOTES:

1. ALL COORDINATES ARE REFERENCED TO UTM NAD83, ZONE 20.
2. THE GROUND TOPOGRAPHY CONTOURS ARE BASED ON A SITE-SPECIFIC LIDAR SURVEY COMPLETED IN 2006.
3. THE RIVER BATHYMETRY CONTOURS ARE BASED ON AN AIRBORNE BATHYMETRIC SURVEY COMPLETED IN 1998.
4. THE CONTOUR INTERVAL OF THE GROUND TOPOGRAPHY AND THE RIVER BATHYMETRY IS 2 METRES.
5. ELEVATIONS ARE IN METRES AND DIMENSIONS ARE IN MILLIMETRES UNLESS NOTED OTHERWISE.
6. THE CONTOUR INTERVAL OF THE GROUND TOPOGRAPHY IS 1 METRE.

LEGEND:

NIC — NOT IN CONTRACT
TOPOGRAPHIC CONTOUR
BATHYMETRIC CONTOUR
HYDROLOGICAL STATION

PROVINCE OF NEWFOUNDLAND PERMIT HOLDER CLASS "A"
This Permit Allows
SNC-LAVALIN Inc.
To practice Professional Engineering in Newfoundland and Labrador. Permit No. as issued by PEGNL N0458 which is valid for the year 2015.

0 100 200 300 400 500 m
SCALE 1:5000

ISSU.	REV.	DATE	DISTRIBUTION & STATUS	ISSU.	REV.	DATE	DISTRIBUTION & STATUS
4	C3	23-MAR-2015	RE-ISSUED FOR CONSTRUCTION				
3	C2	03-MAR-2015	RE-ISSUED FOR CONSTRUCTION				
2	C1	21-JUL-2014	ISSUED FOR CONSTRUCTION				
1	B1	07-NOV-2013	ISSUED FOR BID				

ISSUE REGISTER

No.	REFERENCE DRAWING	No.	REFERENCE DRAWING

No.	DATE	REVISION	A.C. MOD.	R.B. VER.	G.S. APP.
C3	23-MAR-2015	DOWNSTREAM AREA MODIFIED			
C2	03-MAR-2015	SPOIL DISPOSAL AREA MOVED; NOTE 6 ADDED			

********* FOR INTERNAL USE ONLY *********

REVIEW CLASS: | EQUIPMENT TAG NUMBER:

REVIEW DOES NOT CONSTITUTE APPROVAL OF DESIGN DETAILS, CALCULATIONS, TEST METHODS OR MATERIAL DEVELOPED AND/OR SELECTED BY THE CONTRACTOR, NOR DOES IT RELIEVE THE CONTRACTOR FROM FULL COMPLIANCE WITH CONTRACTUAL OR OTHER OBLIGATIONS.

- ☐ 1. REVIEWED AND ACCEPTED NO COMMENTS
- ☐ 2. REVIEWED - INCORPORATE COMMENTS, REVISE & RESUBMIT
- ☐ 3. REVIEWED - NOT ACCEPTED
- ☐ 4. INFORMATION ONLY
- ☐ 5. NOT REVIEWED

LEAD REVIEWER: Date (dd-mmm-yyyy):
NE-LCP MANAGEMENT: Date (dd-mmm-yyyy):
PROJECT MANAGER: Date (dd-mmm-yyyy):

PROFESSIONAL STAMP
REGISTERED PROFESSIONAL ENGINEER — PEG Newfoundland and Labrador — ALVARO CEBALLOS — SIGNATURE — 23-MAR-2015 — DATE — NEWFOUNDLAND & LABRADOR

SNC·LAVALIN

DESIGNED BY A. CEBALLOS	APPROVED Discipline Lead Engineer R. BOUCHARD
DRAWN BY R. MERRI	APPROVED Engineering Manager G. SNYDER
VERIFIED BY N. DAIYAN	
DATE 29-SEP-2012	SCALE 1:5000

CLIENT: nalcor energy — LOWER CHURCHILL PROJECT

PROJECT: LOWER CHURCHILL PROJECT

TITLE: MUSKRAT FALLS
GENERAL ARRANGEMENT OF WORKS
PLAN

SLI DOC No. 505573-3281-4GDD-0021-03
NE DOC No. MFA-SN-CD-2800-CV-GA-0002-01 Rev. C3

FORMAT A0 (mm)

SUB-PKG: 0008-4G01 PLATE 03

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2015.03.20/8:56am

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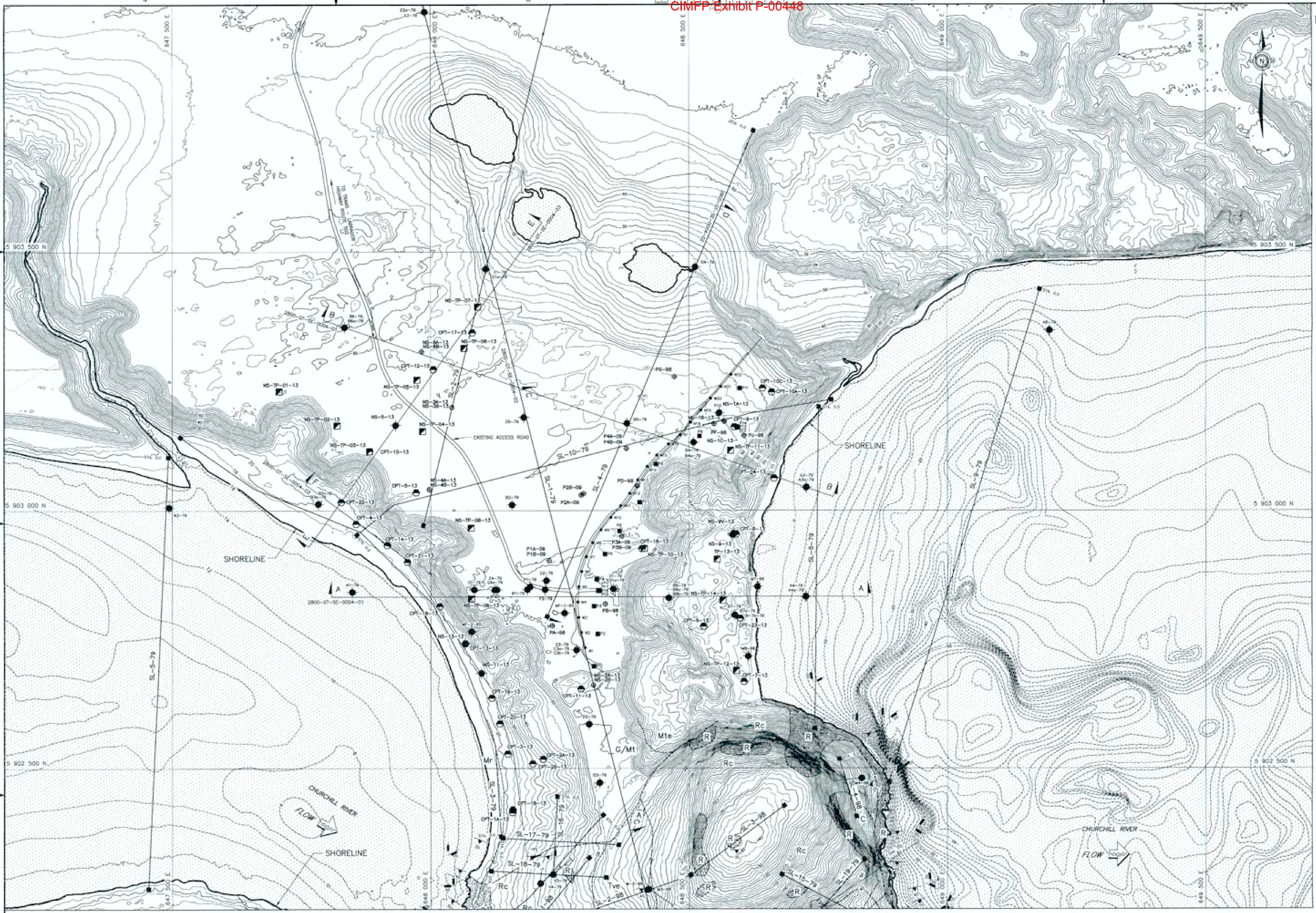


PLAN

NOTES:

1. ALL COORDINATES ARE REFERENCED TO UTM NAD83, ZONE 20.
2. THE GROUND TOPOGRAPHY CONTOURS ARE BASED ON A SITE-SPECIFIC LIDAR SURVEY COMPLETED IN 2006.
3. THE RIVER BATHYMETRY CONTOURS ARE BASED ON AN AIRBORNE BATHYMETRIC SURVEY COMPLETED IN 1998.
4. THE CONTOUR INTERVAL OF THE GROUND TOPOGRAPHY AND THE RIVER BATHYMETRY IS 2 METRES.
5. ELEVATIONS AND DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.

LEGEND:

- TOPOGRAPHIC CONTOUR
- BATHYMETRIC CONTOUR
- SPRING FLOOD WATER LEVEL
- SSSSSSSSS SHEAR ZONE
- SSS SSS ASSUMED SHEAR ZONE
- GEOLOGICAL BOUNDARY
- SEISMIC LINE
- EXPOSED ROCK
- INCLINED JOINT
- VERTICAL JOINT
- INCLINED FOLIATION
- VERTICAL BOREHOLE
- INCLINED BOREHOLE
- PUMPING WELL
- LOST PIEZOMETER
- no-YEAR EXISTING PIEZOMETER
- NS-no-13 2013 BOREHOLE
- CPT-no-13 2013 CPT TEST
- NS-TP-no-13 2013 TEST PIT
- EL ELEVATION
- INT INTERSECTION

SURFICIAL GEOLOGY:

- C COLLUVIAL
- A AEOLIAN
- G GLACIOFLUVIAL
- M MARINE
- T GLACIAL TILL
- O BOG
- R ROCK

- c CONCEALED BY VEGETATION
- e ERODED AND DISSECTED
- p PLAIN
- r RIDGE
- t TERRACE
- v VENEER

PROVINCE OF NEWFOUNDLAND PERMIT HOLDER CLASS "A" PEGNL SNC-LAVALIN Inc.

0 50 100 150 200 m
SCALE 1:2500

ISSU.	REV.	DATE	DISTRIBUTION & STATUS	ISSU.	REV.	DATE	DISTRIBUTION & STATUS
1	C1	07-NOV-2013	ISSUED FOR INFORMATION				

ISSUE REGISTER

No.	REFERENCE DRAWING
MFA-SN-CD-2800-GT-SE-0004-03	EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION - NORTH SPUR SECTIONS - SHEET 3 OF 3
MFA-SN-CD-2800-GT-SE-0004-02	EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION - NORTH SPUR SECTIONS - SHEET 2 OF 3
MFA-SN-CD-2800-GT-SE-0004-01	EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION - NORTH SPUR SECTIONS - SHEET 1 OF 3

No.	DATE	REVISION	MOD.	VER.	APP.

********** FOR INTERNAL USE ONLY **********

REVIEW CLASS: EQUIPMENT TAG NUMBER:

REVIEW DOES NOT CONSTITUTE APPROVAL OF DESIGN DETAILS, CALCULATIONS, TEST METHODS OR MATERIAL DEVELOPED AND/OR SELECTED BY THE CONTRACTOR, NOR DOES IT RELIEVE THE CONTRACTOR FROM FULL COMPLIANCE WITH CONTRACTUAL OR OTHER OBLIGATIONS.

- [x] 1. REVIEWED AND ACCEPTED: NO COMMENTS
- [ ] 2. REVIEWED - INCORPORATE COMMENTS, REVISE & RESUBMIT
- [ ] 3. REVIEWED - NOT ACCEPTED
- [ ] 4. INFORMATION ONLY
- [ ] 5. NOT REVIEWED

LEAD: 18-Nov-2013
MANAGEMENT: 8-Nov-2013
PROJECT MANAGER: 21-Nov-2013

PROFESSIONAL STAMP
NASSER DAYAN 07-NOV-2013

SNC·LAVALIN

DESIGNED BY N. DAYAN
DRAWN BY R. MERRI
VERIFIED BY A. CEBALLOS
DATE 22 SEP 2012
APPROVED Discipline Lead Engineer R. BOUGHAN
APPROVED Engineering Manager G. SNYDER
SCALE 1:2500

CLIENT nalcor energy

PROJECT LOWER CHURCHILL PROJECT

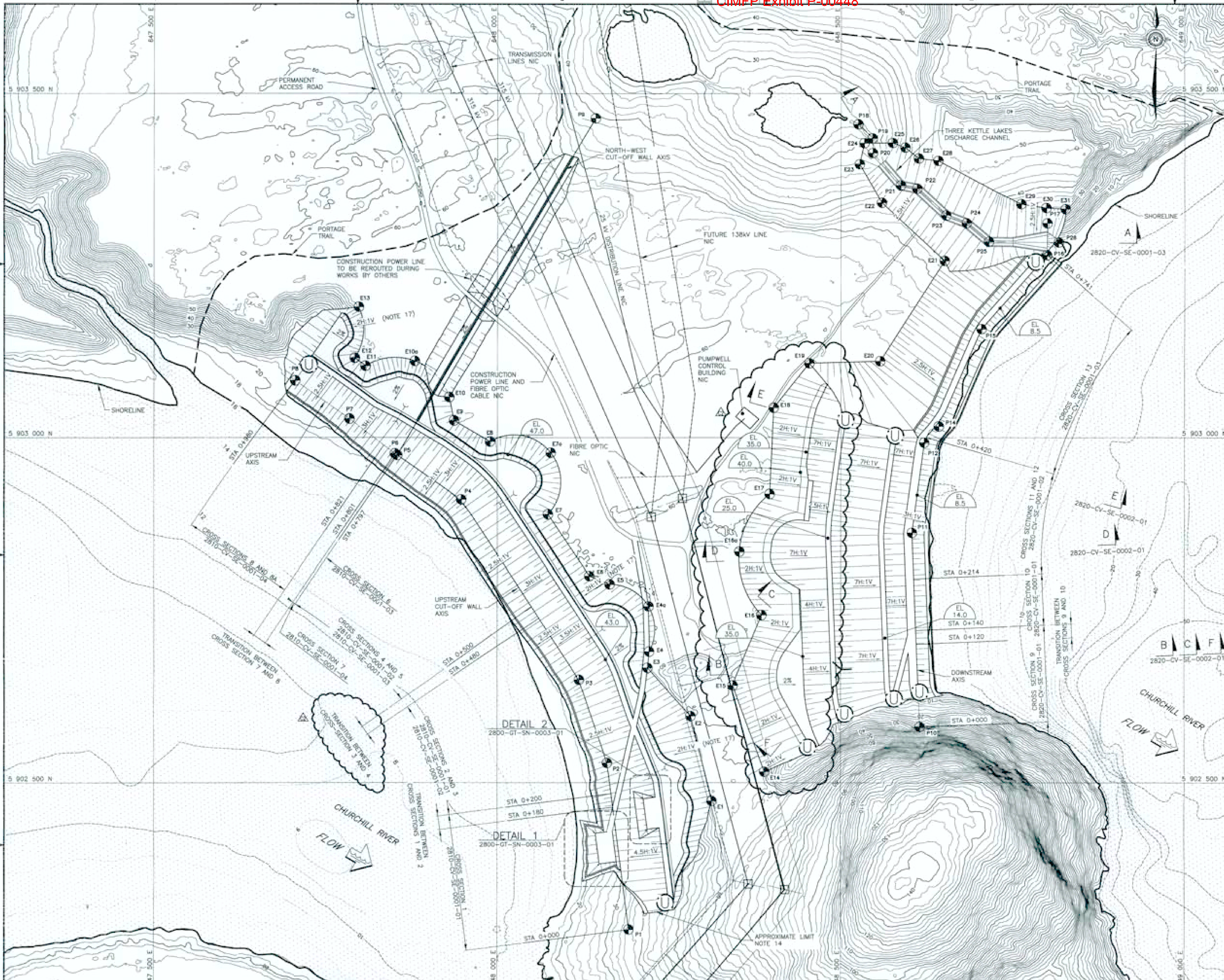
TITLE MUSKRAT FALLS NORTH SPUR STABILIZATION WORKS EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION PLAN

File No. 505573-3281-4000-0008-01

MFA-SN-CD-2800-GT-PL-0012-01 Rev. C1

2013.11.15/3:55pm

SUB-PKG: 0008-4G03 PLATE G04



NOTES:

1. ALL COORDINATES ARE REFERENCED TO UTM NAD83, ZONE 20.
2. THE GROUND TOPOGRAPHY CONTOURS ARE BASED ON A SITE-SPECIFIC LIDAR SURVEY COMPLETED IN 2006.
3. THE RIVER BATHYMETRY CONTOURS ARE BASED ON AN AIRBORNE BATHYMETRIC SURVEY COMPLETED IN 1998.
4. THE CONTOUR INTERVAL OF THE GROUND TOPOGRAPHY AND THE RIVER BATHYMETRY IS 2 METRES.
5. ELEVATIONS AND DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
6. TRANSITION BETWEEN SECTIONS ARE SHOWN ON THE DRAWING AND MAY BE ADJUSTED ON SITE BY THE ENGINEER.
7. THE CONTRACTOR SHALL PREPARE A CONTRACT SPECIFIC ENVIRONMENTAL PROTECTION PLAN WHICH WILL BE SUBJECT TO THE ENGINEER'S APPROVAL.
8. LOCATION OF CUT-OFF WALLS MAY BE ADJUSTED ON SITE BY THE ENGINEER.
9. THE LIMITS OF THE TYPICAL CROSS SECTIONS SHOWN ON THE DRAWINGS ARE APPROXIMATE. FINAL LIMITS WILL BE ESTABLISHED ON SITE BY THE ENGINEER.
10. THE UPSTREAM CUT-OFF WALL AXIS IS DEFINED BY LOCATION POINTS P1 TO P5.
11. THE UPSTREAM AXIS IS DEFINED BY LOCATION POINTS P5 TO P8.
12. THE NORTH-WEST CUT-OFF WALL AXIS IS DEFINED BY LOCATION POINTS P5 TO P9.
13. THE DOWNSTREAM AXIS IS DEFINED BY LOCATION POINTS P10 TO P17.
14. FINAL LIMIT OF STABILIZATION WORKS WILL BE CONFIRMED ON SITE BY THE ENGINEER BASED ON LOCAL CONDITIONS.
15. FOR LOCATION OF FINGER DRAINS SHOWN IN SECTIONS B-B, C-C, D-D, E-E AND F-F, SEE DRAWING MFA-SN-CD-2820-CV-SE-0002-03.
16. ALL WORK SHALL BE CARRIED OUT IN ACCORDANCE WITH THE CONTRACT SPECIFIC ENVIRONMENTAL PROTECTION PLAN (C-SEPP).
17. ABOVE EL 47.00 REGRADING TO 2H:1V IS NOT REQUIRED WHEN NATURAL SLOPE IS 1.8H:1V OR GENTLER.

LEGEND:

- TOPOGRAPHIC CONTOUR
- BATHYMETRIC CONTOUR
- SHORELINE JULY-OCTOBER FOR AVERAGE FLOW OF 1830 m3/s
- PERMANENT ACCESS ROAD
- CONSTRUCTION POWER LINE
- NIC — NOT IN CONTRACT
- BENCHMARK
- LOCATION POINT
- TURNING AREA
- TRANSMISSION TOWER
- CSP CULVERT 600 mm DIA

EXCAVATION POINTS

POINT No	COORDINATES EASTING	NORTHING	RADIUS
E1	648 312.1	5 902 473.4	–
E2	648 281.5	5 902 598.2	–
E3	648 218.8	5 902 668.3	
E4	648 221.2	5 902 691.6	
E4a	648 220.3	5 902 758.5	65.00
E5	648 183.7	5 902 788.3	
E6	648 134.0	5 902 800.1	–
E7	648 073.7	5 902 888.9	
E7a	648 077.6	5 902 978.3	68.00
E8	647 989.4	5 902 994.3	
E9	647 937.0	5 903 024.4	–
E10	647 930.1	5 903 057.9	
E10a	647 879.9	5 903 109.6	83.00
E11	647 808.2	5 903 102.6	
E12	647 792.6	5 903 114.0	
E13	647 798.5	5 903 188.0	–
E14	648 388.7	5 902 515.8	–
E15	648 342.8	5 902 842.9	–
E16	648 384.6	5 902 744.1	
E16a	648 352.3	5 902 834.8	116.00
E17	648 398.2	5 902 918.1	
E18	648 402.0	5 903 042.7	–
E19	648 454.0	5 903 106.4	–
E20	648 557.1	5 903 109.4	–
E21	648 650.7	5 903 255.3	–
E22	648 558.6	5 903 340.1	–
E23	648 527.1	5 903 396.2	–
E24	648 533.8	5 903 427.1	–
E25	648 575.3	5 903 427.5	–
E26	648 593.4	5 903 420.6	–
E27	648 613.0	5 903 404.7	–
E28	648 641.4	5 903 401.2	–
E29	648 762.3	5 903 338.3	–
E30	648 798.5	5 903 332.5	–
E31	648 828.5	5 903 330.8	–

LOCATION POINTS

POINT No	COORDINATES EASTING	NORTHING	RADIUS
P1	648 191.5	5 902 285.7	–
P2	648 159.9	5 902 528.6	150.00
P3	648 119.1	5 902 651.0	150.00
P4	647 947.4	5 902 910.4	150.00
P5	647 854.3	5 902 974.8	–
P6	647 850.9	5 902 977.0	150.00
P7	647 785.0	5 903 028.8	150.00
P8	647 706.1	5 903 081.5	–
P9	648 143.1	5 903 463.8	–
P10	648 613.0	5 902 582.2	–
P11	648 603.3	5 902 861.9	100.00
P12	648 621.2	5 902 992.8	100.00
P14	648 642.5	5 903 016.1	–
P15	648 704.4	5 903 156.0	100.00
P16	648 799.5	5 903 263.5	–
P17	648 799.9	5 903 310.1	–
P18	648 520.1	5 903 454.8	–
P19	648 545.8	5 903 434.2	–
P20	648 546.2	5 903 412.6	–
P21	648 587.1	5 903 365.4	–
P22	648 610.7	5 903 360.7	–
P23	648 652.8	5 903 321.7	–
P24	648 682.7	5 903 310.4	–
P25	648 715.3	5 903 283.2	–
P26	648 816.5	5 903 282.1	–

PROVINCE OF NEWFOUNDLAND PEGL PERMIT HOLDER CLASS "A" SNC-LAVALIN Inc.

SCALE 1:2000 (0 50 100 150 200 m)

ISSUE REGISTER

ISSU.	REV.	DATE	DISTRIBUTION & STATUS
4	C2	23-MAR-2015	RE-ISSUED FOR CONSTRUCTION
3	C1	21-JUL-2014	ISSUED FOR CONSTRUCTION
2	B2	03-APR-2014	ISSUED FOR ADDENDUM 9
1	B1	07-NOV-2013	ISSUED FOR BID

REFERENCE DRAWING

No.	REFERENCE DRAWING
MFA-SN-CD-2800-GT-SN-0003-01	LOGBOOM PLATFORMS SECTIONS AND DETAILS
MFA-SN-CD-2800-CV-SE-0004-01	NORTH SPUR STABILIZATION WORKS – NORTH-WEST AND UPSTREAM CUT-OFF WALLS SECTIONS AND DETAIL
MFA-SN-CD-2800-CV-SE-0006-01 TO 03	NORTH SPUR STABILIZATION WORKS – CUT-OFF WALLS WORK PLATFORM AND CONSTRUCTION STAGES
MFA-SN-CD-2820-CV-SE-0002-01 TO 03	NORTH SPUR STABILIZATION WORKS FINGER DRAINS SHEET 1 OF 3 TO 3 OF 3
MFA-SN-CD-2820-CV-SE-0001-01 TO 03	NORTH SPUR STABILIZATION WORKS DOWNSTREAM – TYPICAL CROSS-SECTIONS SHEET 1 OF 3 TO 3 OF 3
MFA-SN-CD-2810-CV-SE-0001-01 TO 04	NORTH SPUR STABILIZATION WORKS UPSTREAM – TYPICAL CROSS-SECTIONS SHEET 1 OF 4 TO 4 OF 4
MFA-SN-CD-2800-GT-PL-0013-01	PERMANENT ACCESS ROAD – PLAN

REVISION

No.	DATE	REVISION	MOD.	VER.	APP.
C2	23-MAR-2015	DOWNSTREAM AREA MODIFIED	A.C.	R.B.	G.S.
C1	21-JUL-2014	EXCAVATION POINTS ADDED	A.C.	R.B.	G.S.
B2	03-APR-2014	NOTE 17 ADDED	A.C.	R.B.	G.S.

* * * FOR INTERNAL USE ONLY * * *

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- ☐ 3. REVIEWED - NOT ACCEPTED
- ☐ 4. INFORMATION ONLY
- ☐ 5. NOT REVIEWED

LEAD REVIEWER: Date: 8-APR-15

NE-LCP MANAGEMENT:

PROJECT MANAGER: Date: 08 Ap. 2015

PROFESSIONAL STAMP: ALVARO CEBALLOS 23-MAR-2015

SNC-LAVALIN

DESIGNED BY A. CEBALLOS	APPROVED Discipline Lead Engineer R. BOUCHARD
DRAWN BY R. MERRI	
VERIFIED BY R BOUCHARD	APPROVED Engineering Manager G. SNYDER
DATE 29-SEP-2012	SCALE 1:2000

CLIENT: nalcor energy

PROJECT: LOWER CHURCHILL PROJECT

TITLE: MUSKRAT FALLS NORTH SPUR STABILIZATION WORKS PLAN

File No. 505573-3281-4G00-0003-03

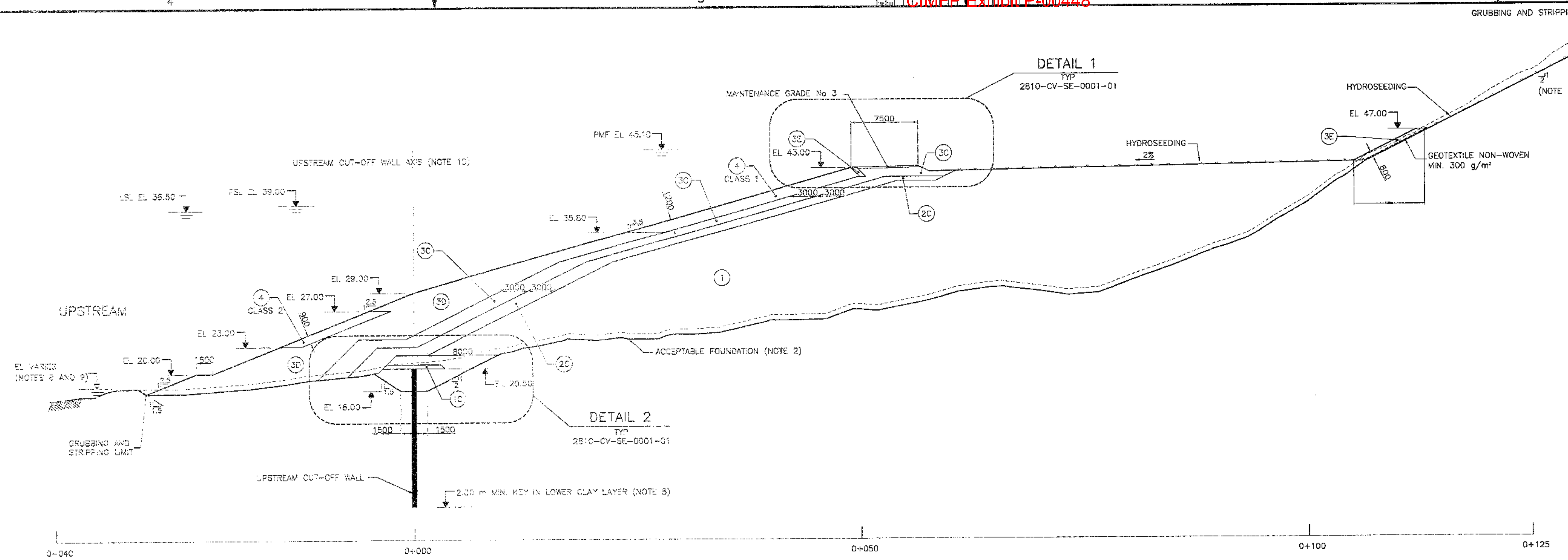
MFA-SN-CD-2800-CV-PL-0009-01 Rev. C2

FORMAT A0 (mm)

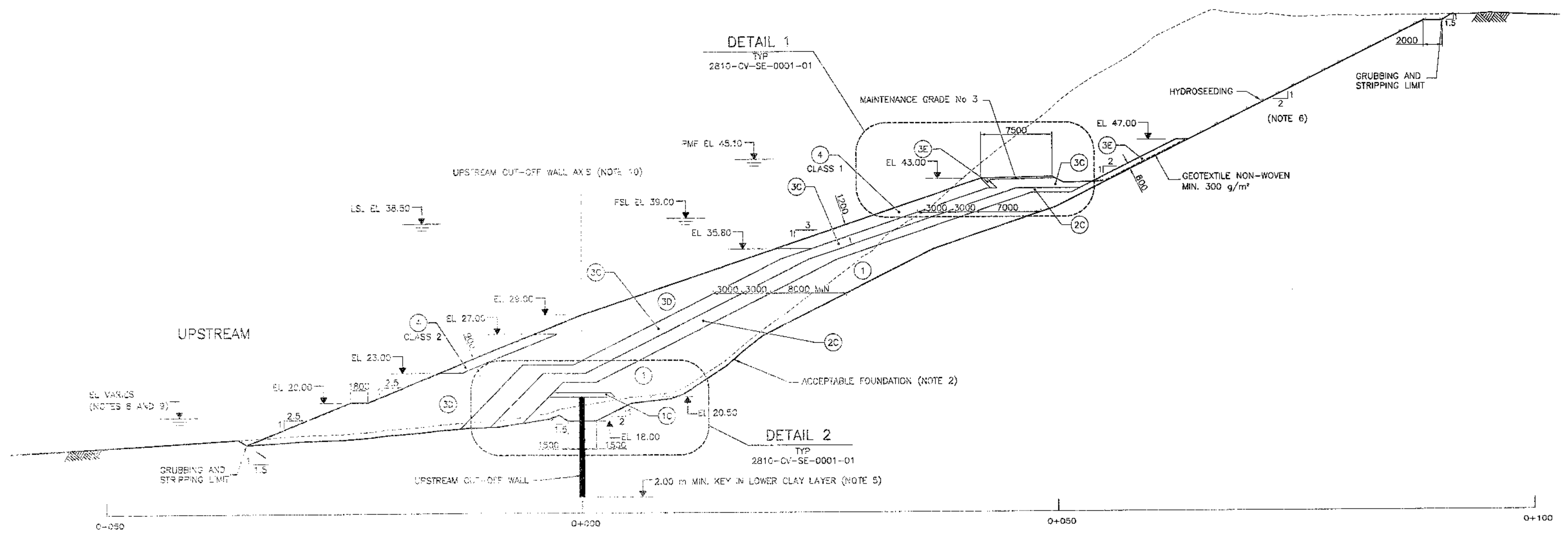
2015.03.30/10:07am

NOTE:

1. FOR NOTES AND LEGEND SEE DRAWING MFA-SN-CD-2810-CV-SE-0001-01



TYPICAL CROSS SECTION 3 (STA 0+200 TO 0+480)
2800-CV-PL-0009-01



TYPICAL CROSS SECTION 4 (STA 0+480 TO 0+797)
2800-CV-PL-0009-01

PROVINCE OF NEWFOUNDLAND PEG PERMIT HOLDER CLASS "A"
SNC-LAVALIN Inc.

0 5000 10000 15000 mm
SCALE 1:200

No.	Reference Drawing
MFA-SN-CD-2800-CV-PL-0003-01	PERMANENT ACCESS ROAD – PLAN
MFA-SN-CD-2800-CV-SE-0001-01	NORTH-WEST AND UPSTREAM CUT OFF WALLS SECTIONS AND DETAIL
MFA-SN-CD-2800-GT-SD-0001-01	EMBANKMENT MATERIALS GRAIN SIZE DISTRIBUTION
MFA-SN-CD-2800-CV-SD-0003-01	HYDROMETEOROLOGICAL DATA
MFA-SN-CD-2810-CV-SE-0001-01	UPSTREAM AREA TYPICAL CROSS SECTIONS SHEET 1 OF 4
MFA-SN-CD-2800-CV-PL-0009-01	NORTH SPUR STABILIZATION WORKS – PLAN

ISSU.	REV.	DATE	DISTRIBUTION & STATUS
2	C1	25-JUL-2014	ISSUED FOR CONSTRUCTION
1	B1	30-Nov-2012	ISSUED FOR BID

ISSUE REGISTER

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- ☐ 3. REVIEWED - NOT ACCEPTED
- ☐ 4. INFORMATION ONLY
- ☐ 5. NOT REVIEWED

LEAD REVIEWER: Date (dd-mmm-yyyy): 30-JUL-2014
NE-LCP MANAGEMENT: Date (dd-mmm-yyyy): 31-JUL-2014
PROJECT MANAGER: Date (dd-mmm-yyyy): 31-JUL-2014

PROFESSIONAL STAMP
PEG ALVARO CEBALLOS 21-JUL-2014

SNC•LAVALIN

DESIGNED BY A. CEBALLOS	APPROVED Discipline Lead Engineer R. BOUCHARD
DRAWN BY R. MERRI	APPROVED Engineering Manager G. SNYDER
VERIFIED BY R. BOUCHARD	
DATE 29-SEP-2012	SCALE 1:200

CLIENT: nalcor energy

PROJECT: LOWER CHURCHILL PROJECT

TITLE: MUSKRAT FALLS
NORTH SPUR STABILIZATION WORKS
UPSTREAM AREA
TYPICAL CROSS SECTIONS
SHEET 2 OF 4

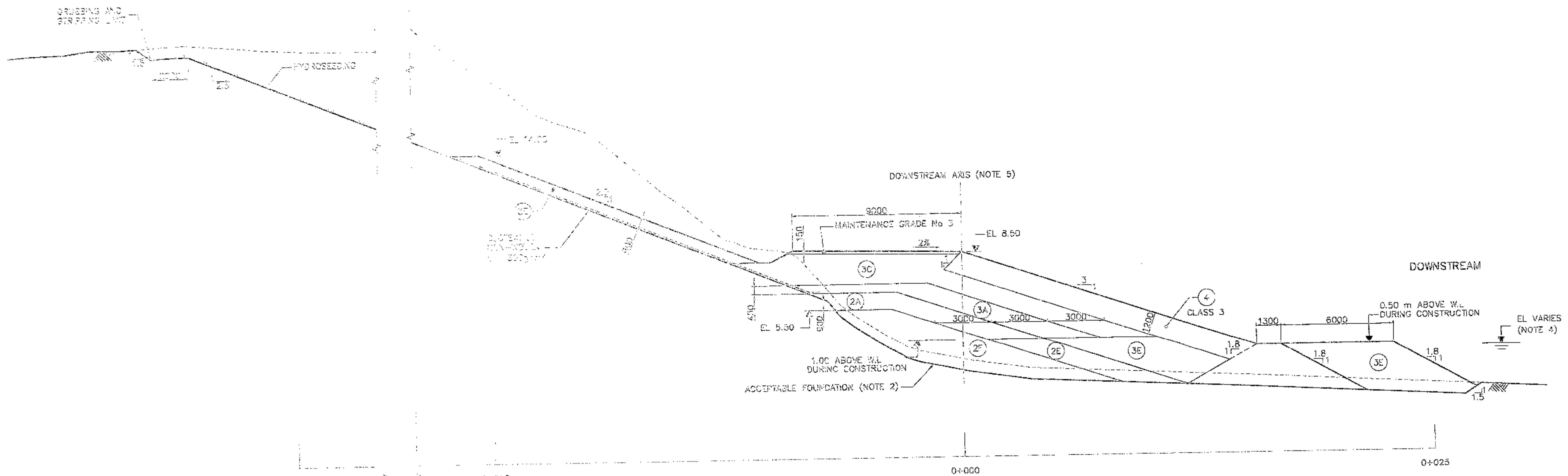
SLI DOC No. 505573-3281-40DD-0004-SH2-01
NE DOC No. MFA-SN-CD-2810-CV-SE-0001-02 REV. C1

SUB-PKG: 0008-4G02 PLATE 05

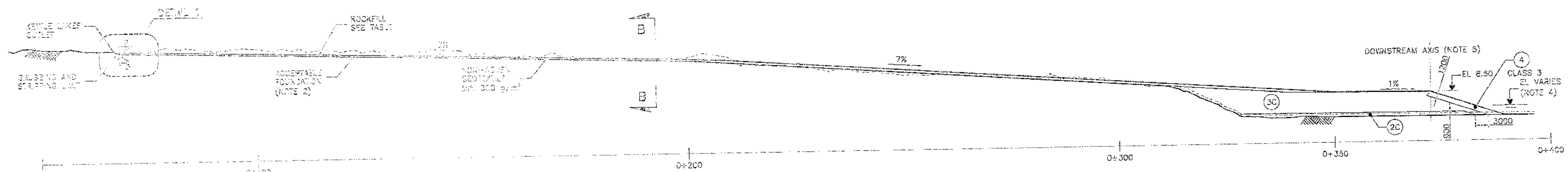
CIMFP Exhibit P-00448

NOTE:

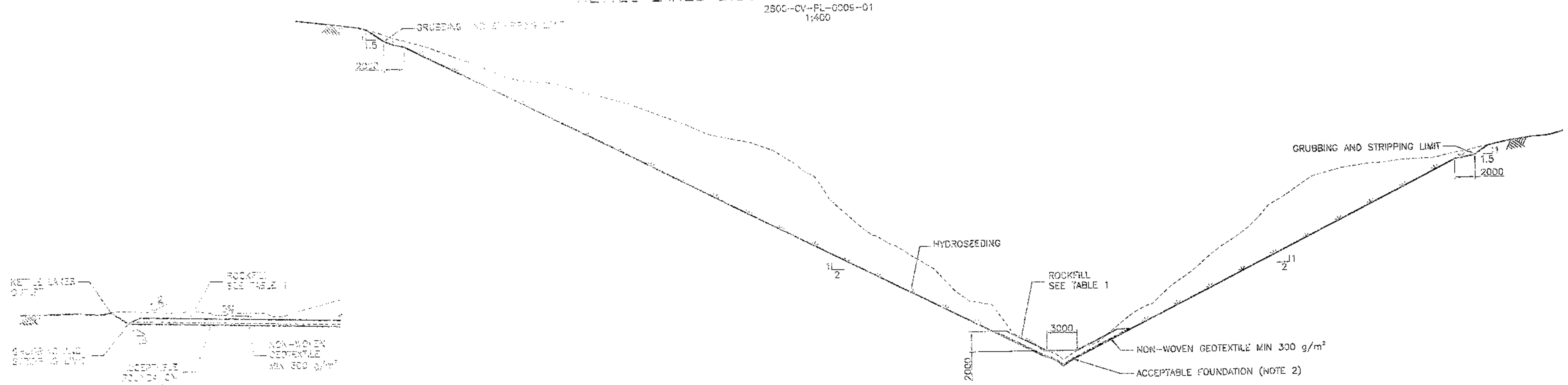
1. FOR NOTES AND LEGEND SEE DRAWING MFA-SN-CD-2820-CV-SE-0001-01



TYPICAL CROSS SECTION 13 (STA 0+420 TO 741)
2800-CV-PL-0009-01
1:100



KETTLE LAKES DISCHARGE CHANNEL, SECTION A–A
2800-CV-PL-0005-01
1:400



TYPICAL CROSS SECTION
SECTION B–B
1:200

DETAIL 1

TABLE 1

KETTLE LAKES DISCHARGE CHANNEL CHARACTERISTICS OF SELECTED ROCKFILL

SLOPE %	ROCKFILL				
	TYPE	DIMENSION mm			THICKNESS mm
		D_{min}	D_{MAX}	D_{50}	
3.0	1	100	250	150	300
7.0	2	200	400	300	600

SCALE 1:100
SCALE 1:200
SCALE 1:400

PROFESSIONAL STAMP

SNC·LAVALIN

CLIENT: nalcor energy

DESIGNED BY: A. CEBALLOS
DRAWN BY: R. MERRI
VERIFIED BY: R. BOUCHARD
DATE: 29-SEP-2012
APPROVED: Discipline Lead Engineer R. BOUCHARD
APPROVED: Engineering Manager C. SNYDER
SCALE: AS NOTED

PROJECT: LOWER CHURCHILL PROJECT

TITLE: MUSKRAT FALLS
NORTH SPUR STABILIZATION WORKS
DOWNSTREAM AREA
TYPICAL CROSS SECTIONS
SHEET 3 OF 3

SLI DOC No. 505573-3281-4GDD-0006-SH3-01

NE DOC No. MFA-SN-CD-2820-CV-SE-0001-03 C1

SUB-PKG: 0008-4302 PLATE 14