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# Lower Churchill Project

# ENGINEERING REPORT
# North Spur – Post Construction Assessment

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# 1 INTRODUCTION

## 1.1 GENERAL PROJECT DESCRIPTION

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

At the site, the river flows in a rock controlled channel to the south of the Rock Knoll. The two falls (Muskrat Falls), about 1 km apart, cause a drop of about 14 m in the Churchill River level from el. 17 m on the upstream side to el. 3 m on the downstream side of the falls. The development comprises, a roller compacted concrete dam, a gated overflow spillway, the powerhouse and a rockfill dam which closes the reservoir on the right bank. The switchyard and the converter station are located southeast of the powerhouse. On the opposite (north) side of the Rock Knoll, a wedge shaped piece of land forms what is referred to as the North Spur, which is a natural dam and it contributes to the reservoir retention.

Access to the hydroelectric development is assured from the south bank while the access to the North Spur is gained from the Trans Labrador Highway.

General layout of the hydroelectric development is shown in drawing MFA-SN-CD-2800-CV-GA-0002-01. See Figure A-1 in Appendix A.

## 1.2 DESCRIPTION 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 maintained for the life of the project.

The North Spur is about one kilometre long between the Rock Knoll in the south and the Kettle Lakes in the north which represent natural boundaries to the North Spur, in terms of both seepage and stability (Figure 1-1).

The early (1965) studies for the Muskrat Falls site recognized the importance of the North Spur as part of the reservoir retention works. A major slide in November 1978, on the downstream face of the Spur, (Figure 1-1), revealed the fragility of this natural deposit and its susceptibility to toe erosion. Maintaining the integrity of the Spur is fundamental to the viability of the project and this fact has been understood from the outset.

## 1.3 PURPOSE OF THE PRESENT REPORT

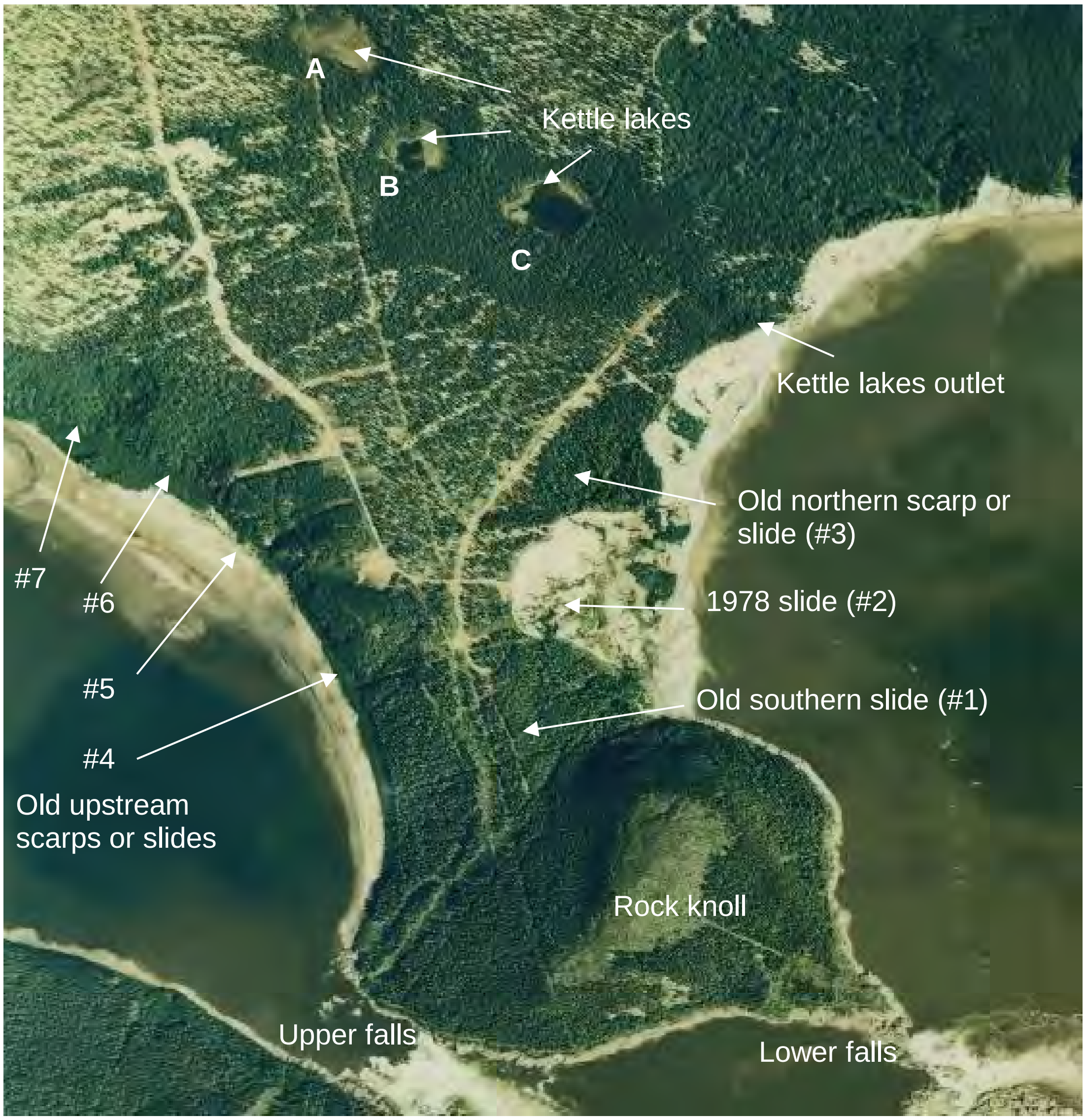
The raising of the headpond and the change in the downstream flow regime for the Muskrat Falls hydro-electric development could adversely affect the stability and integrity of the North Spur. Stabilization works to address this issue have been considered from 1965 to date and the concept has been modified and adapted over the decades to arrive at the current design.

The stabilization works were completed in 2017. Progress on other components of the Muskrat Falls development has resulted in the planned partial raising of the reservoir to el. 23 m and various aspects of the instrument behaviour in the North Spur have been monitored. The purpose of this report is to present:

- A summary of the site characteristics;
- The approach adopted to design the stabilization measures;
- A description of the works;
- The observations to date;
- A review of the conditions to validate the hypotheses used in certain specific studies;
- A revision, as and where necessary, of these studies;

- Recommendations as to any additional work that should be undertaken prior to proceeding with the raising of the reservoir to Full Supply Level (39 m).

**Figure 1-1 : Aerial photo of the North Spur (1988)**



# 2 SITE CHARACTERISTICS

## 2.1 GEOLOGY AND TOPOGRAPHY

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. In the Pliocene, the area was uplifted and the river, descending from the interior plateau, incised a valley through 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 glacial incursions with intervening periods of complete de-glaciation, when the sea re-occupied the valley which had been depressed under the burden of ice.

At some time in the geologic history, the river flowed through a deep gorge to the north of the Rock Knoll. The 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 geomorphologic features. The main deposition, constituting the current landforms, dates from the last glaciation. The buried valley was only detected by the early investigations for the project. 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. The stratigraphic sequence at the North Spur, is shown on Figure 2-2

With the final retreat of the ice, the lower aquifer sands and gravels were deposited first. Under the North Spur, this deposit can have a thickness of up to 160 m. Note that in some areas, moraine is encountered on the bedrock but this is not general. Then followed a period of rising sea levels and a marine environment prevailed in the region allowing the lower clay to be deposited. The deep water fine material, (Lower Clay) reaches a thickness of about 50 m in the vicinity of the Spur. The land began to rebound as the ice load diminished. Consequently, the depth of water decreased and run-off from precipitation and further ice melting created highly variable and seasonal flow conditions under which the stratified drift was deposited and includes intermediate sandy-silt and upper clay horizons.

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

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. The current river alignment passes to the south of the Rock Knoll.

The Churchill River valley, for the most part, runs through a wide valley characterized by extensive surface terraces of overburden material. The exposed overburden consists of sands overlying silty sands, silty and clayey soils as described above. In general the land is higher in the North due to a greater number of contributory streams bringing sediments during depositional phases, which led the river to adopt an initial alignment to the south of the Rock Knoll. The Rock Knoll protected the North Spur from erosion, as the river eroded the land to the east and west of the site. However, the erosion and landslides adjacent to and within the spur have continued and are still active.

The main terrace of the Spur is, in general, at an elevation of 60 m or slightly higher and represents an ice-contact stratified drift deposit. The deposit is characterized by the presence of kettles, local slumping and abrupt stratigraphic change within the first 50 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. The relative surface uniformity is broken by mass wasting features which may approach 1 $km^2$. In fact, the Churchill River valley is affected by large landslides over more than 75% of its length and some show regression for a distance 2 to 4 km.

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 to 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. However, at the southern limit, close to the Rock Knoll, the width of the crest of the North Spur reduces to 80 m.

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

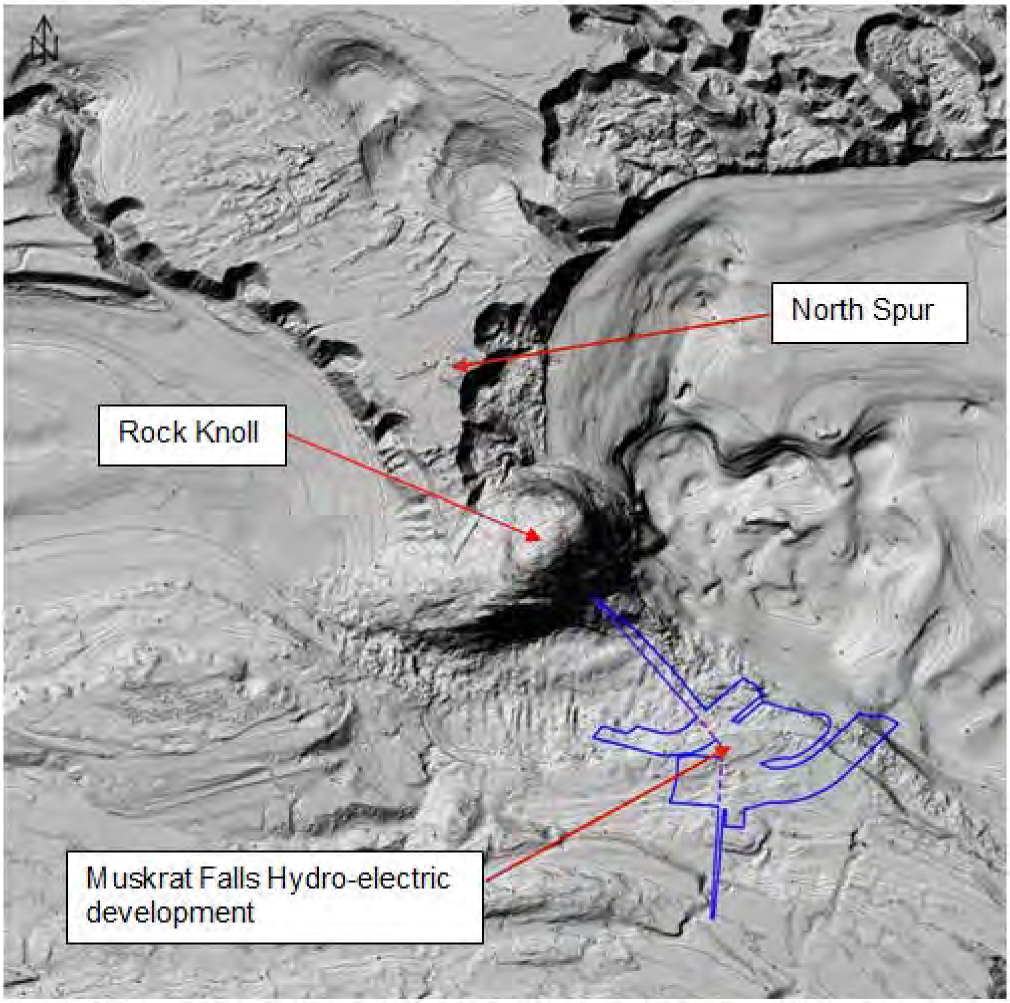
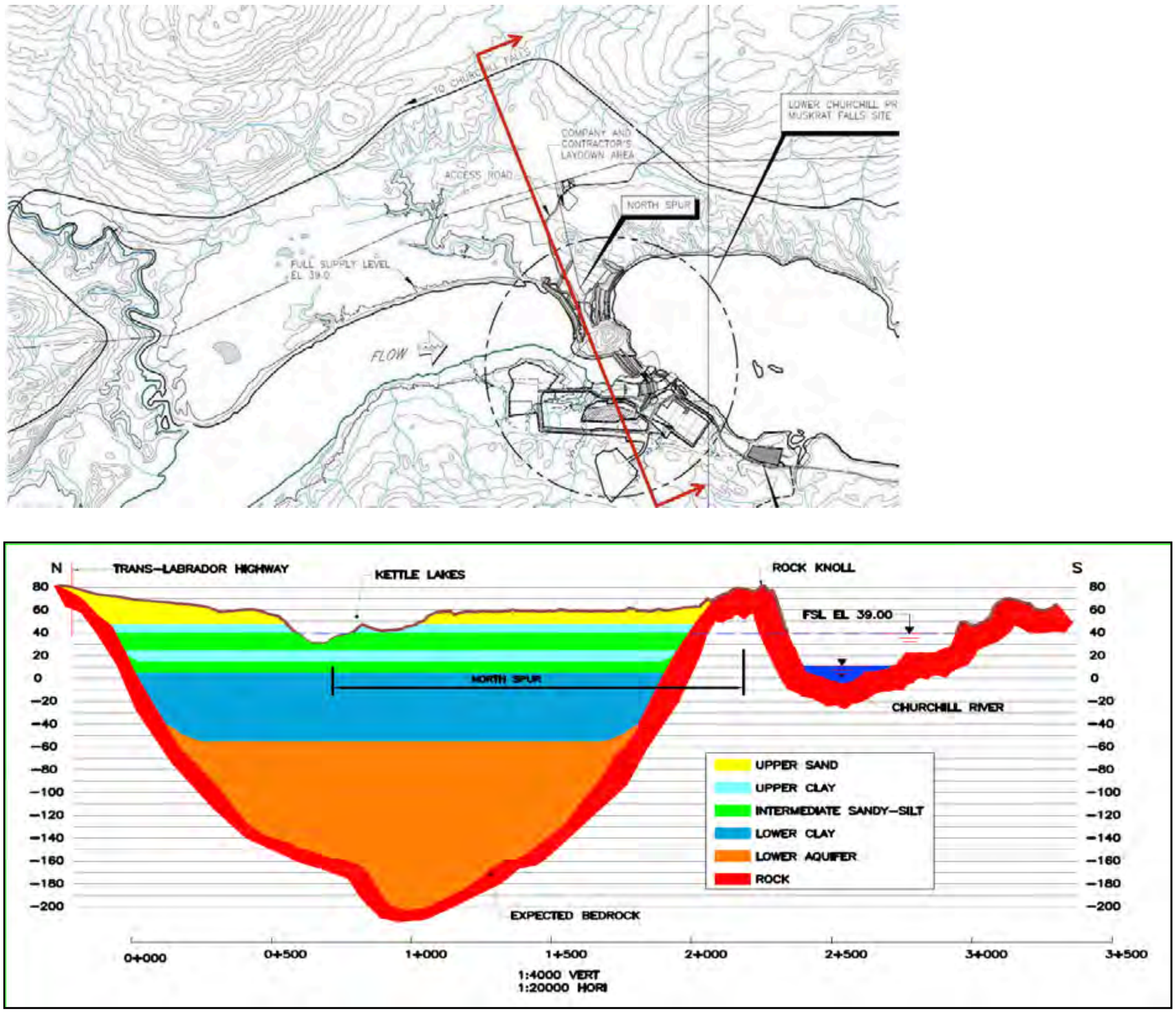


**Figure 2-2: North Spur Stratigraphy – North Spur Cross-Section**




At Muskrat Falls, the level of the river is vertically controlled by the upper falls but the lateral erosion process continues and causes landslides on both sides of the river. This phenomenon is also acting on the downstream side of Muskrat Falls. A deep erosion feature downstream of the lower fall can be seen in Figure 2-1. Under natural (pre-development) conditions, frazil ice forms in the many kilometers of rapids upstream. This frazil ice accumulated in the bay downstream of the falls and forced the water currents to adopt a path below the ice mass, thus producing atypical erosive flow conditions along the bed of the river despite the wide channel at this point. The lower aquifer sand and gravels are exposed in the bottom of this depression.

Most of the estuarine sediments, including the intermediate clay stratum mentioned above, were deposited in slow moving, non-turbulent water, and as a result, are primarily silt and clay size. Thin layers of fine sand, most of which are no more than sand partings, are common.

The water in which the estuarine sediments were deposited is believed to have been brackish rather than fully marine, as the location of deposition was near the head of an estuary and significant additions of melt water fed directly from the retreating ice. Salinity decreased further as the sea level dropped and the shoreline regressed. This brackish environment during deposition, plus the subsequent leaching of the salt from pore water contained in the soil, is known to have a pronounced effect on the physical properties of fine grained soil, such as these estuarine sediments. With the sensitive materials so formed, if a local instability occurs there is a possibility for it to degenerate into a retrogressive landslide.

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 rises to ground surface (elevation 200 m) north of the Trans-Labrador Highway.

## 2.2 GEOTECHNICAL DESCRIPTION OF STRATIGRAPHY

Most of the current knowledge of the Spur stratigraphy and the soil characteristics are derived from the outcomes of the investigation campaigns over the last 50 years but which were carried out mainly in 1979 and 2013. The boreholes and in-situ test locations are shown on Figure A-2 of Appendix A. The hydro-geological information has been obtained from pumping tests, piezometer readings, in-situ permeability tests and CPT dissipation tests.

The stratigraphic model has evolved over time; continuous logs obtained from CPTs and sonic drillings during the 2013 investigations along with conventional boreholes drilled during various investigations provided the data to study the stratified nature of the soil.

The simplified interpretation is illustrated on Figure 2-2; the major soil layers encountered are summarized as:

- Upper sand, generally from elevation 60 m to elevation 45 m to 50 m;
- Stratified drift, including two major deposits of silty sand/sandy silt and silty clay material, generally from elevation 45/50 m to elevation 5/15 m.
- Lower marine clay, generally from elevation 5/15 m to elevation -70 m
- Lower aquifer (pervious sand and gravel layer), generally from elevation -70 m to bedrock.

The nature and the physical and mechanical properties of each soil unit are summarized in the following sections. The stratigraphy is heterogeneous on the North Spur and can change locally.

### 2.2.1 Upper Sand Layer

The upper sand layer covers the surface on the North Spur. This layer mainly consists of compact to very dense, grey fine to medium sand with low fines content.

This layer is mostly dry and well drained except for a perched water table observed above the underlying clay or silty clay layer. No permeability tests were performed in this layer. Using grain size distribution curves and empirical relationships, a value of $1{\times}10^{-4}$ m/s was estimated as the hydraulic conductivity for this layer.

### 2.2.2 Stratified Drift

The stratified drift is a heterogeneous mix of clays, silts and sands with sub-horizontal layering from the marine and estuarine deposition. This unit consists of alternating layers of silty clay of low to medium plasticity (referred to as “upper silty clay”), and silty sand or sandy silt (“silty sand/sandy silt”).

#### *2.2.2.1 Upper Silty Clay Layer*

A low to medium plastic, sensitive, stiff to very stiff silty clay to clayey silt material has been observed within the stratified drift. A summary of material properties for this upper clay layer is presented in Table 2-1. The Liquidity Index values are above unity. The in-situ undrained shear strength obtained by Vane shear tests ranged from 35 to 135 kPa which indicates clay material of firm to very stiff consistency in an intact condition. The average effective stress shear strength parameters of $\phi'$=31° and c'=6 kPa were interpreted from the triaxial and Direct Shear Test (DST) test results.

**Table 2-1: Summary of Material Properties for Upper Silty Clay Layer**

| 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, $S_u$ kPa | 60 – 2 | – | – |
| Sensitivity, in-situ, $S_t$ | 1 – 36 | 10 | 43 |
| Large strain friction angle, $\phi'_{cv}$ $^o$ | 30 – 32 | – | – |
| Effective cohesion, c', kPa | 0 – 10 | – | – |
| Unit weight, $\gamma$ 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 | – | – |

#### *2.2.2.2 Intermediate Silty Sand/Sandy Silt Layers*

The results of sieve analyses on samples recovered from the intermediate silty sand/sandy silt layers indicated a generally fine silty sand material with an average of 27% fines content (passing sieve 0.08 mm). The standard penetration tests carried out in this layer resulted in N values generally higher than 50, which indicates that the layers are in a dense to very dense compacted condition. Three consolidated undrained triaxial tests were conducted on samples from the silty sand/sandy silt layers, during the 1979 investigations, which gave an average effective friction angle ($\phi'$)of 35° to 37° and effective cohesion (c') of 0 kPa under large strain conditions. Two direct shear tests were completed on silty sand and sandy silt samples from borehole NS-1-13, between elevations 28 and 38 m, which resulted in average values of $\phi'$=35° and c'=0.

The presence of silty clay or clayey silt strata interbedded with the intermediate silty sand layer influences permeability test results with values from $10^{-7}$ to $10^{-9}$ m/s.

### 2.2.3 Lower Marine Clay Unit

The lower clay layer is located below the stratified drift and above the lower aquifer (lower sand and gravel layer). This layer consists of clay of low to medium plasticity which exhibits lower values of liquidity index than the upper clay layer and can be classified as slightly sensitive.

The consistency of the clay is stiff to very stiff with in-situ undrained shear strength of 53 to 200 kPa. A summary of material properties for the lower clay layer is presented in Table 2-2.

Based on CPT and vane data, the undrained shear strength at a given elevation is generally similar throughout the North Spur with the Over-Consolidation Ratio (OCR) at about 1.0 below the crest and between 3 and 15 below the upstream and downstream toe where the overburden stresses have been reduced by erosion. It should be noted that some of the material at shallow depth below the toe in areas subjected to previous slides may be in a remoulded state.

**Table 2-2: Summary of Material Properties for Lower Clay Layer (1979 Investigations)**

| 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, $S_u$, kPa | 8 – 96 | _ | _ |
| Sensitivity in-situ, $s_t$ | 2 – 11 | 4 | 35 |
| Large strain friction angle, $\phi'_{cv}$ ° | 33 | _ | _ |
| Effective cohesion, c', kPa | 6 | _ | _ |
| Salt content, g/l | 8 – 22 | _ | 8 |
| Unit weight, $\gamma$ , kN/m$^3$ | 19.2 – 19.5 | _ | 3 |
| Hydraulic Conductivity, k, m/s | $10^{-7}$ – $10^{-9}$ | _ | _ |

### 2.2.4 Lower Sand and Gravel Unit (Lower Aquifer)

The lower aquifer layer is located below the lower clay layer and above the bedrock. It consists of sand and gravel with some cobbles and boulders with a fines content (silt and clay) between 5 and 40%.

Some samples from this layer were found to contain a high proportion (75%) of fines but pumping tests indicated an average coefficient of permeability of $10^{-4}$ m/s and consequently this unit is expected to have, on average, a relatively low fines content.

### 2.2.5 Bedrock

The type of bedrock is generally granite gneiss with pegmatite intrusions. Its depth is very variable across the North Spur as can be observed on Figure 2-2.

# 3 STABILIZATION MEASURES

## 3.1 CONCEPTS AND OBJECTIVES

As mentioned above, assurance of the long-term stability of the North Spur is paramount for the viability of the Muskrat Falls development. Prevention of both large scale and superficial landslides is the objective. Instability in the stratified and sensitive deposits, of which the North Spur is comprised, are related to a high ground water table or other instances of high pore pressures, and initiated principally by erosion at the toe of the slopes. Overloading of the head of a potential slide mass has also to be considered but this is not the usual case under natural conditions.

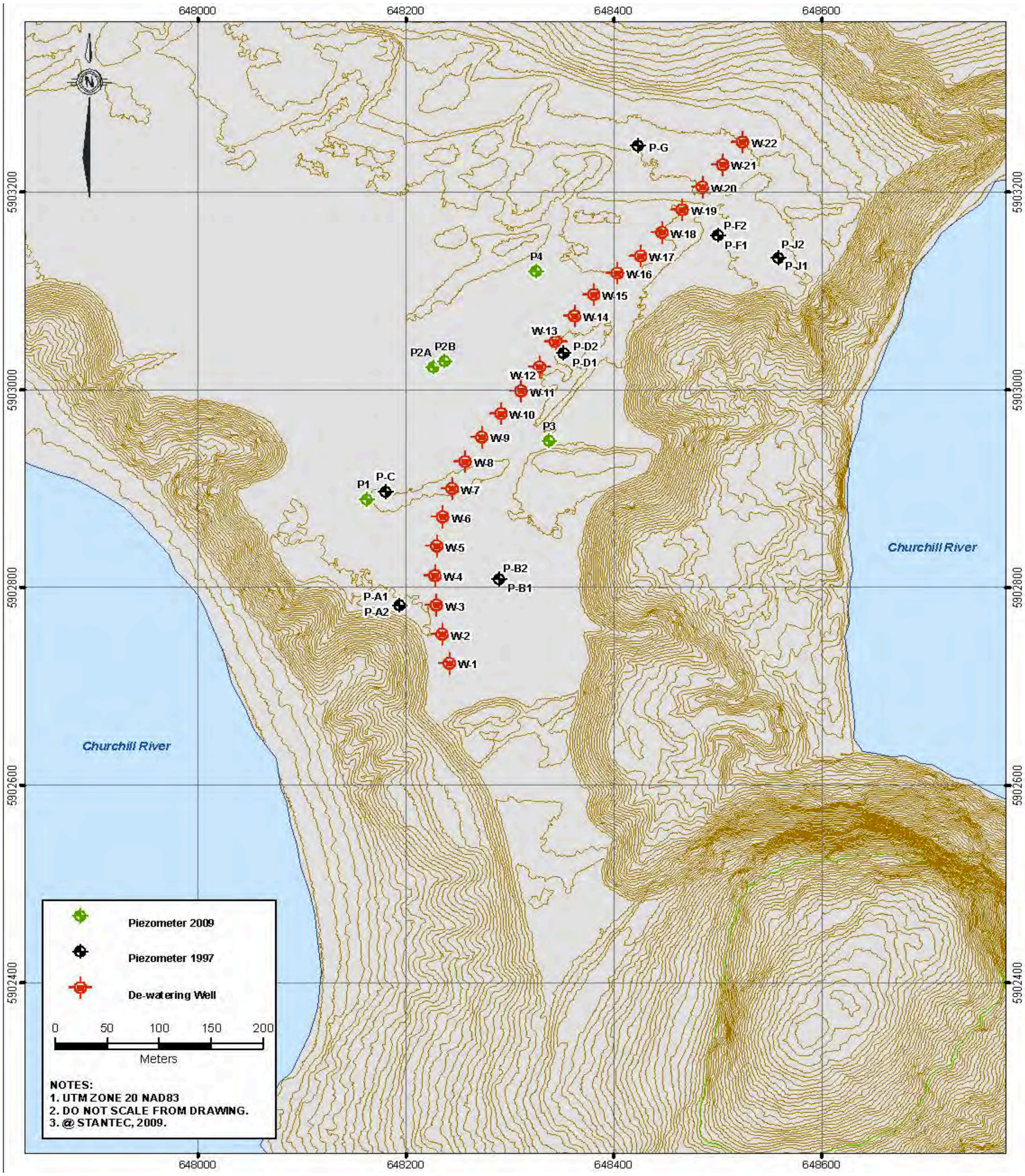
The concepts for the stabilization measures have gone through several iterations since the project was first identified but all include components to:

- Lower the ground water table, including perched water tables, and reduce pore pressures in the soil mass;
- Limit the ingress of seepage from the reservoir;
- Flatten critical slopes, and add embankment fill to enhance stability;
- Protect the toe areas against erosion.

Consideration has also to be given to the potential impact of earthquakes despite the fact that the region is one of low seismic activity.

After the occurrence of the 1978 landslide, temporary measures were deemed necessary to prevent a deterioration of the status of the North Spur until such time as permanent works could be carried out as part of the overall project development. A line of 22 pump wells, installed from the crest of the Spur, was judged to provide sufficient relief of pore pressures. The layout is shown on Figure 3-1. This system has operated since 1981.

**Figure 3-1: Layout of Relief Wells and Piezometer Installations (2009)**



## 3.2 DESIGN OF PERMANENT STABILIZATION WORKS

### 3.2.1 Hydrogeologic Conditions Prior to the Works

Interpretation of the stratigraphic and hydrogeological data permitted the identification of three 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. This water table is mainly recharged by precipitation and water infiltration from top of the Spur. Due to higher permeability of this layer the ground water easily drains towards the upstream, downstream, and to the Kettle Lakes slopes at elevations 40 to 45 m.

A second aquifer, labelled as the “intermediate aquifer”, was identified inside the stratified drift unit. The average piezometric contours are shown in Figure 3-2 indicating water equivalent levels of 30 to 35 m in the center of the North Spur. Under the natural conditions (prior to reservoir filling), a hydraulic gradient can be observed in this figure from the NW and subsequently to the upstream and downstream slopes of the North Spur. In addition, due to a significant vertical gradient, a downward ground water flow, from the perched aquifer to the intermediate aquifer and to the lower aquifer, was identified.

Finally, overlying the bedrock and limited in the upper part by the lower marine clay unit (generally below elevation -70 m), the “lower aquifer” was identified. (Figure 3-3) The piezometric levels in this aquifer were measured during investigations to vary from 15 m and 13.5 m on the upstream side to 4.3 m on the downstream side of the North Spur. As previously described, the river bathymetry shows that a deep depression, with a minimum elevation at about –60 m, exists in the bay downstream of the Spur. Considering the top elevation of the lower aquifer, it is expected that the lower aquifer is connected to the downstream side of the river in this area. The connection is well demonstrated by comparing the variation of the water pressure in the lower aquifer with the variation of the downstream water level in the river. This is illustrated on Figure 3-3 showing the piezometric contours estimated for the lower aquifer unit.

**Figure 3-2: Intermediate Aquifer – Piezometric Contours (1980)**

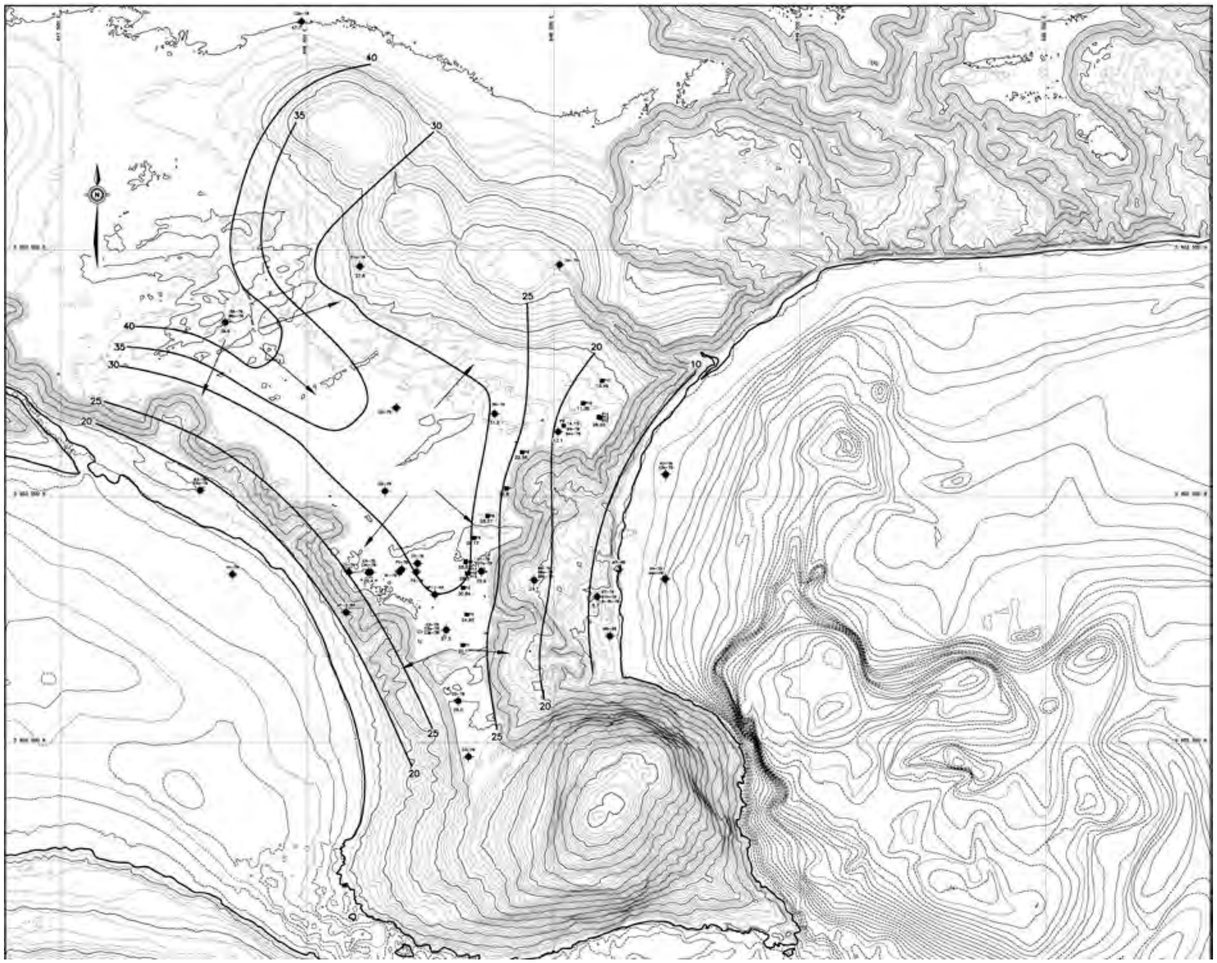
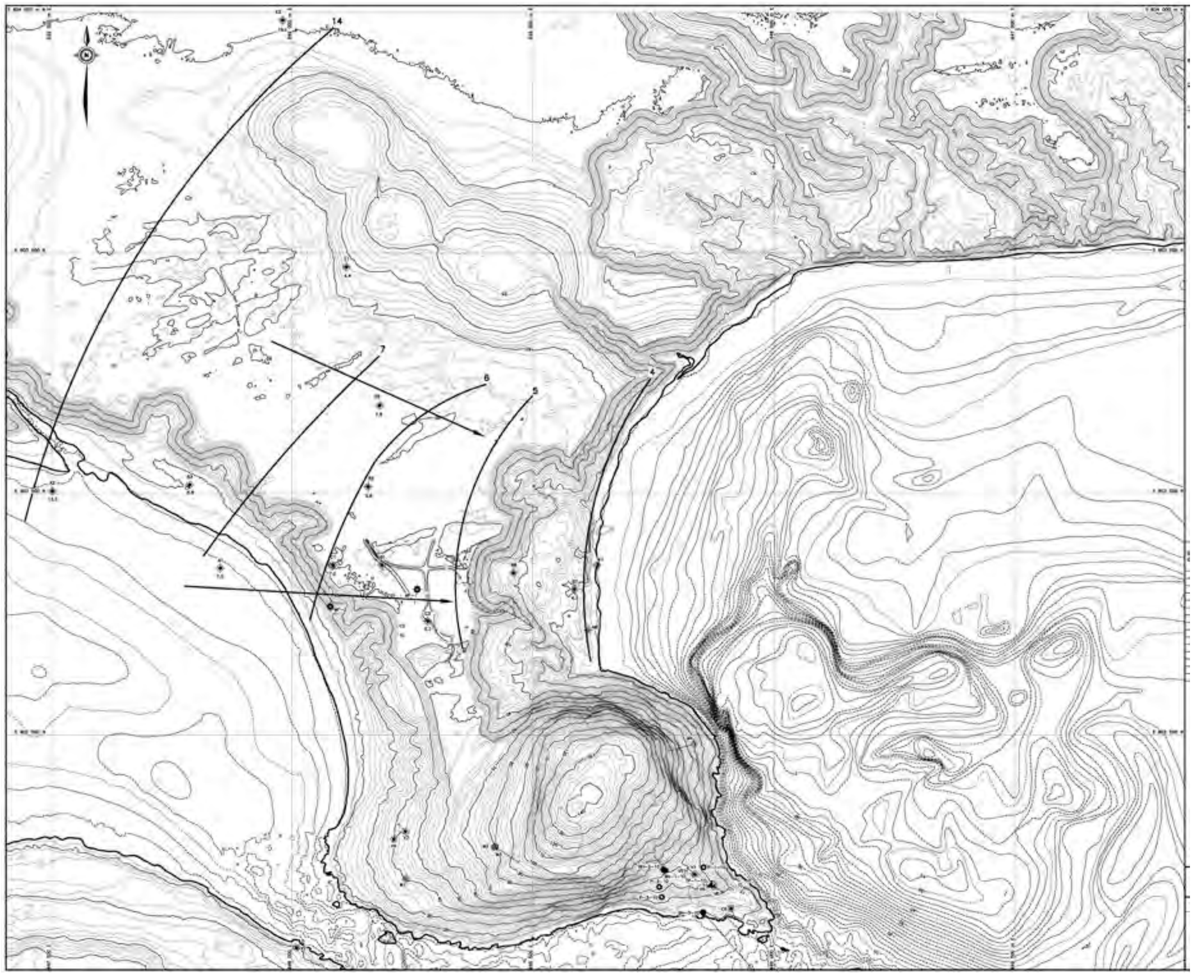


**Figure 3-3: Lower Aquifer – Piezometric Contours (1980)**



Figures 3-2 and 3-3 were prepared in 1980 based on piezometric data measured before influence of the dewatering system installed to reduce the water level in the Intermediate Aquifer. The pump wells had a significant effect in the southern and central part of the downstream side of the North Spur. The wells in the northern part had limited impact but recent refurbishment has increased the yield.

The pump well system, due to the power requirements, operation and maintenance activities etc. is classified as an active system. For long term operations, a passive system is judged to be preferable though continued use of the pump wells has not been discounted at this time.

### 3.2.2 Cut-off walls

In order to reduce the inflow to the North Spur from the NW, a cut-off wall has been incorporated into the design. The orientation of the wall is SW-NE and the depth was established so as to penetrate into the lower clay.

Similarly, in order to reduce infiltration from the reservoir side, a cut-off wall has been constructed from the toe of the upstream slope down to the low permeability clay. The potential for infiltration into the upstream slope from the toe to FSL is addressed by the inclusion of an upstream till blanket. The location of the various components can be seen on Figure 3-4 and in greater detail in Figure A-3

### 3.2.3 Drainage

As mentioned above, a key element in the temporary works to stabilize the North Spur was the series of pump wells to lower the water table. Components of the permanent works have also to play a similar role. A large part of the downstream slope is covered with slide debris. This disturbed material forms a low permeability blanket that hinders drainage. Finger drains installed in trenches cut into the debris have therefore been included in the works. (Figure 3-5) These drain out through the rockfill toe berm.

The flow characteristics for the outflow from the Three Kettle Lakes have been improved by slope trimming and streambed protection in the channel.

**Figure 3-4: North Spur Stabilization Works**

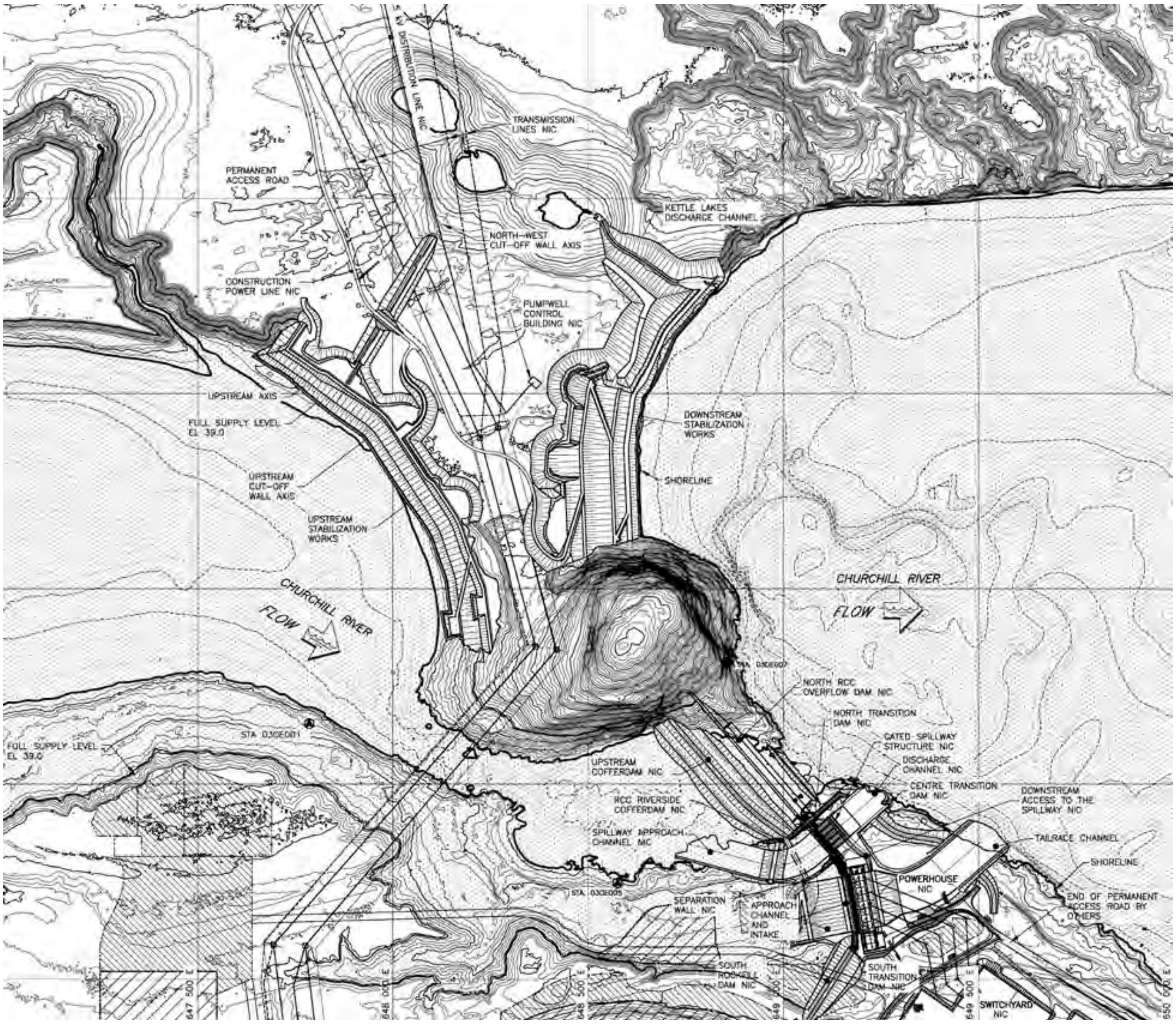
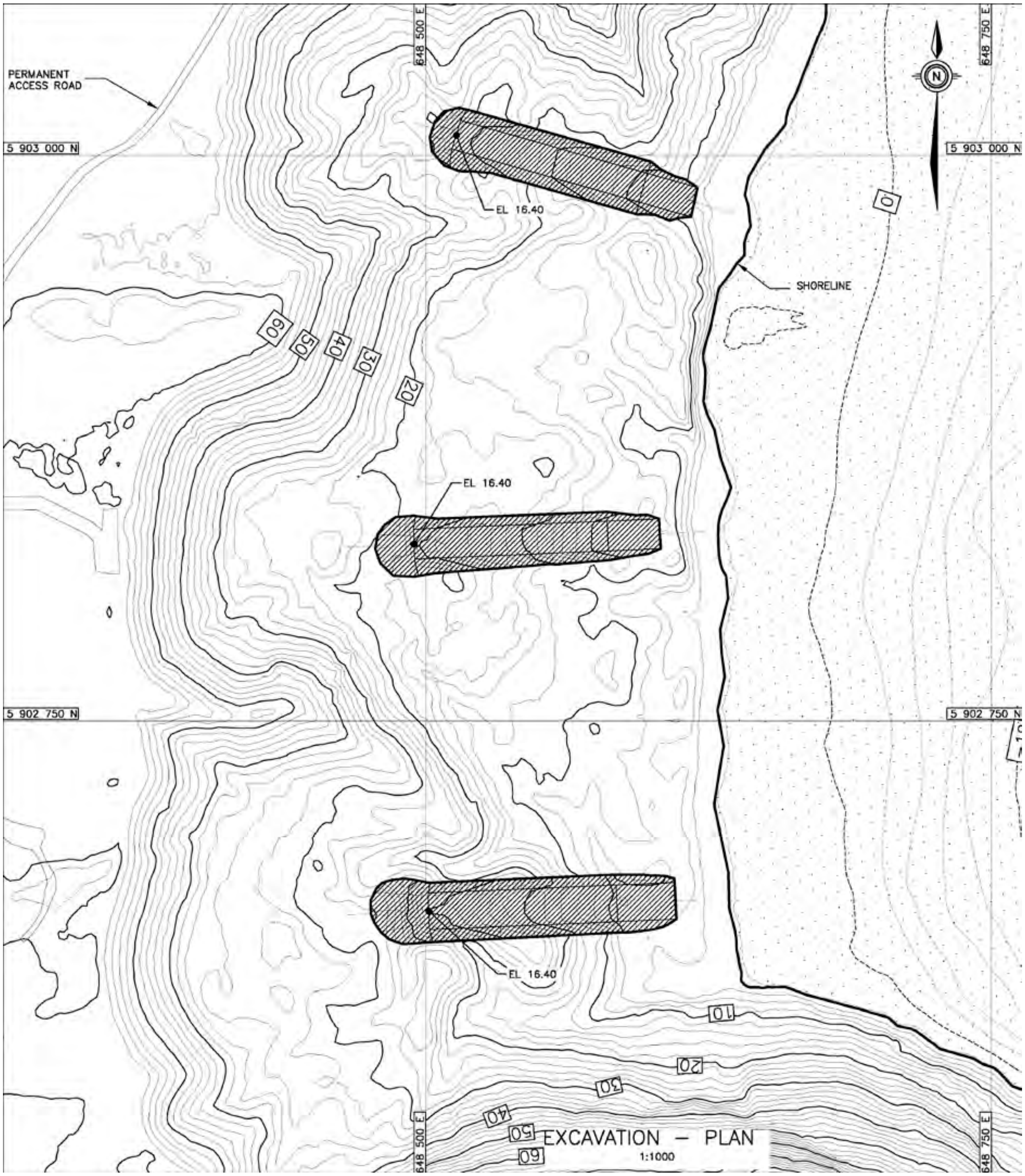


**Figure 3-5: Finger drain location**



### 3.2.4 Slope Stability

Many slopes along the Churchill River banks, including those of the North Spur prior to the stabilization works, are only marginally stable. That is to say with a Factor of Safety of around 1.0. A typical area was observed on the North Spur immediately to the south of the Kettle Lakes outlet channel. Some superficial sliding was noted during a site visit in 2011 and further movement was noted until the realization of the stabilization works.

Slope stability analyses were carried out as part of the design studies and the sections analysed included this area. Back analysis permitted a validation of the strength parameters which, otherwise, were obtained from the laboratory testing that was conducted as part of the 1979 and 2013 investigation campaigns.

In addition to the slope geometry, the soil stratigraphy and the material parameters, the location of the water table and the associated pore pressure distribution is required to carry out the slope stability calculations.

The works included slope cutting for selected steep slopes on both the upstream and the downstream sides and the addition of embankment fill for which the final configuration can be seen in plan on Figure 3-4.

Limit Equilibrium analyses were carried out on all the slopes to demonstrate the adequacy of the proposed works.

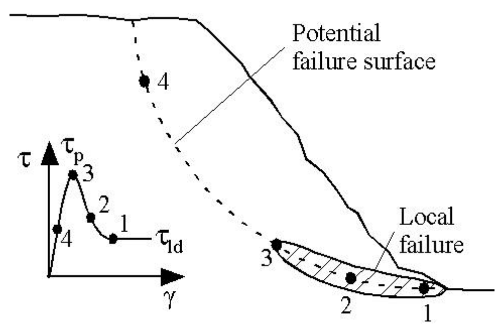
The results of these studies can be found in the Design Report. (MFA-SN-CD-2800-GT-RP-0004-01). This assessment includes a review of the hypotheses used in the Stability Analyses as will be presented in Section 6.1.

### 3.2.5 Progressive Failure

Sensitive clays from Eastern Canada show a strain-softening behaviour in shear and may therefore be susceptible to progressive failure. This issue has generated considerable debate on the subject of the Muskrat Falls development and will be described in detail in the subsequent paragraphs.

Considering the strain-softening behaviour of clays, Skempton (1964) described the failure mode occurring during progressive failure with the following statement: "[…] if for any reason a clay is forced to pass the peak at some particular point within its mass, the strength at that point will decrease. This action will throw additional stress on to the clay at some other point, causing the peak to be passed at that point also. In this way a progressive failure can be initiated and, in the limit, the strength along the entire length of a slip surface will fall to the residual value." Figure 3-6 illustrates how the shear strength along a potential failure surface may vary from peak shear strength to large-deformation shear strength. The soil in the potential sliding mass is therefore subjected to local failure when it reaches its peak shear strength (points 1 to 3 along the potential failure surface in Fig. 3-6), prior to global failure taking place when the entire failure surface is formed (Locat et al, 2011).

**Figure 3-6: Progressive failure along a circular failure surface (from Locat et al, 2011)**



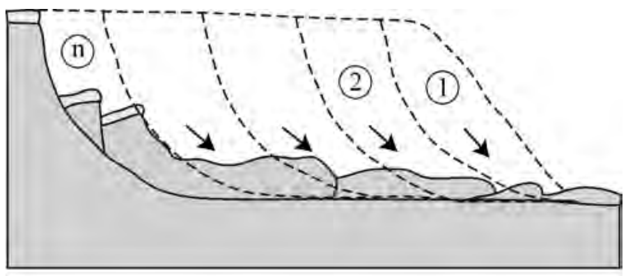

### *3.2.5.1 Single Rotational Landslides*

Rotational landslides can be single or can be the initial instability leading to flow slides. The single rotational landslide is characterised by the fact that a significant portion of the material displaced by the failure stays at the toe of the slope and stabilizes the slope, thereby preventing further instabilities.

### *3.2.5.2 Flowslides*

As indicated in Figure 3-7, a flowslide in sensitive clays results from a succession of slides. There must be an initial slide (failure (1) in Figure 3-7). If the potential energy due to the slump is large enough to remould the clay, it will flow out of the crater if the liquidity index is large enough or the remoulded shear strength small enough. The backscarp thus stands without being supported by debris. If this backscarp is unstable either in undrained or partly drained conditions, there will be another failure (failure (2) in Figure 3-7), and so on until a new condition of stability is attained.

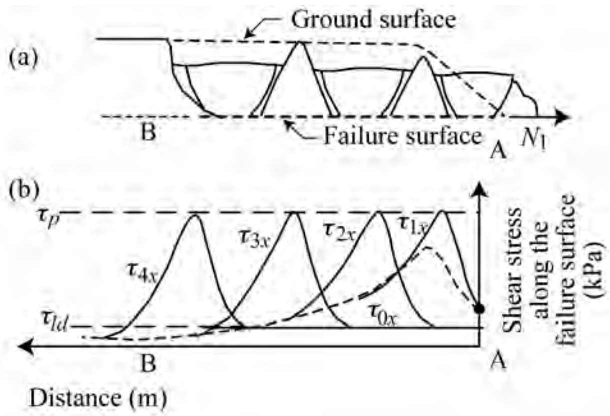
**Figure 3-7: Flowslide in sensitive clay (Locat et al, 2011)**




### *3.2.5.3 Other mechanisms*

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 as shown in Figure 3-8.

**Figure 3-8: Spread Landslide resulting from progressive failure in sensitive clay (Locat et al, 2011)**




Progressive failure may also develop as a result of loading on the slope or beyond the crest of the slope, such as above point B in Figure 3-8. If loading or piling generates a shear stress that locally exceeds the undrained shear strength (e.g. $\tau_{4x}$ in Figure 3-8b), failure may progress downward towards the toe of the slope (Point A in Figure 3-8) and generate a global failure.

Based on observations and past investigations, it is recognised that single rotational landslides and flowslides have occurred along the Churchill River but there is no clear evidence of spreads or other progressive failures.

### *3.2.5.4 Specific Studies*

The subject of progressive failure and the potential for its occurrence on the North Spur were the subjects of a detailed study released in December 2015 (MFA-SN-CD-2800-GT-RP-0001-01). The study included three principal components:

- Seepage analyses to estimate the water pressure distribution in the soil units forming the Spur and to calibrate the model against the observed piezometric conditions;
- Slope Stability analyses to estimate the effect of a surcharge at the crest of the Spur using the Limit Equilibrium Method (LEM)

- Stress Distribution analyses to estimate the stress distribution in the Spur assuming an elastic-plastic behavior model.

The results of this study demonstrated that, for post construction slope geometry and even for conservatively selected pore pressures, the stresses developed along potential failure planes do not exceed the peak strengths that can be mobilized and therefore, downward progressive failure is unlikely to occur. It is sufficient to show that the Factors of Safety against initial slope failure at the toe are adequate. Therefore, in the situation prevailing at Muskrat Falls, the Limit Equilibrium Method of analysis can be applied when establishing the adequacy of the stabilization measures.

However, as in the case of the general stability analyses, slope geometry, soil stratigraphy, the material properties and the pore pressures are the input parameters. The possibility of any significant change in the hypotheses used in the original study needed to be examined post construction and this task formed part of the assessment presented in this document.

### 3.2.6 Dynamic Analysis

#### *3.2.6.1 Preliminary Dynamic Study*

As part of the engineering design, a one-dimensional dynamic analysis was performed to assess the stability of the Spur in case of an earthquake. Based on this preliminary analysis, there was no issue for this aspect. However, an external review performed on the entire project recommended, apart from the dynamic analysis already performed, that the project team should carry out a detailed analysis to examine the impact of topographic effects and to assess cyclic strains.

#### *3.2.6.2 Scope of the Complementary Dynamic Study*

Based on recommendations, a complementary dynamic study was conducted to assess the dynamic stability of the North Spur in the long term, after the implementation of the recommended stabilization measures and after reservoir impoundment. This study was to be divided into six parts:

1. Selection of the most critical section;

2. Revision of the seismic hazard analysis to provide the appropriate Uniform Hazard Spectra (UHS);
3. Updated selection of representative input ground motions;
4. 1D equivalent-linear dynamic response analyses for crest and toe of slope vertical soil profiles;
5. 2D equivalent-linear dynamic response analyses for a cross-section representative of the most critical site conditions;
6. 2D non-linear dynamic response analyses for a cross-section representative of the most critical site conditions.

The details of this study are presented in the report entitled “North Spur Stabilization Work – Dynamic Analysis Study”, SNC-Lavalin, 2015 (MFA-SN-CD-2800-GT-RP-0007-01).

The seismic parameters were obtained from an updated Earthquake Hazard Analysis by Prof. Gail Atkinson.

The Cyclic Resistance Ratios for liquefaction (non-cohesive or sand like) or cyclic softening (clay like) of the various materials comprising the North Spur were obtained from industry standard interpretations of the available Cone Penetration Test (CPT) and Standard Penetration Test (SPT) results.

#### *3.2.6.3 Results*

The results indicate no potential for liquefaction of the granular materials or potential for cyclic softening of the clay at the level of the design seismic event. A cross-section was also submitted to 2D non-linear dynamic response analyses. These analyses confirmed the findings of the equivalent-linear analyses.

Based on the findings of this complementary dynamic study, the North Spur integrity is not expected to be affected by the occurrence of the design seismic event (probability of 1/10 000). This applies to the conditions that were considered and that are described in the report.

The validity of these conclusions are limited by the representativity of the conditions assumed that can differ from the site conditions. The behavior of the granular material can be very sensitive to the degree of saturation. It was assumed that the design earthquake would occur under normal seepage conditions. The normal elevation of the water table in the North Spur was estimated based on the conditions predicted by seepage analyses. The actual and predicted seepage conditions at the various stages of construction and of reservoir filling are examined in the assessment presented in the this document

### 3.2.7 Hydrogeological Study

From the foregoing, it is readily apparent that seepage and pore pressures are the primary conditions governing the long term integrity of the North Spur. In all the studies, pore pressure is one of the principal parameters impacting the results. The geometry of the Spur can be accurately surveyed. The various sections drawn to illustrate overall soil stratigraphy have a reasonably good confidence level though attempted interpolation between the various soundings to define layering in the intermediate stratified drift is difficult due to the heterogeneity. However, the pore pressure distribution and prediction of the reaction of the hydrogeological regime to rising reservoir levels are the most complex elements to determine, also for reasons of the heterogeneity.

To assist with the task of comprehending the regime, a three dimensional hydrogeological model was developed for the area of the North Spur (Hatch, October 2015).

The objectives of the study were to:

- Model and investigate the initial North Spur groundwater seepage patterns;
- Assess the impact of the stabilizing measures on piezometric levels through the body of the Spur at various reservoir levels;
- Provide model results to validate the stabilization works design;
- Provide model results that can be used to determine if the existing pumping system can be discontinued after implementation of the stabilization works;

- Evaluate the effect of reservoir impoundment on the Kettle Lakes at the north end of the Spur; and
- Provide a monitoring and forecasting tool for changes in the piezometric head within the geologic units of the Spur during construction, impoundment and operation.

The 3-D models were developed using the finite element software FEFLOW. One model was developed for the lower aquifer and one for the upper and intermediate aquifer deposits.

The lower aquifer is bounded by the underlying bedrock and the overlying marine clay, the materials are saturated, and thus the simulation is more straightforward in execution and offers a higher degree of confidence in the results.

As far as the intermediate aquifer is concerned, the model was created with six layers, namely:

- An upper sand layer;
- Upper silty clay layer;
- Upper silty sand layer;
- Lower silty clay layer;
- Lower silty sand layer; and
- A lower boundary at the marine clay.

The existence of low permeability slide debris on the downstream slope was acknowledged in one sub-variant.

Calibration of the models was carried out with the historic piezometric data pre-dating the pump well installations, short and long-term pump tests in the wells subsequent to their installation, and piezometric responses in the lower aquifer to a short term flood event in the 1980's. The materials were modelled as isotropic as far as permeability values are concerned. The layering within the intermediate stratified drift and the presence of the other major units is used to include the horizontal bias in the overall conductivity. In other words, although $k_{xx} = k_{yy} = k_{zz}$ for the individual layers, $K_{xx} = K_{yy} > K_{zz}$ for the overall soil mass.

Precipitation and snowmelt contribute to the inflow to the model. A vertical flux was applied to both models to account for this. (4% of annual precipitation for the intermediate aquifer and 0.2% for the lower aquifer)

As mentioned above, there is a noticeable hydraulic gradient (and consequently flow) from the NW part of the Spur. This was modelled as a flux on the northern boundary. Similarly, a gradient is present under natural conditions from the upstream side of the Spur to the downstream, and this will be increased by reservoir impounding. A flux was included as a boundary condition on the upstream side of the model to account for this flow.

Calibration involved varying the relative magnitudes of the two fluxes until reasonable piezometric matching was achieved. However, in addition to the inflow fluxes and the soil permeability values, it was found that the introduction of a downstream blanket of low permeability slide debris was necessary to simulate the conditions in the intermediate aquifer prior to stabilization works.

Considering the effects of heterogeneity, anisotropy, and variations in the infiltration and background piezometric levels during pump tests, the calibration of the models for natural conditions was deemed to be adequate.

The simulation of the anticipated response of the lower aquifer to reservoir impounding demonstrated that the piezometric levels beneath the downstream part of the Spur would rise by an average of about 5 m, but that such pressures would likely not be sufficient to justify the installation of artesian relief wells. Nevertheless, the results indicated the need to monitor the pressures in the aquifer by an adequate array of instruments.

For the intermediate aquifer, the various components of the stabilization works were modeled individually and collectively. In a comparison between the predicted piezometric levels for the reservoir at FSL under the conditions of no intervention, with the values obtained for various combinations of stabilization measures, the following was noted:

- The cut-offs led to a reduction of pressures at all points;

- A combination of cut-offs and upstream blanket led to an increase in pressures due to the reduced drainage to the upstream side (note that in the natural state, some pressures in the Spur exceed the planned FSL);
- A combination of cut-offs, U/S blanket and D/S drains was required to effect a general reduction; and
- The continuation of pump well operation was still beneficial when used in combination with the other components.

Similar findings were made for the intermediate reservoir level of 25 m. To date the maximum water elevation has been 23.5 m and a water level of 23 m has been maintained for a sufficiently long period to gain some appreciation of the piezometric levels under this condition. The readings and a discussion of the predicted values is to be found in section 6.4 of this report.

# 4 DETAILED DESCRIPTION OF STABILIZATION WORKS

## 4.1 GENERALITIES

The majority of the works were completed in 2015 and 2016.

**2015**

Clearing, partial upstream and downstream excavation, upstream and downstream embankments, the construction of the upstream cut-off wall, and a section of the northwest cut-off wall were performed in 2015.

**2016**

In 2016, excavation in both areas (upstream and downstream) was completed. Almost all embankment works were also completed in this period and the construction of the NW cut-off wall and the instrumentation installations were completed in 2016. Construction of the permanent access road and hydroseeding works started late in 2016 season and was suspended until 2017.

**2017**

All embankment construction, permanent access roads and hydroseeding works were completed in 2017.

The aerial photo used to as a basis for Figure 1-1 illustrates the vegetative cover on the North Spur prior to the works. Activities common to all areas were clearing and grubbing, and the construction of access roads and ramps.

Typically with works having a strong geotechnical influence, the knowledge of the ground conditions is never complete despite considerable effort being expended on investigation campaigns. The North Spur was no different. Observation of the conditions encountered during the works was necessary to identify any departures from the design hypotheses, areas where an increased scope of work was required or, conversely, where a less stringent approach would be appropriate. The As-Built drawings and the Construction Report (MFA-SN-CD-2800-GT-RP-0009-01) reflect these changes. Examples are the decisions to:

- Eliminate the bedrock grouting at the contact between the till blanket and the Rock Knoll;
- Incorporate slag cement in the slurry mix for the NW cut-off wall in order to extend the workable time. Note that additives such as Aquafix were also employed to improve properties of the slurry;
- Shorten the NW cut-off wall at its NE extremity;
- Adjust the geometry of the Finger Drains and the thickness of the downstream embankment fill;
- Add more piezometers to the array of instruments.

## 4.2 UPSTREAM WORKS

The works on the upstream side of the North Spur have the dual purpose of creating an impervious barrier against infiltration from the reservoir and stabilizing the natural slopes to prevent landslides.

The impervious barrier consists of two different components. The lower part includes a cement-bentonite cut-off wall (Upstream Cut-Off Wall) excavated through the pervious silty sandy foundation (stratified drift) from elevation 20.5 m and anchored at least 2.0 m into the lower clay layer unit.

The upper part of the impervious barrier is comprised of an inclined till blanket. The upper limit of the till blanket is at least 3 m higher than the FSL, thus preventing reservoir water from overtopping the impervious blanket. The till blanket was placed against the excavated/graded overburden foundation.

The base of the till blanket (zone 1) is connected to the top of the cut-off wall by means of a trench excavated to elevation 18.0 m. The upper contact between the cut-off wall and the till blanket is constructed of zone 1C to assure full bonding between these materials. Upstream typical cross-sections are presented in Figures 4-1 and 4.2.

**Figure 4-1: Upstream Typical Cross Section – Cut**

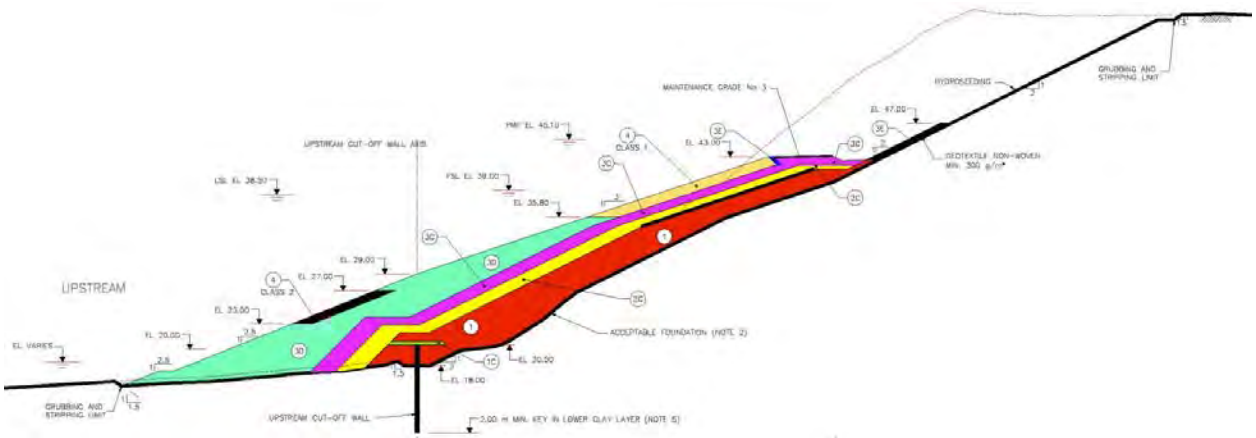
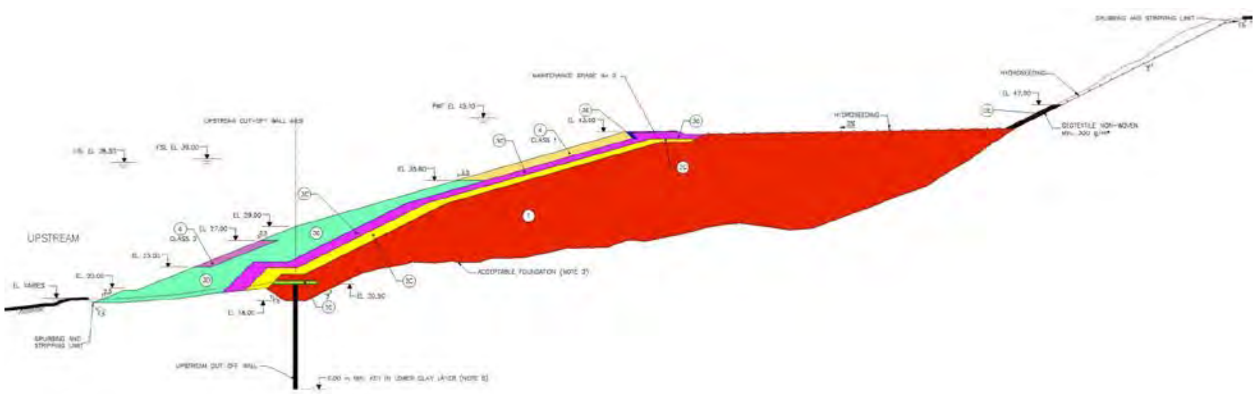



**Figure 4-2: Upstream Typical Cross Section – Fill**




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

For the loading case identified as “during construction and into the final impoundment period of the reservoir” (temporary condition), a factor of safety of 1.1 against uplift was considered assuming a piezometric level of 36 m behind the till blanket. The total weight of the fill materials counteracts the potential uplift pressures beneath the till blanket created by water trapped behind (downstream) the till blanket in the period following construction, and before the reservoir is filled to FSL.

The toe of the rockfill embankment, in contact with the foundation, is reinforced by means of a small berm, 1.8 m wide x 1.8 m high at elevation 20.0 m, constructed with rockfill (zone 3D above water level and zone 3E below water level) to prevent erosion of the embankment during the construction period.

Riprap protects the rockfill embankment against wave action and ice. In the lower part of the slope, the riprap protection (zone 4 class 2) extends from elevation 23 to 27 m, to cater for the period when the river water level will be raised to between elevations 23 m and 25 m to form a temporary headpond during diversion. This is necessary to encourage a stable ice cover during the winter. 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) from 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 slope in the event of the PMF.

The crest of the embankment is 7.5 m wide at elevation 43.0 m. This crest serves as a permanent access road and the surface layer is constituted of crushed stone (maintenance grade No 3 material).

The stabilization works at the north part of the upstream side, beyond the intersection of the upstream cut-off with the NW cut-off wall, are the same as described above except that no provision is made for seepage infiltration from the reservoir. The till blanket is replaced by granular fill material (zone 2G), a product of the overburden excavations.

A permanent access ramp was constructed on the upstream slope to link the top of the North Spur at elevation 60 m with the access road at elevation 43.0 m.

The Quality Control testing results can be found in the Construction Report but a few of the graphs are also included herein (Figures 4-3 to 4-5).

**Figure 4-3: Slurry properties for upstream cut-off**

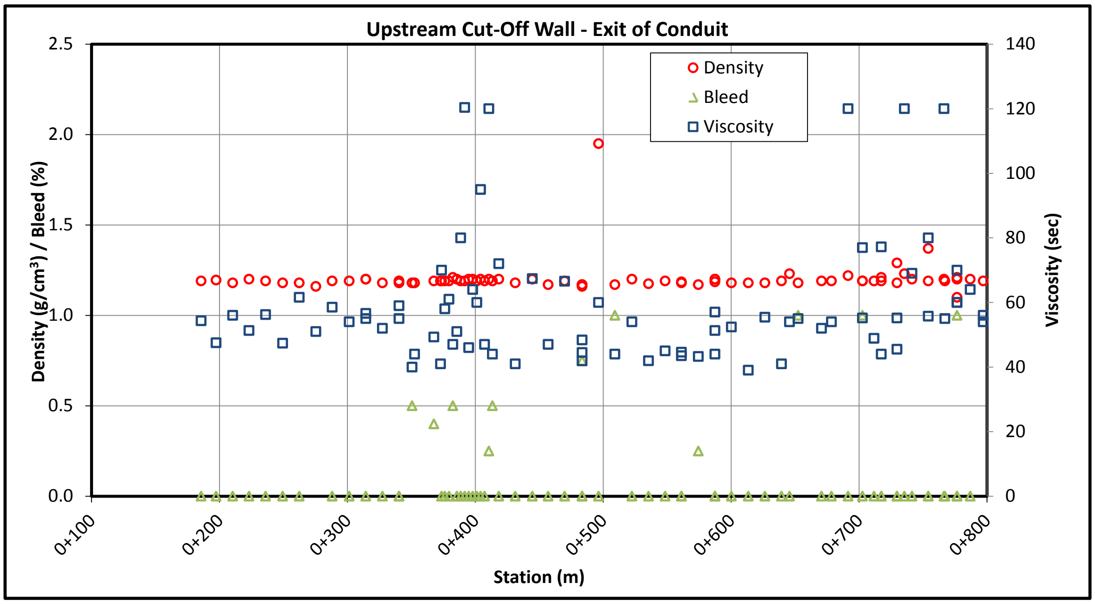



**Figure 4-4: Zone 1 Grain size analyses**

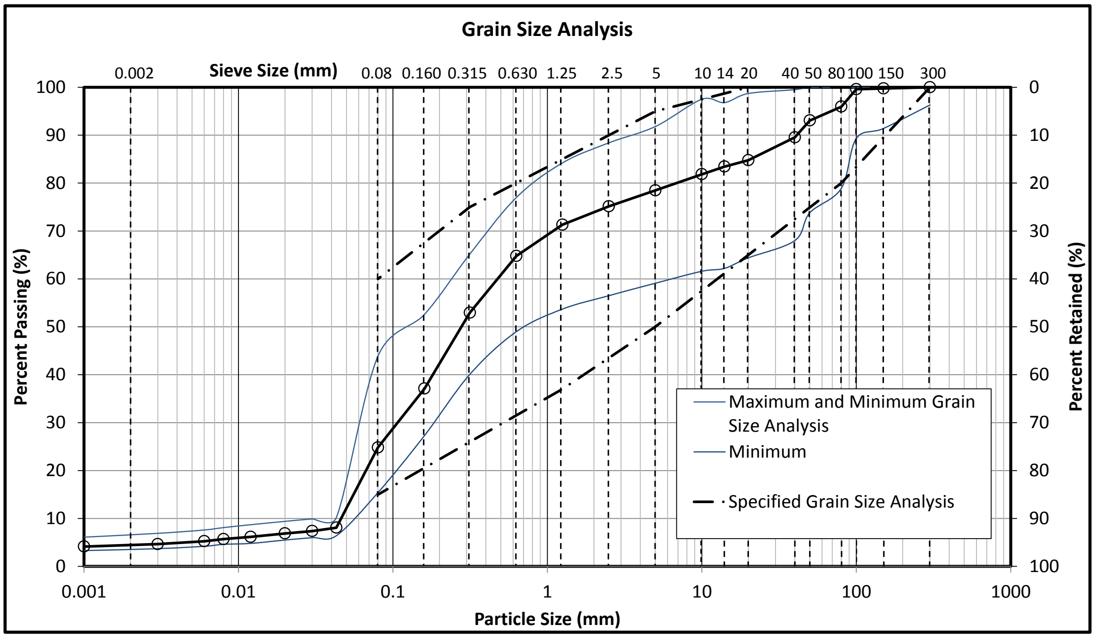
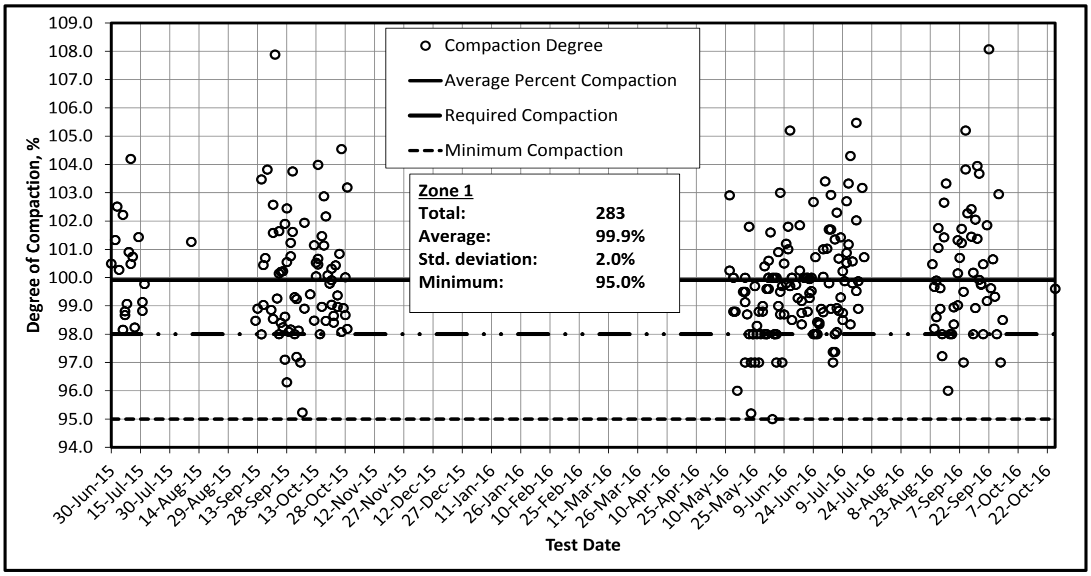


**Figure 4-5: Zone 1, Compaction Results**




## 4.3 NORTHWEST CUT-OFF WALL

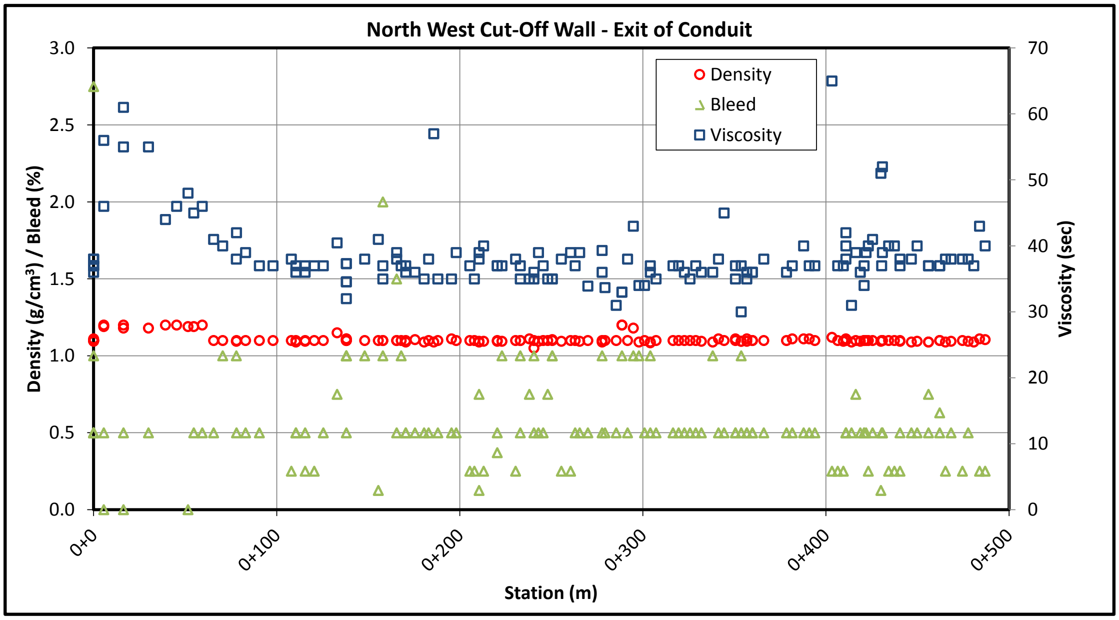
The main objective of the northwest (NW) cut-off wall is to diminish ground water seepage from the Northwest side of the North Spur in order to prevent unfavourable effects on the stability of the downstream slopes.

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

The NW cut-off wall was constructed from elevation 50.0 m and anchored at least 2.0 m in the lower clay layer. The extreme south-westerly portion of the NW cut-off wall was excavated from a temporary platform at elevation 28.5 m.

The trench support fluid and backfill was a cement-bentonite slurry. The mix design, as mentioned above, was modified to include slag cement for constructability reasons. Details of the QC testing can be found in the Construction Report. The graph of Figure 4-6, summarizes the fresh slurry properties.

**Figure 4-6: Slurry properties for NW cut-off**




## 4.4 DOWNSTREAM WORKS

The main objective of the downstream stabilization works is to improve internal and surface drainage in order to control and minimize the piezometric levels and thus increase the factors of safety of the slopes against sliding.

After clearing and stripping, much of the colluvium that constituted the debris from the 1978 slide was excavated to remove the irregularities and create a uniform slope at 7H:1V as a foundation for subsequent fill placement. The upper slopes (mainly the upper sand) were cut back to 2H:1V. At the northern extremity adjacent to the Kettle Lakes Outlet, the entire slope was adjusted to between 2H:1V and 2.5H:1V.

An internal drainage system (finger drains), below the existing downstream slopes was designed to collect and evacuate seepage appearing in the bottom of the main gullies eroded since 1978 into the slide debris. The invert of these finger drains was established in the field to penetrate, as much as possible, the remoulded, intact and low permeability slide debris. The backfill was zoned to provide filtering and draining characteristics in the gullies and consists of selected crushed stone drain material (zone 3A) enveloped in a granular filter (zone 2A), all embedded in a semi-pervious sand (zone 2F). Seepage water is evacuated through the riprap placed along the river shore. The layout of the finger drains can be seen in Figure 3-5.

In the toe area, from the river bed to 1 m above the water level at the time of construction, the embankment is constructed of dumped materials of zones 3E and 2E protected by zone 4 rip-rap. The toe of the latter is widened by a berm of 1.3 m and further protected against erosion by a 6 m wide zone of zone 3E rockfill dumped to 0.5 m above water level. This permits observations of scour to be made in the first few seasons and corrective action taken, if necessary, before any damage occurs to the main body of fill.

The stabilization works, between 1.0 m above water level and elevation 8.5 m, consist of a layer of riprap (zone 4 class 3) placed at a slope of 3H:1V, over successive layers of rockfill transition (zone 3C), drain materials (zone 3A) and granular filter (zone 2A).

Two separate main areas (south and north) were considered for dimensioning the stabilization works above elevation 8.5 m. In the south, from the Rock Knoll at Stn. 0+000 to Stn. 0+420, the stabilization works, from elevation 8.5 m to elevation 35.0 m, consist of a layer of granular filter (zone 2A) overlain by a layer of drain material (zone 3A) and 2 layers of rockfill (zones 3C and 3D) to prevent surface erosion. The filter material was placed either on graded in-situ overburden or on graded backfill built with semi-pervious clean sand (zone 2F) obtained from the required excavations. The thickness of the filter material (zone 2A) is 900 mm and the thickness of the drain and rockfill layers are 450, 450 and 900 mm respectively. Between elevations 35.0 and 40.0 m, a 3E rockfill, 600 mm wide, was placed against the graded slope.

In order to avoid migration of fine particles from the foundation, a geotextile was placed between the graded slope (acceptable foundation) and all rockfill materials.

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 was regraded at 2H:1V and protected with placed rockfill (zone 3E) placed over a geotextile up to elevation 40.0 m.

Berms are located at nominal elevations 8.5 m, 14 m and 25 m. The lowest berm at elevation 8.5 m serves as permanent access road along the river shore. The berm at elevation 14 m serves as an access road to piezometers installed from that level to ensure their protection in the event of the PMF. The berm at elevation 25 m gives access for inspection. A permanent access road was constructed on the 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 northern part of the downstream shore, from Stations 0+420 to 0+740, where instabilities had been observed on the steep slope, the stabilization works consisted 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) over a geotextile up to elevation 14.0 m, thus 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.

Downstream typical cross-sections are presented in Figures 4-7, 4-8 and 4-9. Grain size analyses for zones 2A and 3A are shown in Figures 4-10 and 4-11.

**Figure 4-7: Downstream Typical Cross-Section – Cut Above Elevation 25 m**

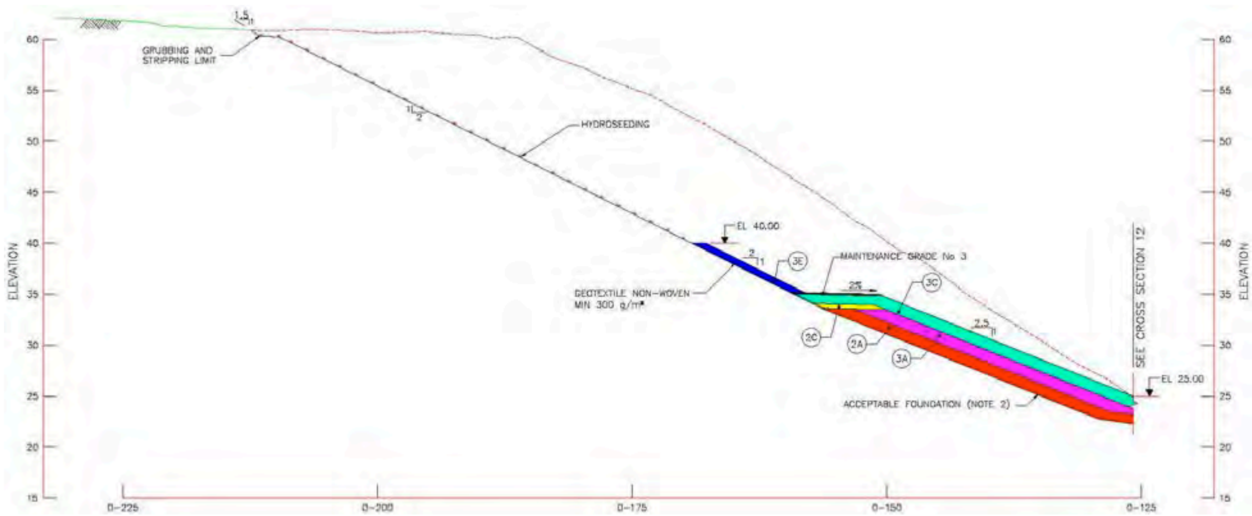



**Figure 4-8: Downstream Typical Cross-Section – Fill Above Elevation 25 m**

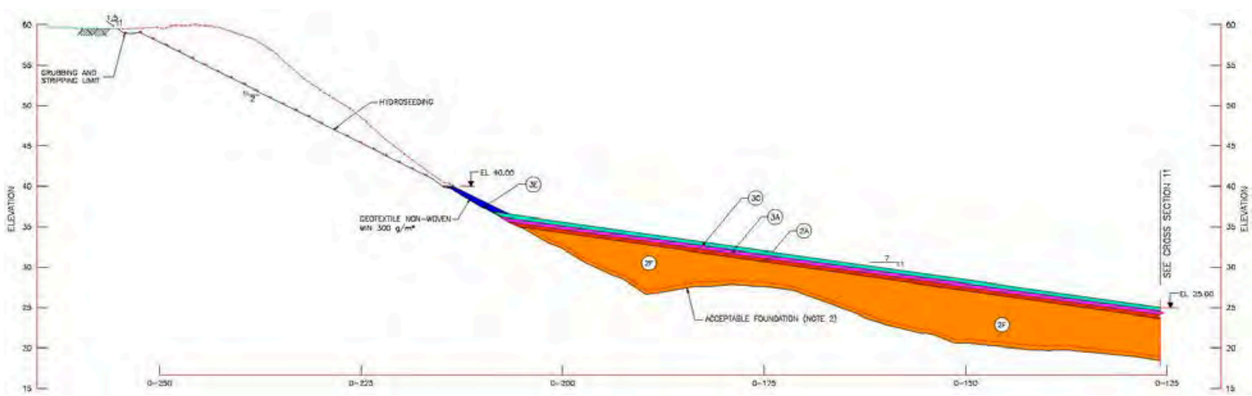



**Figure 4-9: Downstream Typical Cross-Section – Cut and Fill Below Elevation 25 m**

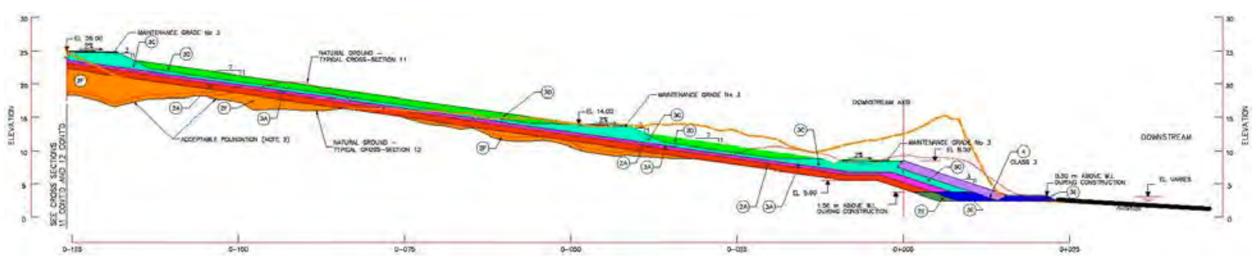


**Figure 4-10: Zone 2A, Grain size analyses**

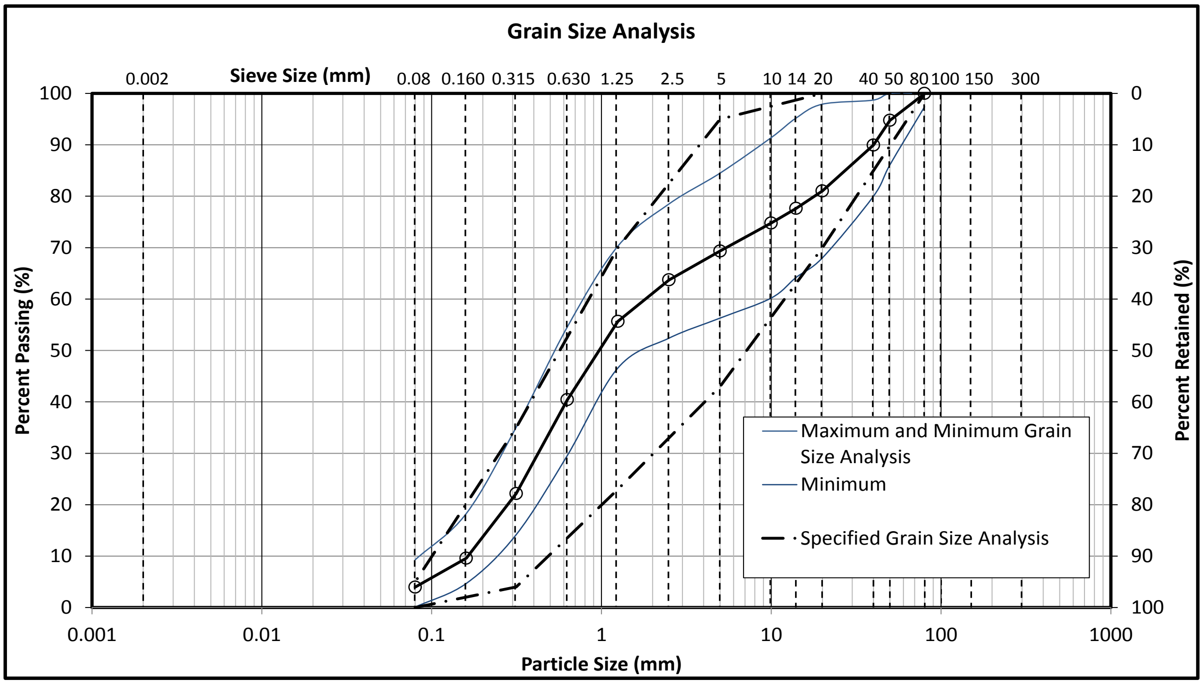
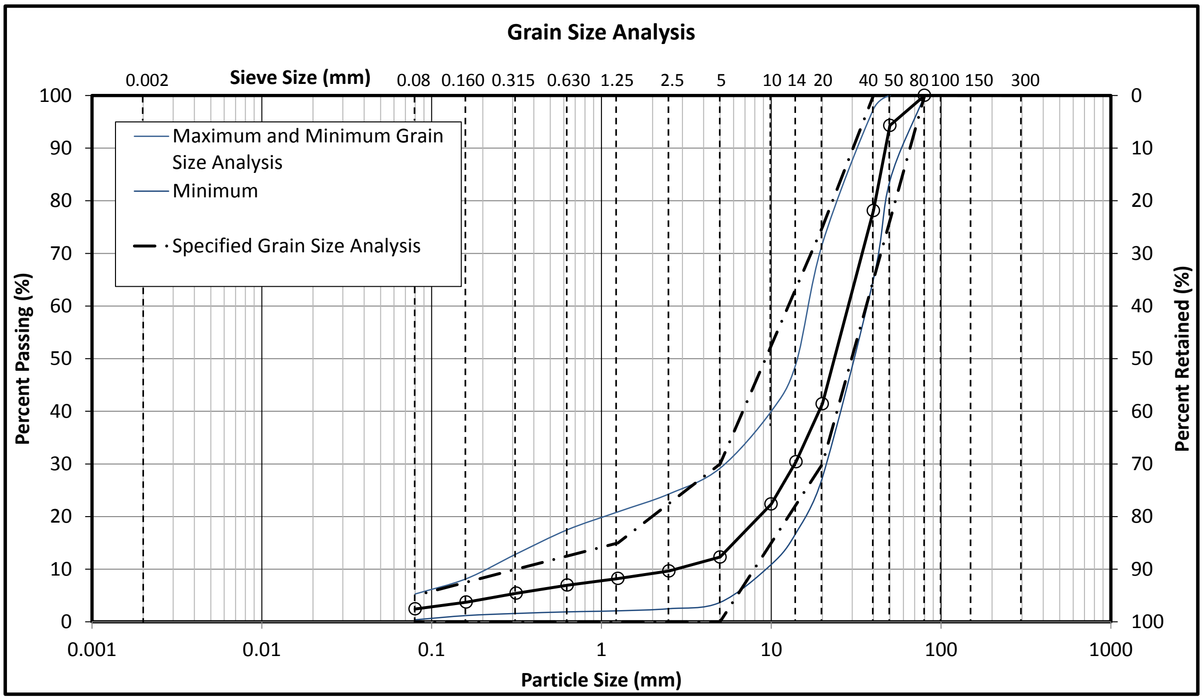


**Figure 4-11: Zone 3A, Grain size analyses**




## 4.5 KETTLE LAKES AREA WORKS

The purpose of the construction works in this area is to stabilize the natural slopes and to facilitate the flow of water in the Kettle Lakes outlet channel.

The slopes of the valley, from the outlet of the Lower Kettle Lake down to the river bank, were regraded and stabilized with slope and invert protection constructed at the bottom of the valley, from the river shore. The protective fill was constructed over a geotextile placed against the acceptable foundation after grubbing and stripping operations. The thickness and particle size of the erosion protection on the slopes of the channel was defined with consideration for the longitudinal slopes of the discharge channel and the anticipated flow. The lateral slopes of the discharge channel were regraded at 2H:1V. A stilling basin was built at the exit of the channel to prevent erosion of the materials.

### 4.6 NORTH SPUR FOUNDATIONS

All embankments were constructed over overburden or bedrock foundations.

Prior to the excavation to reach the acceptable foundation levels, grubbing and stripping, which consisted of the removal of organic matter and contaminated materials, were required.

### 4.7 PERMANENT ACCESS ROAD

Permanent access to the North Spur is gained by a road from the Trans-Labrador Highway to the crest of the Spur and from there to the upstream and downstream areas of the Spur.

The road was constructed with a running surface of Maintenance Grade No. 3 material.

### 4.8 INSTRUMENTATION

Instrumentation of the North Spur includes:

- Standpipe and vibrating wire piezometers;
- Inclinometers;
- One flowmeter.

Several piezometers had been installed over the years, particularly around the time of the pump well installations. Some have ceased to function due to lightning strikes and for other reasons. New instruments have been installed as part of the stabilization works to enhance monitoring capabilities. These included some direct-push vibrating wire instruments where the materials were appropriate for this type of installation. All piezometers permit remote parameter monitoring by transducers and cables which are connected to the main data acquisition system located in the Pumpwell Control Building on the crest of the North Spur. The data acquisition system is remotely accessed from the Nalcor server via the existing optical fiber converter.

Inclinometers and the flowmeter require manual on-site reading.

## 4.9 HYDROSEEDING

Hydroseeding works are designed to prevent erosion of the upper sand and silty sand layer on the upstream and downstream slopes, and in the kettle lakes exit channel above the limit of the embankment fill. These were completed in 2017. The adequacy of the vegetative cover is being monitored and the possible need for additional erosion measures in the future should not be overlooked.

## 4.10 PUMP WELL REFURBISHMENT

In parallel with the stabilization works, maintenance of the existing pump wells was carried out to ensure that they kept working during the construction period. However, additional refurbishment of the system would be required for long term reliability of the pumpwell system. As stated in the North Spur Design Report, the stabilization works are designed with the assumption that the installed pumping system will no longer be required.

As the hydrogeological model indicated, the value of the local drawdown provided by the wells is still judged to be of some benefit, However, the ability to continue to use the wells, if required, was recommended by the Advisory Board as part of the contingency planning. Therefore, ensuring their performance in the short to medium term was the aim of the maintenance. Following reservoir filling and monitoring over a period a few years, the long term use and need for refurbishment of the wells will be re-evaluated. It is to be noted that even during normal operation and particularly during the 2016 inadvertent shut down period, the yield of the pump and the piezometric response can be different from one well to another. Consequently, the efficiency of each pump well is to be considered in the refurbishment plan.

# 5 BEHAVIOUR OF THE NORTH SPUR SINCE CONSTRUCTION

## 5.1 GENERAL

Site inspections are conducted on a regular basis (weekly or bi-monthly) by an engineer. Visual inspection is an important complement to instrument monitoring as the latter often only gives data for discrete points and an overall appreciation of behaviour is required. That being said, measurement of parameters such as pore pressure can only be obtained from instruments. Values of these parameters may change due to changes in behaviour or as a result of outside influence such precipitation or temperature. It is therefore important to record measurements at regular intervals and this can only be accomplished by automated data acquisition systems that include meteorological data. The existing meteorological station located at the Muskrat Falls site addresses this requirement, and it has been confirmed that this gauge will remain in place during the operational phase of the project.

To evaluate the pump well system operation, instrumentation was added in each well to measure the water level and the fluctuation in time.

The instrument layout can be seen on Figure 5-1

## 5.2 PIEZOMETERS

To better understand the distribution of piezometric pressures within the North-Spur, a series of sections have been drawn with piezometric heads added. In addition, the evolution of the values can be appreciated in the plots of piezometric level vs time. These are shown in the Figures in Appendix B.

As far as piezometric levels are concerned, the following comments can be made:

- The response of each piezometer has to be analyzed with regard to its location, elevation and type of installation, and to the stabilization works carried out in the vicinity;
- The reservoir level has risen from around 17 m in October 2016 to 23 m in early 2018.

**Figure 5-1: Instrumentation**

- For the region immediately downstream of the U/S cut-off and blanket, the piezometers are showing a stable situation, or a pressure increase. This is noticeable in the graphs of P17-16, and P15-16, respectively. The increase was not unexpected as the flow direction shown by the hydraulic gradient in Figure 3-2 has been effectively blocked by the presence of the upstream blanket.
- The exception is the instrument (P14-16) located in the narrow southern part of the Spur where the proximity and the effect of downstream drainage are noted. Moreover, the surface area for vertical infiltration is less. The readings of this instrument indicate a noticeable decrease since installation.
- Piezometer P18-16 is also located behind the embankment fill but in an area beyond the cut-off walls. The readings, as expected, track the rising reservoir level.
- Piezometers P08-16 and P09-16 are located in the narrow southern part of the Spur, mid way between the upstream and downstream toes. The readings indicate a decrease in pressure since installation.
- In October 2016 and well into 2017, a majority of the pumps were unintentionally stopped by an electrical supply/control problem. After several months, the issue was recognized and rectified, which led to decrease from the peak piezometric levels and a return to previous values by February 2018. This event amply illustrates the need for remote monitoring to enable response in a timely fashion to changes in the behaviour.
- Not all the piezometers in the vicinity of the pump well system indicate influence of the pump shut-down and those that do are to varying degrees.
- PF-1-97, installed on the downstream side of the line of wells at elevation 2 m, shows a gradual increase of about 2 m with no fluctuation relative to the pumps.
- NS-1-13, relatively close but at elevation 31 m, has decreased by 3 m since construction.
- PG-97, on the upstream side of the line of pump wells responded to the pump shut-down and has yet to decline to previous levels. P4-A-09 and P4-B-09 reacted and have returned to previous levels with no net change.

- P1-B-09 installed at elevation 19 m showed a minor response but, P1-A-09 installed at el. 36 m indicates an almost constant value of around 47 m, clearly related to the perched water table in the upper sand.
- PB-1-97 and PB-2-97, installed respectively at el. 2 m and el. 13 m, show no connection with the pump activity despite being adjacent (in plan) to what used to be the most productive of the pump wells, W4. The readings have been essentially constant since construction.
- For the instruments on the upper part of the downstream slope, P10-16, P11-16 and P12-16 indicate stable values; even a small decrease is noted. However, for P13-16 installed at el. -4 m, the response seems to reflect an influence of the pump well shut-down with a net increase of 4 m of head to el. 20 m. Note that the bases of the well screens were set at around 0 m elevation.
- Nearer the downstream toe, instruments P01-16, and P03-16, installed at around elevation 0 m, show fairly stable water pressure heads of 10 m.
- At P02-16, P04-16 and NS-2A-13, the piezometers follow the downstream river level as they are located close to or within the lower aquifer.
- A series of piezometers were installed to gain insight into the groundwater regime in the vicinity of the NW cut-off. Three boreholes date from 2013 i.e. the period prior to construction and two were executed in 2016. Decreases of 4 m and 2 m respectively were noted shortly after construction at NS-4A-13 and NS-4B-13 but the piezometric levels are still far above the reservoir level. Levels, also above the reservoir are constant at NS-3A-13, NS-3B-13, NS-6A-13, NS-6B-13, P19-16 and P20-16.

## 5.3 INCLINOMETERS

Six inclinometers were installed as part of the 2016 works. No significant movements have been recorded to date.

## 5.4 FLOW METER

One flow meter is installed in a corrugated pipe culvert at the outlet of the Lower Kettle Lake to be read on a seasonal basis (summer/fall only). A pressure device (Microdiver) is also used on to monitor the water level variation at the entrance to the culvert.

## 5.5 VISUAL INSPECTIONS

In the period since construction some superficial erosion has been noted but no evidence of movement.

There is a spring located above the crest of the upstream blanket (at around station 0+650) such that water is observed to flow over the fill and to freeze during the winter period.

# 6 IMPACT OF OBSERVED BEHAVIOUR ON DESIGN STUDIES

## 6.1 SLOPE STABILITY

Stability analyses had been carried out as part of the design studies as described in section 3.2.4 of the present report. These were reviewed post-construction to ascertain whether any condition including ground topography, materials and pore pressures are different from the original hypotheses and thus warrant a revision of the analyses.

On the upstream side at section 4 (see Figure C-1 in Appendix C), the observed and predicted conditions are as envisaged and no revision is required.

On the downstream side, an additional analysis at section 9 has been carried out in order to verify the current state of the North Spur in the location of the 1978 slide. The profile is As-Built and the phreatic surface is derived from the instrument readings. An example of the software output can be seen in Figure C-2. Examination of this figure indicates that the analysis, which selects the most critical potential failure surface(s), focuses on the upper slope in the sand layer and the slope protection at the toe. The lowest value for the Factor of Safety is 1.46 for a shallow potential slide in the sand which has no significant effect on the overall downstream slope.

Section 13 is located in the Northern sector of the downstream slope, in an area unaffected by the 1978 slide but subject to superficial surface slides as was noted during reconnaissance of the site in 2011 during the design phase. Figure C-3 shows the results of an analysis using the pre-stabilization works profile. The calculated Factor of Safety of 0.97 (or around unity) is consistent with the above mentioned observation of shallow slope failures and serves to validate the selection of material parameters.

Figure C-4 shows the same area with the post-construction slope, and protection zones at the toe. Using the phreatic surface which was derived from piezometric measurements, the analysis indicates a minimum Factor of Safety of 1.54. Figure C-5 is taken from a sensitivity analysis and shows that local increase of pore pressure in the mid-height area of the slope can reduce the calculated Factor of Safety to unity. Figure C-6 illustrates how the above situation can degenerate into sequential slips or progressive failure. While this situation is unlikely, it is considered prudent to provide additional piezometer in this area to allow pore pressures to be monitored at that location.

Monitoring of pore pressures in the natural soil stratigraphy during impounding and during operation is obviously to be included in surveillance plan. Validation of the array of piezometers in the North Spur was described in Section 6, and it is considered that there is sufficient coverage to have confidence in the assessment of the slope stability using data from those instruments.

Figures C-7 and C-8 illustrate the results of analyses to evaluate the stability of the underwater slope downstream of section 9. The stability may potentially be affected by pore pressures in the lower aquifer. However, as a comparison of the two figures shows, a proportional increase of local pressures to account for the future reservoir raise from 23 m to FSL at 39 m has only limited influence. Nevertheless, the piezometers located within the lower aquifer will be closely monitored during reservoir filling to validate this assumed pressure distribution.

## 6.2 PROGRESSIVE FAILURE

As mentioned in section 3.2.5, the potential for progressive failure in the sensitive clay and silty clay strata was the subject of a specific study. This study was carried out prior to the execution of the stabilization works and was based on the best available data at the time to establish the design hypotheses. A review of this data was required to ensure the validity of the conclusions formulated. This review has now been carried out. The pore pressures used in the analyses were conservatively assumed to be at least as high as those obtained from the SEEP-W model and are greater than values measured post construction or extrapolated to FSL. The material properties and the geometry of the completed works correspond essentially to those used in the analyses. Consequently, no information garnered during the construction or of the subsequent behaviour indicates a need to re-visit the issue of progressive failure.

## 6.3 DYNAMIC ANALYSIS

The potential impact of pore pressures on the results of the analyses was evaluated. The Memorandum dated August 1st, 2018, a copy of which is to be found in Appendix D, presents the findings. Essentially, for the range of pore pressures anticipated during the raising of the reservoir to the FSL of 39 m, no significant change in the results is expected. Consequently, dynamic analyses indicate that the design earthquake loading will not induce liquefaction in the granular materials or cyclic softening in clay-like materials for the anticipated soil and piezometric conditions.

## 6.4 HYDROGEOLOGY

As mentioned in section 3.2.7, two 3-D models were developed to simulate the hydrogeologic regime and to attempt to predict the response to the rising reservoir, namely at natural river level of 17.5 m±, at the intermediate level of 25 m during construction, and at the FSL of 39 m.

The lower aquifer is confined and seemingly easier to model. Based on the data available from the winter 2017/2018 impoundment at el. 23 m, the lower aquifer shows little or no reaction to the increase in reservoir level from the natural condition (17 m±) to 23 m. The model had predicted an increase of 2.2 m on average for the piezometers installed in the lower aquifer. Aided by the model, a conclusion was reached that the relief wells penetrating the lower aquifer would not be required. Monitoring to date confirms this approach.

For the intermediate aquifer, correlation of predicted and measured values is weak. The model is not adequate to simulate the behaviour and to predict the water pressure distribution in the North Spur. The Table 6-1 presents the change in piezometric value due to the rise from el. 17.5 m to el.25 m (23 m for measurements).

**Table 6-1: 3-D Hydrogeological Model, Comparison between model and observations**

| Piezometer | Model (el. 25 m)<br>A | Observation (el. 23 m)<br>B<br>Neighbouring Piezometer |
|---|---|---|
| | Delta (m) | Delta (m) |
| B6-79 | 0.7 | 0 |
| E1-79 | 0.6 | -0.8 |
| D4-79 | -1.1 | none in vicinity |
| B5-79 | -0.5 | -0.4 |
| C4-79 | -2.4 | -0.4 |
| C3-79 | -6.8 | 0.2 |
| D1-79 | -9.8 | 1 |
| B8-79 | -2.7 | -0.4 |

A = Delta predicted by the model for the selected piezometer including pump operation and all stabilization measures

B = Delta observed in neighbouring piezometer for the same condition as A

The water level in the 3rd of the Three Kettle Lakes was predicted to drop by about 1 m; in fact no change has been noted. Furthermore, the reservoir level is at el. 23 m and the lake level could be expected to be even lower.

It is difficult to develop a detailed comparison as the piezometric values used for the initial calibration came from instruments that are no longer functioning. The complexity of the stratigraphy with micro depositional features within the already highly variable 3-D principal units cannot be deduced from the boring and sampling conducted at the site. The record of some piezometers does not show a response to barometric pressures, indicating an aquiclude or an unsaturated state. The aleatory response of piezometers to the pump well operations and the apparent random nature of the aquifer in terms of depth confirm the difficulty in the numerical simulation. An apparent downward gradient indicates that the pressures are not those that could be expected from a hydrostatic regime below a defined single groundwater table. The real monitoring data, progressively acquired during the reservoir filling, will be more valuable. Therefore, an update of the model of the intermediate aquifer is not recommended at this time. The model composition and the hydrogeologic numerical simulation of the North Spur could be re-visited as more data becomes available in the early stages of impounding to el. 39 m, if the trends merit and additional information to manage the hydrogeologic regime is deemed necessary.

# 7 CONCLUSIONS

The assessment has indicated that the stabilization works were carried out in accordance with the design intent and the performance to date with the reservoir at the interim level of 23.0 m is satisfactory.

Specific studies carried out prior to the construction have been re-examined, with a view to evaluating any effect on the results of:

- post-construction North Spur geometry;
- material properties; and,
- anticipated pore pressures.

No change to the conclusions of these studies was found to be warranted.

Stability analyses indicate that the Factor of Safety for all stabilized slopes of the North Spur works are satisfactory. At the northern sector of the downstream slope the sensitivity analysis showed that an increase in pore pressure could potentially reduce the Factor of Safety and additional piezometers (2) should be considered for mid-height of the slope in that area.

Minimizing the risk for progressive failure is achieved by preventing an initial slope failure. It was found that adequate instrumentation is in place to ensure that the conditions are being monitored.

Current monitoring tends to confirm that relief wells in the lower aquifer were likely not a requirement based on the 23 m water level behavior, however final assessment should be made following final impoundment. As far as the intermediate aquifer is concerned, the numerical simulation, while useful, cannot provide detailed predictions against which the piezometer readings can be compared. Alert levels should be established progressively with data accumulation during impounding and based more on trends and rates of change than unique values.

The North Spur in its current configuration is therefore deemed to be fit-for-purpose. Reservoir raising may proceed at the planned filling rate, with the current surveillance plan and normal precautions.

# 8 RECOMMENDATIONS

The initial draft of this report was completed in April 2018, and a presentation was made to LCP on the findings and recommendations. The recommendations made at that time have been acted on and include:

- Installation of two additional piezometers in the northern sector of the downstream slope to enhance the monitoring in that area.
- Assessment and maintenance of the pumpwell system to allow determination of the extent of work that would be required to undertake a full refurbishment, should it be required.
- Inspection of the North Spur by the geotechnical team following winter and correction of any surface erosion prior to the on-set of winter conditions.

While declaring that the North Spur is fit for purpose in comprising part of the water retention facilities of the Muskrat Falls project, it should be noted that the existing surveillance plan will need to be updated for the impoundment period, and then to function as the operational surveillance plan once the reservoir has been raised to el 39 m.

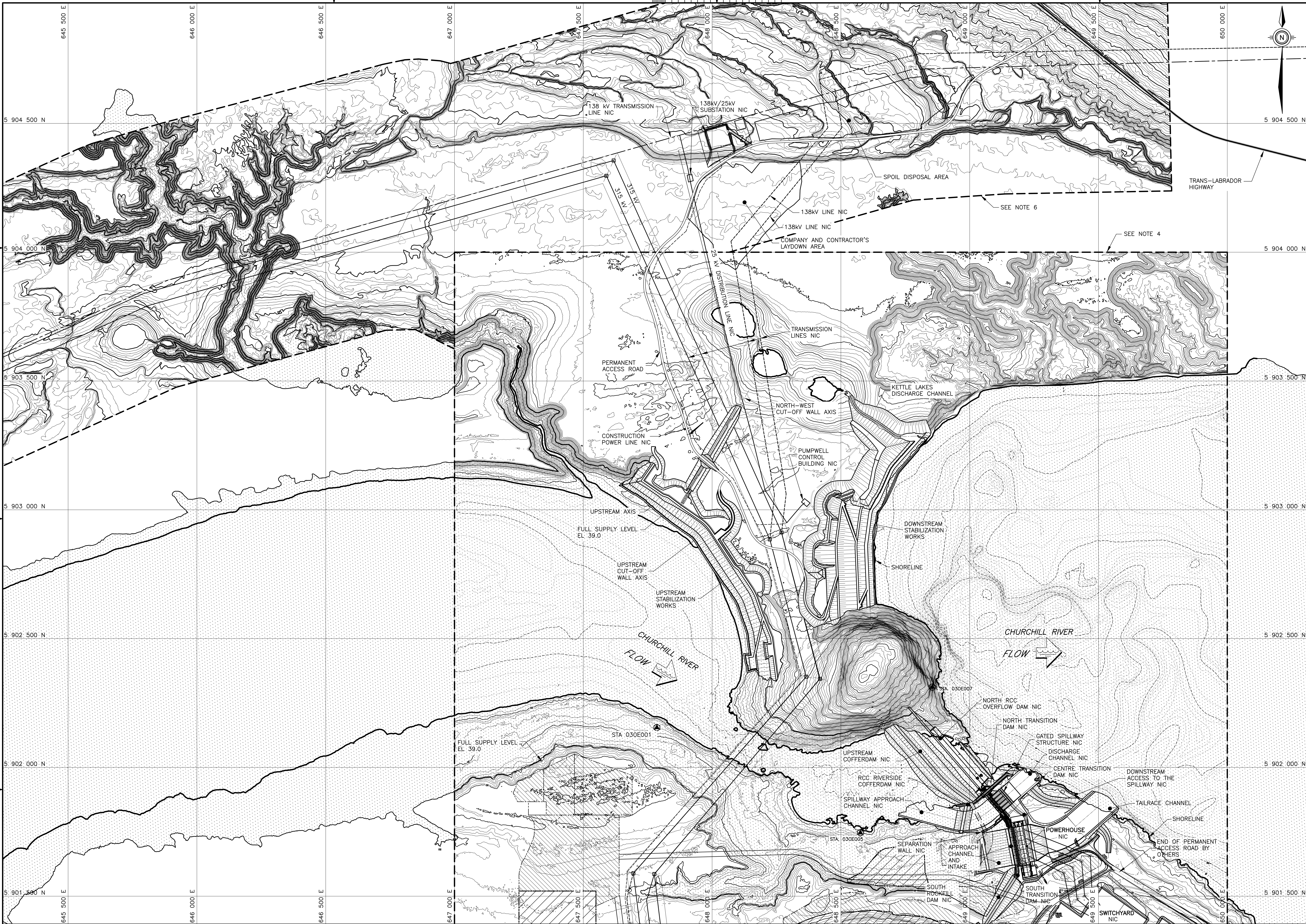
Following impoundment, it may be some time before the behaviour of the North Spur, as monitoring by the instrumentation, stabilizes. Continued review of the observations should take place to assess whether changes are recommended to the frequency of observation, number and location of instruments and to develop triggers or indicators of concerns about the response the raising of the reservoir. The final decision on whether or not relief wells are required in the lower aquifer will also be made post-impoundment.

It is recommended that the North Spur be treated as a dam, and that access to the structure by the public be limited, as it would be with any water retaining structure. This does not preclude access to the portage trail, the Knoll or recreational access.

It is also recommended, that prior to impoundment, and for the first few years after impoundment, that a visual inspection of the North Spur be carried out by qualified dam safety engineers, and that any deficiencies (such as local erosion or small surface slippage) be addressed prior to the following winter.

## APPENDIX A

## PROJECT LAYOUT AND NORTH SPUR INVESTIGATIONS




# 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

Figure A-1

0 100 200 300 400 500 m

SCALE 1:5000

| ISSU. | REV. | DATE | DISTRIBUTION & STATUS | ISSU. | REV. | DATE | DISTRIBUTION & STATUS |
|---|---|---|---|---|---|---|---|
| 5 | C4 | 15–FEB–2016 | RE–ISSUED FOR CONSTRUCTION | | | | |
| 4 | C3 | 23–MAR–2015 | RE–ISSUED FOR CONSTRUCTION | | | | |
| 3 | C2 | 03–MAR–2015 | RE–ISSUED FOR CONSTRUCTION | | | | |
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|---|---|---|---|

| No. | DATE | REVISION | MOD. | VER. | APP. |
|---|---|---|---|---|---|
| L1 | 30–MAR–2018 | AS–BUILT | | | |
| C4 | 15–FEB–2016 | PERMANENT ROAD SHOWN IN DOWNSTREAM AREA; DOWNSTREAM AREA MODIFIED | A.C. | R.B. | G.S. |
| C3 | 23–MAR–2015 | DOWNSTREAM AREA MODIFIED | A.C. | R.B. | G.S. |
| C2 | 03–MAR–2015 | SPOIL DISPOSAL AREA MOVED; NOTE 6 ADDED | A.C. | R.B. | G.S. |

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- ☐ 5. NOT REVIEWED

LEAD REVIEWER: Date (dd-mmm-yyyy):

NE-LCP MANAGEMENT: Date (dd-mmm-yyyy):

PROJECT MANAGER: Date (dd-mmm-yyyy):

PROFESSIONAL STAMP

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–0021–05

NE DOC No. MFA–SN–CD–2800–CV–GA–0002–01 Rev. L1

FORMAT A0 (mm)

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PLAN

## NOTES:

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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
- ANGLE INCLINED JOINT
- VERTICAL JOINT
- ANGLE INCLINED FOLIATION
- VERTICAL BOREHOLE
- INCLINED BOREHOLE
- W9 PUMPING WELL
- P6 LOST PIEZOMETER
- no-YEAR EXISTING PIEZOMETER
- NS-no-13 2013 BOREHOLE
- CPT-no-13 2013 CPT TEST
- CPT-no-15 2015 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 |

Figure A-2

0 50 100 150 200 m

SCALE 1:2500

| ISSU. | REV. | DATE | DISTRIBUTION & STATUS |
|---|---|---|---|
| 3 | L1 | 30-MAR-2018 | AS-BUILT |
| 2 | C2 | xx-JAN-2016 | RE-ISSUED FOR INFORMATION |
| 1 | C1 | 07-NOV-2013 | ISSUED FOR INFORMATION |

ISSUE REGISTER

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|---|---|
| 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. |
|---|---|---|---|---|---|
| L1 | 30-MAR-2018 | AS-BUILT | | | |
| C2 | xx-JAN-2016 | 2015 CPT TEST LOCATIONS ADDED. | A.C. | R.B. | G.S. |

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LEAD REVIEWER: Date (dd-mmm-yyyy):

NE-LCP MANAGEMENT: Date (dd-mmm-yyyy):

PROJECT MANAGER: Date (dd-mmm-yyyy):

PROFESSIONAL STAMP

SNC·LAVALIN

DESIGNED BY N. DAIYAN

DRAWN BY R. MERRI

VERIFIED BY A. CEBALLOS

DATE 22 SEP 2012

APPROVED Discipline Lead Engineer R. BOUCHARD

APPROVED Engineering Manager G. SNYDER

SCALE 1:2500

CLIENT nalcor energy LOWER CHURCHILL PROJECT

PROJECT LOWER CHURCHILL PROJECT

TITLE MUSKRAT FALLS NORTH SPUR STABILIZATION WORKS EXISTING GEOLOGICAL AND GEOTECHNICAL INFORMATION PLAN

File No. 505573-3281-4GDD-0008-03

MFA-SN-CD-2800-GT-PL-0012-01 Rev. L1

FORMAT A0 (mm)

**SUB-PKG: 0008-4G03 PLATE G04**

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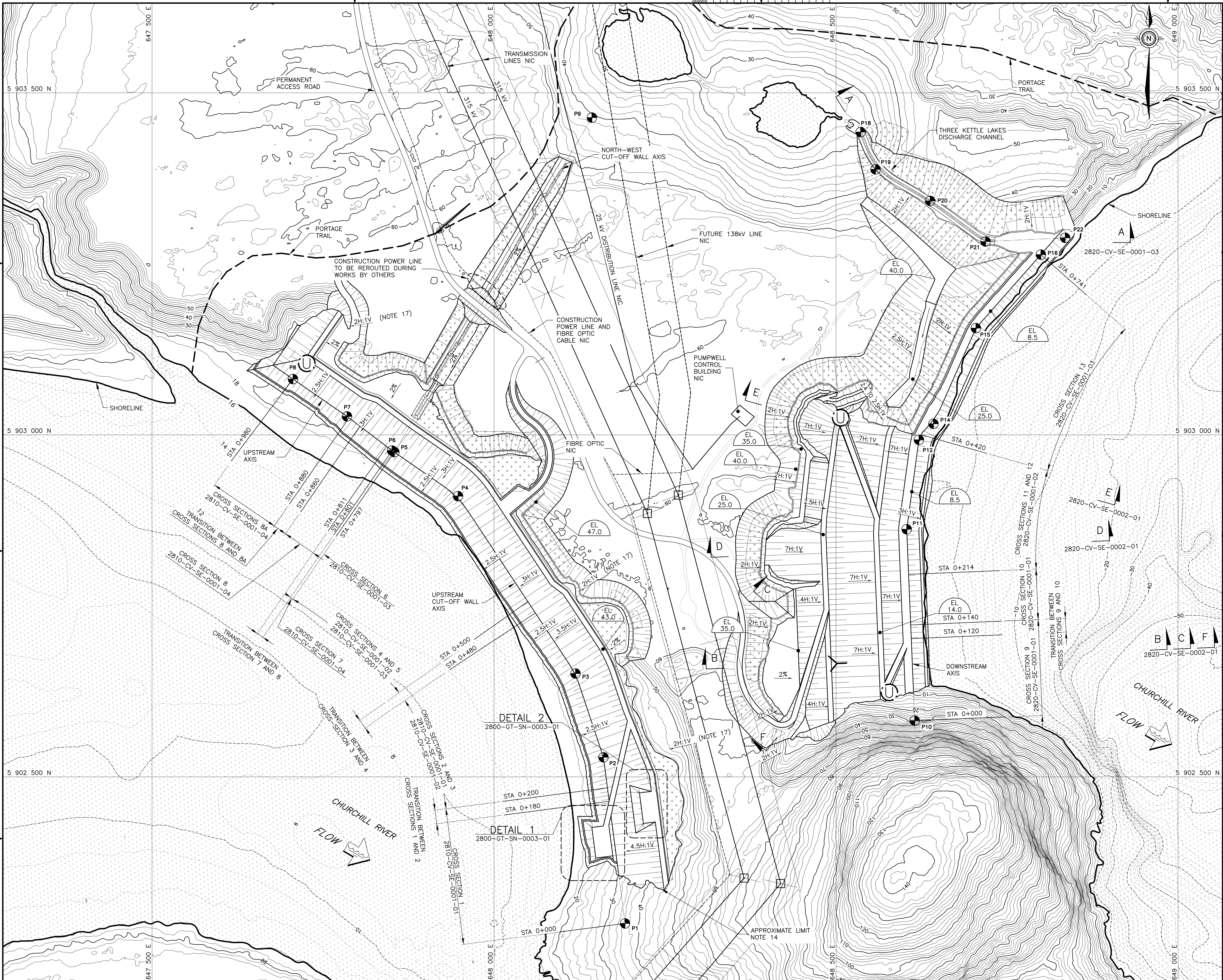
2018.04.11/11:23am




NOTES:

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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.
7.
8.
9.
10. THE UPSTREAM CUT-OFF WALL AXIS IS DEFINED BY LOCATION POINTS P1 TO P5.
11. THE UPSTREAM AXIS IS DEFINED BY LOCATION POINTS P1 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 P16.
14.
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.
17. ABOVE EL 47.00 REGRADING TO 2H:1V IS NOT REQUIRED WHEN NATURAL SLOPE IS 1.8H:1V OR GENTLER.
18. ALL ACCEPTABLE FOUNDATION IN THE DOWNSTREAM AREA IS COVERED WITH GEOTEXTILE WHEN IN CONTACT WITH 3A, 3C OR 3D ZONE MATERIAL.

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
HYDROSEEDING

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 | 5 903 026.9 | 150.00 |
| P8 | 647 706.1 | 5 903 081.5 | – |
| P9 | 648 143.1 | 5 903 463.8 | – |
| P10 | 648 615.0 | 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 | – |
| | | | |
| P18 | 648 536.0 | 5 903 442.6 | – |
| P19 | 648 557.7 | 5 903 388.1 | 100.00 |
| P20 | 648 637.6 | 5 903 342.0 | 600.00 |
| P21 | 648 718.6 | 5 903 282.6 | 150.00 |
| P22 | 648 835.0 | 5 903 288.0 | – |

Figure A-3

SCALE 1:2000

ISSUE REGISTER

| ISSU. | REV. | DATE | DISTRIBUTION & STATUS | ISSU. | REV. | DATE | DISTRIBUTION & STATUS |
|---|---|---|---|---|---|---|---|
| 5 | C3 | 15-FEB-2016 | RE-ISSUED FOR CONSTRUCTION | | | | |
| 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 5 | | | | |
| 1 | B1 | 07-NOV-2013 | ISSUED FOR BID | 6 | L1 | 30-MAR-2018 | AS-BUILT |

| No. | 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-DD-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 |

| No. | DATE | REVISION | MOD. | VER. | APP. |
|---|---|---|---|---|---|
| L1 | 30-MAR-2018 | AS-BUILT | | | |
| C3 | 15-FEB-2016 | PERMANENT ROAD SHOWN IN DOWNSTREAM AREA: DOWNSTREAM AREA MODIFIED (STA 0+420 TO 0+471). | A.C. | R.B. | G.S. |
| 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 *********

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

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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 G. SNYDER
SCALE 1:2000

CLIENT nalcor energy LOWER CHURCHILL PROJECT

PROJECT LOWER CHURCHILL PROJECT

TITLE MUSKRAT FALLS NORTH SPUR STABILIZATION WORKS PLAN

File No. 505573-3281-4GDD-0003-05

MFA-SN-CD-2800-CV-PL-0009-01 Rev. L1

FORMAT A0 (mm)

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# APPENDIX B

# INSTRUMENTATION LAYOUT AND RESULTS

Figure B-1: Locations of instrument profiles

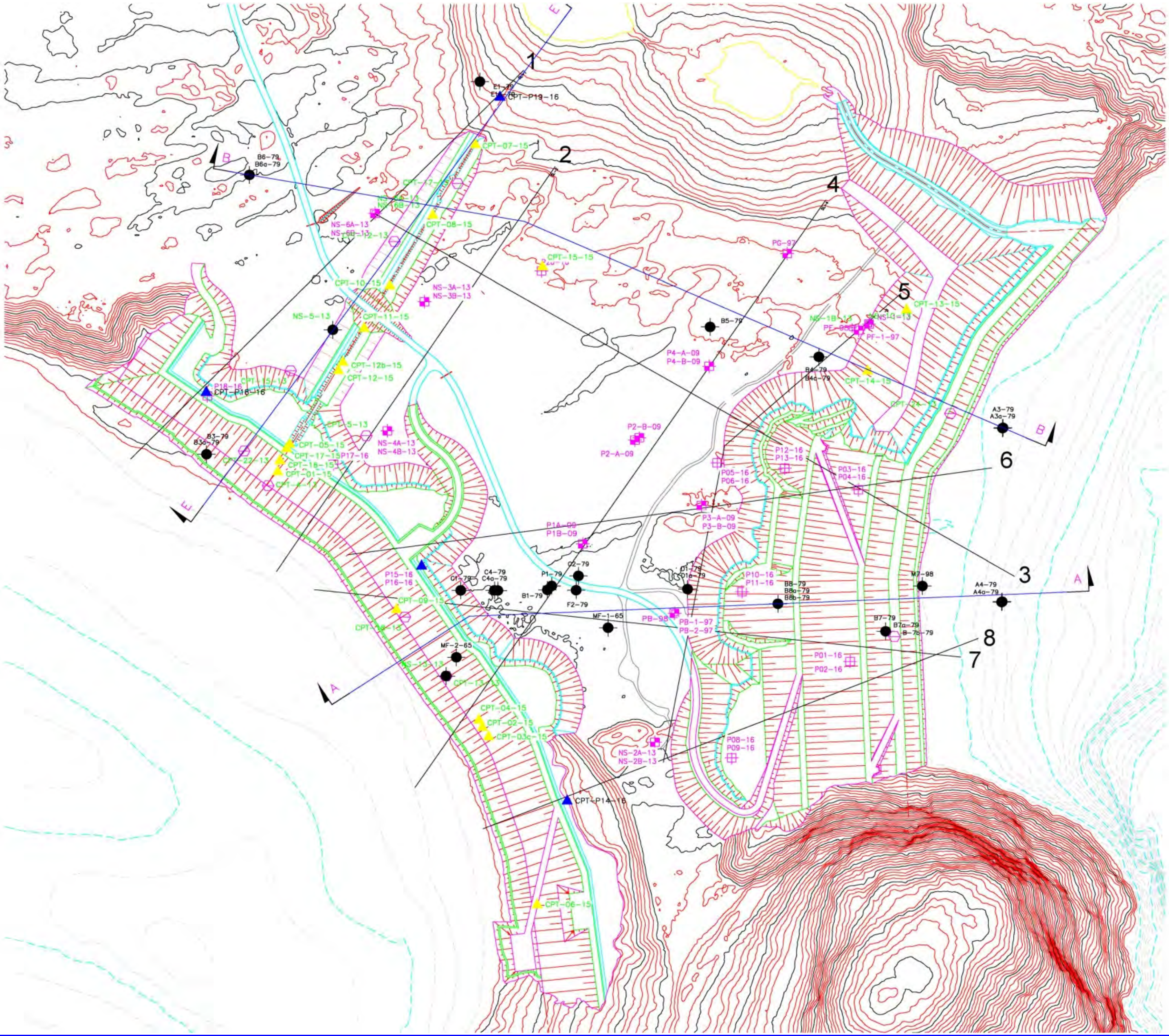


Figure B-2: Section 1

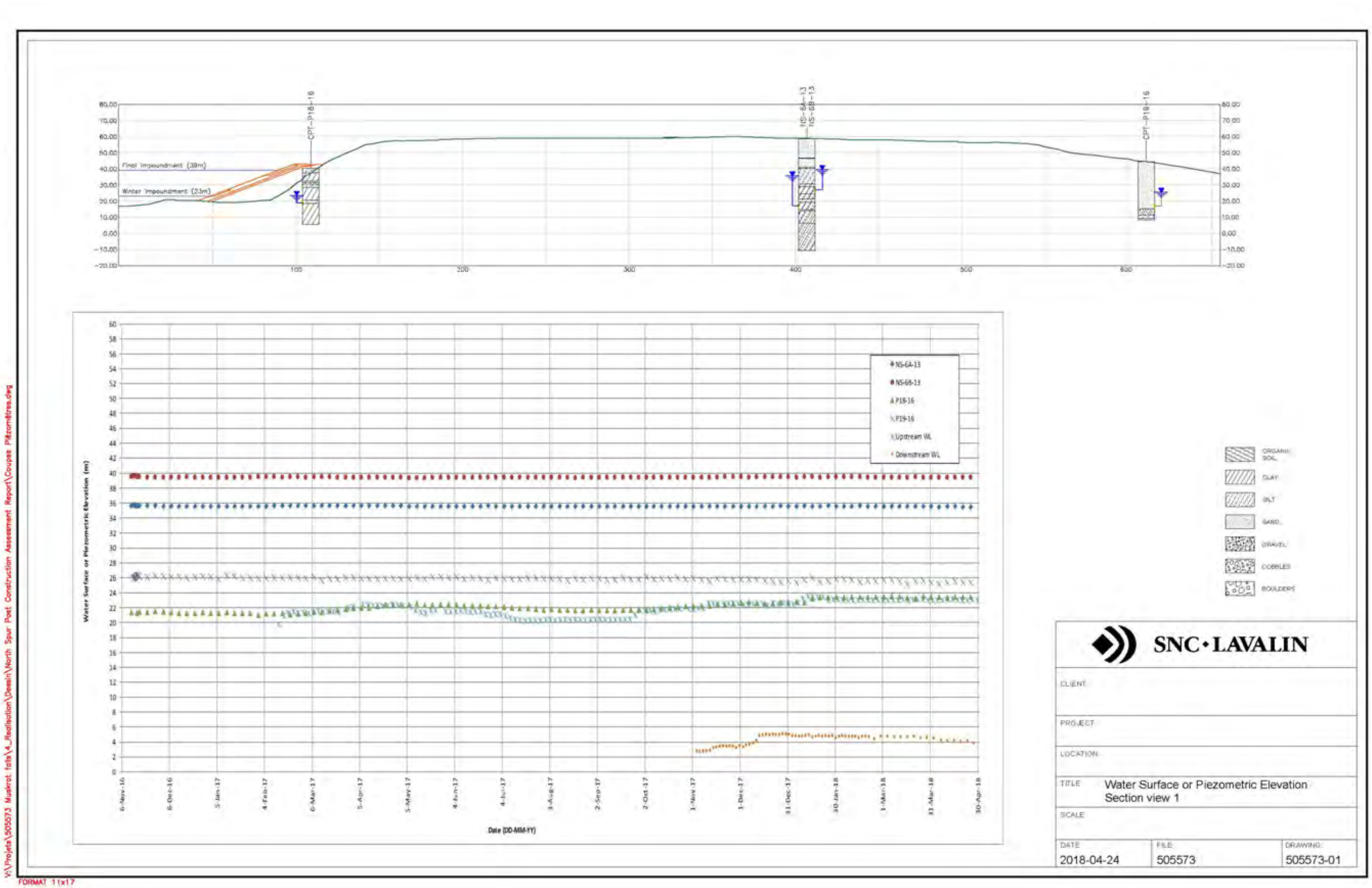
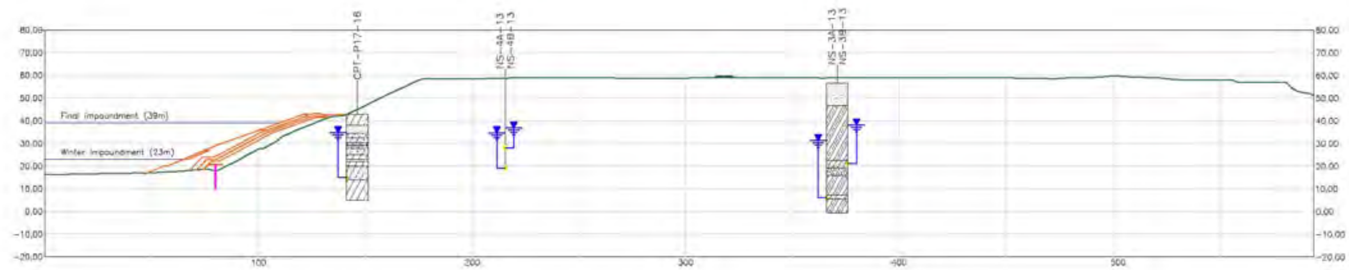
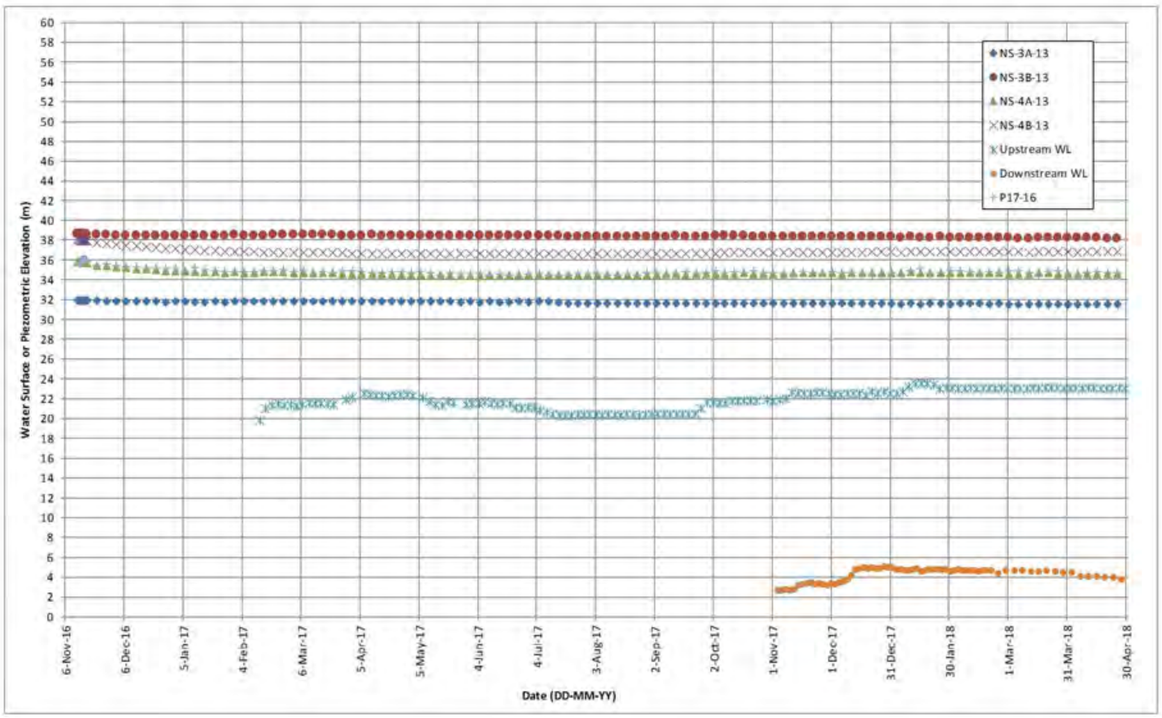


Figure B-3: Section 2





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| PROJECT | | |
| LOCATION | | |
| TITLE Water Surface or Piezometric Elevation Section view 2 | | |
| SCALE | | |
| DATE 2018-04-24 | FILE 505573 | DRAWING 505573-02 |

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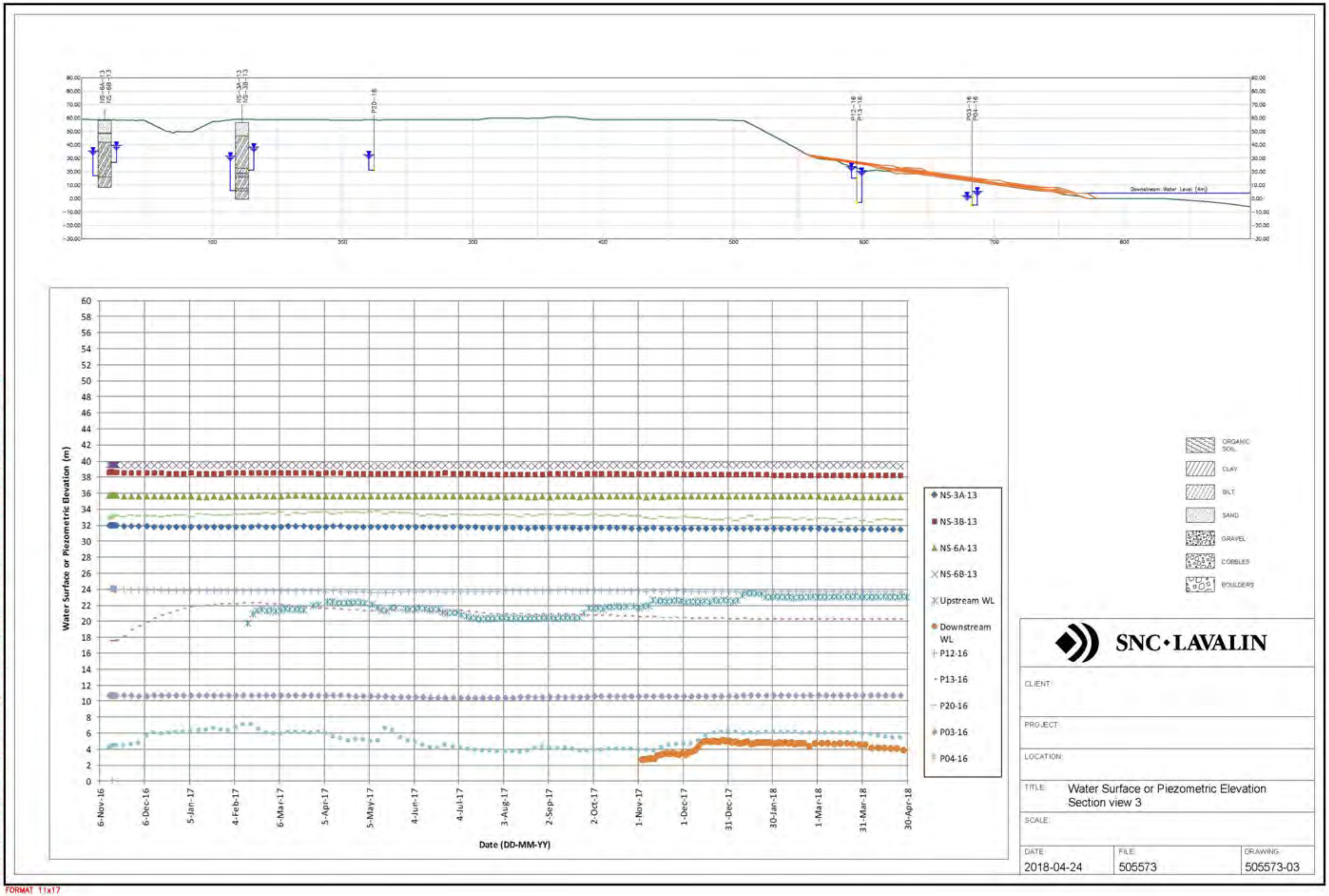
FORMAT 11x17

Figure B-4: Section 3



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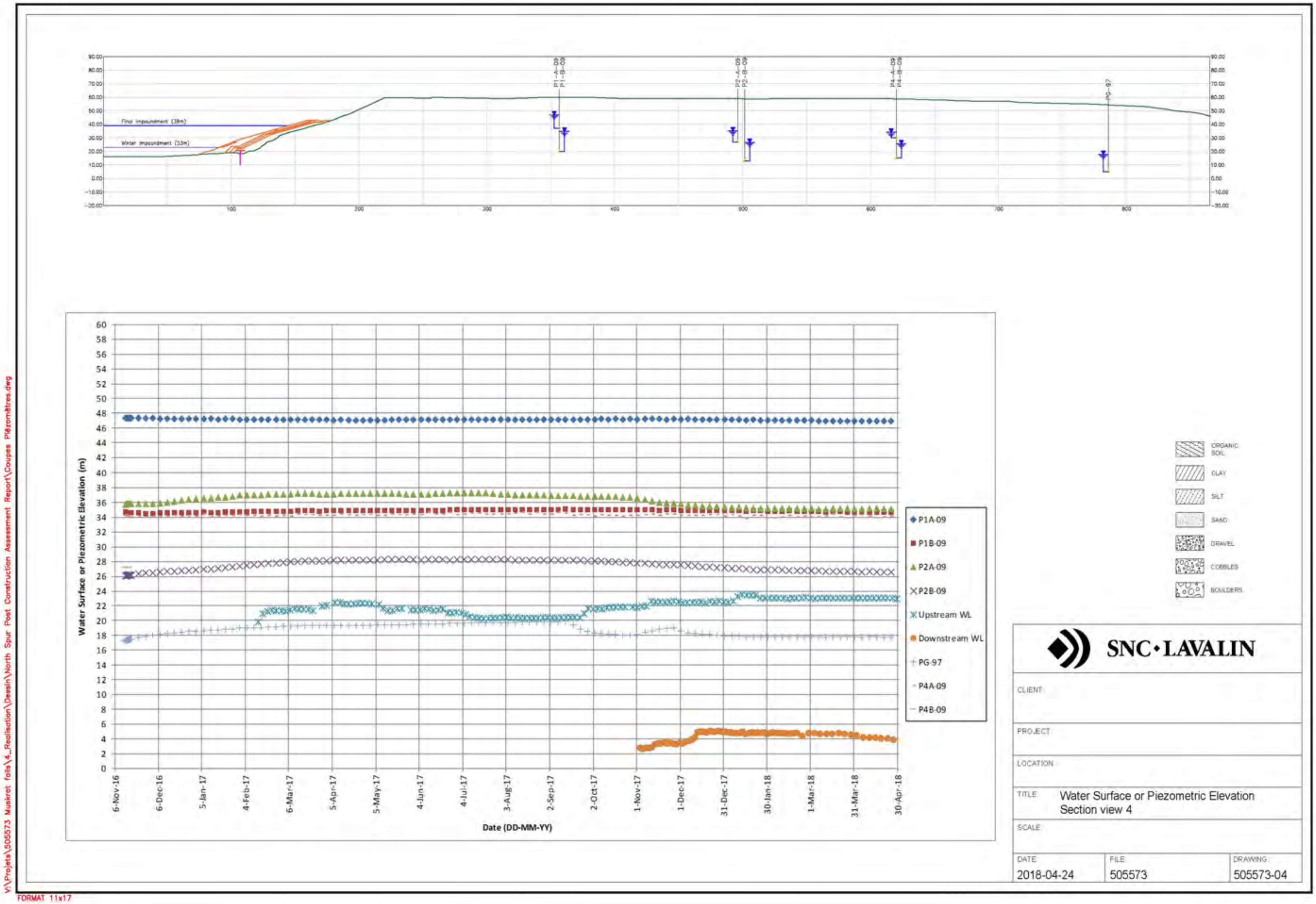

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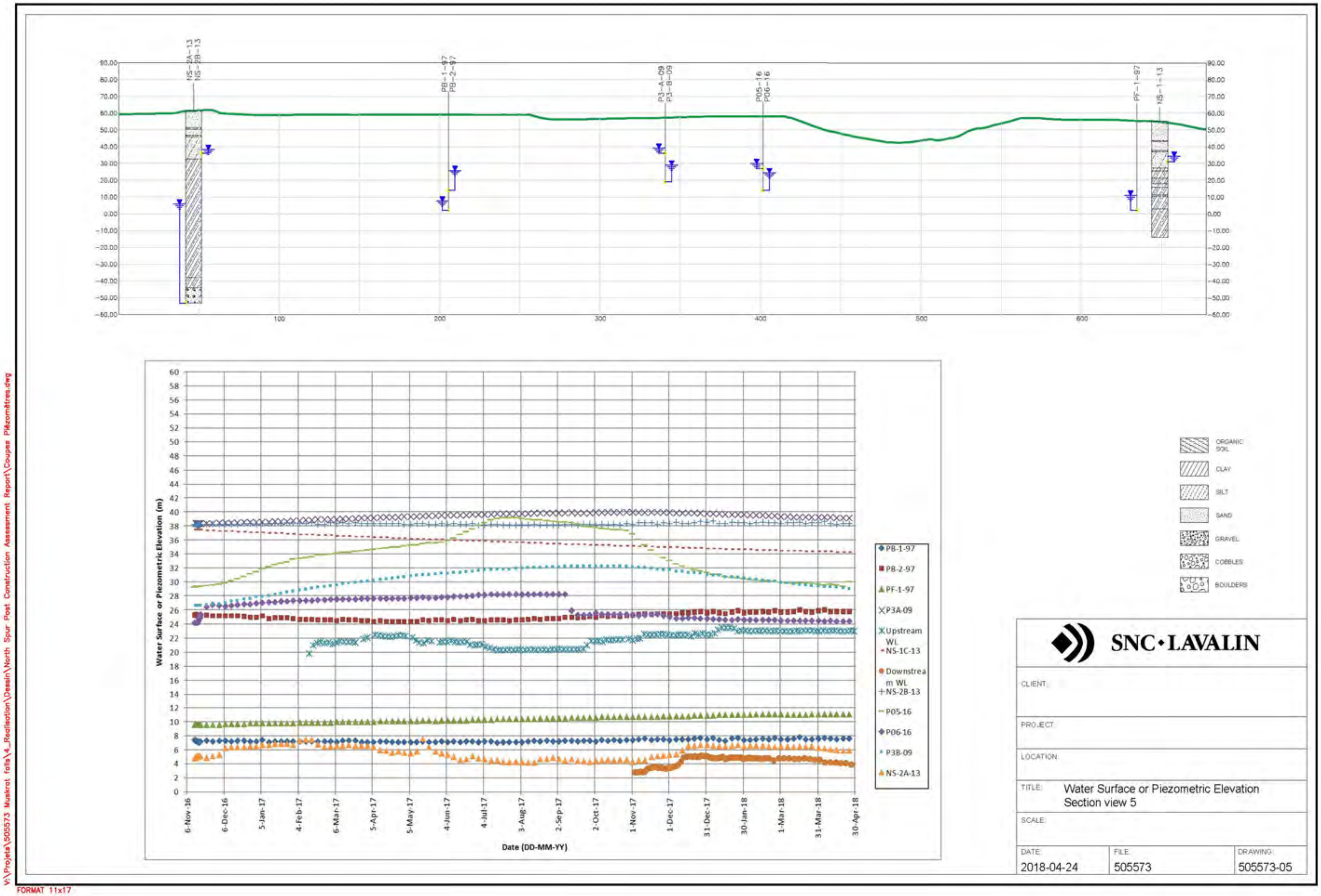


Figure B-7: Section 6

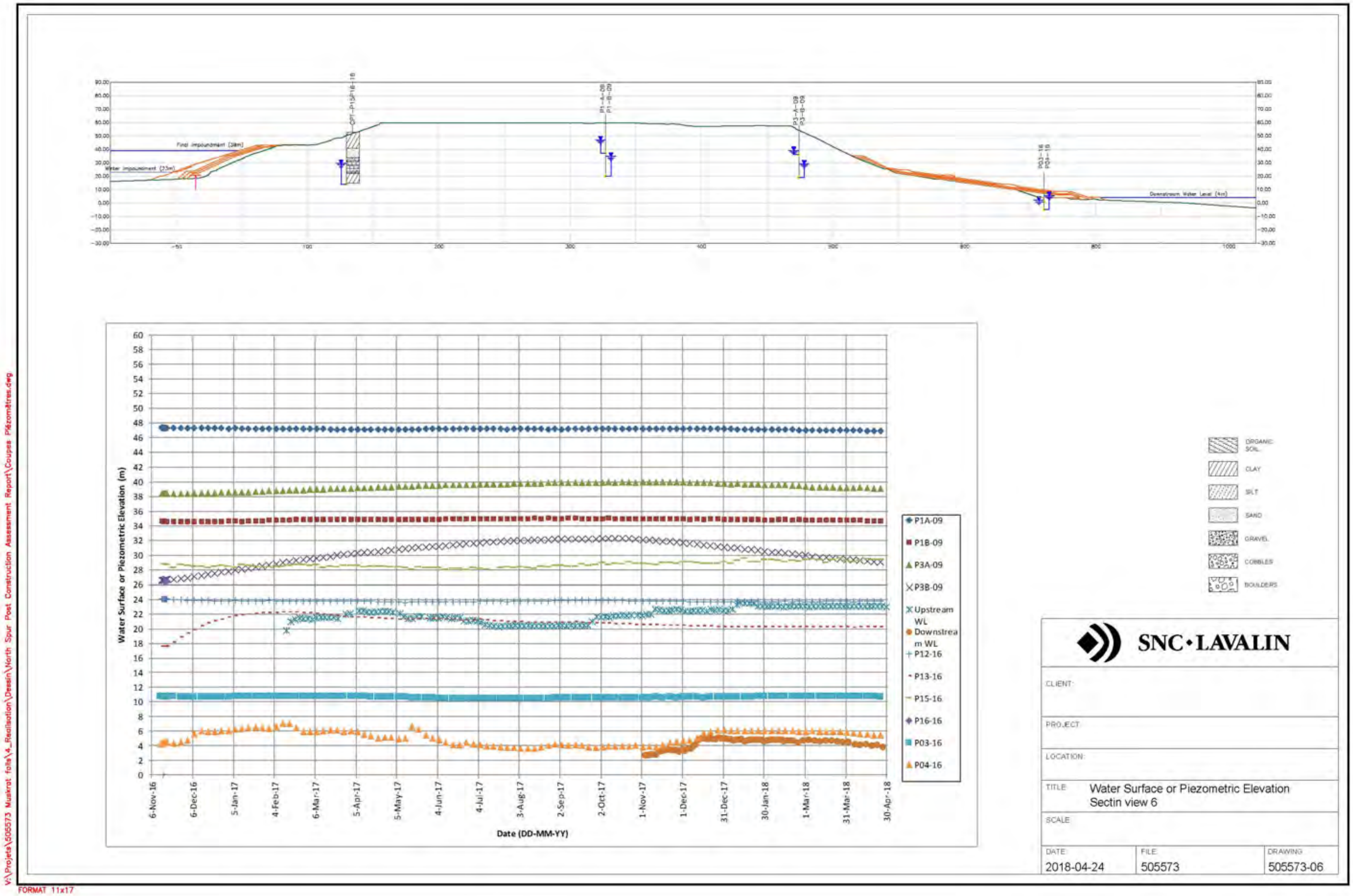
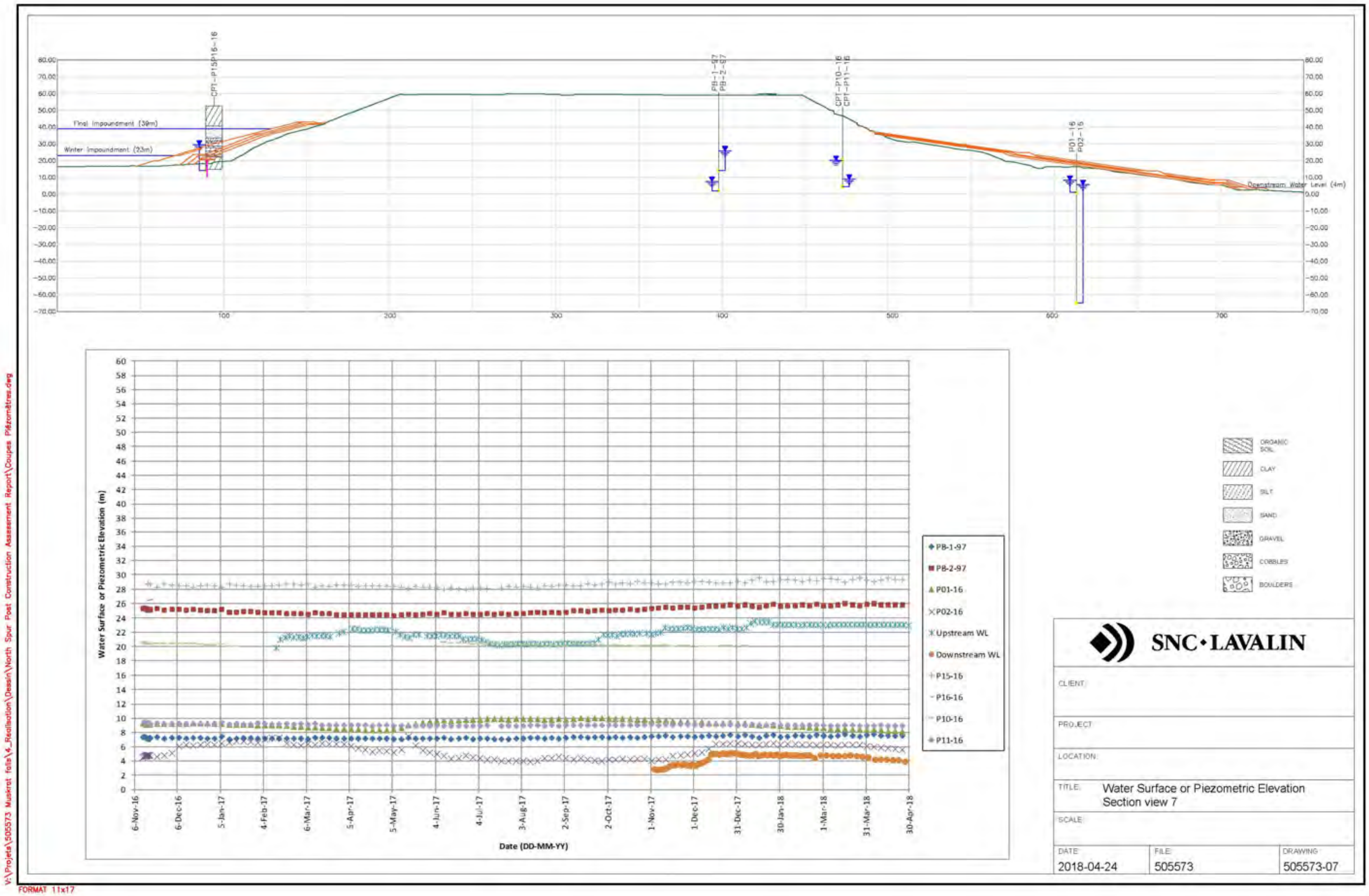


Figure B-8: Section 7



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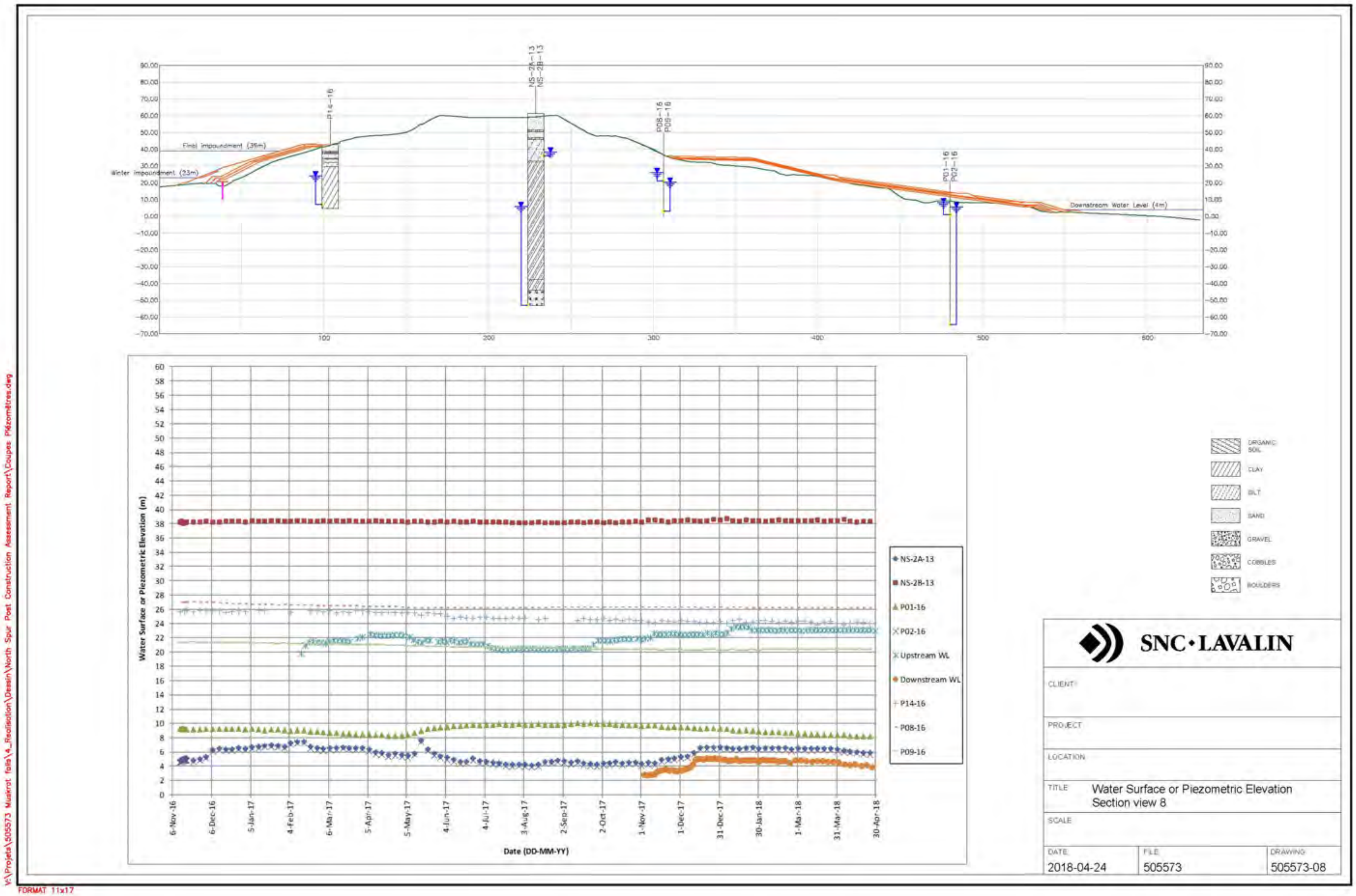


Figure B-10: Section A-A

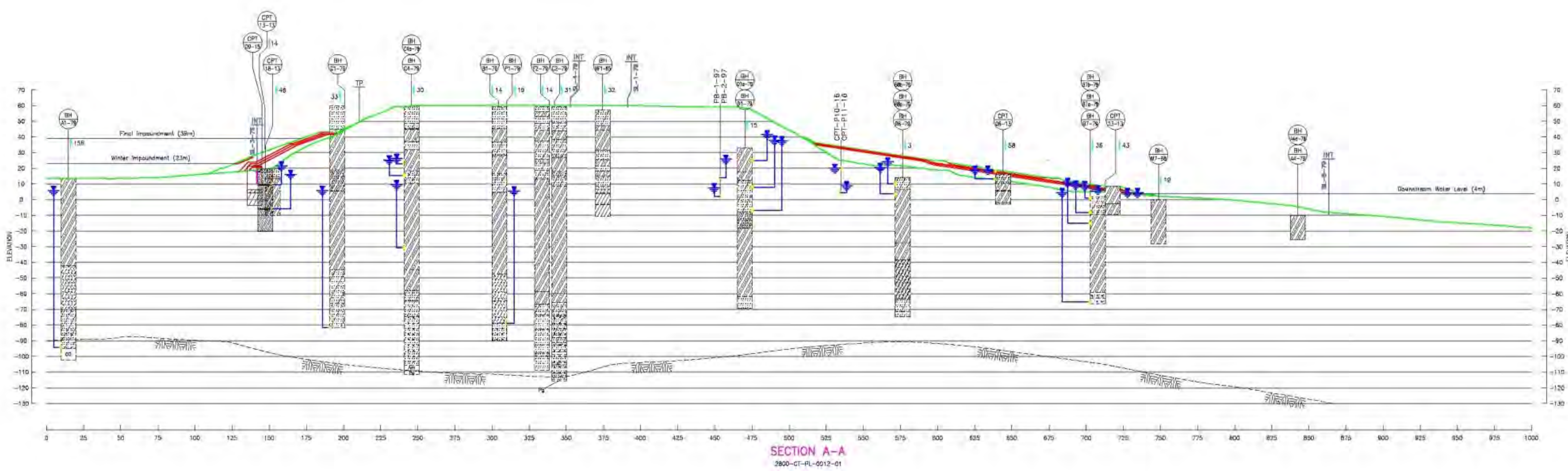


Figure B-11: Section B-B

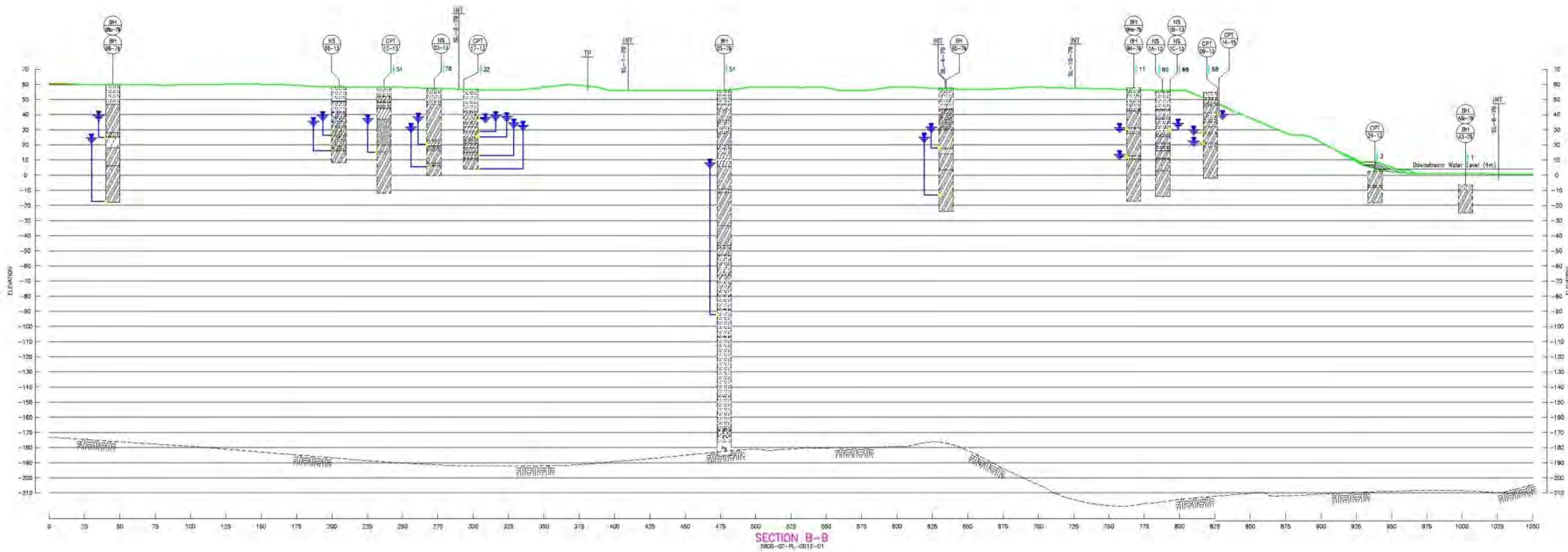
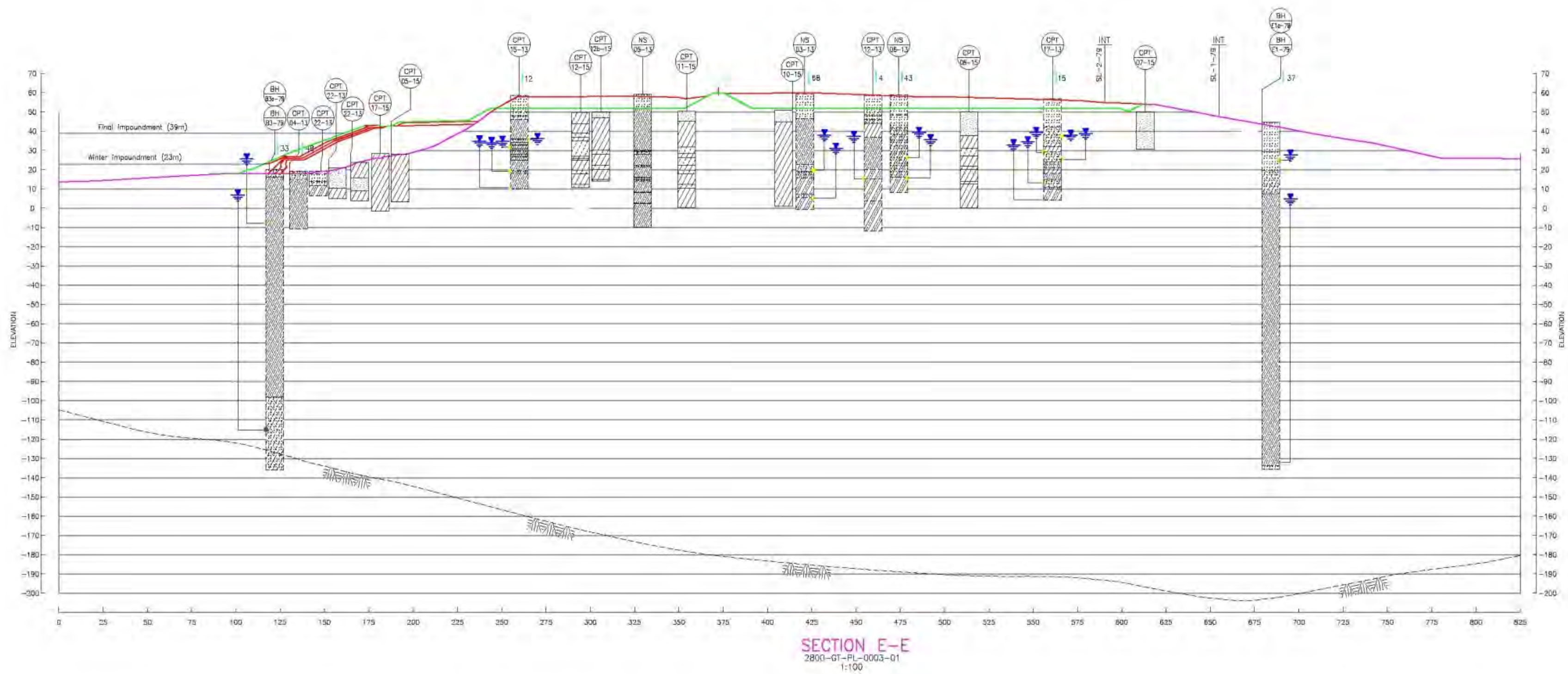
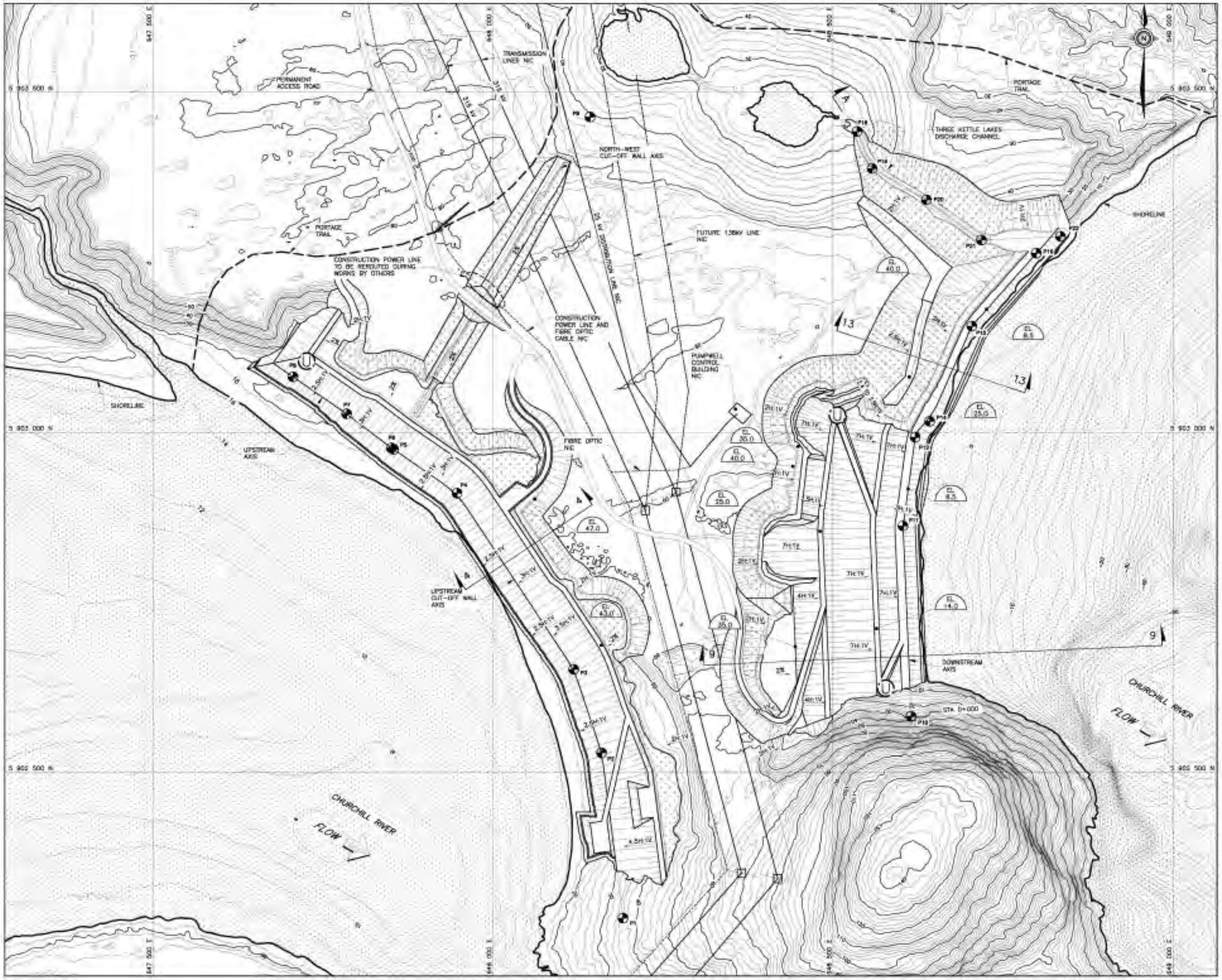


Figure B-12: Section E-E



# APPENDIX C

# RESULTS OF STABILITY ANALYSES FOR THE ASSESSMENT




PLAN

Figure C-1

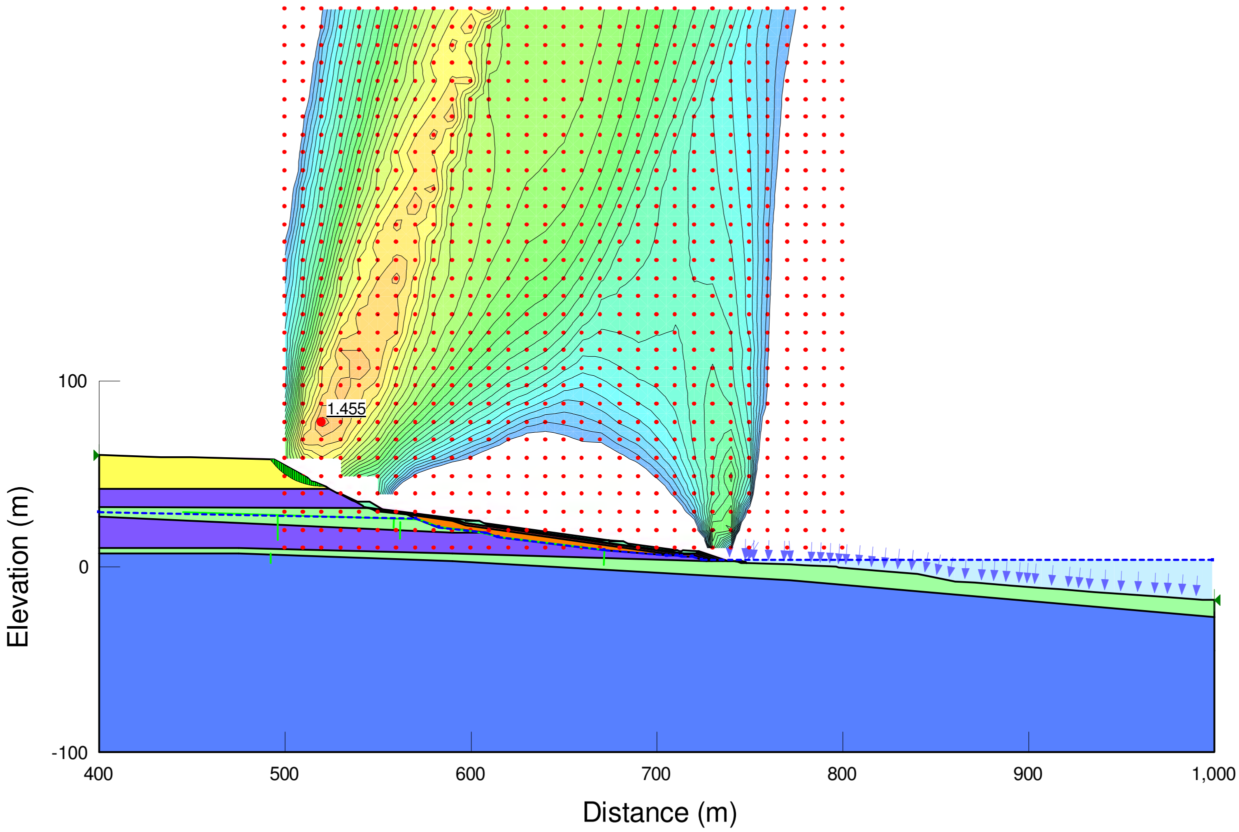



Figure C-2 Downstream Area – Cross Section 9 – Current Conditions

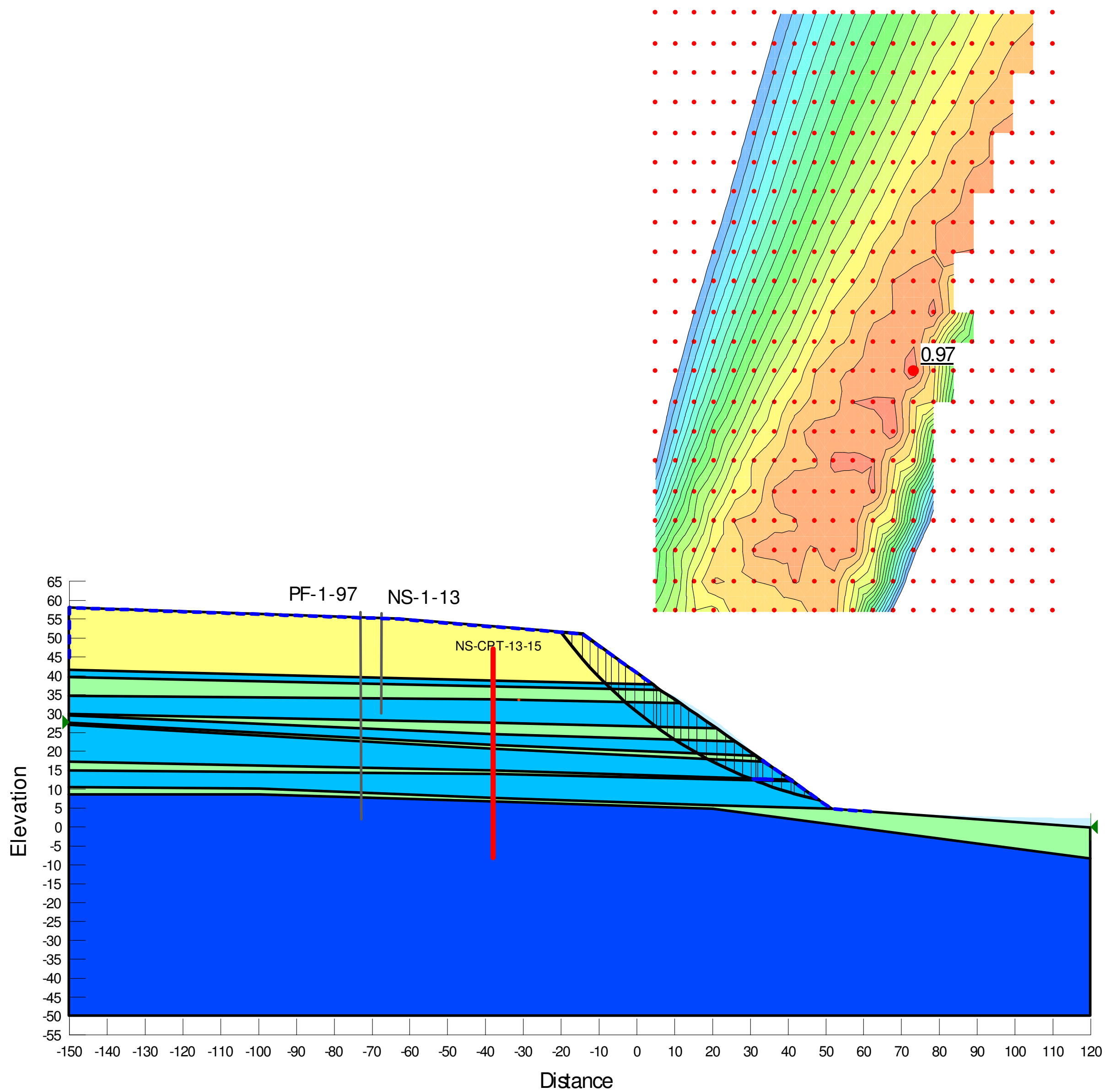



Figure C-3 Downstream Area – Cross Section 13 – Before Stabilization Works

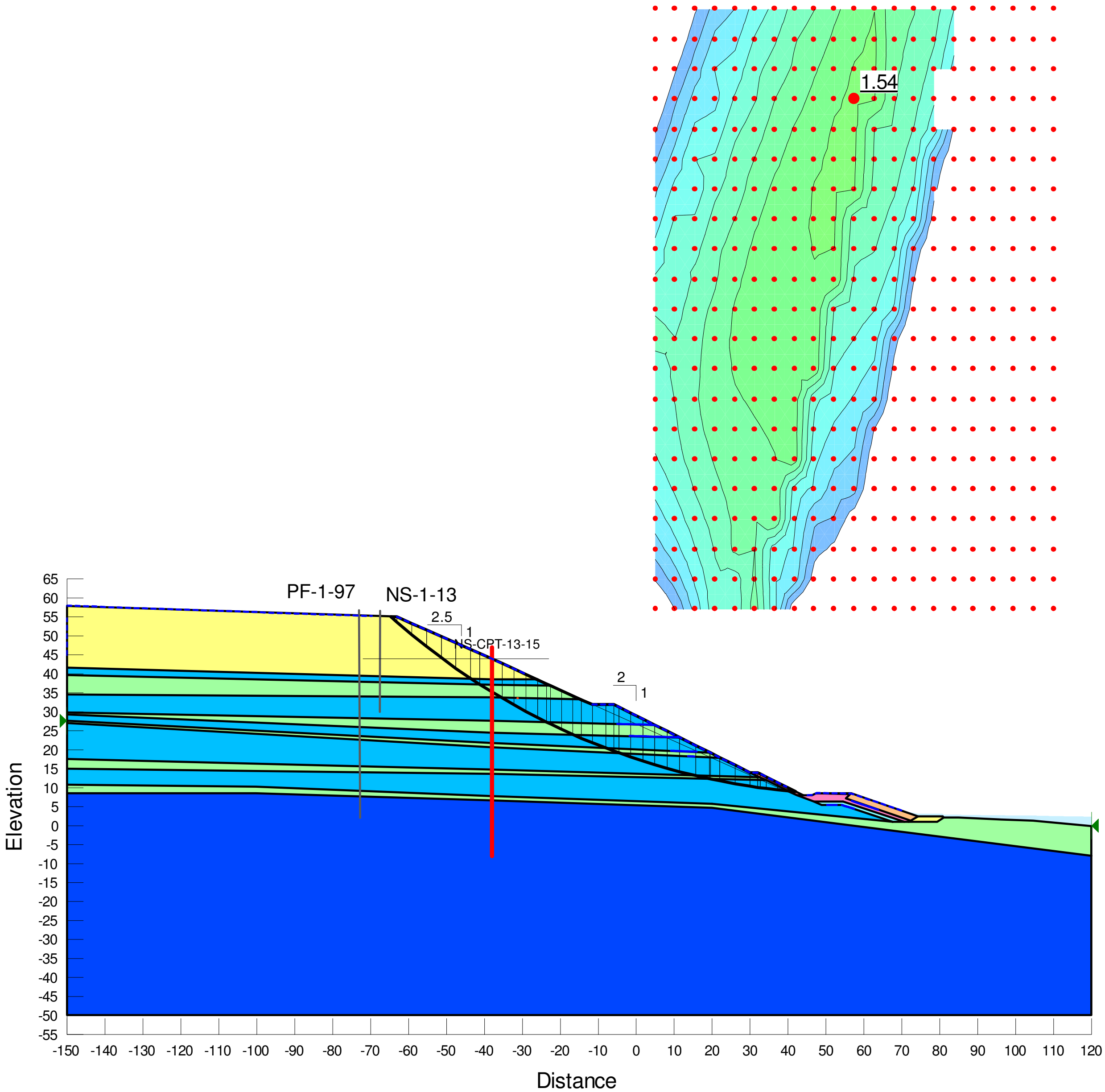



Figure C-4 Downstream Area – Cross Section 13 – Current Conditions

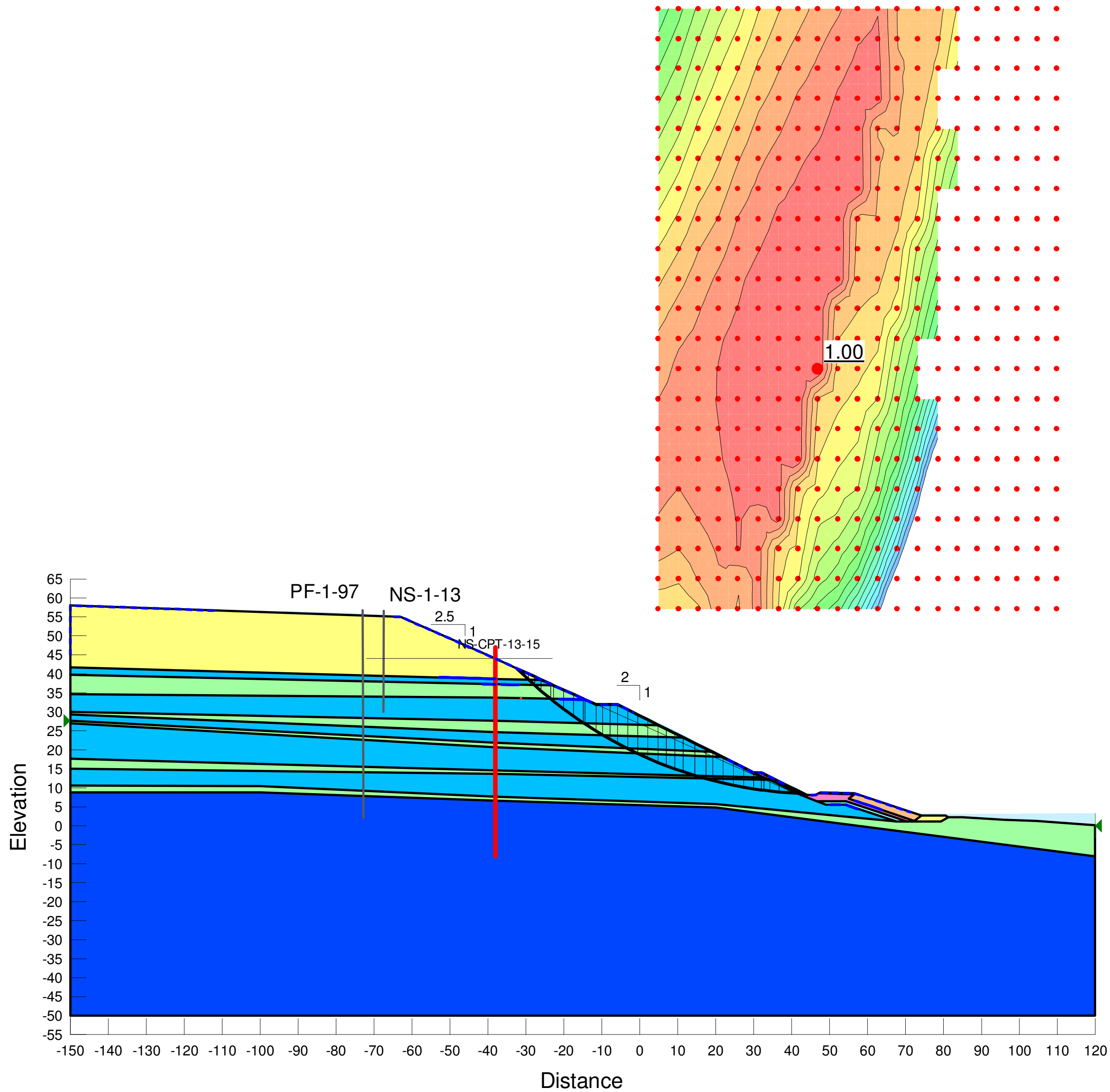



Figure C-5 Downstream Area – Cross Section 13 – Sensitivity Analysis, FS ± 1.0

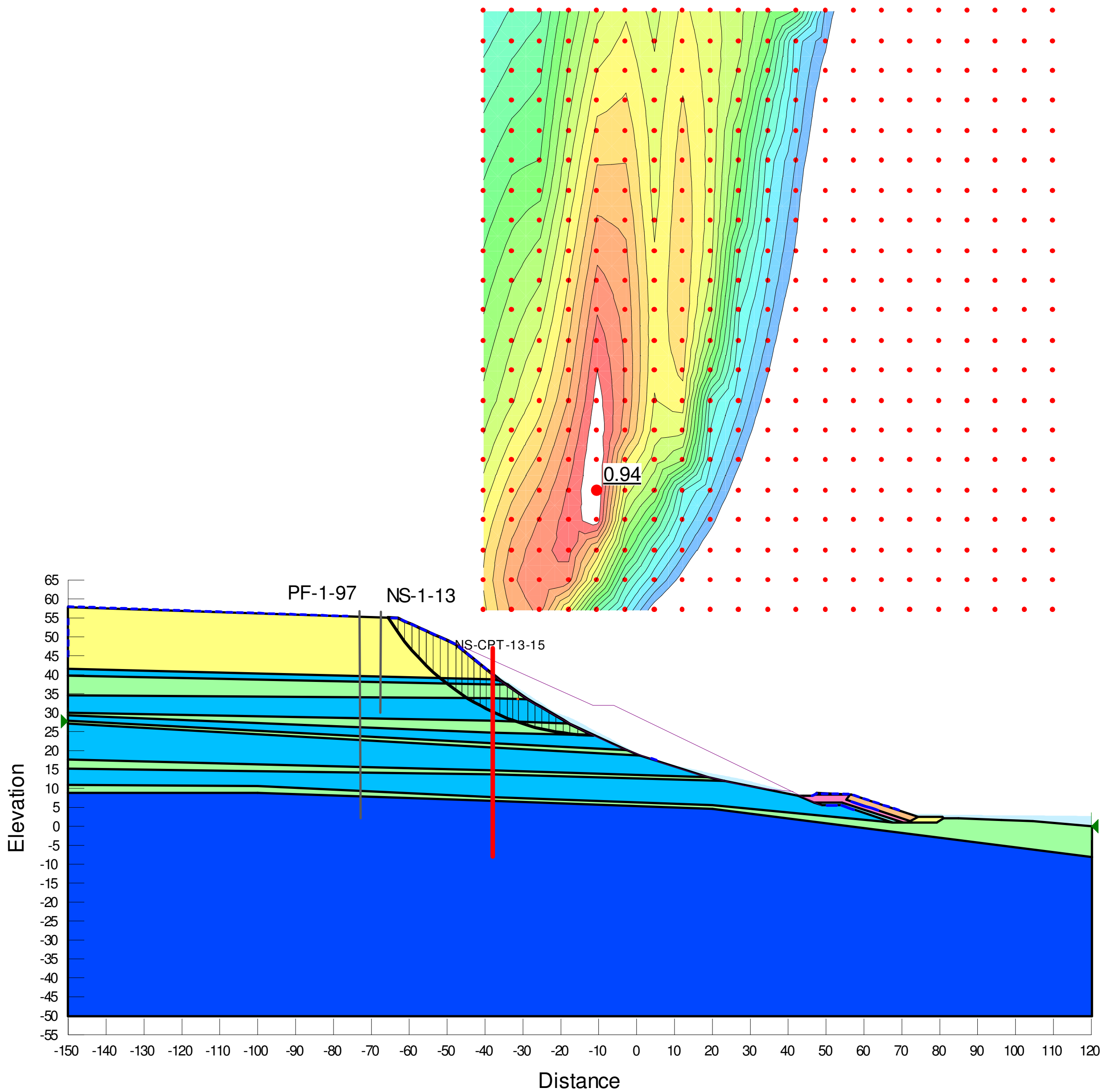



Figure C-6 Downstream Area – Cross Section 13 – Progressive Failure

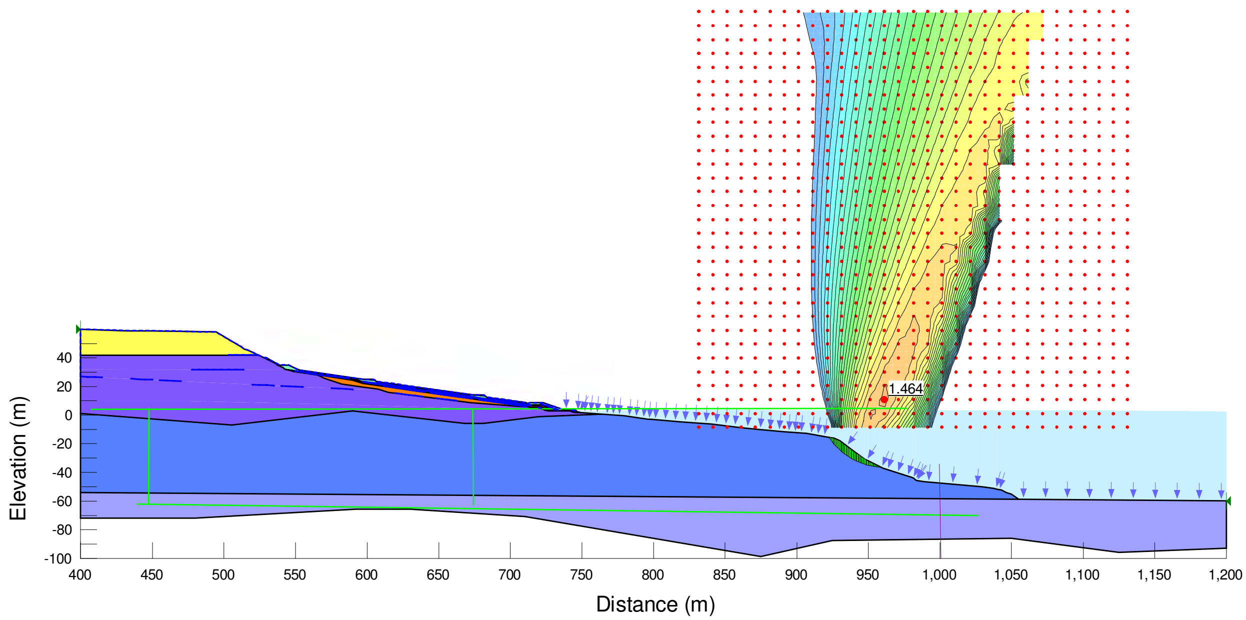



Figure C-7 Downstream Area – Cross Section 9 – Depression – Current Conditions

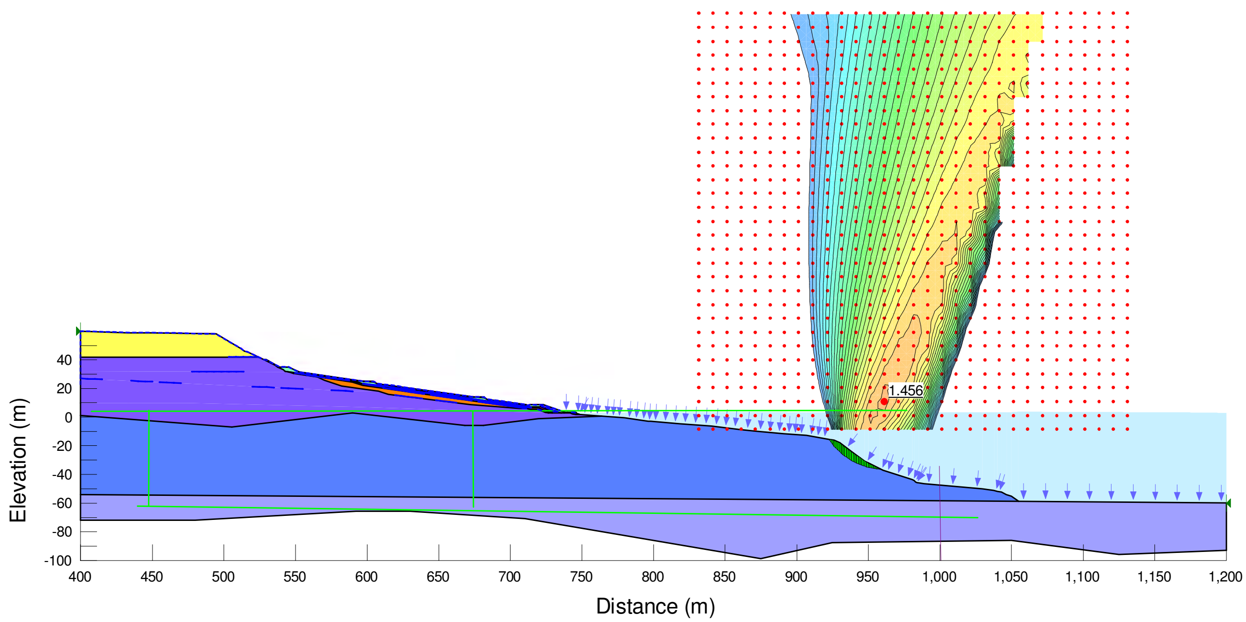



Figure C-8 Downstream Area – Cross Section 9 – Depression –Pressure in Lower Aquifer increased in proportion to FSL

# APPENDIX D

## MEMORANDUM CONCERNING EFFECT OF PORE WATER PRESSURES ON THE DYNAMIC ANALYSIS STUDY RESULTS

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Internal Note

Certifiée ISO 9001

| | | | |
|---|---|---|---|
| **TO :** | Régis Bouchard, ing. | **DATE :** | August 1st, 2018 |
| **C.C. :** | Anthony Rattue, ing., Greg Snyder, P.Eng., Alvaro Ceballos, ing. | | |
| **FROM :** | Denise Leahy, ing. | **REF. :** | 505573-3281-4GER-xxxx-PA |
| **Verified by:** | Amir Zamani, ing. | | |
| **SUBJECT :** | North Spur Stabilization Works – Dynamic Analysis Study - Effect of water pressure | | |

This internal note is presenting general comments about the effect of water pressure on the empirical methods that were used in the Dynamic Analysis Study (SLI, 2015) of the North Spur.

In the dynamic analyses of SLI (2015), the water pressure distribution in the North Spur assumed after the mitigation measures and filling of the reservoir was based on levels predicted from previous seepage analyses performed for the expected normal long term seepage conditions. More recent data indicate that the water pressure could differ from what was assumed in the analyses: lower than assumed in some areas and greater in others. The effect of these differences on the conclusions of the dynamic study will be discussed below.

# 1.0 EMPIRICAL METHODS

As presented in chapter 5.0 of the Dynamic Analysis Study Report (SLI, 2015), in the empirical methods, the imposed seismic loading (CSR) is compared to the cyclic resistance loading (CRR) to which the material can resist without undergoing liquefaction (for granular materials) or cyclic softening (for clay-like materials). A factor of safety is calculated as FS = CRR/CSR.

## 1.1 Imposed seismic loading

The imposed seismic loading is represented by the Cyclic Stress Ratio (CSR) estimated using site specific dynamic response analyses. The site is represented by either a one-dimensional vertical soil column (1D) or a two-dimensional section (2D) using equivalent-linear total stress analyses or non-linear effective stress analyses, as described below.

## 1.2 Cyclic resistance loading

The Cyclic Resistance Ratio (CRR) is estimated based on SPT or CPT tests for granular material, and based on plasticity and undrained shear strength for clay-like material.

The estimation of CRR in SLI (2015) was based on the relationships proposed by Idriss and Boulanger (2008) to estimate CRR for liquefaction (granular materials) and for cyclic softening (clay). This estimation is generally conservative as it represents a lower bound of the liquefaction or cyclic softening cases of an important case history database.

In the procedure proposed by Idriss and Boulanger (2008), CRR and CSR are normalised relative to the effective overburden stress, $\sigma'_{vc}$.

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## 2.0 Effect of water pressure on CRR

### 2.1.1 CRR for granular material

The estimation of CRR for granular material was based on SPT normalised index $(N_1)_{60cs}$ and on CPT normalised tip resistance $q_{c1N}$. These in situ tests were realized under the water pressure conditions before mitigation measures and filling the reservoir. Both indices $(N_1)_{60cs}$ and $q_{c1N}$ are normalized for an overburden pressure of 100 kPa and represent the density or compactness of the material at the time of the test.

In the dynamic study, the effect of the changes of the water pressure conditions between before and after the mitigation works and the filling of the reservoir was neglected. In general, the water pressure in the long term were believed to be somewhat higher after filling of the reservoir than at the time of testing; the effective overburden pressure would then be somewhat lower and could lead to a slight reduction of the density but it is generally neglected. Should the water pressure be lower than assumed, it is conservative to neglect the slight potential increase in density.

### 2.1.2 CRR for clay-like material

CRR for cyclic softening of clay-like material was estimated based on undrained shear strength, Su, interpreted from CPT tip resistance based on correlations with undrained shear strength measured in vane field tests. CRR was estimated based on Su/s'v for the water pressure and the effective overburden pressure at the time of the in situ Su measurement. These in situ tests were realized under the water pressure conditions before mitigation measures and filling of the reservoir. Based on CPT data, the undrained shear strength at a given elevation is generally similar throughout the North Spur with the OCR at about 1.0 below the crest and between 3 and 15 below the downstream toe. It was assumed that the combined effect of unloading of the crest and water pressure changes would not reduce significantly the Su/s'v ratio.

## 3.0 Effect of water pressure on CSR

As noted above, in the procedure proposed by Idriss and Boulanger (2008), CRR and CSR are normalised relative to the effective overburden stress, $\sigma'_{vc}$. CSR is estimated based on the maximum cyclic shear stress $\tau_{max}$profile as:

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}}$$

Three types of dynamic response analyses were carried out: 1D and 2D equivalent-linear analyses, and 2D non-linear analyses; these analyses provide the $\tau_{max}$ profile (1D) or distribution (2D). In general, the position of the water table has no direct effect on the calculation of the $\tau_{max}$ profile. Therefore, the main effect to be expected should be on CSR estimated based on the $\tau_{max}$ profile and normalized relative to $\sigma'_{vc}$.

However, in 2D analyses, it has an effect on the dynamic properties of the soil materials and therefore on the outcome of the dynamic analyses.

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## 3.1 Effect on dynamic properties

In 1D analysis, Vs and Gmax profiles were estimated based on two seismic CPT profiles and on triaxial seismometer measurements carried from the crest. Therefore, for the 1D analyses of the crest profiles (upstream S1 and downstream P1), the position of the water table had no effect on dynamic properties.

For the P2 downstream toe profile, no triaxial seismometer test nor deep borehole were performed; a Gmax distribution was estimated as a function of the effective overburden stress based on P1 and S1 profiles, and the results were extrapolated to section P2 based on the assumed long term normal water pressure distribution as illustrated on Figure C-18 of SLI (2015).

Such a common Gmax – $\sigma'_v$ relationship was also required for the 2D analyses for a smooth transition between the crest and toe profiles. However, given the depth of the water table and the limited variation expected compared with the water pressure distribution assumed in the analyses, the effect on Gmax should be limited and the effect on the $\tau_{max}$ profile is expected to be negligible, except in the shallow zones when the water table is high.

## 3.2 Comparison between upstream and downstream conditions

The 1D equivalent-linear analyses on section P1 (top of the hill conditions of Section 13- see Figure 1 - based on CPT-09 profile) were performed for two water table elevation conditions: the downstream conditions at 15 m and the upstream conditions at 30 m. The Upstream conditions were modeled using the same stratigraphic profile as the P1 Downstream (WL 15 m) analyses but assuming a normal reservoir elevation of 39 m and a water level of 30 m in the slope below the crest. The comparison of the results illustrates the effect of the water pressure conditions on the estimation of CSR (see Figure 2): the upstream conditions are more severe because the water table elevation is higher; the saturated zone is reaching higher in the stratified drift and the effective vertical stress is lower; therefore the CSR is higher while CRR is the same for both conditions.

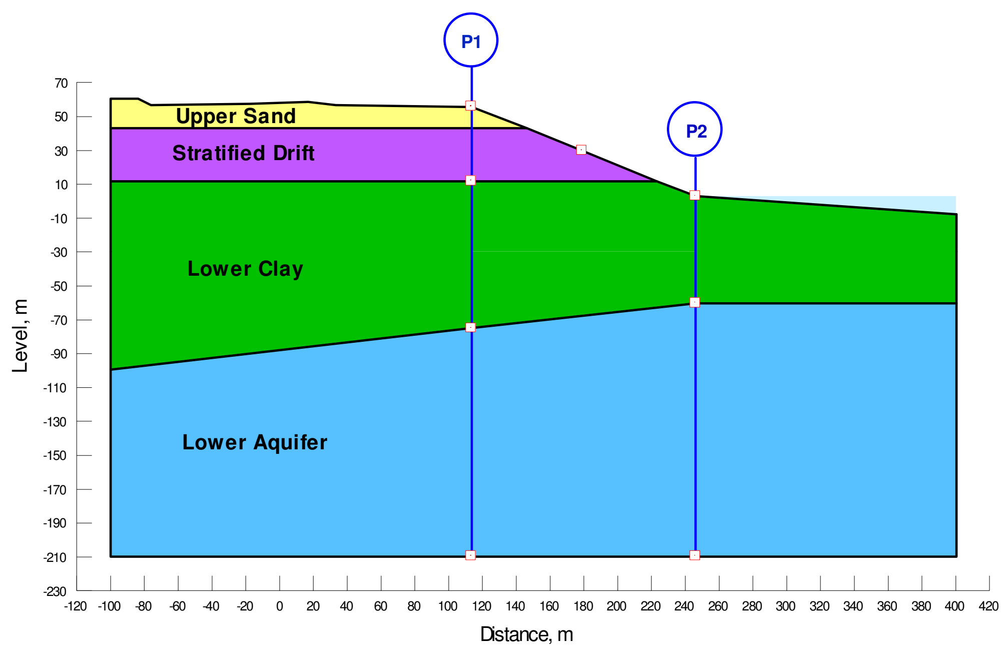


**Figure 1: Section 13 - 2D Stratigraphic Model**

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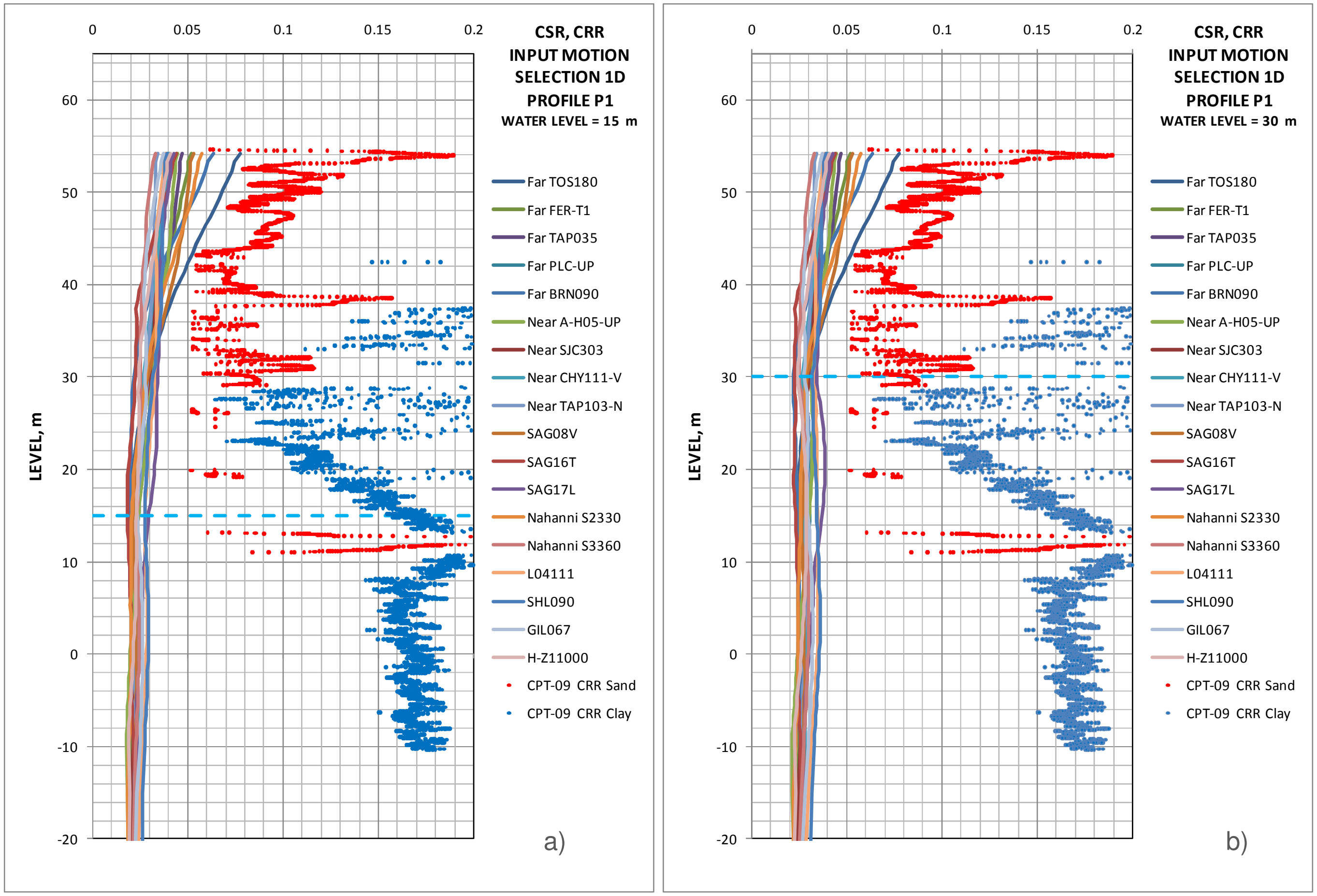


**Figure 2 - CRR and CSR for P1 Profile –Input Motions Selected for 1D Analyses**
**a) Downstream (WL 15 m); b) Upstream (WL 30 m)**

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## 4.0 Conclusion

The effect of the water pressure distribution on the outcome of dynamic response analyses as performed in SLI (2015) Dynamic Analysis Study was summarised.

A variation of water pressure would have no effect on the estimation of CRR for both sandy and clayey materials. The estimation of CRR for sandy material was based on material density as measured in situ for the conditions before the mitigation measures and the filling of the reservoir. The effect of a variation of the effective overburden pressure on the material density was neglected. The estimation of CRR for clayey material was based on in situ Su measurements normalised by the effective overburden pressure at the time of testing.

A variation of the water pressure distribution would have some effect on the estimation of the material dynamic properties since the shear modulus Gmax is estimated as a function of the effective overburden based on shear wave velocity profiles measured below the crest. The effect on the $\tau_{max}$ profile is expected to be negligible, except in the shallow zones in the case where the water table is high.

The main effect to be expected from a variation of the water pressure relative to that assumed in the SLI (2015 study should be on CSR estimated based on the $\tau_{max}$ profile and normalized relative to $\sigma'_{vc}$:

$$\mathrm{CSR} = 0.65\frac{\tau_{\max}}{\sigma'_{vc}}$$

The effect on the factor of safety FS = CRR/CSR based on 1D dynamic responses analyses was illustrated by the comparison of results on section P1 for two water table elevations: the downstream conditions at 15 m and the upstream conditions at 30 m. The upstream conditions are more severe because the water table elevation is higher; the saturated zone is reaching higher in the stratified drift and the effective vertical stress is lower; therefore the CSR is higher while CRR is constant. However the relative effect on FS is weak as can be seen on Figure 2.

This cannot be generalised for any water pressure variation but for most of the North Spur, the variation between assumed and measured long term water pressure distribution is expected to be relatively low compared with the order of magnitude of the effective overburden pressure and therefore, the conclusions of the SLI (2015) study should apply. If significant water pressure variation is expected in relatively shallow zones, complementary verification analyses would be advisable.

## 5.0 References

Beaty, M. and Byrne, P.M. (2011) UBCSAND Constitutive Model, Version 904aR. Beaty Engineering LLC and University of British Columbia. February, 2011.

Idriss, I.M. and Boulanger, R.W. (2008). Soil Liquefaction during Earthquakes. MNO-12, Earthquake Engineering Research Institute, Oakland, CA.

SNC-Lavalin (SLI, 2015) Lower Churchill Project - North Spur Stabilization Works - Dynamic Analysis Study. Engineering report SLI Document No. 505573-3281-4GER-0005-00 / Nalcor Reference No. MFA-SN-CD-2800-GT-RP-0007-01 Rev. B1 / 2015-10-05.