LOWER CHURCHILL RIVER RIVERBANK STABILITY REPORT

PREPARED FOR

Grand Riverkeeper Labrador, Inc.

BY

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EXECUTIVE SUMMARY

On specific questions regarding the formation of the Churchill River Valley and comments on stability issues related to the North Spur

The intent of this Report is to explain the extraordinary features of the Churchill River Valley, and to comment on North Spur stability regarding the proposed future impoundment.

The Churchill Riverbed. The soil properties related to lean clay formations in the Churchill River Valley have a significant impact on the assessment of slope stability and the safety factors related to it. The North Spur, in its present state, has numerous large landslide scars, of which some are due to recent landslide events, indicating that erosion and land-sliding — as in the rest of the valley — is an ongoing geological process.

The shape of the Churchill River bed, both upstream and downstream of Muskrat Falls, differs in an exceptional way from usual riverbed formations. Along a stretch of at least 30 km, the Churchill River Valley most often has a width of about 1000 m. Yet it may locally vary from a minimum width of 600 m up to a maximum of 1500 m. Except for an area immediately downstream of Muskrat Falls, the riverbed is notably shallow. Even in places where the water current was observed as being significant, the water depth was only about 0.4 m.

The exceptional depth of the riverbed — about 70 metres — immediately downstream of Muskrat Falls is due to a whirlpool in which the water current is so strong that sedimentation of the eroded marine sediments originating from the upper Churchill Valley cannot take place.

Stability of the Riverbanks. Modern research requires that the stability analysis of long slopes with sensitive clay must carefully take into consideration the risk of “brittle slope failure”. As the proposed impoundment would exert a gigantic external lateral pressure — locally, on the cut-off wall, and on the under-consolidated mixed sediments behind it — a careful study related to progressive failure is an unavoidably necessary measure.

Friction as such is normally a dependable factor promoting stability, but in the current case the voids in the loose mixed soil are filled with soft and sensitive clay material, the shear strength of which cannot be predicted by current or past active vertical effective pressures.

The properties of the very lean Upper Clay layers of the North Spur differ from those of normal clays. In normal clays, the clay content is usually considerably in excess of the void volume of the more coarse-grained material. In very lean clays in which the coarse-grained portions of the soil have a loose granular structure, shear deformation [from sideways pressure] will tend to decrease the pore volume containing clay and water. This brings about an inherent tendency to *soil liquefaction* and loss of stabilising friction, allowing soils to slide.

Many of the clays of the Churchill Valley have the tendency for this unusual kind of liquefaction. This tendency makes the results of standard soil investigations — and the subsequent determination of safety factors in respect of slope stability — very unreliable. This caution particularly applies if the stability analysis is based on the conventional Plastic Equilibrium Mode.

Dependable stability analyses must therefore emphasise the potential risk of progressive failure formation due to the intrinsic tendency to liquefaction, particularly regarding the Upper Clay layers of the North Spur.

The fact that the Churchill River Valley has developed in the way it has, and continues to do so, substantiates the validity of the geotechnical conditions described above, and which are dealt with in greater detail in the body of this Report.

Mathematical Analysis of Stability. In order to illustrate the specific stability conditions along the riverside slopes of the Churchill River Valley, a mathematical stability analysis of a typical riverside situation has been carried out in Appendix A (See also Figure 4.4). The results of this analysis are commented on in Section 4.23.

As the clay content of the mixed clayey soil layer is extremely low — the soil mainly consisting of sand and silt — the stability investigation is chiefly based on the frictional resistance of the mixed soil. Two cases have been analysed. In Case A the groundwater level is assumed to be at the ground surface, yielding a rough safety factor of only 1.09. In Case B the groundwater level is taken as being at 5 metres below the ground surface, yielding a safety factor of about 1.43.

These results demonstrate the decisive effect of varying groundwater conditions in the soil mass behind a typical riverside slope in the Churchill River Valley. This analysis also indicates why the steep riverside slopes may, at least transiently, remain stable.

Such mathematical calculations are of limited value without the support of appropriate soil testing. Such investigation must be based on *rapid, un-drained* direct shear tests on virtually undisturbed clay samples, since progressive failures tend to develop at high rates of shear deformation. The diameter of the test samples should not be less than 100 mm. These direct shear tests should not be deformation-controlled, i.e. they should be carried out in such a way that the development of failure surfaces remains unconstrained.

Landslide Risk at the North Spur. The soil masses behind the riverbank slopes of the Churchill River have exerted their vertical pressures for millennia. And yet, as explained in this report, it is when even moderate changes of lateral loading [sideways pressure] take place — such as hydraulic pressure change, seismic activity, gradually failing lateral support, or creep deformations — that the propensity to liquefaction and the resulting loss of shear resistance can occur, releasing enormous landslides of the kind at Edward Island a few years ago.

To reduce or eliminate landslide risk at the dam, NALCOR intends to install a cut-off wall — a watertight membrane — to help stabilise the upstream slope of the North Spur. This would, of course, be advantageous for increasing the effective pressure on granular soil layers that truly abide by the normal laws of frictional resistance.

However, the behaviour of a mixed soil with *lean clay content* may be totally different, as will be shown in Section 2. Reduced porosity caused by additional shear deformation may result in liquefaction and instability — and in this case the shear deformation and resulting loss of shear resistance may in turn generate a tendency to liquefaction along the entire length of a potential failure surface, resulting in a condition of *global progressive failure*.

In fact, considering the type of sensitive behaviour of the lean Upper Clay No. 2 in the North Spur, the local concentration of hydraulic pressure at the proposed cut-off wall may even create a highly disadvantageous condition. Critically, local concentrated loading is the most common and most effective *triggering agent* in the development of extensive progressive landslides, i.e. slides extending more than 70 to 100 metres.

Conclusion. The contention of this document is not that the North Spur dam containment is bound to fail. Yet, considering the enormous threat to populated areas that would result from a breakage of the North Spur ridge, all stability analyses related to the impoundment must prove that the possibility of such a failure has been definitely excluded.

In the opinion of this engineer, not all of the relevant and appropriate analyses have yet been carried out with robust favourable results. Thus a catastrophic landslide on the North Spur of the Muskrat Falls dam must still be treated as a possible, foreseeable event.

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REPORT

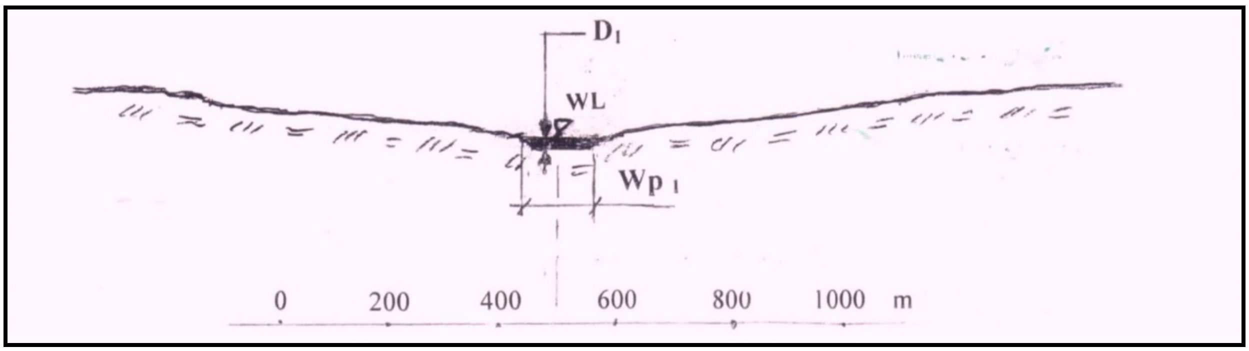
On specific questions regarding the formation of the Churchill River Valley and comments on stability issues related to the North Spur

1. General

The Churchill River Valley in Labrador (Newfoundland) differs from most river valleys as seen by the author of this article — whether it be observations on land or from high up in aeroplanes or helicopters.

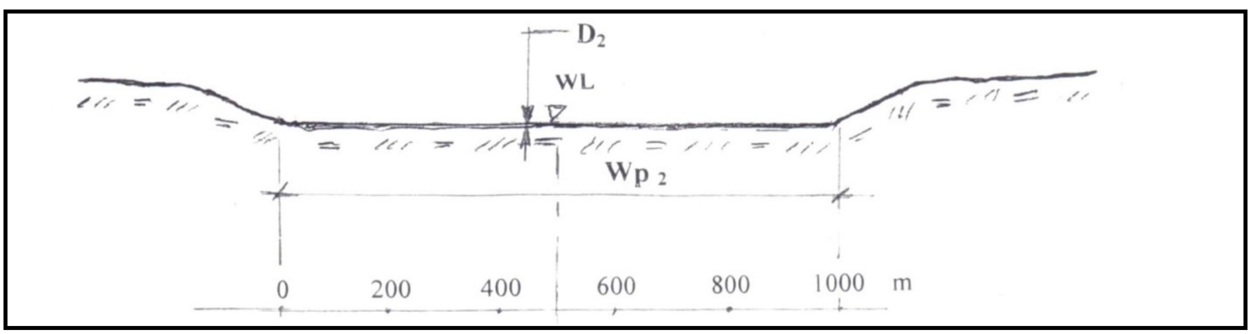
Except when passing through lakes, the width of a normal riverbed in looser sedimentary formations is related to its gradient and the amount of flowing water per second. Even in wide, flat, and in the direction of flow gently sloping areas, rivers tend to meander developing a riverbed width corresponding to water-flow and current riverbed gradient.

Figure 1.1. Riverbed formation under normal geological conditions, i.e. stable soils with little tendency to liquefaction or weakness in any specific sedimentary layer



Wp_1 = Riverbed width (≈ Wet perimeter), D = Mean water depth (WL = Water level).

Figure 1.2. Riverbed formation of the type occurring in the Churchill River valley with a remarkably wide but shallow riverbed

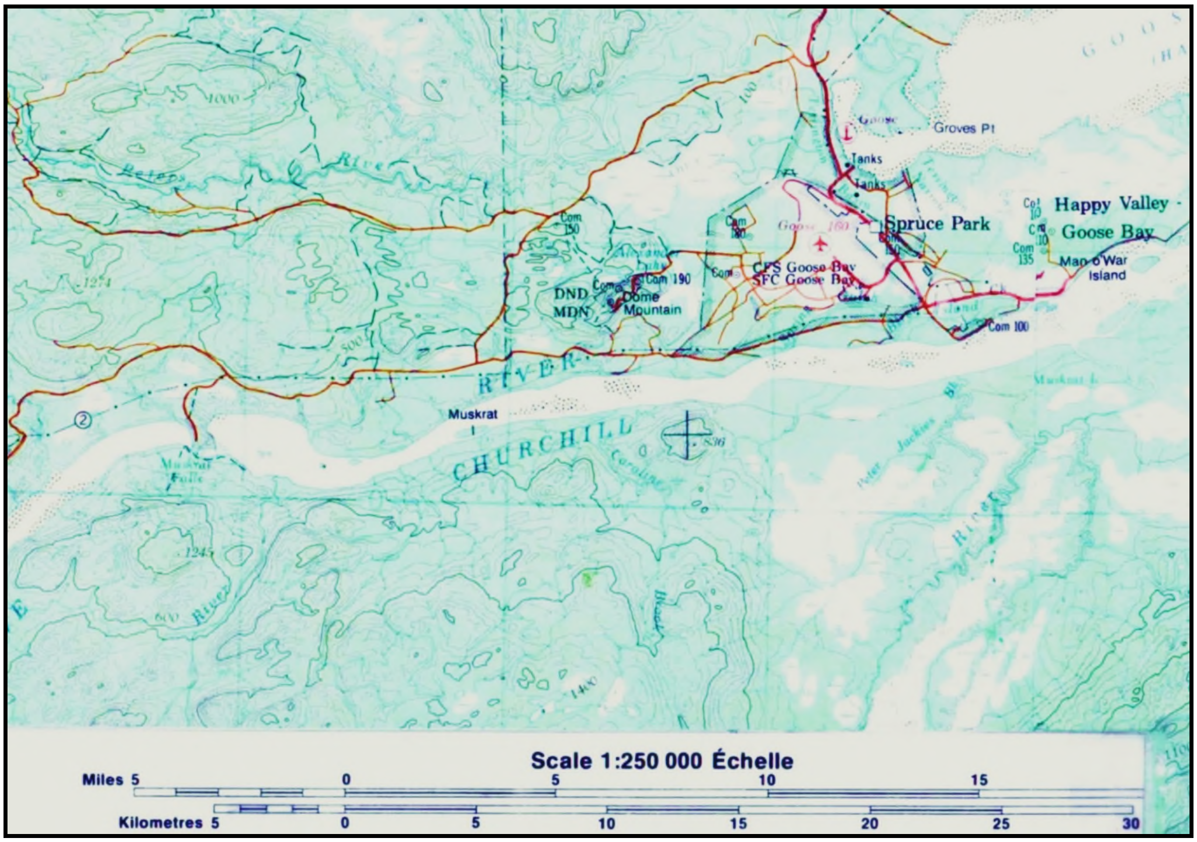


Denotations as in Figure 1.1 above.

The width of the Churchill riverbed, upstream and downstream of Muskrat Falls, differs in an exceptional way from normal riverbed formations conforming to the description pertaining to Figure 1.1.

In the Churchill River Valley, which in principle is shaped as shown in Figure 1.2, the riverbed — along a stretch of at least some 30 km — normally has a width of about 1000 m. Yet, it may locally vary from minimum width of 600 m up to a maximum of 1500 m.

Figure 1.3. Map showing the Churchill riverbed upstream and downstream of Muskrat Falls



(Copied from part of map produced by the Canada Centre for Mapping, Department of EM&R)

Except for an area immediately downstream of Muskrat Falls, the riverbed is notably shallow. Even in places where the water current was observed as being significant, the water depth was only about 0.4 m.

1.1 Whirlpool below Muskrat Falls.

The exceptional depth of the riverbed — about 70 metres — immediately downstream of Muskrat Falls is due to a whirlpool in which the water current is so strong that sedimentation of the eroded marine sediments originating from the upper Churchill Valley cannot take place.

Figure 1.4. Arial photo of discoloured water in the whirlpool immediately below Muskrat Falls

This whirlpool is the reason for the exceptional water depth immediately below the North Spur. The discolouring of the water is due to the presence of soil particles transported by the streaming water.

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2. On the extreme sensitivity of lean clays

Fat clays, i.e. soils rich in clay particles < 0.002 mm, are known to develop extreme sensitivity if subjected to groundwater percolation over time. However, clay sensitivity also depends on various other factors such as:

2.1 The type of biotite

This refers to the chemical nature of the "flat" crystals forming the clay constituents. (Terzaghi & Peck [1]). There are four common types clay biotite, namely:

a) Montmorillonite

b) Illite

c) Kaolinite

d) Chlorite

2.2 The Liquidity Index

The Liquidity Index (I_L) of a soil expresses the relationship between the actual (natural) water content (**w**), the Liquid Limit ($\mathbf{w_L}$) and the Plasticity Limit ($\mathbf{w_P}$). The water content of a soil is defined as the ratio — usually in terms of per cent (%) — between the weight of water and the weight of the dry material in the probe. The Liquidity index is defined as:

$\mathbf{I_L = (w - w_P)/(w_L - w_P)}$ .... **Equation 1**

where the parameter $\mathbf{w_L}$ represents Liquidity Limit — i.e. the water content at which the clayey soil material behaves as a liquid on being heavily remoulded.

$\mathbf{w_P}$ is the limit of plasticity defining the water content at which the clay ceases to be plastic.

The difference $\mathbf{w_L - w_P}$ signifies the range of soil plasticity and is denominated the Plasticity Index ($\mathbf{I_P}$) or the range of plasticity. Hence

$\mathbf{I_P = w_L - w_P}$ (Figures 3.3 and 3.4) .... **Equation 2**

Thus, if the water content (w) of a clay layer exceeds the liquid limit ($\mathbf{w_L}$), the value of the Liquidity index I_L will be greater than 1 (unity), signifying a point at which the soil, when excessively sheared, tends to turn into a viscous slurry — i.e. losing a significant part (or practically all) of its shear resistance.

2.3 Void Ratio and Porosity

The relative clay content of a mixed natural clayey-sandy soil layer may be expressed as the percentage relationship between the current volume of clay contained in the voids (ΔV) of the coarse granular material and the total volume of mixed soil.

2.3.1 *Critical void ratio in granular soils*

The void ratio (**n**) of a granular soil is defined as

$\Delta V/V$ = n .... **EQUATION 3A**

where V is the *total soil volume.*

Another related parameter is porosity (e)

e = $\Delta V/VS$ .... **EQUATION 3B**

where V_S is the volume of the *solid material* content.

The relation between the parameters **e** and **n** is expressed by the equations:

n = e/(1+e) .... **EQUATION 4A** **OR**

e = n/(1-n) .... **EQUATION 4B**

When a loose granular soil is sheared, its porosity (n) tends to decrease involving reduced pore volume and a lower value of the porosity number. This process continues under increasing shear strain until the pore volume change gradually ceases at a value denoted n_{crit} — i.e. a value of n at which the void volume remains constant under further shear deformation. The parameter (n_{crit}) is of crucial importance in the current context and is known in geotechnical engineering as the "critical void ratio". (Terzaghi & Peck [1])

In water-saturated soils, decreasing void volume inevitably leads to the build-up of excess pore water pressures and the related loss of frictional shear resistance — i.e. possibly to the extent that even a granular soil such as sand may momentarily liquefy.

This constitutes the well-characterised phenomenon called *soil liquefaction*, liable to take place in loosely compacted, saturated sandy (and silty/sandy) soils when subjected to significant shear strain or to the effects of vibration, by which pore void volume (porosity) decreases radically. (Vibrations may result from earthquakes, blasting, piling or vibratory activity).

Yet, although the excess pore water pressures generated by such activities may bring about soil liquefaction — i.e. total loss of friction between the soil particles — the reduction of shear resist-ance may in general only be partial.

If, on the other hand, a densely compacted soil with an identical granular structure is sheared, the porosity (n) would instead increase up to a value corresponding in principle to the critical void ratio.

2.3.2 *Liquefaction in lean clays*

The behaviour of mixed soils of sand, silt and clay strongly depends on the volume of the clay particles (V_{Clay}) in relation to the concurrent void volume of the coarser material, i.e. the ratio $V_{clay}/\Delta V$.

If, for instance, the clay content V_{Clay} is significantly greater than the void volume ΔV — i.e. **$V_{clay} >> \Delta V$** — then obviously the soil matrix will typically behave as clay, the granular soil particles being immerged in the clay without significant inter-granular contact contributing to shear resistance. Hence, the strength parameters of such a soil will then correspond to those of clay that has been exposed to the same pre-consolidation pressure.

On the other hand, if the volume of clay (i.e. clay particles including adsorbed water) initially is equal to or smaller than the concurrent void volume of the granular material, i.e. $V_{clay} \leq \Delta V$, the properties of the soil matrix become extremely complex and highly dependent on the consolidation process and possible ongoing change in the relationship between the void volume **(**ΔV) and the coexisting clay volume (V_{clay}).

Hence, in the early stages of sedimentation, such a soil features high porosity $n = \Delta V/V_S$ and the clay that fills the voids remains extremely soft. However, as the normal pressures increase due to accumulating sediments, the stiffer structure of the granular material gradually tends to carry more and more of the increasing normal stresses, while the clay content remains soft and under-consolidated. A consolidation process of this kind ends up in a condition in which a major portion of the vertical load is carried by the granular soil matrix.

This implies in turn that the degree of consolidation of the void clay is not related to the total effective sedimentary load, as was the case when $\mathbf{V_{clay}} >> \mathbf{\Delta V}$.

As a result, the mixed soil finally consists of two components with markedly different strength characteristics, i.e.:

a) A stiff but relatively porous, largely symmetrically loaded granular soil structure carrying a major part of the current vertical load;
b) Voids filled with soft clay material, the shear strength of which has little correlation with the actual effective vertical load.

Thus a mixed sandy, silty clayey soil of this kind is likely to exhibit high sensitivity when subjected to agents prone to causing liquefaction in granular soils. When sheared, the void volume of such a soil decreases, generating excess pore pressure and resulting in reduced effective stress resistance in the granular soil structure.

The shear resistance of the mixed soil may then be radically reduced — especially if, in addition, the soft void clay content is inherently sensitive or “quick”.

In fact, a lean mixed clayey sandy soil of this kind can liquefy — even to the extent that most of its shear strength is lost — with the residual resistance being reduced to a small fraction of its initial shear strength.

2.4 Conclusions

The implication of the above is that, for markedly *lean clays*, the shear strength of the clay content may not relate to the vertical effective pressure in the normal way. This means that the shear resistance of a soil of this kind can be far less — especially under shear strain — than what would be normal for a soil with higher clay content.

Yet, the main problem of the lean clay condition is its impact on sensitivity. If the pore volume of the coarse grained material is above the critical void ratio — i.e. when void volume decreases under shear deformation — the effects of significant shear deformation (as well as vibratory impact loading) is likely to generate a phenomenon very similar to hydraulic “liquefaction” in sands.

An important and complicating feature in this kind of liquefaction is that its duration may be highly drawn-out, depending as it is on time-related factors such as drainage conditions, low permeability, and the thickness of adjacent clayey soil layers. In thick sedimentary clays, liquefaction and/or loss of shear strength of this kind can be a very long-lasting phenomenon.

The potential for liquefaction of this kind can make the results of soil investigations of the standard type extremely unreliable, and hence the results of slope stability analyses are uncertain.

Soil investigations in lean clay soil material require undrained direct shear laboratory testing to determine the minimum residual resistance for the relevant rates of load application.

The large strain residual shear resistance (S_R) is thus a crucial parameter for predicting both the additional disturbance load to trigger a slide as well as the potential extent (i.e. the degree of disaster) of progressive landslides in long natural slopes.

A mixed largely granular soil with small clay content is featured by having:

a) A stiff but relatively porous symmetrically loaded granular soil structure carrying a major part of the current (or previously existing) vertical load;
b) Voids filled with soft clay material, the shear strength of which has little relevance to the effective vertical pre-consolidation pressure.

The brittleness of the lean clay soil may also, at least partially, depend on the proportions of illite and kaolinite present in the clayey substance.

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3. Relevance of the phenomena described in Section 2 for clays in the Churchill River Valley

3.1 General

The stability conditions in natural slopes are closely related to their geological and hydrological history.

The loose soil formations, in which the Churchill River has cut its course, consist of sediments deposited in sea and fjords during the Great Ice Age. At this time, parts of the present landmass were still deep below the present sea level due to the settlement of the earth crust because of the enormous weight of glacial ice sheets measuring kilometres in thickness.

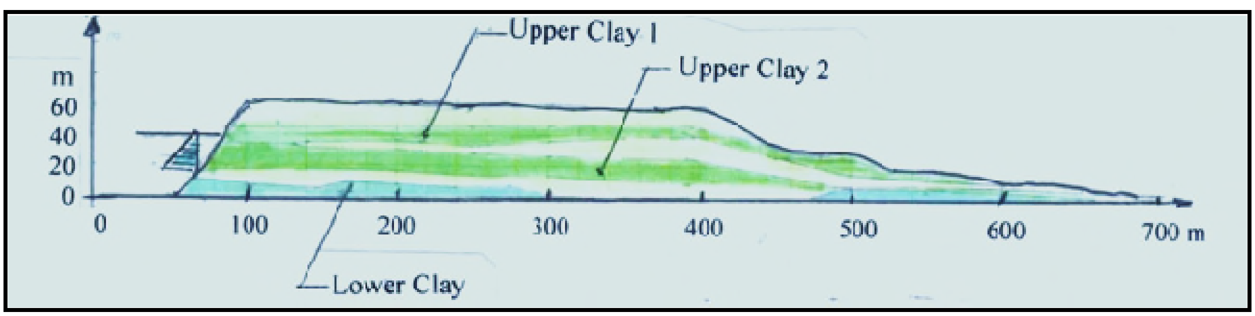
These maritime deposits — emerging from the regressing sea — were later to become part of southeast Labrador as in, for instance, the Churchill River Valley.

3.2 Structure of sedimentary deposits in the Churchill River Valley

The following description of the sedimentary structure that later shaped the valley of the Churchill River is based on information on posters at the IWLSC - Conference in Québec City 2013 [2] and sparse geotechnical data presented in the *NALCOR Report to the Independent Engineer* (2014-07-21). [3]

The Grand Riverkeeper Labrador Inc. also made it possible for the author of this article to perform local observations of soil exposures on land, from boat on the river and from air by helicopter in October 2014. The soil profiles exhibit massive layers of sands, silty sands, silty clays and clays.

Figure 3.1. Section B-B through the North Spur showing main sedimentary features according to the *NALCOR Report* [3]



As indicated in Table 1, the layers of Upper Clay 1 and 2 consist of lean silty clays with a permeability k ≈ 1 x 10^{-7} m/sec.

The different soil layers, being marine deposits, are likely to be similar over wide areas. For instance, the Upper Clay layer 1, near the present water level downstream of Muskrat Falls, can be widely observed.

However, streaming water, varying wave conditions, and topography have brought about differences in granular content and the thickness of contemporary deposits, especially in the upper part of the soil profile.

3.3 Classification of the upper clay layers by permeability

The *NALCOR Report to the Independent Engineer* contains little detailed geotechnical information about the different soil layers in the North Spur or in layers beyond the riverside escarpments.

However, the water permeability of the soil layers in Section B-B of the North Spur is listed in Table 1. The values are in accordance with the *NALCOR Report to the Independent Engineer* (2014 07 27). [3].

Figure 3:1 shows Section B-B through the North Spur in the cited *NALCOR Report*, in which the values of water permeability of six soil layers are defined below:

Table 1. Permeability of soils in the North Spur

Layer 1	**Sand** – permeability **k ≈ 1 x 10^{-4}** m/sec, Upper sand
Layer 2	**Silty clay-1** – permeability **k ≈ 1 x 10^{-7}** m/sec, Upper clay 1
Layer 3	**Silty sand** – permeability **k ≈ 0.8 x 10^{-5}** m/sec, Upper intermediate Silty Sand Drift
Layer 4	**Silty clay-2** – permeability **k ≈ 1 x 10^{-7}** m/sec, Upper clay 2
Layer 5	**Silty sand** – permeability **k ≈ 0.8 x 10^{-5}** m/sec, Lower intermediate Silty Sand Drift
Layer 6	**Clay-2** – permeability **k ≈ 1 x 10^{-8}** m/sec, Lower clay to great depth.

The values of soil permeability are of crucial interest in the current context, as they enable defining the character of the soils in a general way. Applying the well-known Hazen formula, the likely relations between clay, silt, and sand content can be appraised.

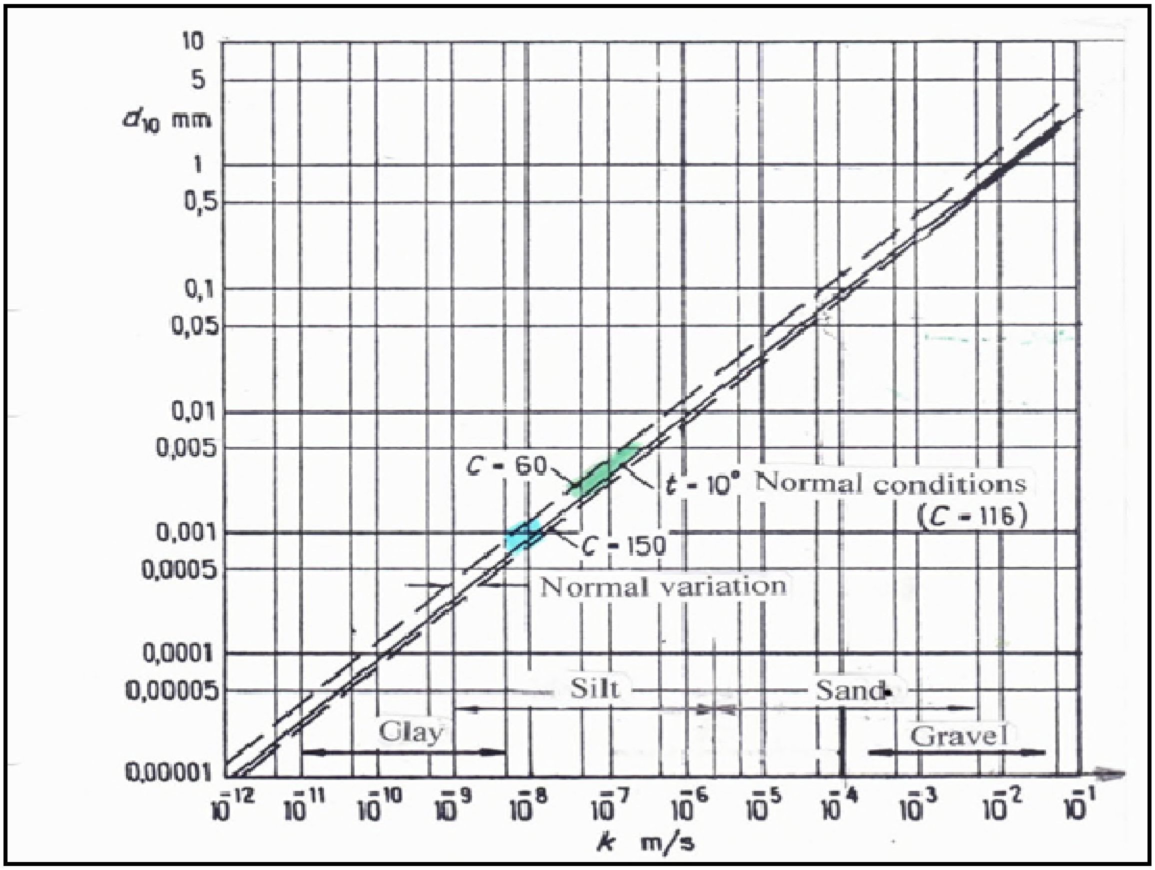
A. Hazen was a scientist who made early, thorough studies of water filtration in soils. His work was largely focussed on the relationships between the water permeability **k** (m/s) of soil filters and the mean particle grain-size in these filters. References to the Hazen relationships are repeatedly made in the well-known basic geotechnical textbook by Karl Terzaghi and Ralf B. Peck, *Soil Mechanics in Engineering Practice*. [1]

Figure 3.2 below shows the results of analyses in accordance with the Hazen's formula published in New York in 1925 as *The Filtration of Public Water Supplies*. [5]

The permeability values of the soil layers in Section B-B of the North Spur listed in Table 1 are as mentioned in accordance with the *NALCOR Report*.

For the Upper Clays Nos. 1 and 2, the permeability (**k**) is stated to be 1 x 10^{-7} m/sec. Hence, by the Hazen relationship displayed in Figure 3.2, this value of **k** would correspond to the silty material represented by the green marking added by the present author.

Figure 3.2. Using Hazen's formula, an analysis of the relationship between soil permeability k (m/s) and mean particle size



Hazen A. (1892), "Physical properties of sands and gravels with reference to their use in filtration" [4] and Hazen A. (1925), *The Filtration of Public Water Supplies*, New York. [5]

Furthermore, the figure shows that the permeability of pure clays ranges between k ≈ 10^{-11} and $10^{-8.5}$ m/sec — the typical permeability being about one hundredth to one thousandth times less than the permeability given by NALCOR for Upper Clays Nos. 1 and 2. [3].

Hence, Figure 3.2 clearly indicates that the Upper Clay Layers 1 and 2 do not contain a sufficient volume of clay to actually fill all voids in the mixed sandy, silty, clayey soil, such that the mean permeability (10^{-7}m/sec) is far greater than that of a pure clay material at the same consolidation pressure.

(The blue marking in Figure 3.2 refers to the Lower Clay layer No. 2, which is much richer in clay content).

It is therefore evident that the void system in the Upper Clay layers cannot be completely and fully filled with normally consolidated clay material. In other words, parts of the void system must still be open to filtering water. This explains the relatively high permeability (10^{-7}m/sec) of the Upper Clays 1 and 2 of the North Spur.

Conclusion: The values of permeability given in Table 1 by NALCOR [3] clearly indicate that the Upper Clay layers in Section B-B of the North Spur belong to the very lean and sensitive types of clay discussed and defined in Section 2 above.

3.4 Classification of the upper clay layers based on their liquidity and plasticity limits

The *NALCOR Report to the Independent Engineer* contains little information regarding soil properties for specific identifiable soil layers that would make it possible to perform valid studies of stability related to the North Spur. However, sparse overall geotechnical data and information were given on posters at the IWLSC Conference in Québec City (2013). Note that this information is also defined in very general terms, as seen in the table below:

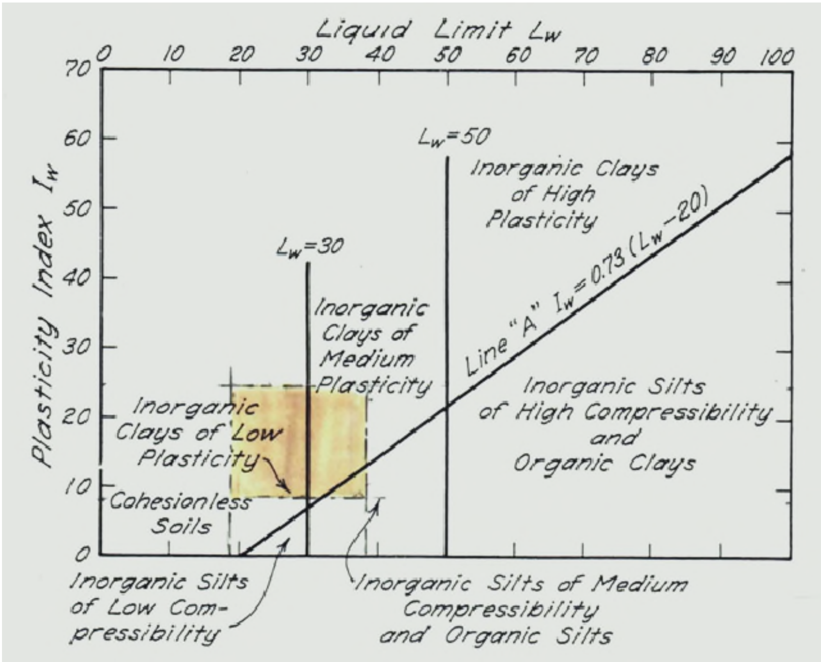
Table 2. ***Soil data according to posters at the IWLSC Conference in Québec (2013)***

Water content w %	21% – 41%	Index of Liquidity (I_L)	0.7 – 3.0
Liquid limit w_L %	19% – 39%	Index of Plasticity (I_P) %	8% – 25%
Plastic limit w_P %	13% –23%	Sensitivity in-situ	2 – 28
Undrained shear strength	40 – 120 kPa	Effective cohesion c’	0 – 10 kPa
Unit weight at natural water content, γ_m	18.4 – 19.7 kN/m^3	Initial void ratio e_o	0.93 – 1.06

As the values in Table 2 exhibit wide ranges of soil sample properties, the precise and coherent values applicable to specific soil layers of interest are not presented. The values given in Table 2 are therefore of little value for geotechnical analysis, e.g. for the assessment of hazards related to slope stability.

If the properties of the mixed clayey soils given by NALCOR are diagrammed by means of a Casagrande Plasticity Chart [6], they will all fit within the yellowish square shown in Figure 3.3. This area corresponds to a wide spread of different soils with properties ranging from stable inorganic clays of medium plasticity to unstable mixed inorganic clays of very low plasticity, bordering to extremely lean mixed clayey soils. For instance, the shear resistance and the sensitivity of the soils represented by the yellow rectangle may be radically different. (References 1 and 8 also define and describe so-called border-line materials).

Figure 3.3. Relation between Liquid Limit (w_L) and Plasticity Index (I_P) for soils



Note: The yellow rectangular area in the Figure has been added to the diagram by the author of this report. (Plasticity chart in accordance with A. Casagrande 1932, *"Research on the Atterberg limits of soils"*, Public Roads 13, pp 121-136). [6]

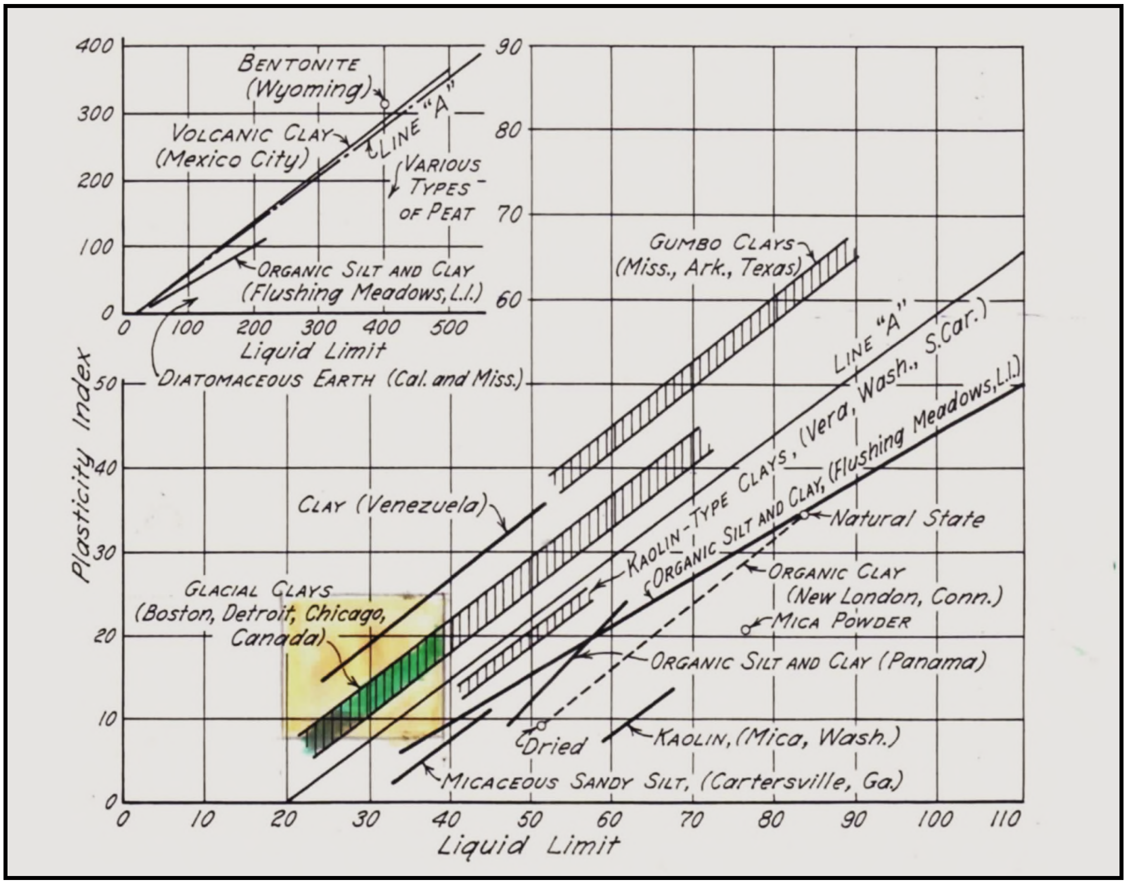
Yet, although the NALCOR data cannot be used for the detailed assessment of slope stability, they still confirm the conclusions previously made in Section 3.3 regarding the lean clay content in the Upper Clay layers 1 and 2 — and especially if the accumulated knowledge contained in the Casagrande Plasticity Chart in Figure 3.4 is considered.

The plasticity chart, in Figure 3.4 below, demonstrates how the properties of clays may largely be related to their geographic location — primarily because of differing contents of the main types of clay substance, such as montmorillonite, kaolinite and illite. (Section 2.)

As already mentioned, the yellow area represents the ranges of soil data given by NALCOR (IWLSC, Québec, 2013) [2]. However, the coloured and striated areas within the yellow rectangle apply, according to the Casagrande Chart, to soils in Canada and Northern USA [6]. This means that the NALCOR data for mixed soils given in Table 2 are very likely represented by the narrow green area within the yellow rectangle in Figure 3.4 below.

Furthermore, the greyish area included within the NALCOR range of soil properties applies to extremely lean sandy (silty) clays in these geographical regions. They feature very low Liquid Limits (**w_L**) and high Limits of Plasticity (**w_P**) — thus having Plasticity Index values as low as **I_P** ≈ **8**%. (**$I_P = w_L - w_P$**).

Figure 3.4. Relation between Liquid Limit (w_L) and Plasticity Index (I_P) for a wide range of mixed soils



The yellow area corresponds to the data given by NALCOR [2] in respect of Liquid Limits and the Plasticity Indices applying to the Upper Clays Nos. 1 and 2 shown in Figure 3.1.

Note: The colouring of specific areas in this figure was done by the present author. (Plasticity chart according to A. Casagrande 1932, *"Research on the Atterberg limits of soils"*, Public Roads 13, pp 121-136. [6]

Conclusion: The low values of I_P indicate the presence of extremely lean clays mixed with sand and silt — i.e. precisely the types of lean sensitive clay discussed under Item 2.32 above.

3.5 Classification of clay layers based on site observations

As mentioned under item 3.1, the Grand Riverkeeper Labrador Inc. made it possible for the author to perform local observations of soil exposures and slides on land, from boat on the Churchill River and from air by helicopter in October 2014. The following comments are restricted to clay formations near river water level (WL) — as for instance seen in Figure 3.5 below.

Figure 3.5. Clay exposure a few hundred metres upstream of the large 2014 landslide shown in Figure 4.3 (Photo: Eldred Davis)

3.5.1 *Clay exposure just north of the 2014 slide shown on Figure 4.3*

Figure 3.5 shows a large exposure of the type of lean clay discussed in Section 2. The effect of merely repeated stamping with a foot was enough to cause visible wavelike movement of the clay surface, indicating sensitivity and high propensity to soil liquefaction.

Another important, easily identifiable property of this clay layer was its propensity to being eroded. The large erosion scar on the picture has been caused by water trickling during rain. Many other smaller recent erosion scars were to be seen — of which one is right in the centre of the riverside scarp in the picture.

This propensity to erosion indicates that the clay content in the soil is low, and that the strength of clay material in the voids of the coarser soil structure does not correspond to the vertical pressures that once existed — i.e. before the wide riverbed valley developed as a result of erosion by the river.

In other words, the lean clay in the exposure matches the description in Section 2.3, and which is briefly defined in Section 2.4. This means that the clay content of the mixed soil is highly under-consolidated, thus indicating propensity to soil liquefaction. The specific features of the Churchill River Valley with its shallow riverbed and steep unstable riversides are mainly due to this specific soil condition.

Similar observations were also made along the clay exposure at the foot of the most recent downstream slide in the North Spur.

3.5.2 *Another clay exposure, north of the 2014 slide*

Figure 3.6 below illustrates another feature of the clay exposures. Although the scarp forming the riverbank is only 1 to 1.5 metres high — the river being very shallow here — deep cracks have developed in the clayey soil, indicating impending local failure. This means that the cohesive strength of the mixed silty, sandy clay is only in the order of 4 to 6 kN/m^2, since the shear resistance to avoid failure is about $c \approx 0.20 \times \rho \times g \times H = 0.2 \times 19 \times 1.5 \approx 6$ kN/m^2.

However, considering that this soil layer, way back in the past, was subjected to effective pressures corresponding to the weight of, at least, some 30 metres of overlying marine sediments, its cohesive shear strength ought to exceed the current values by far.

For instance, a clay with a normal illitic clay content, exposed to effective pressures of that magnitude in the past, should have a shear strength in the order of 70 kN/m^2 — i.e. about ten times higher than the actual cohesive strength of the soil in the clay exposures close to the current river water level. (See the mathematical expression below.*)

This striking incongruity also points to the specific kind of lean clay, the properties of which are described in Sections 2.3 and 2.4 above.

Figure 3.6. Clay exposure close to river water level
The 2014 landslide is seen in the background. (Photo: Eldred Davis, 27 October 2014)

* $c \approx 0,45 \times w_L \times \rho \times g \times H > 0.45 \times 0.26 \times 19.7 \times 30 \approx 70$ kN/m^2, where w_L is the Liquidity Limit. Formula according to Hansbo S., 1957. [7]

* * * * *

4. Implications for the Churchill River Valley of the properties of its markedly lean clays

4.1 General

As mentioned in Section 3.1, the loose soil formations, in which the Churchill River has cut its course, consist of maritime deposits that were exposed in the final phases of the Great Ice Age.

As of today, the Churchill River still cuts its way through these soil layers, whose properties have formed the spectacular, unusual shape of this river valley (see Fig. 1.2). Two important soil properties to be considered in this context are:

a) The coarser layers of sediment (sands and silts), which are highly sensitive to erosion.

b) The Upper Clays 1 and 2 (e.g. as diagrammed in Figure 3.1 of the North Spur), whose properties are dealt with in Sections 3.1 to 3.4 above. These studies show that Upper Clay layers 1 and 2 consist of mixed sandy/silty soils with very sparse clay content.

The properties of the exposed marine clay layers downstream of Muskrat Falls, discussed in Section 3.5, are mainly based on observations and physical inspection.

Figure 4.1. One of many smaller "superficial" slides between Goose Bay and Muskrat Falls (Photo: Eldred Davis)

4.2 Erosion processes

In the current context, the clay layer spaced above and below the Churchill River water level (i.e. the WL on October 27, 2014) is of particular interest, since the depth of the wide and shallow riverbed seems generally to be largely restricted by this layer, whereas the riverbanks rise steeply up to the surface level of the original marine sediments. (Cf. Figure 4.1).

There are basically two kinds of erosion processes in the Churchill River valley.

4.2.1 *Short-term erosion*

Figure 3.5 clearly indicates the progress of erosion. At high water levels, i.e. exceeding the top of the clayey soil layer, the foot of the sandy, silty slope above the clay is eroded by the streaming water.

As the critical angle of friction is surpassed, the uniformly graded sands and/or silts in the slope slide in smaller or larger blocks into the river, soon getting washed away by the water current. Yet, as may be concluded from the Figures 3.5 and 3.6, the lean clayey formations shown are also subject to erosion, although to a much lesser extent than the overlying more uniformly graded granular soils.

However, in a somewhat extended time perspective, the clayey layer also gets worn away, thus undermining the foot of the current slope. This process also results in earth slips and slides of the type shown in Figure 4.1.

In many places, the progress of erosion is so fast (i.e. in geological terms) that vegetation does not even manage to get a foothold before the next slide event — a phenomenon evidenced by the many barren slopes and sandy exposures that can be seen in the Churchill River Valley.

4.2.2 *Longer-term widening of the Churchill Valley by massive landslides*

As evidenced by the enormous landslide at Edward Island upstream of Muskrat falls, and by the recent 2014 landslide on the north bank downstream of Muskrat Falls, the widening of the Churchill River valley also takes place in the form of gigantic landslides involving soil masses distant from the riverside slope. These steps in the valley-widening process may have been so extensive that, if the area had been more populated, they would have been labelled as major catastrophes.

In geological terms, these types of giant landslides have been going on for thousands of years, i.e. in principle ever since the marine sediments emerged from the regressing sea. Yet, the easily eroded masses of soil that have slid into the river have been washed away relatively quickly.

The massive landslides of this kind can be explained as follows:

The short-term erosion effects, described in the previous section, increase the shear stresses in the sensitive lean clay layers that can be seen in Figures 3.5 and 3.6. (This clay layer is also clearly represented in Figure 4.4).

In other words, the gradually failing lateral support at the riverside slope generates a massive build-up of shear stress — at a point further away from the riverbank — in the clay layer carrying the overlying soil masses.

Figures 4.2a and 4.2b. The Edward Island landslide (200?). These two photos have been taken from different angles and aerial positions. The enormous extent of the landslide can be seen by considering that the encircled areas represent the same locality. (Photos: Cabot Martin)

Figure 4.3. The 2014 large landslide south of Muskrat Falls: widening of the Churchill River Valley by large "quick-clay" slides. (Photo: Cabot Martin)

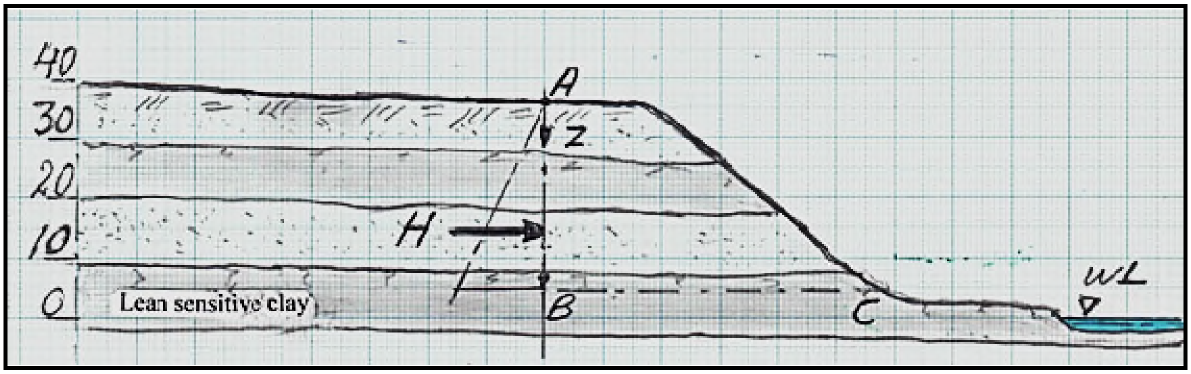
In addition, the deviatory deformation related to the increase in shear stress causes a creep movement down the slope that in turn generates vertical cracks or corresponding tensile extension and loosening of the granular soil structure. Both of these processes may give rise to faster penetration and improved access of water to the deeper soil layers, likely causing large increases in hydraulic pressure during periods of heavy rainfall or melting snow.

The total shear deformation due to all these effects may well result in *liquefaction* of the kind dealt with in Section 2. (Paragraph 2.4).

4.2.3 *Analysis exemplifying the long-term widening process in the Churchill River Valley*

Figure 4.4 shows the typical condition, with marine sediments of mostly uniformly graded sands and silts resting on a clay layer with properties similar to those in the Upper Clay layer 2 (in Figure 3.1 related to the North Spur).

Figure 4.4. Typical slope section close to the 2014 landslide between Goose Bay and Muskrat Falls. This section is assumed to have marine sediments similar to those in the North Spur.



The object of the analysis in Appendix A is to demonstrate the propensity for large landslide formation related to the current geological conditions. The soils above level B are taken to consist mainly of uniformly granular friction material (sands and silts) with a mean friction value of ϕ^{o}. Two cases are analysed, with the details given in Appendix A.

Case A Groundwater level (GWL) at ground surface.

Case B Groundwater level (GWL) at 5 m below ground surface.

4.2.4 *Conclusions from the exemplification analysis*

On the basis of mainly frictional resistance, the safety factors are estimated to be:

Case A $\mathbf{F_s}$ = 6502/5985 ≈ **1.09**

Case B $\mathbf{F_s}$ = 7006/4885 = **1.43**

The mean shear stress along failure surface B→C is then, using the plastic approach:

$\tau_{BC} \approx H_{z=30m} / L_{BC} = 5985/50 \approx \mathbf{120\ kN/m^2}$

According to the *NALCOR Report* [2], the Liquid Limits (W_L) for the mixed soils range between 20% and 39%.

However, if the clay content in the lean bottom clay layer had significantly exceeded the current void ratio, having for instance a Liquid Limit W_L = 40%, then the mean shear strength along failure surface B→C — estimated with Hansbo's formula applying to illitic clays — would be less than the calculated shear stress = 120 kPa.

c ≈ $0.45 \cdot \sigma' \cdot W_L$, where σ' denotes the pre-consolidation pressure [7], then when

W_L = 40%, **c** = 0.45 x 9.9 x 30 x (40/100) ≈ **53.5** kN/m^2; and for

W_L = 19%, **c** = 0.45 x 9.9 x 30 x (19/100) ≈ **25.4** kN/m^2

Point A — Considering that, in this condition, the required shear resistance to avoid failure is at least τ_{BC} = 120 kN/m^2 — i.e. more than 2.2 times the available strength of a normally consolidated illitic clay with c = 53.5 kN/m^2 — it is thus apparent that the stability of the steep riverside slope, and the soil structure behind it, almost totally depend on the frictional capacity of the lower clay layer.

Therefore the effect of even a minor propensity to liquefaction is an inherent landslide hazard.

Point B — Another vital condition in this context is the acute impact on the stability of the riverside soil masses of changes related to the groundwater conditions. For instance, a five-metre change in the groundwater level (GWL) reduces the safety factor $\mathbf{F_s}$ from **1.43** in Case B to **1.09** in Case A.

4.2.5 *The 1967 Rollsbo landslide, 20 km north of Gothenburg, Sweden*

The author of this article had the opportunity to study an investigation of the Rollsbo landslide (a large landslide having an area of about 20,000 m^2), carried out by the Swedish National Road Administration (SNRA). The slide event took place when steel pipes for the installation of vertical drainage were being driven with a pile-rammer machine.

When reviewing the soil conditions, the author detected that the failure surface, located by the SNRA, was mainly confined to a narrow sandy clay layer with unusually low clay content. The failure surface was surprisingly *not* located in the sensitive, normally consolidated soft clay that dominated the soil profile.

It was thus evident that the lean sandy clay was weaker and more sensitive to disturbance, with a greater inherent propensity for landslide development, than the surrounding fatter, sensitive clay layers of the normal kind in the area.

4.3 Conclusions from the analysis in Appendix A

The example in the previous section demonstrates that the steep riverside slopes may be stable, at least transiently, due to the fact that the clay content in the mixed riverbed layer — dealt with in Sections 3.51 and 3.52 and exhibited in Figures 3.5 and 3.6 — is so low that the shear resistance in potential failure surfaces is essentially related to friction in the coarse, sandy/silty material.

Thus under most circumstances the total shear resistance is only to a minor extent related to the cohesive strength of the under-consolidated clay in the voids of the granular soil structure.

Simple friction is normally a dependable factor promoting stability in soils, but in the case of the Churchill River sediments the voids of the loose mixed soil are filled with soft and sensitive clay material. The strength of this material is not compatible with — i.e. does not correspond to — current or past active vertical effective pressures. However, the void clay itself may have contributed in some measure to the looseness of the granular structure of the mixed soil.

As has been described earlier, when loose soils of this kind are subjected to shear deformation, the void volume of the dominantly coarse-grained and loosely layered soil tends to decrease. This leads in turn to excess pressure build-up in the void clay (or in void water), possibly resulting in liquefaction, or at least in a drastic loss of friction and shear strength, as explained in Section 2 above. (See the Conclusion in Section 2.4).

The crucial issue in this context is the fact that liquefaction is related to the properties of the under-consolidated pore clay itself, i.e. not only to water saturation.

Extreme excess pore water pressures in "quicksand" normally tend to abate quickly due to permeability, but in clayey soils — depending on layer thickness and how low its permeability — the dissipation of pore pressure may be a long process, possibly extending over years or even decades.

The extraordinary evolution of the Churchill River Valley, described in Section 1, is due to the lean character of, in particular, the riverbank clay layer, dealt with in Section 3. This clay is highly sensitive and prone to liquefy on being subjected to additional shear deformation that in turn may progressively result in great loss of shear resistance over large distances. According to numerous soil studies, the properties of this clay layer, found along long stretches of the river, are also those of the Upper Clay layer 2 in the North Spur.

Importantly, the sensitivity of the clays in the Churchill River Valley is of a specific, distinct nature that should *not* be confused with sensitivity of highly over-consolidated clays, such as those of the over-consolidated clays common in the Québec area. (The sensitivity of normally consolidated and slightly over-consolidated Scandinavian clays is also different from that of the lean clays in the Churchill River Valley).

For Churchill River Valley clays, the correct geotechnical approaches to (a) soil investigation, (b) choosing an appropriate type of stability analysis, and (c) choosing appropriate stability criteria, *are not the same* as for other clays.

Similarly, the forces that trigger landslides, the progressions of the slides, and the configuration of finished slides in the Churchill River Valley are not likely to be congruent with those of land-slides elsewhere.

Landslides in the Churchill River Valley are mainly of two kinds:

1) Small, essentially superficial, slides along the riverside due to ongoing undercutting of the riverside slopes by streaming water. At some sites these slides recur so frequently that vegetation does not manage to get a foothold before the next slip event takes place, as evidenced by the many barren sandy slope exposures along the river.

2) Large landslides of a progressive or retrogressive nature involving elevated ground further away from the riverbank. Such slides may be triggered by various agents, including human activity. However, in places little affected by human activity, as in the Churchill Valley, the most likely causes of large landslides include seismic activity, heavy and prolonged precipitation, and pressure changes in groundwater aquifers.

The landslides depicted in Figures 4.2 and 4.3 are good examples of the second kind of river valley formation, which is further diagrammed in Figure 1.2.

How do these large landslides begin? As shown above, the mixed lean clayey soils possess an inherent propensity to liquefaction when subjected to shear deformations, as detailed in Sections 2.4 and 4.2.2 above.

The major component of instability in the lean-clay condition is its impact on sensitivity. If the pore volume of the coarse material is above the critical void ratio (n_{crit}) — i.e. in the state when void volume decreases under shear deformation — then significant increases in shear stress (as well as that of vibratory impact loading) are likely to produce results similar to the hydraulic liquefaction seen in sands. The soil's residual shear resistance may drop to just a small fraction of its initial shear resistance.

It is vital to observe that these landslides *cannot* be predicted by means of the conventional Plastic Equilibrium Mode analysis that was the dominant approach to slope stability analysis during much of the 20th century. Modern research has shown that this analytical model *does not apply* to long slopes in sensitive clays. In this case the Plastic Equilibrium approach is neither physically nor mathematically valid. (Cf References in Appendices B and C).

* * * * *

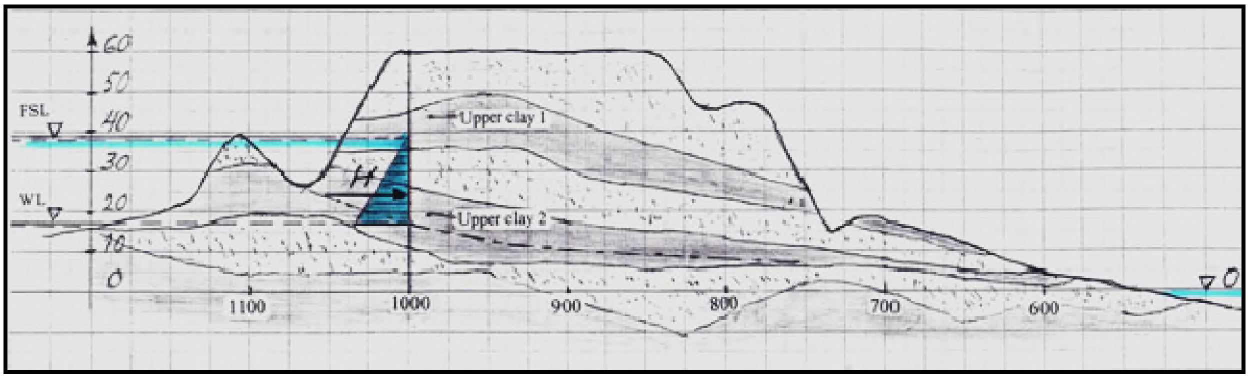
5. Implications of the properties of markedly lean clays for North Spur stability following impoundment

The soil properties related to lean clay formations in the Churchill River Valley have a significant impact on the assessment of slope stability and the factors of safety related to the same.

The North Spur in its present state has numerous large landslide scars, of which some are due to recent landslide events, indicating that erosion and land-sliding — as in the rest of the valley — are ongoing processes. In this context the problems of stability in the North Spur are primarily connected with Upper Clay layers 1 and 2. (See Table 1 in Paragraph 3.3 and Reference 3). Could the enormous lateral force of impounded water provoke a major landslide?

These issues are, of course, well-known problems with the potential to preclude construction of the Muskrat Falls dam. They have been carefully examined by NALCOR and SNC·Lavalin:

Figure 5.1. Section K–K, NALCOR [3]



In this diagram the force H represents an additional load due to impoundment from level +17 m to +39 m. At level +17, the value of H, acting on the soil mass above the potential slip surface shown above, amounts to 2.420 kN/m or 24,200 metric tons over a width of 100 m. (Vertical scale / horizontal scale = 2.5/1.0).

This potential landslide threat must be met with carefully designed engineering solutions. The risk-reduction methods incorporated into NALCOR's plan for the North Spur section of the dam containment include:

1) Pre-consolidation of critical clay layers by lowering the groundwater pressures in the relevant aquifers.
2) Installing a watertight membrane (cut-off wall or COW) against the upstream face of the North Spur ridge.
3) Establishing erosion protection banks in various places.

5.1 Pre-consolidation of clay layers

As indicated by the discussion in Sections 2, 3 and 4, the lean clay layers in the Churchill River Valley are of an unusual — and from a geotechnical point view very problematic — nature.

The problem with markedly lean clays that have a loose, coarse-grained structure, is that although frictional resistance to symmetrical top-down pressures can be maintained over long time periods (centuries, millennia), the result of a momentary increase of shear stress and the resulting lateral shear deformation may produce liquefaction in the mixed soil, whereby all, or most, of the resistance to a sideways slide may be lost. This is a condition with an inherent propensity to progressive failure development, possibly resulting in a massive landslide.

The potential for this kind of liquefaction makes the results of standard-type soil investigations and laboratory testing unreliable. Slope stability analysis and related safety factors that are based on such "standard" results may therefore be totally inaccurate.

The fact that the Churchill River Valley has developed in the way it actually has validates these geotechnical observations. The soil masses behind the riverside slope have exerted their vertical pressures for several millennia, and yet even moderate changes in lateral loading conditions — such as water pressure change, seismic activity, gradually failing lateral support, creep deformations, and the subsequent loss of shear resistance because of the propensity to liquefaction — can release enormous landslides of the kind seen at Edward Island.

The impoundment of water up to Level +39 means subjecting the clay layers at Level +10 to an additional load of 3960 kN/m, representing an active additional external load of 39,600 metric tons over a width of 100 m.

The corresponding force on the soil mass above level +17, acting on the potential failure surface indicated at Section 1000 in Figure 5.1, is 2420 kN/m, i.e. an additional horizontal force capable of generating significant lateral shear deformation and loss of shear resistance.

5.2 Establishment of a watertight membrane (cut-off wall)

NALCOR's current design for the dam calls for a cut-off wall (COW) to reduce pore water pressures and water percolation in the soils of the North Spur. The installation of a watertight membrane — done by injecting bentonite — would of course be advantageous for safely increasing the pressure on soil layers that truly abide by the normal laws of frictional resistance in granular soils.

However, the behaviour of a mixed soil with lean clay content may, as has been demonstrated in previous chapters, be totally different. The reduced porosity caused by shear deformation may simply result in liquefaction, whereby the loss of shear resistance may in turn generate additional liquefaction further along the potential failure surface, thus resulting in a condition of global progressive failure.

In fact, considering the type of sensitive behaviour of the lean clay in Upper Clay layer 2, the local concentration of additional hydraulic pressure at the COW is even likely to create a highly disadvantageous condition, because local (concentrated) loading is the most common and *most effective triggering agent* in the development of extensive progressive landslides — i.e. slides potentially longer than 70 to 100 metres.

The many documented landslides in the Churchill River Valley are in fact precisely due to the presence of this specific type of lean clay, formed under marine sedimentary conditions during the most recent Ice Age.

Conclusion: The potential effect of applying high local stress along the watertight membrane (COW) should be thoroughly investigated for the possibility of progressive failure. In particular, potential failure surfaces like the one indicated in Figure 5.1 should be intensively studied.

Also, it must be recognized that the Plastic Equilibrium Mode failure model has no relevance in this context.

5.3 Erosion protection banks

Generally speaking, erosion protection is a useful measure for slope stabilisation. Yet, in respect of the risks related to progressive landslide development, stabilisation of the toes of the slopes has relatively little effect.

* * * * *

6. Concluding remarks

The contention of this document is *not* that the North Spur dam containment is bound to fail. Yet, considering the enormous threat to populated areas that would result from a breakage in the North Spur ridge, the possibility of such an event must be eliminated.

Modern research requires that the stability analysis of long slopes with sensitive clay must carefully consider the risk of “brittle slope failure”. As the impoundment represents a gigantic external force to be exerted locally on the COW, a meticulous study of potential progressive failure is an unavoidable and crucial measure to ensure the safety of the dam.

Friction is normally a dependable factor for stability. However, in the current case the voids in the loose mixed soil are filled with soft and sensitive clay material, the strength of which does not correlate well with current or past active vertical effective pressures.

The properties of the very lean Upper Clay layers in the North Spur differ from those of normal clays, in which the clay content is usually considerably in excess of the void volume of more coarse-grained material.

In very lean clays, the loose granular structure of the coarse-grained portion of the soil undergoes a decrease in the pore volume when subjected to shear deformation. This is the reason for the inherent propensity to soil liquefaction of such clays.

The likelihood of liquefaction of this kind makes the results of standard-type soil investigations, and the associated determination of safety factors in respect of slope stability, very unreliable. This applies in particular if calculations are based on the Plastic Equilibrium Mode of analysis.

Dependable stability analyses must therefore consider the potential risk of progressive failure formation due to the intrinsic tendency to liquefaction, particularly regarding Upper Clay layer No. 2 of the North Spur. Such analysis must be based on undrained direct shear tests on virtually undisturbed clay samples, the diameter of which should not be less than 100 mm.

These direct shear tests should not be deformation-controlled in such a way that the development of failure surfaces is in any way constrained, thus establishing the crucially important value of the residual shear resistance of the lean clayey soil. This can be achieved with the test specimen being confined by rings, as demonstrated for instance in Reference 9 (Proceedings of the 11th ICSMFE, San Francisco, 1985).

One way of testing for residual shear resistance is to retrieve large, virtually undisturbed samples from relevant soil exposures and then apply the original vertical effective pressure on the specimen before shearing the same at an appropriately high rate of shear deformation. Shear deformation in an ongoing progressive landslide tends to be rapid.

Vane shear tests such as those performed by Aas, G. (1966) may also be instructive in the present context. [10]

Furthermore, the analyses of potentially extensive slides must not be based on the concept of a Plastic Equilibrium Mode, as this failure mode is not valid under current conditions.

Studies of progressive failures in highly sensitive Scandinavian clays indicate that the Plastic Equilibrium Mode of analysis is not applicable when potential landslides extend more than 70 to 100 metres — the distance largely depending on the depth of the failure surface below ground level.

Furthermore, the possibility of progressive failure developing in soil layer 6 — the Lower Clay extending to a great depth according to Reference 2 — should also be investigated.

* * * * *

APPENDIX A

Analysis of the section in Figure 4.4

The objective of the following analysis, based on the section shown in Figure 4.4, is to demonstrate the propensity for large landslide formation related to the current geological conditions. The soils above Level B are taken to consist mainly of uniformly granular friction material (sands and silts) with a mean friction value of ϕ^{o}.

The void ratio for loose mixed-grained sands of current type is assumed to be (n) = 40%. This is in accordance with Terzaghi & Peck (Reference 1 , Article 6, Table 6.3).

Hence, the porosity e = n/(n–1) = 0.40/0.60 = 0.6667 = 66.67%

Rock density γ_R = 26.5 kN/m^3, Water density γ_{H20} = 10 kN/m^3

The water content **w** = $W_{water/}$**/**W_{rock} = $e\cdot\gamma_{H2O}$**/**γ_R = 66.67 x 10/26.5 = 25.16%

Density (water saturated) $\boldsymbol{\gamma_w}$ = (w+1)**/**(w+γ_{H20}/γ_R) = (0.2516 + 1)/(0.2516 + 10/26.5)
= 1.2516**/**(0.2516 + 0.3774) = **19.90** kN/m^3

Density (under water $\boldsymbol{\gamma_w'}$ = γ_w– 10 = 19.9 – 10 = **9.90** kN/m^3

Dry density $\boldsymbol{\gamma_d}$ = γ_R**/**(1+e) = 26.5/1.667 = **15.90** kN/m^3

or, $\gamma_d = (1\text{-}n)\cdot\gamma_R$ **/**1 = (1– 0.40) x 26.5 = 15.90 kN/m^3

Internal friction value ϕ = 30^o, Length B→C = 50 m

Effective cohesion c' in the clay layer = 6 kN/m^2. (Cf Section 3.52)
(According to Ref. 2, c' is in the range of 0 to 10 kN/m^3).

As the clay content in the clay layer is extremely low — the soil mainly consisting of sand and silt — the stability investigation is tentative, mainly based on frictional resistance in the lean sandy clay. The currently effective cohesion is taken to be only c' = 10 kN/m^2. [2]

Case A

Groundwater level (GWL) at ground surface, i.e. z = 0, z_B = 30 m,

Horizontal earth pressure (p kN/m^2) at a depth of z = 30 m

$p_Z = \boldsymbol{\gamma_w'}\cdot z\cdot \tan^2(45 - \phi/2) + \gamma_{H20}\cdot z\cdot$ = 9.9 x z x 0.33333 +10 z
$H_{z=30\,m} \approx 0.33333\cdot\gamma'\cdot z^2/2 + \gamma_{H20}\cdot z^2/2$ = 0.33333 x 9.9 x 30^2/2 + 1 x 10 x 30^2/2
= 1485 + 4500 = **5985 kN/m**

Mean shear stress along surface B→C (plastic approach):
$\boldsymbol{\tau_{BC}} \approx H_{z=30\,m}$**/**$L_{BC}$ = 5985/50 ≈ 120 kN/m^2

Shear resistance along surface B→C

$R_{z=30\,m} = {}_{o}\int^{50} c'\cdot dx + \gamma_w'\cdot[z_{30}\cdot{}_{o}\int^{20} dx + {}_{20}\int^{50} z_{30}/2\cdot dx)]\cdot \tan\phi$
$=10\cdot 50+(19.9–10)\cdot[(30\cdot 20 +30\cdot 30/2)]\cdot \tan 30^o = 500 + 9.9 \times [600 + 450]$ **x** $\tan 30$
$= 500 + 10395 \times 0.5774 =$ **6502 kN/m**,

roughly rendering a safety factor of only

$F_{s\,(a)} = 6502/5985 \approx 1.09$

Case B

Groundwater level (GWL) at 5 m below ground surface, i.e. $z_w= 5$, $z_B = 30$ m,

Horizontal earth pressure (p kN/m²)

$p_Z = \gamma_d\cdot z_w\cdot\tan^2(45 – \phi/2)+ \boldsymbol{\gamma_w}'\cdot (z– z_w)\cdot\tan^2(45 – \phi/2) +\gamma_{H20}\cdot(z– z_w)$
$(\tan^2(45 – \phi/2) = 0.33333)$
$H_{z=30\,m} \approx 0.33333\,\gamma_d\cdot[(z_w^2/2+ z_w\cdot(z_{30} –z_w)] + 0.33333\,\gamma'\cdot(z– z_w)^2/2 + \gamma_{H20}\cdot(z– z_w)^2/2$
$= 0.33333 \times 15.9 \times [5 \times 5/2 + 5 \times 25)] + 0.33333 \times 9.9 \times (30–5)^2/2 + 1 \times 10 \times (30–5)^2/2$
$= 66.3 + 662.4 + 1031.3 + 3125.0 =$ **4885 kN/m**

The total *shear resistance* based on friction and effective cohesion ($c' = 10$ kN/m²) with GWL at z = 5 m is:

$R_{z=30\,m} = {}_{o}\int^{50} c'\cdot dx + [{}_{o}\int^{25} \boldsymbol{\gamma_d}\cdot z_w \mathbf{+} {}_{25}\int^{50} \boldsymbol{\gamma_w}'\cdot(z_{30}– z_w)]\cdot\tan\phi\cdot dx =$
$= 500 +[(15.9\cdot 5+9.90\cdot 25)\cdot 25+9.90\cdot 25/2\cdot 25)]\cdot\tan 30 =$
$= 500 + 25\cdot[79.5+247.5+123.75]\cdot\tan 30 =$
$= 500 + (79.5+247.5)\cdot 25\cdot\tan 30\cdot+123.75\cdot\tan 30\cdot 25 =$
$= 500 + (327 + 123.75) \times 0.5774 \times 25 = 500 + 11269 \times 0.5774 = 500 + 6506 =$ **7006 kN/m**

This renders a safety factor of

$F_{s\,(b)} = 7006/4885 = 1.43$

APPENDIX B

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Research institutes in which Progressive Failure analysis is recognized as a professional method and which have a documented capacity for performing progressive failure analyses include the following:

Luleå Technical University, Luleå, Sweden

Norwegian University for Technical & Natural Sciences (NTNU), Trondheim, Norway

Norwegian Geotechnical Institute (NGI), Oslo, Norway

Laval University, Québec City, Québec, Canada

Queen's University, Kingston, Ontario, Canada

APPENDIX C

Comprehensive list of references

List of recent publications on the subject of brittle slope failure, presented at world conferences and symposia

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