Safety and Reliability of the Muskrat Falls Dam, in Light of the *Engineering Report* of 21 December 2015 by Nalcor/SNC-Lavalin

Submitted to the
NL Public Utilities Board's

Investigation and Hearing into Supply Issues and Power Outages on the Island Interconnected System

on behalf of

Grand Riverkeeper Labrador, Inc.

by

Dr. S. Bernander

13 October 2016

Dr. S. Bernander
Adjunct Professor (Retired)
Luleå Technical University, Sweden

Email <stig.bernander@telia.com>
Tegelformsgatan 10, SE 431 36, 36 MÖLNDAL, Sweden
Tel +46-31-871104 Cell +46-72-3954646

FOREWORD

The following comments should be regarded as complementing the previous report by the undersigned, *Lower Churchill River Riverbank Stability Report,* dated 26 November 2015 [1], concerning the *Nalcor Report to the Independent Engineer on the Lower Churchill Project, North Spur Updated, 21-JUL-2014* [2]. This *Riverbank Stability Report* was prepared as expert testimony on behalf of the Grand Riverkeeper (Labrador) Inc., as are the current *Comments*.

When the report of 26 November 2015 was written, the undersigned had not yet had the possibility to review either the Nalcor/SNC- Lavalin report of 21 December 2015 titled *Engineering Report*, *North Spur Stabilization Works, Progressive Failure Study* [3] or the *Lower Churchill Project. North Spur Stabilization Works – Design Report* [4] of 30 January 2016 by the same authors.

Having now reviewed the 21 December 2015 ***ENGINEERING REPORT*** in detail, the undersigned author finds that his previous comments on the stability of the Muskrat Falls dam containment, especially its North Spur, remain relevant.

The current review is largely focused on specific issues that have been presented more fully in this Nalcor/SNC-Lavalin ***ENGINEERING REPORT*** [3]. References will also be made to this Reviewer's earlier *Riverbank Stability Report* [1].

EXECUTIVE SUMMARY

The stability of the North Spur as a dam containment structure is a complicated issue. Imminent impoundment and the creation of a reservoir will bring about new conditions in the North Spur: (a) strong lateral stresses from the weight of water, stresses that will fluctuate with the water level in the dam, and (b) a rise in the water table everywhere in the reservoir area, most notably in the areas adjacent to the dam.

It must be asked, Have Nalcor's engineers adequately accounted for these new conditions in preparing their stability analysis, especially with regard to the possibility of a progressive (downhill) landslide? Have they used appropriate techniques to estimate the risks?

The following criticism of the ***ENGINEERING REPORT*** of 21 December 2015, presented here by this Reviewer, does not constitute any prediction of likely failure of the North Spur caused by impoundment, other man-made stress, or seismic activity.

However, the strong criticism is made that the stability analyses in the **REPORT** fail to address the effects of important aspects of basic geotechnical design and of modern research in the field.

- The **REPORT** appears to rely exclusively on the assumption that an ideal elastic-plastic stress-strain relation is applicable to the sensitive porous soils in the North Spur.
- The geotechnical data for the North Spur presented in the **REPORT** do not suggest such an elastic-plastic physical relationship.

Thus this Reviewer finds such an assumption to be highly questionable. Further:

- The **REPORT** does not present any results from stress/strain deformation tests, or any other evidence, that might indicate that the ideal elastic-plastic relationship is likely to be valid. The Report does not, for instance, address the decisive effects on the shear resistance of a soil due to the relation between the in-situ porosity of a soil and its critical porosity.
- Considering the initial emphasis in the **REPORT** on the possibility of progressive failure, stress-deformation data are absolutely indispensable for predicting landslide hazard in long slopes with sensitive soils.
- Instead of such data, however, the **REPORT** offers the output of a computer model that extrapolates from static conditions and long-term percolation.
- Nor does the **REPORT** deal with the drastic effects on residual shear strength related to stress/strain reversals in porous silty/sandy soils (and in granular soils with very poor clay content).
- The **REPORT** gives no valid explanation for studying only horizontal failure planes in the North Spur when investigating the effects of the enormous water pressure that will be permanently imposed by the impoundment of water above the Muskrat Falls dam.

Hence it is this Reviewer's assessment that safety factors based on this stress-strain model, including those offered in the **REPORT**, are not well founded and cannot be accepted without further supporting evidence. The inevitable conclusion is that the safety and reliability of the Muskrat Falls dam have not been demonstrated.

This Reviewer strongly recommends a dynamic testing procedure for accurately assessing the porosity of potentially sensitive North Spur soils.

The most reliable way to investigate the porosity of loose soils in-situ is by subjecting them to heavy vibration and assessing the resulting changes. The Reviewer therefore recommends that investigators drive a series of piles in a concerted manner into the North Spur east of the cut-off wall and measure the resulting soil settlement.

This kind of dynamic testing makes it possible to estimate the reliability of the computer model employed in the **REPORT**. If the resulting safety factors are found to be significantly less, then further remedial actions can be planned and carried out in a timely fashion.

Additional mitigatory measures would involve the compaction of the under-consolidated silty clay soils of the North Spur to the point that they are no longer vulnerable to liquefaction under dynamic loading conditions.

In view of the catastrophe that would envelop downstream communities in the event of a breach in the North Spur, these issues deserve the most careful scrutiny and decisive action by those entrusted with leadership of the Project.

Until and unless they are satisfactorily resolved, the reliability of the Muskrat Falls generating station in meeting the electrical needs of Newfoundland cannot be presumed.

Gothenburg, 13 October 2016
Stig Bernander

1. GENERAL CONSIDERATIONS

The Nalcor/SNC-Lavalin ***ENGINEERING REPORT*** is a wide-ranging and, from many points of view, a comprehensive geotechnical study based on conventional mid-20th century modes of analysis in soil mechanics, many of which this writer has supported when used in appropriate settings.

For instance, in-situ conditions based on long-term stress change, long-lasting hydrology, or extremely slow rates of additional change of loading may normally be well analysed using the conventional procedures generally applied in the ***ENGINEERING REPORT***, which from this point and on will be referred to as the **REPORT** or Reference [3].

The author of the current comments, herein named the “Reviewer”, will focus on items and conditions that may question or undermine the reliability of studies based on conventional modes of stability analysis — such as the Limit Equilibrium Mode (denoted LEM in the **REPORT**).

According to the basic assumptions stated on pages 34 and 35 of the **REPORT**, the “elastic-plastic” stress-strain relationship is at the heart of the failure analyses that it describes.

A condition of decisive importance regarding the validity of LEM analyses is the relation between the in-situ porosity (**n**) of a soil layer and what in soil mechanics is defined as the *critical soil porosity* ($\mathbf{n_{crit}}$). This relationship, and how it applies to the types of soil in the North Spur, is treated in some detail.

If LEM analysis is found to be not appropriate, what other methodologies may be used to estimate the risk of slope failure? These Comments then turn to recent research in progressive failure in long slopes and how the risk of such failure may be assessed with several new technologies.

2. On progressive failure development

In modern research on landslide hazards, the geotechnical phenomenon denoted "progressive failure" cannot in any way be either predicted or precluded by analyses based on the Limit Equilibrium Mode (LEM). This is due to the fact that progressive failure simply cannot take place in materials with stress-strain (deformation) relationships of the kind called *elastic-plastic* in the **Report** — i.e. materials being nearly perfectly plastic under large deformations. (Refer to Sections 5 and 6 of Reference [1], where these issues are treated in more detail).

Under what conditions can progressive failure occur? Such landslides occur in soils in which powerful deformations are succeeded by a drastic reduction of shear resistance, as exemplified by curves C and D in Figure 2.1. (In contrast, elastic-plastic soils deform linearly with increasing shear stress, as in curve A). Further, as is highlighted in Sections 5.3 and 5.4, serious loss of residual shear resistance — liquefaction — may also result from deviatory deformation or from reversals of stress and deformation that are independent of current stress levels.

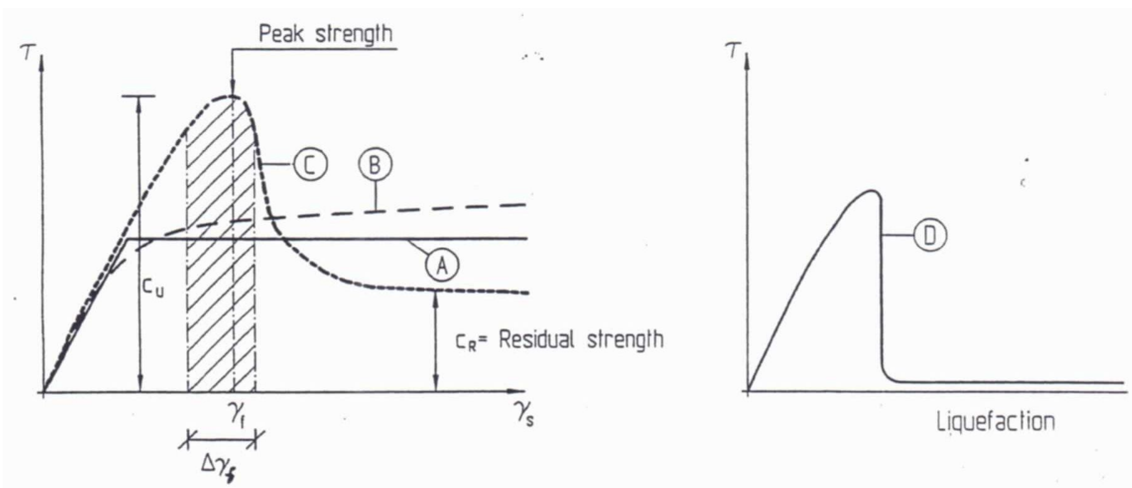


Figure 2.1. Deviatory stress-strain (deformation) relationships of different kinds

A) Elastic-plastic (LEM) relationship

B) Long-term perfectly drained (LEM) condition

C) Sensitive undrained condition

D) Liquefaction, e.g. due to deviatory deformation in loose soils and sensitive clays

Hence, in materials with properties like those in cases (A) and (B) in Figure 2.1, progressive failure is simply not possible, whereas in cases (C) and (D) progressive failure may be a likely event.

Both forward progressive (downhill) and retrogressive (uphill with lateral spread) failures can be triggered by deviatory shear deformation caused by an external load or simply by reversals of stress and strain. These additional load effects may be due to a variety of causes, including human activity, hydrological change, water-filled deep cracks (due to ongoing creep movement), erosion, vibration, or seismic action. The weight of water behind a dam wall is certainly such an effect.

A surprising feature in many extensive progressive landslides is that the slope studied may have remained stable for centuries or millennia, and yet, a seemingly insignificant local load has managed to destabilize a wide area, measuring hundreds of metres in width and length. Landslides of this kind are frequent in Canada, Scandinavia, in post-glacial regions in Europe, and in tropical areas with laterite clays.

The huge landslide at Edwards Island in 2010 — in the Churchill River Valley upstream from Muskrat Falls — is a striking example. In this case, the sensitivity-generating landslide hazard is related to the high porosity of the soil layers, which is an extreme but typical feature common for the soils in the Churchill River Valley [5].

As has been emphasized in previous reviews [1,6], the crucial issue in this context is:

> *Do the stress-strain curves of the soils in the North Spur correspond to curves A and B in Figure 2.1, or is it possible that deformations due to additional loading may result in stress-strain (deformation) relationships such as those of curves C or D?*

The formation and ongoing geological development of the Churchill River Valley render clear evidence that the properties of its marine sediments have *not* been of an elastic-plastic nature in the past — and nor will they become plastic in the future without extraordinary remedial measures.

The progressive failure issue is further dealt with in Sections 5.3, 5.4, and 6.1 below.

3. ABOUT CRITICAL VOID RATIO AND CRITICAL POROSITY IN SOIL SENSITIVITY

3.1 The Stratified Drift of the North Spur — the Upper Silty Clays

The values of Liquid Limits, Unit Weights and Void Ratios shown in **Table 1** below are valid for soils within the ranges of data as presented in **Table 2-1** on page 17 of the Nalcor/SNC-Lavalin **REPORT** [3].

Table 1. Types of soil in the Stratified Drift, the properties of which range between the values given in **Table 2-1** for the Upper Silty Clays. (**REPORT**, page 17).

Type of soil Stratified Drift	Water content **w** %	Plastic Limit **PL** %	Liquidity Index **LI**	Corresponding Liquid Limit **LL** %	Unit weight γ_w kN/m^3	Void ratio ***e***	Porosity **n**
Type **1a**	43	13	2.8	**38.8**	**17.71**	**1.14**	**0.53**
Type **1b**	43	15	2.0	**36.5**	**17.71**	**1.14**	**0.53**
Type **1c**	43	25	1.3	**40.6**	**17.71**	**1.14**	**0.53**
Type **2a**	35	13	2.8	**35.9**	**18.56**	**0.93**	**0.48**
Type **2b**	35	15	2.0	**32.5**	**18.56**	**0.93**	**0.48**
Type **2c**	35	25	1.3	**34.4**	**18.56**	**0.93**	**0.48**
Type **3a**	30	13	2.8	**34.1**	**19.19**	**0.80**	**0.44**
Type **3b**	30	17	2.0	**32.0**	**19.19**	**0.80**	**0.44**
Type **3c**	30	25	1.3	**30.6**	**19.19**	**0.80**	**0.44**
Mean values	31	19	1.3	**29.5**	**19.06**	**0.82**	**0.45**

Relationships

$w = n/[(1-n)\times\gamma_R] = e/\gamma_R$ $\quad n = e/(1+e) \quad e = n/(1-n)$ $\quad$ w = water content

Density, H_2O-saturated $\quad \boldsymbol{\gamma}_w = (w+1)/(w+1\times\boldsymbol{\gamma}_{H20}/\boldsymbol{\gamma}_R)$ $\quad \gamma_{H20}$ = Density of water = 10 kN/m^3

or: $\boldsymbol{\gamma}_w = n\cdot\boldsymbol{\gamma}_{H20} + (1-n)\times\boldsymbol{\gamma}_R$ kN/m^3

Dry density $\quad \boldsymbol{\gamma}_d = (1-n)\,\boldsymbol{\gamma}_R$ kN/m^3

Assumed density of rock material $\quad \boldsymbol{\gamma}_R = 26.5$ kN/m^3

For comparison, see Terzaghi and Peck [7], Article 6, **Table 6.3**, “Index Properties of Soils”. The values shown in **Table 1a** below are typical of sands:

Table 1a. From Terzaghi and Peck [7]

Type of soil	Porosity	Void ratio	Water content	Water-saturated unit weight
	n	***e***	**w** %	***γ*** (kN/m^3)
*Uniform sand, **loose***	**0.46**	**0.85**	**32**	**18.**9
Uniform sand, dense	0.34	0.51	19	20.9
*Mix-grained sand, **loose***	**0.40**	**0.67**	**25**	**19.9**
Mix-grained sand, dense	0.30	0.43	16	21.6

As can be readily concluded by comparison between **Table 1** and **Table 1a**, all values of initial void ratio, porosity, and water content for the Type 1 and Type 2 soils indicate a looser composition than even those attributed to loose sands by Terzaghi-Peck. The unit weights of these soils, i.e. 17.7 to 18.6 kN/m^3, are all below those of a loose uniform sand, confirming a loose composition. According to the **REPORT**, the Upper Clays belong to the Stratified Drift, which is referred to as a "heterogeneous mix of clays, silts and sands ..."

The unit weights of the Type 3 soils in **Table 1** also fall below the Terzaghi-Peck value for loose mix-grained soils, as 19.2 kN/m^3 is less than 19.9 kN/m^3. The initial void ratios ranging between 0.81 and 0.90 are all in excess of 0.67, values that apply to loose mix-grained sand.

Furthermore, the water content for all of the Type 1 and Type 2 soils, including the average value, exceeds the Liquid Limit, a condition which is indicative of high sensitivity.

***Conclusion*.** The soil properties in **Table 1** are consistent with the very specific formation of the Churchill River Valley in the past and its ongoing development. These soils tend to be loose and non-compacted, and they have been susceptible to repeated landslides over a long period of time. The North Spur itself has scars of at least nine significant slides. For the most recent large slide in the North Spur, in 1978, Nalcor's own consultants found that the silty clay layer had developed multiple failure surfaces and liquefied over a long lateral distance [4]. Note that with respect to a future progressive slide, it does not matter whether this earlier landslide had a progressive component or not. Impoundment will create major new stresses not previously felt in the North Spur. **What is important is that liquefaction has occurred and could occur again**.

3.2 The Lower Clay Layer

This section deals with a study (similar to the one in Section 3.1) regarding the soil properties of the Lower Clay layer. In **Table 2** below, the values of liquid limits, unit weights, and void ratios are all applicable to soils with Water Content, Plastic Limit, and Liquidity Index as presented in Table 2-2 of the **REPORT**.

Table 2. Types of soil in the Lower Clay formation, the properties of which range within the values of soil data in **Table 2-2** for Lower Clay. (**REPORT**, page 19).

Type of Lower Marine Clay	Water content **w %**	Plastic limit **PL %**	Liquidity index **LI**	Correspond. Liquid limit **LL %**	Unit weight kN/m^3	Void ratio ***e***	Porosity **n**
Lower cl Ia	**45**	13	2.0	**35.5**	17.53	1.19	0.54
Lower cl Ib	**45**	15	1.5	**37.5**	17.53	1.19	0.54
Lower cl Ic	45	17	1.0	45.0	17.53	1.19	0.54
Lower cl IIa	**35**	11	2.0	**28.5**	18.56	0.93	0.48
Lower cl IIb	**35**	13	1.5	**29.8**	18.56	0.93	0.48
Lower cl II c	35	15	0.9	37.4	18.56	0.93	0.48
Lower cl IIIa	**30**	10	2.0	**25.0**	19.19	0.80	0.44
Lower cl IIIb	**30**	13	1.5	**26.5**	19.19	0.80	0.44
Lower cl III c	30	16	0.9	31.7	19.19	0.80	0.44
Mean Values	29	21	0.6	39.9	19.33	0.77	0.43

Table 2 indicates that almost all values of the water content significantly exceed the corresponding values for the Liquid Limit (**LL**), indicating a high sensitivity. Yet, the mean value of the Liquidity Index is 0.6 (i.e. below 1.0). However, although this may appear to be a reassuring condition, the fact that **LL** varies widely between 0.1 and 2.0 indicates that layers with high sensitivity also occur in the Lower Clay formation — a fact that allows the possibility of developing a progressive failure.

(Note that a mean value in this context simply denotes the mean result from a number of tested soil samples. It does not necessarily represent the average resistance or the mean sensitivity of the soil mass of interest).

Finally, there is a relationship between quick clay and the desalination of marine sediments due to the percolation of fresh water. This is a well-known long-term risk factor for the development of quick clay; in the North Spur this risk is associated with the Lower Clay layer. The effects of such water seepage may have to be considered at a later date, but at the present time it is the high porosity of the soils in the Stratified Drift that presents the greatest danger.

4. SHEAR STRENGTH — DEPENDENCE ON DIVERSE EFFECTS

A basic principle of analysis in soil mechanics is that the values of peak shear strength, residual shear resistance, and stress-strain (deformation) relations are not fixed or invariable properties of the tested soils. Rather, they remain dependent on various internal and external factors that are of particular concern when the possibility of progressive failure is considered.

Several of these parameters are *rate-related*, because they are highly dependent on:

- the rates of load application and the rates of stress change during landslide development;
- the rates of dissipation of excess pore pressure, e.g. the thickness and permeability of the soil layers neighbouring the developing failure surface.

Other important factors include:

- the relationship between current porosity (**n**) and the value of critical porosity ($\mathbf{n_{crit}}$);
- the over-consolidation ratio (OCR); and
- whether or not a failure surface (or shear band) has already developed.

According to the **REPORT**, the peak shear strengths of the North Spur soils have largely been measured by vane tests. In this context, it is of interest to refer to the diagram in **Figure 4.1** published by Aas, 1966 [9]. The diagram shows how peak strength and residual shear resistance may relate to the angle of torsion and the speed at which the vane is turned.

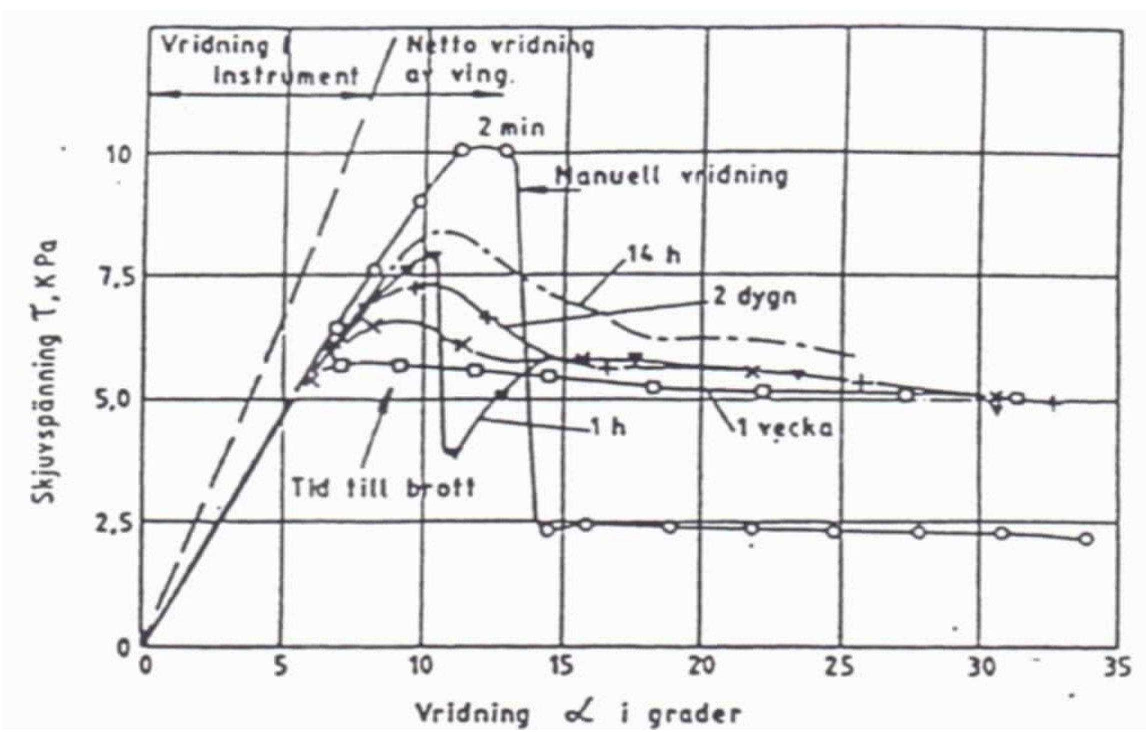


Figure 4.1. Stress-strain (deformation) curves for consolidated, undrained vane tests at different strain rates [9]. Legend: brott = failure, Vridning = torsion, dygn = day, vecka = week, grader = degrees.

Figure 4.2 illustrates a corresponding relationship found in direct shear laboratory tests between peak and residual shear resistances at different rates of load application. (Note that the residual shear resistance in the triggering phase of a possible progressive failure may not be identical to the remoulded undrained shear strength). The right-hand graph demonstrates another important effect, namely the impact of the current over-consolidation ratio (OCR).

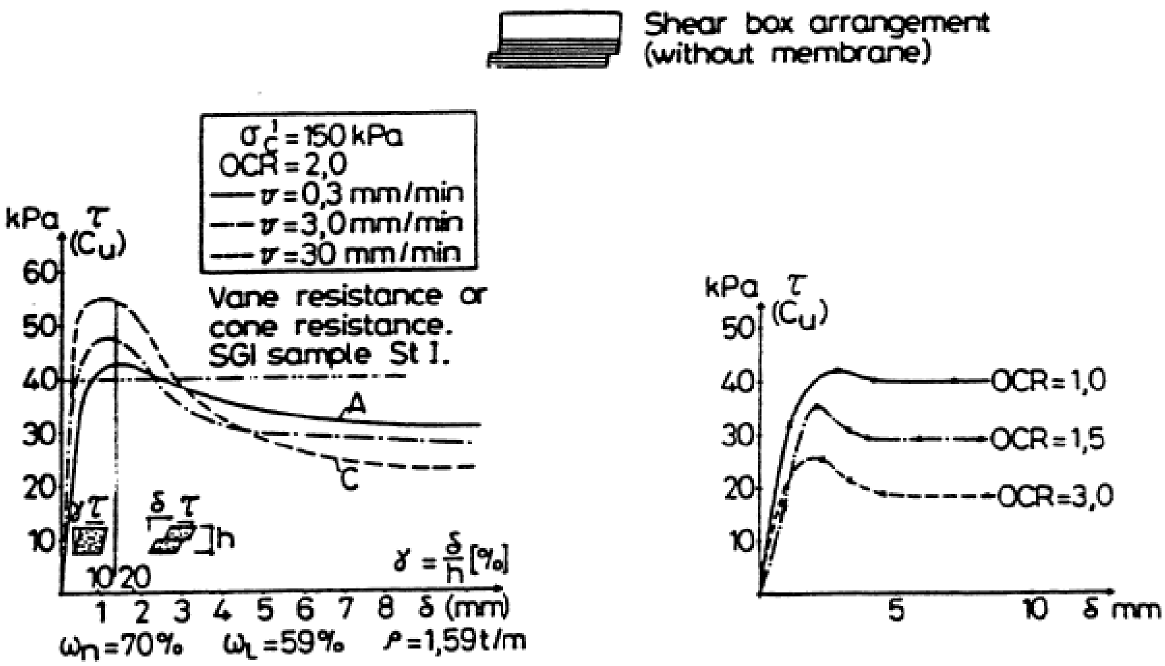


Figure 4.2. Typical test results from consolidated undrained direct shear tests on a normally consolidated Swedish clay. Note that deformation on the horizontal axis is represented both in terms of angular strain and slip displacement in millimetres.*

In this regard the Reviewer finds it anomalous that the **REPORT** does not contain diagrams of the stress-strain (deformation) relations for soil samples that are typical of identifiable critical layers in the Stratified Drift.

This Reviewer believes that, even now, Nalcor/SNC-Lavalin must present such diagrams. These are not likely to correspond to the "elastic-plastic" relations that they have generally applied.

From a safety point of view, the above soil data constitute an unclear and unsatisfactory situation, since sensitivity, low residual shear resistance, and possible subsequent liquefaction are the preconditions for potential progressive failure development.

***Conclusion*.** Without relevant stress-strain diagrams, it is not possible to have a realistic understanding of the safety factors with regard to possible progressive failure development. This is a striking omission in the **REPORT**.

---

* It may be noted that the clay samples in these tests were confined by means of mutually unconnected horizontal rings, thereby avoiding the effect on the test results related to the rubber enclosure that is normally used in laboratory tests of this kind. Bernander and Svensk, 1985 [10].

5. ADDITIONAL COMMENTS ON THE ANALYSES INTERPRETED FROM THE NALCOR/SNC-LAVALIN ENGINEERING REPORT

5.1 About failure surfaces

In the **REPORT**, stress distribution, possible slope failures, and safety factors are predicated on:

> ... *shear stresses along various horizontal surfaces passing through the two Upper Clay Layers and through the Lower Clay*.

Thus the soil models used for stability and stress distribution analyses are based on perfectly horizontal stratification. This is a questionable assumption for a number of reasons.

The interpreted soil layer stratigraphy before and after the 2013 soil investigations — presented at the IWLSC Conference, 2013 [11] — as well as the interpreted stratigraphy of other sections through the North Spur, are heterogeneous and very different from one another. This condition indicates that the sedimentary structure of the North Spur remains highly variable and uncertain, implying that the horizontal stratigraphy adopted in the stability analyses does not correspond very closely with actual conditions.

The **REPORT** does not present any rational justification for basing its Numerical Finite Element analyses on a macro soil model with perfectly horizontal stratification.

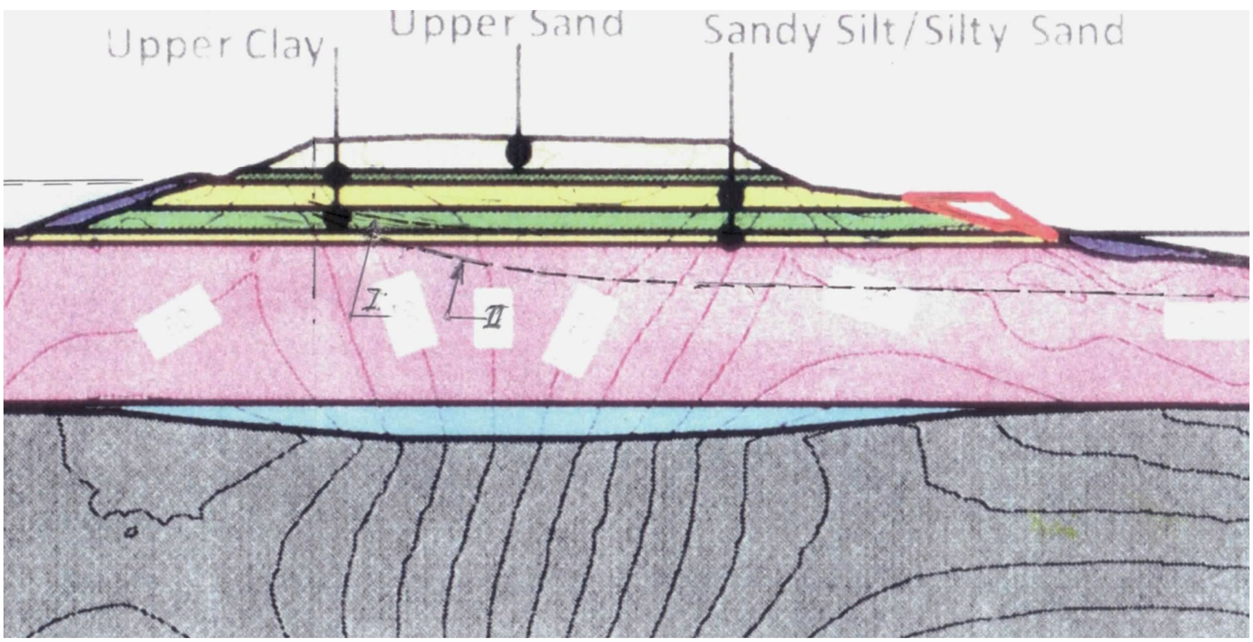


Figure 5.1. Potential failure planes (I and II) possibly leading to progressive failure development.

In soil mechanics there exists no rule stating that developing failure surfaces are even likely to be horizontal. This is true irrespective of whether the ground surface above is sloping or not.

A forward-acting failure development near the cut-off wall (COW) may, for instance, initially progress along the sedimentary orientation in the Stratified Drift, but may just as well develop more steeply through the Upper Clay and then progress further into sensitive layers in the Lower Clay formation. See, for example, the potential failure planes I and II in **Figure 5.1**.

According to **Figure 8** in posters at the IWLSE Conference 2013 [11], the lower contour of Upper Clay 2 slopes about 3 metres along the length coordinate x ≈ 200 m (near the COW) to x ≈ 350 m. This is an inclination of about 2%.

As the thickness of the Upper Clay layer near the COW is about 5 metres, the slope of a linear potential failure surface increases to 8/150 = 5.3%. If the shape of the failure plane is assumed to be parabolic, then the slope of the failure surface close to the COW will be some 10.6%. The shear stress (**τ**) due to vertical stress over such an inclination is in the order of

$$\boldsymbol{\tau} = \sigma'_v \times \sin 0.106.$$

Considering that $\boldsymbol{\gamma}_w = n + (1\text{-}n)\,\boldsymbol{\gamma}_R$, including the weight of percolating pore water above water level $\mathbf{W_L} = +39$, the vertical effective stress ($\boldsymbol{\sigma'_v}$) may be roughly estimated to:

$$\boldsymbol{\sigma'_v} \approx (59\text{-}46)\times[0.36\times10 + (1\text{-}0.36)\times26.5] + (46\text{-}39)\times[0.41\times10 + (1\text{-}0.41)\times26.5] +$$
$$+ (39\text{-}23)\times[0.48\times10 + (1\text{-}0.48)\times26.5\text{-}1\times10] =$$

$$= 13 \times 20.56 + 7 \times 19.74 + 16 \times (17.98 - 10) = 267.3 + 138.2 + 127.7 =$$

$$= 553.2 \text{ kN/m}^2$$

Hence the shear stress (**τ**) at a beginning failure plane of parabolic shape may amount to

$$\boldsymbol{\tau} = \sigma'_v \times \sin 0.106 = 553.2 \times 0.1058 \approx 58.5 \text{ kN/m}^2.$$

The impoundment from water level W_L= +17 m to W_L = +39 m represents a horizontal force above level +17 of H_w ≈ 2420 kN/m.

Assuming that the length of a triggering zone for progressive failure formation is taken to be 50 m (by experience a reasonable assumption), then the mean shear stress roughly amounts to

$$\boldsymbol{\Delta\tau} = 2420 \div 50 = 48.4 \text{ kN/m}^2.$$

The maximum value is likely to be about 50% higher than the average value, i.e.

$$\boldsymbol{\Delta\tau_{max}} \approx 1.50 \times 48.4 = 72.6 \text{ kN/m}^2.$$

Hence, the total local shear stress could be in the order of 72.6 + 58.5 = 131.1 kN/m^2. Note that this value is higher than almost all the intact undrained shear strength measurements, $\mathbf{s_u}$ = 35 to 135 kN/m^2, shown in **Table 2-1** of the **Report**.

Nor are the corresponding shear strengths very reassuring for the Lower Clay: $\mathbf{s_u}$ = 53–200 kN/m^2 (**Table 2-2)**, as steeper failure surfaces could well develop in this clay formation.

***Conclusion*.** The **Report** presents no valid justification for presuming only horizontal failure planes through soil layers in the North Spur. The rough analysis made above does not claim to render a precise account of the risk of forward (downhill) progressive failure, but it does demonstrate the need to perform a thorough study of failure planes other than horizontal ones.

5.2 On safety factors based on "elastic-plastic" LEM analysis

Section 3.2.3 of the **REPORT** cites a prominent Québec scientist:

> *Conventional limit equilibrium methods, applied to progressive landslides, generally give factors of safety for spreads well above unity and therefore cannot explain observed ground movements* (Locat 2013 [12]).

The only way the Reviewer can interpret this statement is that Dr Locat is sceptical of the validity of using "conventional limit equilibrium methods" (LEM) for predicting the stability conditions in the North Spur — and if so she is quite right. The **REPORT** does, in fact, fail to show that the stress-strain properties necessary for LEM analysis to be valid are present in the porous soils of the North Spur.

The same considerations apply to progressive landslides in Scandinavia. **None of the extensive landslides known to this Reviewer were predicted — or could even be explained in hindsight — by using stability analyses based on the conventional elastic-plastic LEM mode**.

In this respect, *all* analyses made by the Reviewer, e.g. in Refs. [13,14,15], have clearly shown that as soon as the length of a potential landslide exceeds 50–80 metres, depending to some extent on the depth of the failure plane, safety factors based on LEM become seriously unreliable. Indeed, the dynamic changes during a progressive failure are the hallmark of this phenomenon.

Conclusion. The Reviewer is compelled to doubt the reliability of safety factors in the downhill stability analyses of the eastern slope as shown in **Figure 5-2** of the **REPORT**. Unless they can be supported by additional modes of testing, these safety estimates should not be accepted as well-founded and relevant to the physical situation of the North Spur.

5.3 Effects of seismic activity

The potential effects of earthquakes have been investigated in SNC-Lavalin's *Lower Churchill Project. North Spur Stabilization Works – Design Report* of 30 January 2016 [4].

A crucially important question becomes: Have the seismic analyses also been based on elastic-plastic LEM relations? Or have they been based on the sensitive, brittle properties of loose silty sands and loose mixed layers with little clay content, as are found in the Stratified Drift?

As engineers are well aware, seismic actions on structures made of elastic-plastic materials (of the kind assumed in the **REPORT**) are normally quite harmless. However, If the affected structures consist of brittle material, such as brickwork without tough reinforcement, catastrophic events can and do take place. (See, for example, Section 2 of this Reviewer's *Riverbank Stability Report*, 2015 [1]).

The crucial questions in this context are:

- Are the materials involved highly stressed, i.e. close to peak resistance or exerted to significant strain or deformation irrespective of absolute stress levels?

- Are the soils highly sensitive or prone to liquefy, the vital issue being whether the in-situ porosities of the soil layers are higher than the critical porosity?
- Is there any potential risk of reversals of stress and strain, e.g. due to seismic effects?

In this context, it is worrying that the *Design Report* [4] offers no test results showing the impact on residual shear resistance of deviatory deformation and of stress /strain reversals. Instead it offers a computer model, similar to the one previously described, that may not be relevant to the dynamic conditions found in North Spur soils.

As has already been touched upon, the porosity of a soil may be of crucial importance. If the current porosity of a soil exceeds its critical value, $\mathbf{n} > \mathbf{n}_{crit}$, then the soil is prone to massive loss of shear resistance or to liquefaction when sheared or exposed to stress-strain reversals related to vibration, pile driving, seismic activity, etc. (See Terzaghi-Peck, Article 17 [8] and the following extracts from that article).

Spontaneous Liquefaction and True Quicksands

Experience indicates that spontaneous liquefaction most commonly occurs in fine silty sands. This fact, combined with the observed performance of true quicksands, suggests that the aggregate formed by the sand grains possesses a *metastable structure;* that is, the structure is stable only because of the existence of some supplementary stabilizing influence. A clean sand deposited under water is stable, although it may be loose, because the grains roll down into stable positions. In a sand capable of spontaneous liquefaction, some agent must interfere with this process.

**************************:

Although clean sand deposited under water has a stable structure even if loose, sand deposited simultaneously with silt may develop a metastable structure. The depressions between the grains of sand on the surface of the sediment are partly filled with loose silt which prevents the sand grains from reaching stable conditions. Subsequent consolidation under static pressure, with no lateral strain, is resisted by friction at the points of contact between the grains of sand. However, if slip at the points of contact occurs, for instance on account of a shock with an intensity exceeding a certain threshold value, the metastable structure breaks down and liquefaction takes place. The resulting failure appears to be progressive, starting at one point and proceeding by a chain reaction.

A metastable structure in a natural sand deposit is very difficult to detect, because the structure collapses during sampling and subsequent transportation. Yet, if a layer of true quicksand is located beneath the base of a structure or of an earth dam, it is a potential source of danger. Experience suggests that true quicksands may occur in layers or large lenses between layers of loose or moderately dense sands. Such occurrences are probably the result of seasonal variations in the silt content of the turbid water which transported the sand to the site of deposition. Hence, if a dam is to be built above a thick layer of loose sand, the sand should be compacted as described in Article 50 because it may contain zones of true quicksand.

The succeeding section of Article 17 deals with "Liquefaction under Reversals of Stress and Strain", which is a subject of particular relevance with regard to seismic effects. **The soil data in the REPORT, the specific slide-prone character of the North Spur, and the unique postglacial development of the Churchill River Valley all strongly indicate the risk of soil porosities being generally too high to be safe from seismic risk, i.e. $\mathbf{n > n_{crit}}$.**

If the issue has not yet been researched, it should be a priority to find out whether the 2010 slide at Edwards Island, the 2013 slide downstream of Muskrat Falls, or the 1978 North Spur slide were related to any concurrent seismic activity. If the answer is yes, then the proposed stabilization works may require radical revision.

Conclusion. The computer model of a "design seismic event" carried out by Nalcor's engineering team may be of little relevance if it is based on the assumption that North Spur soils are elastic-plastic in nature. Further, neither the current **REPORT** [3] or *Design Report* [4] offer any empirical data on the behaviour of these soils when subjected to the stresses typical of seismic events.

5.4 Stress analysis based on seepage

In the analyses of steady-state conditions — such as in-situ stress distribution — this type of drained soil analysis may be useful.

However, stability criteria and safety factors cannot be based on effective stress seepage analysis in the context of the fast development of progressive failure in deformation-softening soils, because in this case *total stress conditions* apply.

During the rapid stress changes in the different phases of progressive failure, the water content of the soil is trapped in the pore system, and there is no time for water to seep away. Thus, when transient conditions or the effects of additional loads are investigated in highly sensitive soil formations, effective stress distribution based on long-term seepage has little relevance. Similarly, although finger drains may be useful for promoting drained conditions, they constitute no effective guarantee against progressive failure development.

Although frictional resistance is generally a reliable stabilizing parameter, it must be emphasized that the crucially necessary condition for this physical law to hold true is the fulfillment of Equation 1a or Equation 1b below.

a) Even in cases, where the additional load — causing shear deformation — is of a static nature, it is imperative that the in-situ porosity (**n**) does not exceed the critical porosity ($\mathbf{n_{crit}}$):

$\mathbf{n} < \mathbf{n_{crit}}$ Equation 1a

or in terms of void ratio ($\boldsymbol{e}$)

$\boldsymbol{e} \leq \boldsymbol{e}_{\mathbf{crit}}$ Equation 1b

where **n** and $\boldsymbol{e}$ relate to one another as

$\boldsymbol{e} = n/(1-n)$ or $\mathbf{n} = e/(1+e)$

If the condition specified by Equation 1a or Equation 1b is not fulfilled, even a slow increase in static load — or deviatory deformation — may reduce frictional resistance to the extent that liquefaction occurs.

b) Furthermore, when the additional stresses involve reversal changes of stress or strain — when shear stresses alternate between $\pm$ $\boldsymbol{\Delta\tau_{x,y}}$ or axial stresses alternate between $\pm\boldsymbol{\Delta\sigma_x}$ or $\pm\boldsymbol{\Delta\sigma_y}$ — then liquefaction can occur *even if* the conditions specified by Equations 1a and 1b are fulfilled. The porosity, in fact, has to be somewhat less than its critical value. (See Terzaghi and Peck [8]).

As indicated below in Section 6.5, finger drains constitute no valid guarantee against failure due to stress-strain reversals from seismic action.

It may be noted here that the same situation applies to the banks of a new reservoir during an emergency draw-down of impounded water. Unstable soils are not able to drain fast enough following the changes in pressure, and multiple small slides inevitably occur.

5.5 General considerations on progressive failure analysis

Page 8 of the **REPORT**, lines 9 to 22, is indicative of the Nalcor engineering team's conception of progressive landslide failure. At the same time it reveals that the team is not well acquainted with the research in the field of soil mechanics that has occurred during the past 50 years, and especially since the turn of the century.

Lines 9–15 of Page 8 in the **REPORT** run as follows:

> There is no approved and accepted method to estimate in advance a safety factor before a progressive failure landslide occurs. The cases presented in the literature are always related with a landslide that has already occurred and so all cases presented are examined through a back calculation analysis. After the fact, the safety factor (SF) is known to be 1.0 or slightly below (0,999) and back calculation analysis methods use this fact and assume an unstable conditions immediately before the landslide.

Although there is much to be said about this passage, the Reviewer will focus on three points:

- It is true that, to date, there are still no general, official prescriptions concerning progressive failure analysis, but this is mainly due to the intricacy of the problem. The issue often relates to complex geological features and stress-strain (deformation) properties that are often not easy to determine in a generally applicable way. Yet this does not mean that it is an impossible task to define and analyse the problem.
- Furthermore, the difficulty of doing so cannot be a valid reason for neglecting the issue.
- It is a common misconception that progressive failure analysis can be investigated only in hindsight, i.e. by back-analysis of a near-identical landslide that has already occurred. This approach is misleading from several points of view.

For instance, practically all established and usable values of shear strength of clays have, since early in the 20th century, been determined by both back analyses of smaller slides and by applying differing methods of soil investigation, such as tests involving direct shear, compression, fall cone, and triaxial compression or vane boring in-situ**.** The results of these various procedures are rate-dependent and must therefore be carried out at specified rates of load application in order to determine the actual shear strength of the soil. (See for instance **Figure 4.1** in Section 4 above).

In very much the same way, applicable large strain (deformation) resistance values can be derived both from laboratory testing at relevant rates of loading and from back-analyses of extensive landslides in similar — but not necessarily identical — soil conditions.

Moreover, analyses of progressive failure — including the quantification of the final extent (the degree of disaster) of a number of slides — have shown that the residual shear resistance has often been only about 30% of the maximum shear stress. It is obviously imprudent not to apply this information when predicting slope stability under similar conditions.

Thus, examination of dynamic changes in shear resistance offers a safer prediction model than using elastic-plastic LEM procedures, which are known to be unreliable for potentially large landslides (> 50 to 70 metres) under sensitive soil conditions (see **Figure 5.2** below).

In addition, it is crucial to be aware that both progressive and retrogressive landslides develop in several phases at distinctly different rates of loading or of changes in stress. The properties of the stress-strain parameters occurring in these phases are normally very different. For instance, the values of both the peak shear stress and the residual resistance — which govern the triggering phase — are quite different from those acting in the late phase which determine the final extent of an extensive landslide. (Cf **Figure 4.1**).

Studies by this Reviewer [13,14,15] demonstrate how the risk of a progressive landslide can be estimated from basic geotechnical parameters. In this context it may be noted that as early as 1983–1985 Skanska Ltd had made seven predictive stability studies of extensive slopes in western Sweden, all on the basis of progressive failure formation. Four of the studies were made on behalf of the Swedish Geotechnical Institute and three in the course of ongoing Skanska Ltd projects. In only two of the seven projects were the safety factors with respect to the triggering load found to be insufficient, thus necessitating remedial measures.

Similarly, recent literature on progressive landslide failure has been published by a number of authors and institutes such as Locat (Québec), Picarelli et al. (Italy), NGI (Oslo), NTNU (Trondheim, Norway), Luleå Technical University (Sweden), and Skanska Ltd, (Sweden). Further, Puzrin, Germanovitch, Saurer, et al. (Switzerland) have published several reports on slide propagation in submerged slopes.

Conclusion. Contrary to the SNC-Lavalin statement cited above, reasonable prediction of progressive slope failure can be made without reference to a previous landslide under identical circumstances. **Analytical difficulty cannot be cited as a justifiable reason for not carrying out studies of possible stability problems in the North Spur**. It must be emphasized that the impoundment of water will put new shear stresses on sensitive soils in which retrogressive failures have already occurred. The risk of progressive failure is a very real one.

5.6 Maximum potential landslide extension using LEM

An interesting example of false prediction of slope stability by conventional LEM analysis was established in the study of the landslide at Bekkelaget, Norway by Aas, 1983 [16]. (See **Figures 5.2** and **5.3** below). The Bekkelaget landslide was also referred to and commented on by this Reviewer [13,15].

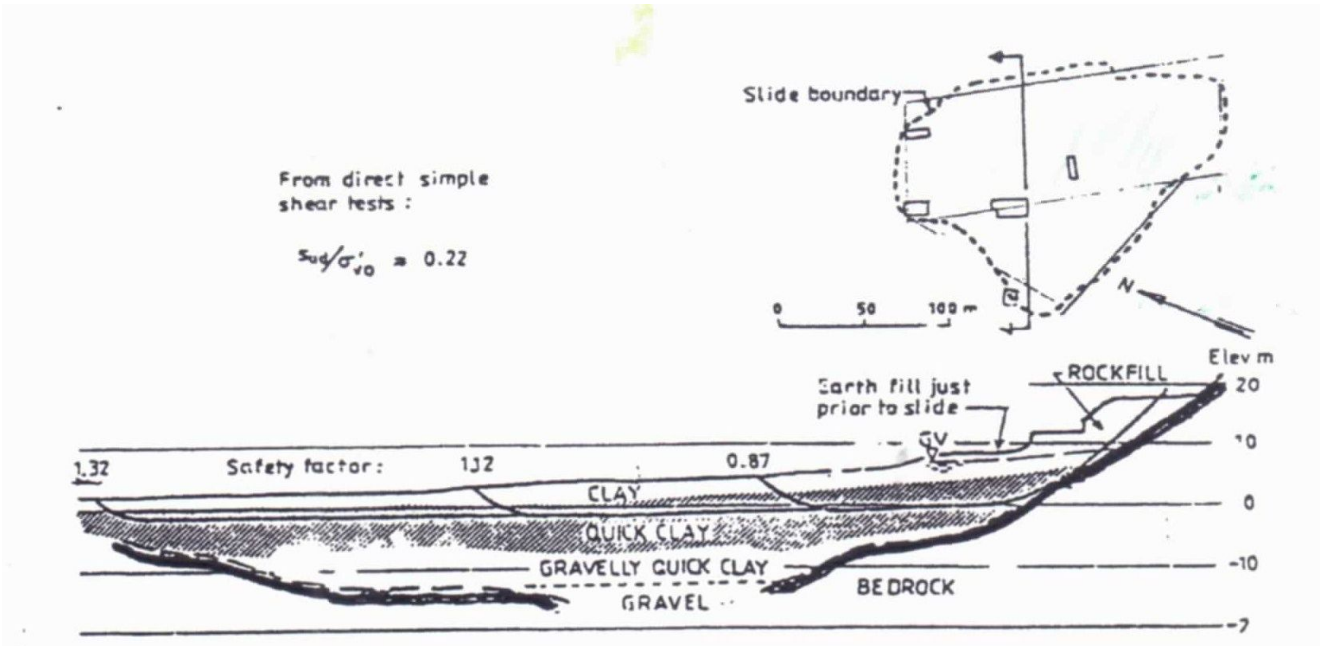


Figure 5.2. The Bekkelaget landslide, Norway. Analysis by Aas (1983) [16]. The odd circumstance to be noted here is that the slide actually developed along the 200-metre-long failure surface with the highest safety factor, F_S =1.32, and not along the short failure surface with an insufficient safety factor of F_S = 0.87, i.e. less than 1.00.

Investigations by the Reviewer have shown that, when slip circles in sensitive soils extend more than 50–70 metres, safety factors based on LEM analysis may become seriously unreliable.

Further examples given in Ref. [15] show clearly that, depending on various parameters (such as geometry, time, stress-strain relationships, etc.), safety factors based on progressive failure analysis may be as low as 25% of the corresponding safety factors calculated using LEM analysis.

In this context we may turn to Nalcor's analysis of the downstream (eastern) slope of the North Spur. A cross-section of the North Spur is diagrammed in **Figures 5-2a** and **5-2b**, page 38 of the REPORT. Note that the length of the chord of the slip circles shown in the figures extends nearly 200 metres — a clear indication that LEM methods for assessing safety are of limited usefulness.

Table B.III - Downhill progressive slide - triggering loads -

Comparison of slope hazard based on PrFA analysis and
slope hazard based on conventional (IPFA) analysis.

2009 05 22
File: Triggering load - Synopsis II
Author: Stig Bernader

Legend: **c** - Peak shear resistance
c(lab) Laboratory shear strength
c(R) Residual resistance
to in situ shear stress

dn = Down-slope displacement at x = L
L = Influencial length of force N_l
$q_{crit} = N_{crit}/H$
t(el) Elastic limit

\# Safety factor based on ideal- plastic equilibrium analysis of a slopeof length L

	Input data Depth to failure surface **H = 20 m**									**IPFA analysis**			**Results from IPFA analysis #**				**#Con-ventio-**	**Progressve f. PrFA analysis ***		
Case reference number	**Gra-dient (Gr)** tan(Gr)	**In situ shear stress to** kPa	**Peak shear resist-ance** kPa	**Lab shear resist-ance** kPa	**Assoc. shear defor-mation** %	**Elas-tic limit t(el)** kPa	**Recid. shear resist. cR/clab** 1	**Recid. shear resist. cR** kPa	**Density of clay g** kN/m3	**Ep-Ea =4H c** kN/m	**H** m	**sin Gr** 1.00	**qf** kN/m2	**Max. height of fill** m	**Fpr = qcrit/q q=18** kN/m2	**q** kN/m2	**nal safety factor Fc q=18**	**qcrit** kN/m2	**x = L (crit) at Ncrit** m	**Fpr/Fc**
Case 1	**0.050**	15.98	30	25	3.0	16	**0.400**	**10.0**	16.0	2000	20.0	0.0499	150.43	**8.36**	**1.081**	18.0	**2.23**	19.5	111.8	**0.484**
Case 2	**0.060**	19.17	30	25	3.0	16	**0.400**	**10.0**	16.0	2000	20.0	0.0599	132.88	**7.38**	**0.887**	18.0	**1.91**	16.0	112.7	**0.464**
Case 3	**0.070**	22.35	30	25	3.0	16	**0.400**	**10.0**	16.0	2000	20.0	0.0698	114.98	**6.39**	**0.645**	18.0	**1.67**	11.6	112.9	**0.385**
Case 4	**0.080**	25.52	30	25	3.0	16	**0.400**	**10.0**	16.0	2000	20.0	0.0797	97.30	**5.41**	**0.391**	18.0	**1.53**	7.0	104.0	**0.256**
Case 4a	**0.080**	25.52	30	25	3.0	16	~~0.400~~	**5.0**	16.0	2000	20.0	0.0797	97.41	**5.41**	**0.363**	18.0	**1.55**	6.5	100.0	**0.235**
Case 5	**0.050**	15.98	30	25	3.0	16	**0.600**	**15.0**	16.0	2000	20.0	0.0499	153.46	**8.53**	**1.226**	18.0	**2.20**	22.1	118.5	**0.557**
Case 6	**0.060**	19.17	30	25	3.0	16	**0.600**	**15.0**	16.0	2000	20.0	0.0599	134.77	**7.49**	**0.994**	18.0	**1.88**	17.9	119.2	**0.528**
Case 7	**0.070**	22.35	30	25	3.0	16	**0.600**	**15.0**	16.0	2000	20.0	0.0698	115.82	**6.43**	**0.713**	18.0	**1.65**	12.8	119.2	**0.433**
Case 8	**0.080**	25.52	30	25	3.0	16	**0.600**	**15.0**	16.0	2000	20.0	0.0797	97.14	**5.40**	**0.432**	18.0	**1.50**	7.8	110.3	**0.288**
Case 9	**0.050**	15.98	30	25	3.0	16	**0.800**	**20.0**	16.0	2000	20.0	0.0499	154.84	**8.60**	**1.425**	18.0	**2.19**	25.6	121.6	**0.651**
Case 10	**0.060**	19.17	30	25	3.0	16	**0.800**	**20.0**	16.0	2000	20.0	0.0599	137.31	**7.63**	**1.176**	18.0	**1.85**	21.2	127.9	**0.636**
Case 11	**0.070**	22.35	30	25	3.0	16	**0.800**	**20.0**	16.0	2000	20.0	0.0698	117.26	**6.51**	**0.845**	18.0	**1.61**	15.2	130.1	**0.526**
Case 12	**0.080**	25.52	30	25	3.0	16	**0.800**	**20.0**	16.0	2000	20.0	0.0797	96.87	**5.38**	**0.505**	18.0	**1.46**	9.1	120.8	**0.346**
Case 13	**0.050**	15.98	30	25	3.0	16	**0.900**	**22.5**	16.0	2000	20.0	0.0499	100.00	**5.56**		18.0		25.6		
Case 14	**0.060**	19.17	30	25	3.0	16	**0.900**	**22.5**	16.0	2000	20.0	0.0599	100.00	**5.56**		18.0		21.1		
Case 15	**0.070**	22.35	30	25	3.0	16	**0.900**	**22.5**	16.0	2000	20.0	0.0698	118.50	**6.58**	**0.956**	18.0	**1.58**	17.2	139.4	**0.605**
Case 16	**0.080**	25.52	30	25	3.0	16	**0.900**	**22.5**	16.0	2000	20.0	0.0797	96.64	**5.37**	**0.568**	18.0	**1.43**	10.2	129.7	**0.398**
Mean values							0,2								**0.87**		**1.87**			**0.49**
Deviations - max/min			**%**																**34.2**	**-51.5**

Figure 5.3. Relationships between safety factors determined by Progressive Failure Analysis and elastic-plastic LEM analysis. Note especially the column with a red heading [15].

Conclusion. The data presented demonstrate the inadequacy of Limit Equilibrium Mode analysis to calculate safety factors for the North Spur. The Nalcor authors have not presented a true Progressive Failure Analysis in their **REPORT**, and there is no indication that such work has ever been carried out.

5.7 Regarding soil properties in the North Spur and over-consolidated clays in Eastern Canada

In the **REPORT**, reference is often made to landslide conditions in Eastern Canada (EC), as if the geology and soil properties of the Churchill River Valley (CRV) were a uniform part of this vast area. However, as this Reviewer and others have pointed out, the consolidated clays typically found in EC are different both in origin and in physical properties from the mixed marine sediments of the CRV. No conclusions drawn from one can be applied to the other. [See Sections 1.1, 1.2, 3.1.2, 3.1.3 of these Comments, and Sections 5.1, 5.2, and 5.3 of the **REPORT**.]

For instance, according to the **REPORT**, the fact that most landslides in EC are classified as retrogressive spreads is used to exclude most types of slope failure in the CRV other than spreads and flow slides. Further, the fact that the main failure surface in spreads often tends to incline gently is used to support a methodology of investigating only failure development along horizontal surfaces. (See Section 5.1 and the end of Section 5.2 of the **REPORT**).

However, in reality the properties of the highly over-consolidated fat clays — widespread in Eastern Canada — have little in common with the under-consolidated mixed lean clays or porous silty/sandy soils such as those in the Stratified Drift of the North Spur. Nor do EC clays conform to the generally porous marine sediments common in the Churchill River Valley. (See Sections 2 and 3 of the Reviewer's previous 2015 Report [1]).

In the retrogressive spread slide of about 8 hectares that occurred at Saint-Barnabé-Nord, the ratio of clay to silt varied from about 70%/27% to 30%/60%, whereas the sand content was mostly less than 5% and very rarely in excess of 10%. In contrast, the clay content of the Upper Clays and mixed silty sands of the Churchill River Valley is far below 30%. (Section 3, Ref. [1]).

Moreover, the permeability values (**k** = m/sec) in Saint Barnabé-Nord ranged from 1×10^{-9} to 5×10^{-9} m/s, whereas the **k**-values of the Upper Clays in the North Spur are about 1×10^{-7} m/s. This implies that the mixed Upper Clays in the Stratified Drift are from 20 to 100 times more permeable than the clays in Saint-Barnabé-Nord.

In other words, the properties of the soils in Saint-Barnabé-Nord were those of true clays, and their sensitivity was due to high over-consolidation ratios (OCRs) and not to high porosity. Note that the high OCRs imply that the current vertical stress is considerably less than the original consolidation pressure [17].

In clear contrast, the sensitivity of soils in the North Spur is related to the in-situ soil porosity (**n**) being markedly greater than the value of the critical porosity ($\mathbf{n}_{crit}$). Such types of soil may liquefy when subjected to a moderate deviatory deformation or because of minor repetitive stress-strain reversals — and that remains true irrespective of the prevailing stress level.

Conclusion. The sensitivity of the soils in the Churchill River Valley is of a totally different nature and origin than that of the highly over-consolidated clays of Eastern Canada.

5.8 *A proposal for realistic testing of the porosity of soils in the Stratified Drift*

As stated in the quotation from Terzaghi and Peck on Page 14 [8] above:

> *"A metastable structure in a natural sand deposit is very difficult to detect, because the structure collapses during sampling and subsequent transportation."*

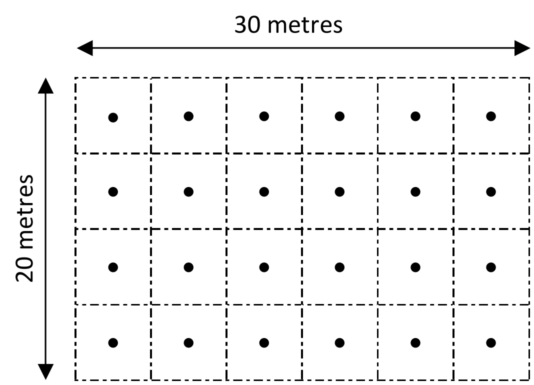
As shown in previous Sections, both the data presented in the **REPORT** and the general character and development of the Churchill River Valley strongly indicate that the in-situ porosities (or void ratios) of some soils of the North Spur are probably critically high. If this is the case, then the safety factors presented in the **REPORT** are of little relevance. Considering the enormous catastrophe that would envelop downstream communities in the event of a breach in the North Spur, the true status of soil porosity in the North Spur should be verified in-situ, and verified beyond any shadow of a doubt.

A practical way to accomplish this goal is to carry out tests in which the soil profile is subjected to violent vibratory treatment and the subsequent changes are carefully measured. Such a test yields a more dependable measure of the actual in-situ porosities of soil layers.

This Reviewer suggests the following in-situ stress test (provided of course that such a test has not already been carried out).

Proposed Testing Procedure

1) Within an area of say 20 metres x 30 metres, 24 piles are driven by a rammer in straight lines at 5-metre centres. A positive feature of such a test area is that it need not necessarily obstruct or interfere with ongoing construction work.



Suggested pattern of test piles

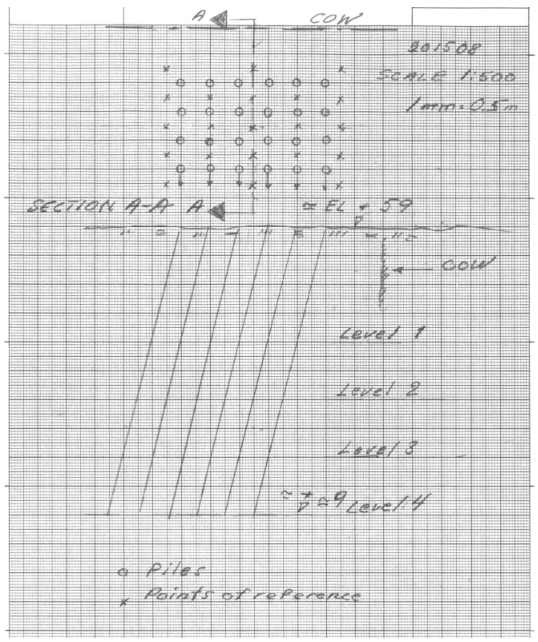
2) The piles may consist of 0.3 m diameter steel pipes fitted with splices every 10 metres. The pile tips should be flat and closed by a perforated steel plate to allow dissipation of water. Alternatively, other methods of drainage may be employed. A point of reference for each pile and its precise level must be fixed and registered.

3) All piles are driven 20 metres to elevation ≈ +39, i.e. about 20 m below the ground surface level. The sequence in which the piles are driven is not crucial. The settlements of all reference points are then accurately measured, and excess pore water pressure is allowed to subside by drainage through the perforated bottom plates or by other means.

4) All piles are then driven another 10 metres to elevation ≈ +29, i.e. about 30 m below the ground surface. The settlements of the reference points are measured and excess water pressure is again dissipated.

5) All piles are driven another 10 m to elevation ≈ +19. The settlements of the fixed points are again recorded and excess water pressure dissipated. At this point the total soil settlement indicates roughly the amount of vibratory compaction of the loose Stratified Drift.

6) All piles are driven another 10 m to elevation ≈ +9. The additional settlements generated in the Lower Clay are measured. Some degree of vibratory compaction may also be expected in this layer. Below, the recommended test pattern and depth levels are diagrammed together:



Impact tests of this kind are the best way to get a realistic notion of the true in-situ porosity of such soil layers. The above procedures yield a reliable indication of the effects on soil porosity of heavy vibratory impact. From the measurements of soil settlement it is possible to evaluate the inherent sensitivity of the soil profile, i.e. how sensitive the layers are to deviatory deformation and to stress/strain reversals such as those caused by such large triggering loads as changes in water levels and seismic activity.

If the settlements generated in the Stratified Drift and Lower Clay prove to be minute or moderate, then the reliability of the results of analyses made in the **REPORT** will be generally confirmed.

If, on the other hand, the settlements indicate a high degree of compaction — i.e. the mean in-situ porosity (**n**) is clearly in excess of the critical porosity ($\mathbf{n_{crit}}$) — **then it will be necessary to strengthen the affected soil structures**. As per Terzaghi and Peck [8], the recommended technique would be vibratory compaction, to be carried out over a wide area of the North Spur east of the cut-off wall.

6. Summary

Although the Nalcor/SNC-Lavalin **Report** is a mostly comprehensive geotechnical study, in the opinion of this Reviewer it is deficient in important aspects of the laws of soil mechanics and in current research in this field. The following shortcomings may be noted:

6.1 On progressive failure

In Section 3 of the **Report** there is generally correct wording about the possibility of progressive and retrogressive failure formation. Yet, apart from a number of references to the literature on the subject of "Progressive Failure", there is no evidence in the **Report** of any actual progressive failure analyses having been performed. Nor have any results from stress-strain (deformation) testing, which are indispensable for performing such analysis, been presented in the **Report**.

Progressive failure analysis requires that soil parameters — especially the stress/deformation relationships — applicable to each of the different phases of landslide development be defined and implemented in the analysis.

This means, for instance, that even if FLAC analysis (Fast Lagrangian Analysis of Continua, a computer model) is utilized, each phase of a progressive (or retrogressive) landslide has to be studied separately, applying the specific relation between stress and deformation that is valid in the phase being studied.

6.2 On the general application of elastic-plastic (LEM) analysis

The studies in the **Report**, aiming at certifying acceptable safety against the initiation of possible progressive failure development in the downstream slope, are all based on elastic-plastic soil behaviour. Yet there is no evidence in the **Report** that this stress-strain relationship has been validated for the porous soils of the North Spur.

This is extremely unsatisfactory. One of the best-established facts about the soil conditions in the North Spur (and generally in the Churchill River Valley) is the finding that the soil layers do *not* comply with, or abide by, the kind of elastic-plastic behaviour that is generally assumed in the **Report**.

The geotechnical data presented in the **Report**, e.g. in Table 2-2 on page 19, indicate that these soils, especially in the Stratified Drift, have a marked potential propensity to liquefy — to lose most of their shear resistance — when subjected to deviatory deformation or stress-strain reversals. Note that such liquefaction has, in fact, recently taken place in similar soils in the Churchill Valley, causing large landslides [5]. It is negligent to say simply that all of these were retrogressive landslides, *whereas none of them took place under the future conditions of reservoir impoundment*.

Again, such liquefaction is due to the in-situ porosity being generally greater than the critical porosity. (See Section 2). The use of LEM and drained analyses is, according to basic rules in soil mechanics, justifiable only as long as it proven that the actual soil porosity in-situ ($\mathbf{n}$) is not too different from the critical soil porosity ($\mathbf{n}_{\mathbf{crit}}$).

If this proves not to be the case in the North Spur, then there will be an urgent need for soil compaction over large areas of the North Spur. (Cf Terzaghi and Peck [7,8] and the quotation in Section 5.3, as well as the compaction test proposal in Section 5.8).

6.3 Horizontal failure planes

Stability modelling in Sections 5 and 6 of the **REPORT** is based on horizontal failure surfaces through the Upper and Lower Clay formations. Yet there is no rule in soil mechanics exempting failure planes that are not horizontal. In fact, failure planes do not as a rule favour horizontal propagation. On the contrary, progressive landslide initiation is typically triggered by locally steep failure surfaces in the initiation zone.

As indicated in Section 5.12 above, failure surfaces may well develop both in the lower Upper Clay layer and along sensitive drifts in the massive Lower Clay formation. Dependable stability analysis must therefore include any type of failure surface propagation, based on verified stress-deformation relationships.

6.4 Maximum potential landslide extension using LEM

The engineering team's proposals for the stabilisation of the eastern or downstream slope of the North Spur are shown in cross-section in Figure 5-2 on page 38 of the **REPORT**. Several slip circles are indicated by dashed lines on the potentially vulnerable slope. Note that the chord length of the slip circles, representing the maximum displacement of a landslide, is almost 200 metres.

Investigations by this Reviewer [13,14,15] have indicated that when slip circles in sensitive soils extend more than 50 or 70 metres, safety factors based on LEM analysis become very unreliable, especially with respect to concentrated additional loading. (See also Section 5.6).

6.5 Finger drains

Although finger drains are useful for promoting and maintaining drained conditions over time, they constitute no guarantee against progressive failure development.

During the rapid stress changes in the different phases of progressive failure formation, the water content of the soil is virtually trapped in its pore system. There is little or no time for water to percolate in any direction. Hence, if the porosity (**n**) is in excess of the critical porosity ($\mathbf{n}_{crit}$), soil liquefaction may take place whether or not finger drains are present.

6.6 Investigation of in-situ porosity conditions in soil layers

When evaluating the results from the testing of initial void ratios, the difficulty of obtaining undisturbed soil samples must be taken into account. In particular, the in-situ void volume of soil material with high porosity is easily affected by the sampling procedure. (Cf the Terzaghi-Peck quotation in Section 5.3, also Section 5.8).

6.7 Required testing

The soil investigations presented in the **REPORT** comprise mostly laboratory testing carried out in 1979 and 2013. Relatively few dynamic tests were done in-situ. The detailed computer model that is employed is explicitly based on elastic-plastic conditions and LEM analysis. Dynamic stress conditions are extrapolated from static ones.

However, as is well-recognised, at least several of the soils of the North Spur are not of the elastic-plastic type. Furthermore, LEM analysis cannot model or predict potential failures of the downhill progressive kind.

It is noted that the scars of nine major landslides are visible on the two sides of the North Spur as far as the Kettle Lakes.* The most recent of these, on the downstream slope in 1978, involved liquefaction of slip surfaces in the Stratified Drift over a long lateral distance. All experts agree that without engineering intervention, the North Spur will continue to suffer landslides and degrade as a natural barrier to the Churchill River.

Bearing this in mind, it is striking that the authors of the **REPORT** have not offered the results of dynamic hydro-geological testing that would better quantify the risk of a progressive failure. Without such results, the safety factors presented in the **REPORT** cannot be accepted as best engineering practice.

This Reviewer has proposed, in Section 5.8, a practical method for making a simple, effective in-situ assessment of the stability of the North Spur even while construction proceeds. If the soil settles significantly under vibrational stress, then the safety factors and proposed stabilization works in the **REPORT** may be judged inadequate. If however, the soil settles very little, then the assumptions of the **REPORT** may be considered to be confirmed.

The Reviewer urges that this testing be done immediately, before construction makes significant changes to current water levels.

6.8 Potential mitigation

If the tests recommended in Section 5.8 demonstrate a risk of North Spur failure following impoundment despite the proposed stabilization works, then additional stabilization would be required. This Reviewer suggests — tentatively, until the data are better known — that this would best be done by compacting the upper soils of the North Spur over a wide area.

The time required for such compaction, and its interaction with the construction program, is a further compelling reason for carrying out the required vibrational testing immediately.

---

* There are at least two giant older scars of so called “bottle-neck slides”, one of which now forms the Kettle Lakes depression. Bottle-neck landslides occur in highly sensitive soils [18].

7. *Conclusion*

The Nalcor/SNC-Lavalin **Engineering Report** of 21 December 2015, subtitled "North Spur Stabilization Works, Progressive Failure Study", offers a detailed examination of the suitability of the North Spur as a dam. It concludes that, following a series of measures to stabilize its slopes against further landslides, the North Spur will form a safe and reliable part of the impoundment wall.

This Reviewer has commented in detail on this **Report** and its conclusions. They are summarized here:

- The **Report's** stability analysis is based on *inappropriate assumptions* about the soil characteristics of the North Spur, failure planes, and dynamic stresses.
- The **Report**, despite its subtitle, does not offer a study of potential progressive failure, and *recent relevant research in this field is ignored*.
- The **Report's** computer model is based on *inappropriate data and assumptions* that stress response under static conditions can be used to model dynamic ones.
- The stabilization measures proposed in the **Report** — principally to maintain vulnerable soils in a semi-drained state — *are likely to be of little relevance* to the deficiencies noted above.

In view of these deficiencies — and noting that large flowslides involving liquefaction of silty clay are a notable feature of the Lower Churchill Valley, and noting that very large slides of this kind occurred in 1978 on the North Spur itself, in 2010 at Edwards Island, and in 2014 on the north bank just five kilometres downstream of Muskrat Falls[*] — this Reviewer believes that the information that Nalcor has made public to date is not sufficient to conclude that the Muskrat Falls dam is both safe and reliable. Thus the Reviewer recommends that a renewed analysis of the risk of a progressive failure be initiated at once for the North Spur.

The Reviewer recommends that the first component of such an analysis should be an empirical in-situ test of the North Spur: its response to the heavy vibration of pile-driving, as detailed in Section 5.8.

If the mixed layers of the Stratified Drift are found to settle and compact upon such heavy vibration, then these layers must be considered susceptible to possible liquefaction and progressive flow-sliding under the major shear stresses that will follow impoundment.

In such a case, new geo-engineering studies must be carried out with a view to quantifying the risk and stabilizing the vulnerable soils. It is likely that this would involve compaction of the upper soils of the North Spur over a wide area and a major revision of the current construction program.

[*] This last landslide has good video documentation, found at https://www.youtube.com/watch?v=LIcL_pN4NlQ.

REFERENCES

[1] Bernander S. *Lower Churchill River, Riverbank Stability Report*, 26 November 2015. Previously submitted as expert testimony to the NL Public Utilities Board.

[2] Lower Churchill Project — North Spur Updated*, Nalcor Report to the Independent Engineer*. 21 July 2014.

[3] Nalcor–SNC/Lavalin. (21 December 2015) ***ENGINEERING REPORT***, North Spur-Stabilization-Works, Progressive-Failure-Study. 128 pages.

[4] SNC-Lavalin. (30 January 2016) "Lower Churchill Project. North Spur Stabilization Works – Design Report". Nalcor Doc. No. MFA-SN-CD-2800-GT-RP-0004-01. 264 pages.

[5] AMEC Earth & Environmental. (2011) "Geotechnical Investigation: Edwards Island Landslide, Churchill River, Labrador". Nalcor Contract #LC-EV-007. 32 pages.

[6] Bernander S. *Further Comments on the Updated Nalcor Report of 21 July 2014*. Dated 7 January 2016.

[7] Terzaghi K and Peck RB. "Index Properties of Soils"*,* Article 6, p. 28, *Soil Mechanics in Engineering Practice*, Second Edition 1967. (John Wiley & Sons Inc. New York, London, Sidney).

[8] Terzaghi K and Peck RB. "Shearing Resistance of Cohesionless Soils", Article 17, pp. 107–111. *Soil Mechanics in Engineering Practice*, Second Edition 1967. (John Wiley & Sons Inc. New York, London, Sidney).

[9] Aas G. (1966). "Special Field Vane Tests for the Investigation of Shear Strength of Marine Clays". *Report*, Norwegian Geotechnical Institute, Oslo.

[10] Bernander S and Svensk I, et al. (1985). "Shear strength and deformation properties of clays in direct shear tests at high strain rates". *Proc. 11th ICSMFE*, San Francisco, Vol. 2/B/5, pp. 987–990.

[11] Geotechnical data related to the Muskrat Falls Dam Project on posters at the IWLSC Conference (Québec 2013).

[12] Locat A, Jostad HP, Leroueil S. (2013) "Numerical modeling of progressive failure and its implication to spreads in sensitive clays". *Canadian Geotechnical Journal*, 50(9), pp. 961–978.

[13] Bernander S. (2000). *Progressive Landslides in Long Natural Slopes*. Licentiate Thesis. 2000:16 *ISSN:1402–1757 * ISRN: LTU–LIC–00/16–SE.

[14] Bernander S. (2008). *Downhill Progressive Landslides in Soft Clays*. 2008:11|ISSN:1402–1528| ISRN: LTU–FR–08/11–SE.

[15] Bernander S. (2011). *Progressive Landslides in Long Natural Slopes. Formation, Potential, Extension and Configuration of Finished Slides in Strain-Softening Soils*. Doctoral Thesis. ISBN: 978-01-7439-283-8. ISSN: 1402-1544 www.ltu.se.

[16] Aas G. (1983). “A Method of Stability Analysis applicable to Natural Slopes in Sensitive and Quick Clays”. *Proc. Symposium on Landslides*, Linköping 1982, Swedish Geotechnical Institute, Report no 17, pp. 7–25.

[17] Singh A and Mitchell JK. (1968) “General stress-strain-time function for soils”. *J Soil Mech Found* Div 94 (SM 1), ASCE, pp. 21–46.

[18] Bernander S. “Outline of Serious Concerns on the Adequacy of Landslide Analysis at the North Spur, Muskrat Falls” (including an Executive Summary), as presented to Ms C. Blundon, Public Utilities Board, NFL. Dated January 2014.

References to further professional reports or comments (2014) by S. Bernander on slope stability related to the North Spur or to the Churchill River Valley in general:

[19] Bernander S. “Comments on Nalcor’s Report to the Independent Engineer of 21 July 2014”. Dated 14 September 2014.

[20] Bernander S. PowerPoint presentation (Appendix III) in an assembly hall in Saint John's on 30 October 2014 and at the Memorial University on 31 October 2014.