Riverbank stability in loose layered silty clays

Comments on the North Spur Dam at Muskrat Falls in Churchill River, Labrador, Newfoundland.

Stig Bernander and Lennart Elfgren
A schematic view of the North Spur Ridge illustrates the stability problem. When the water level is raised with 22 m, from +17 m to +39 m, a force $N_w$ starts to act on the cut-off wall. The question is if the shear stresses $\tau - \tau_o$ on a possible slip surface (red dotted line) are big enough to hold the force $N_w$ in equilibrium. Here $\tau_o$ is the in situ stress before the water level starts to rise.

The lower left figure illustrates the material properties of the soil with a shear stress/strain ($\tau/\gamma$) diagram with a maximum shear stress $s$ and a residual softened shear stress $s_R$. The red dotted line indicates the classic ideal plastic assumption with no reduction of the shear strength. When the force $N_w$ starts to act on the cut-off wall, the soil behind the wall starts to deform ($\gamma$) and shear stresses ($\tau$) are growing according to the stress-strain diagram. When the shear stresses reach the maximum value $s$, they start to soften and are finally reduced to the residual value $s_R$ close to the wall.

The maximum value the wall may carry is $N_{\text{crit}} = \int(\tau - \tau_o)dx$ as illustrated in the lower right figure. In order for the slope to remain stable, $N_{\text{crit}}$ must be at least as large as $N_w$. The main objective of a stability analysis is to calculate $N_{\text{crit}}$ for varying material properties. If unrealistic ideal plastic properties are assumed (green dotted line), there will in many cases falsely be no apparent stability problem.
Preface

This report aims to summarize some issues regarding the stability of riverbanks made up of glacial marine sediments. The report consists of an introduction and of three appended reports written by Stig Bernander arguing for the need of an up-to-date analysis of the risk for a progressive failure of the proposed dam at the North Spur at Muskrat Falls in Churchill River in Labrador, Newfoundland, Canada.

Möldal and Luleå in July 2017

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Abstract

The differences are outlined in landslide analysis between the classic limit equilibrium method with assumed plastic properties of the soil and a progressive analysis applying softening material properties.

The risk for failure is studied in the dam at the North Spur riverbank ridge at Muskrat Falls in Churchill River in Labrador, Newfoundland, Canada. A sloping failure surface is much more critical than the horizontal surfaces which have hitherto been studied. Results from new analyses have now been obtained applying softening material properties probable for the ridge. The results indicate safety factors lower than 0.5, i.e. there is a high risk that the ridge will fail if the water level is raised to the proposed level.

Three reports are appended where Stig Bernander argues in detail for the need for a proper progressive failure analysis based on measured material properties. He also proposes how such properties may be obtained and gives an example of a way to stabilize the ridge if the soil properties show a softening behaviour. Finally examples of progressive failure analyses are included using probable material properties.

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IV. Spreadsheet Analysis, 2017-06-01
Stability of the Hydropower Dam at Muskrat Falls studied by Stig Bernander

Case 3: \( \tau_0 = 21.1 \text{ kPa} \quad s = 60 \text{ kPa} \quad s_R = 12 \text{ kPa} \quad s/s_R = 5 \quad N_{cr} = 866 \text{ kN/m} \)
Safety factor \( F = N_{cr} / N_w = 866 / 2420 = 0.357 < 1 \)

Case 4: \( \tau_0 = 41.1 \text{ kPa} \quad s = 70 \text{ kPa} \quad s_R = 14 \text{ kPa} \quad s/s_R = 5 \quad N_{cr} = 521 \text{ kN/m} \)
Safety factor \( F = N_{cr} / N_w = 521 / 2420 = 0.215 < 1 \)
Notation

**Upper case Roman letters (in alphabetical order)**

\[ E = \text{Total earth pressure} = E_0 + N \ (\text{kN/m}) \]

\[ E_0 \quad \text{In-situ earth pressure (kN/m)} \]

\[ E_{p\text{Rankine}} \quad \text{Critical down-slope earth pressure resistance at passive Rankine failure (kN/m)} \]

\[ F_s^l \quad \text{Safety factor for local failure} \ (N_{cr} / N_q) \]

\[ F_s^{HI} \quad \text{Safety factor for global failure} \ (E_{p\text{Rankine}} / E) \]

\[ G \quad \text{Secant modulus in shear (GPa)} \]

\[ H_{X_i \rightarrow X_{i+1}} \quad \text{Height of element} \ i \rightarrow i + 1 \ (\text{m}) \]

\[ K_0 \quad \text{Ratio between minor and major principal stresses} \]

\[ K_p \quad \text{Rankine coefficient for lateral passive earth resistance} \]

\[ L_{cr} \quad \text{Limit length of mobilization of shear stress at} \ N_{cr} \ (\text{m}) \]

\[ N_{cr} \quad \text{Critical load effect initiating local slope failure (kN/m)} \]

\[ N_q \quad \text{Additional load in the direction of the failure plane (kN/m)} \]

\[ N \quad \text{Earth load increment due to additional load (kN/m)} \]

\[ V_p \quad \text{Volume of pores} \]

\[ V_s \quad \text{Volume of solids} \]

**Lower case Roman letters (in alphabetical order)**

\[ b \quad \text{Width of element considered (m)} \]

\[ s, s_u \quad \text{Un-drained peak shear strength} \ (\text{also sometimes called} \ S, S_u, c, c_u) \ (\text{kPa}) \]

\[ s_R \quad \text{Residual shear strength} \ (\text{also sometimes called} \ S_R, c_R) \ (\text{kPa}) \]

\[ e = V_p / V_s \quad \text{void ratio} \]

\[ g \quad \text{Gravity} \ (9.81 \text{ m/s}^2) \]

\[ m \quad \text{mass} \ (\text{kg}) \]

\[ n = V_p / (V_p + V_s) \quad \text{porosity} \]

\[ q \quad \text{Additional vertical load} \ (\text{kN/m}^2) \]

\[ w \quad \text{water ratio} \ (= e \cdot \rho_w / \rho_d) \]
**Greek letters (in alphabetical order)**

- β  
  - Slope gradient at coordinate x (°)
- γ(x, z)  
  - Deviator shear strain at point x, z
- γ_{el}  
  - Deviator strain at elastic limit
- γ_{f}  
  - Deviator strain for shear stress peak value
- γ  
  - Load from weight of soil ρg (kN/m^3)
- δ_{cr}  
  - Critical displacement in terms of axial deformation (m)
- δ_{N}  
  - Down-slope displacement in terms of axial deformation generated by forces N (m)
- δ_{r}  
  - Down-slope displacement in terms of deviator deformation (m)
- ρ  
  - Soil density (kg/dm^3)
- v  
  - Poisson coefficient
- τ_{el}  
  - Shear stress at elastic limit (kPa)
- τ, τ(x, z)  
  - Total shear stress in section x at elevation z (kPa)
- Δτ_{x_i-x_i+1}  
  - Shear increment from step i to i + 1 (kPa)
- τ_0(x, z)  
  - In situ shear stress in section x at elevation z (kPa)

**Masses, Volumes and Ratios**

- Void ratio e = V_g/V_s
- Porosity n = V_p / (V_p + V_s)
- Water ratio w = m_w / m_s = e·ρ_{aq}/ρ_s

**Definitions**

Clay has a particle size less than 0,002 mm; silt has a particle size less than 0,63 mm and sand has a particle size less than 2 mm.
1. Glacial sediments

The location of the studied riverbank at Muskrat Falls in Churchill River is given in Figure 1.1. A view of the falls and the North Spur is given in Figure 1.2.

The stability conditions in natural slopes are closely related to their geological and hydrological history. Slopes in the northern hemisphere of clay (particle size less than 0.002 mm) and silt (particle size less than 0.63 mm) are made up of glacial and post-glacial marine deposits that emerged from the regressing sea after the last glacial period some ten thousand years ago. Hence, the sediments deposited at the end of this period in sea and fjords are now found in valleys and plains above present sea level, forming deep layers of soft and silty clays, silts and sands.

As the ground gradually rose above the sea level, the strength properties of the soils and the earth pressures in the slopes have, by consolidation and ongoing creep movement, slowly accommodated over time to increasing loads due to changing hydrological conditions. Apart from the retreating free water level, this metamorphosis consists of dry crust formation, increased downhill seepage pressures, falling ground water table and the due increase of effective stresses in the soil mass. Chemical deterioration may also have affected soil strength and sensitivity.

The properties of different soil layers may vary considerably from loosely layered sands and clayey silts to over consolidated clays, see Bernander (2011) and Appendix I.
Figure 1.2. Muskrat Falls with the North Spur. The Spur ends with a granite rock close to the falls, Rock Knoll. Section A denotes a studied part of the ridge. Dury (2017).

2. Soil properties

Glacial soils may be extremely sensitive and thus liquefy when remoulded (quick clays/sands). In tests the clays exhibit a peak strength after which the soil structure collapses leading to a corresponding reduction in the stress.

A typical shear stress/shear strain relationship for a sensitive deformation softening clay is shown in Figure 2.1, Bernander et al (2016). For different deformation rates the relationship may vary widely. The ratio \( s_R/s \) between the residual stress \( s_R \) and the maximum stresses \( s \) may vary considerably for different clays. In the figure also the case with an ideal plastic behavior is indicated with a dotted blue line. This kind of behavior is often assumed in the classic simplified limit equilibrium method used for analysis of stability.

Not only for clays but also for silts and sandy clays, where the inter-particle friction plays a greater role than the cohesion, there may be a considerable softening, Terzaghi et al. (1996).

It should again be pointed out that the properties may vary considerably. The fat clays in eastern and central Canada generally differ considerably from those of the meager silty clays and clayey silts in and around Churchill River Valley.

The soil layers for the studied ridge at Muskrat Falls are illustrated in Figure 2.2. They are described in Leahy (2015) and Ceballos (2016) and are further discussed in Appendix I.

The upper sand layer consists mainly of dense grey fine to medium sand with low fines content. The layers under are a heterogeneous mix of clays, silts and sands from marine and estuarine deposits named the stratified drift. The lower clay layer is located below the stratified drift and is clay of low to medium plasticity. In the studied section A in Figure 1.2, the soil layers are slightly inclined - sloping downwards with about 4% from the upstream side of the ridge towards the downstream side.
Figure 2.1. Stress-strain ($\tau$/$\gamma$) and stress-deformation ($\tau$/$\delta$) relationships in a typical deformation softening soil. The full red line indicates a softening behavior while the blue dotted line indicates an assumed ideal plastic behavior. Stage I is the condition before $\tau$ reaches $\tau_{\text{max}} = s$. Stage II is the subsequent deformation softening part with a residual shear strength of $s_R$. The ratio $s_R/s$ is often denoted the sensitivity of the soil. Bernander et al. (2016).

Figure 2.2. The different layers in the studied ridge at Muskrat Falls in Section A in Figure 1.2, Dury (2017).

In Table 2.1 some values are given from tests on two of the layers in the North Ridge, the upper silty clay layers in the stratified drift and the lower marine clay, respectively, Leahy (2015). It may be noted that the remolded undrained shear strength $s_R$ (denoted $S_{ur}$ in the table) varies considerably adopting values between, 2 – 60 and 8 – 96 kPa respectively. These values correspond to the value $s_R = 17kPa$ in Figure 2.1. Further, in Table 2.1, the sensitivity $S_t$ is defined as the ratio of the intact undrained shear strength denoted $S_u$ to the remolded undrained shear stress denoted $S_{ur}$ that is $S_t = S_u / S_{ur}$ with values varying between 1 - 36 and 2 - 11 respectively. Possible stress-strain diagrams for the upper silty clay layers...
are illustrated in Figure 2.3. As no deformation properties are given in Leahy (2015) the
stiffness values are guessed.

Table 2.1 shows that there are soil layers - both in the Upper Silty Clay and the Lower
Marine Clay formation - with risk of liquefaction or massive loss of shear resistance - and in
which failure surfaces may develop, see further Appendix II and III.

**Table 2.1. Material Properties for Upper Silty Clay Layers and Lower Marine Clay Layer at
North Ridge, Leahy (2015)**

<table>
<thead>
<tr>
<th>Property</th>
<th>Upper Silty Clay</th>
<th>Lower Marine Clay</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>General range</td>
<td>Average No. of tests</td>
</tr>
<tr>
<td>Percent finer than 2 microns</td>
<td>35 - 45</td>
<td>- 19</td>
</tr>
<tr>
<td>Water content, w %</td>
<td>17 - 43</td>
<td>31 199</td>
</tr>
<tr>
<td>Liquid limit, LL %</td>
<td>17 - 43</td>
<td>30 168</td>
</tr>
<tr>
<td>Plastic limit, PL %</td>
<td>13 - 22</td>
<td>19 168</td>
</tr>
<tr>
<td>Plasticity index $PL = LL - PL$ %</td>
<td>2 - 22</td>
<td>11 168</td>
</tr>
<tr>
<td>Liquidity index, $LI = (w - PL)/(LL - PL)$</td>
<td>0,6 - 2,8</td>
<td>1,3 168</td>
</tr>
<tr>
<td>Intact undrained shear strength, $S_u$, kPa</td>
<td>35 - 135</td>
<td>- -</td>
</tr>
<tr>
<td>Remolded undrained shear strength, $S_{ur}$, kPa</td>
<td>2 - 60</td>
<td>- -</td>
</tr>
<tr>
<td>Sensitivity, in situ, $S_t = S_u/S_{ur}$</td>
<td>1 - 36</td>
<td>10 43</td>
</tr>
<tr>
<td>Large strain friction angle $\phi_{cv}$, o</td>
<td>30 - 32</td>
<td>- -</td>
</tr>
<tr>
<td>Effective cohesion, $c'$, kPa</td>
<td>0 - 10</td>
<td>- -</td>
</tr>
<tr>
<td>Salt content, g/l</td>
<td>0,8 - 1,5</td>
<td>- -</td>
</tr>
<tr>
<td>Unit weight, $\gamma$, kN/m³</td>
<td>18,4-19,7</td>
<td>- 11</td>
</tr>
<tr>
<td>Hydraulic conductivity, $k$, m/s</td>
<td>$10^{-7}$ - $10^0$</td>
<td>- -</td>
</tr>
</tbody>
</table>

Notes: The Liquid limit, $LL$, and the Plastic limit, $PL$, are measures of the water content in a
fine grained soil. They were originally defined by Albert Atterberg (1846-1916) and refined by
3. Failure risk

Existing slopes are basically stable, as long as they remain undisturbed by human activity and unaffected by significant intrinsic deterioration phenomena.

However, deterioration of shear strength and especially increasing sensitivity in the uphill portion of a long slope – e.g. because of long-time upward ground water seepage – is prone to make the entire slope acutely vulnerable to progressive failure. This is frequently a precondition in Canadian and Scandinavian landslides, many of which have been triggered by documented – yet seemingly trivial – human interference.

Hence, in long natural slopes of soft sensitive clays, the real slide hazard cannot be defined in the conventional way by the principle of plastic equilibrium. Results of analyses considering deformation and deformation-softening clearly indicate that the true degree of safety can only be correctly assessed by investigating the response – in terms of progressive failure – of clearly defined disturbance conditions, Bernander (2011).

A traditional analysis for failure in a long slope is given in Figure 3.1. When the shear stress $\tau$ in a possible failure surface is smaller than the maximum shear capacity $\sigma$ the slope is regarded as safe, see e.g. Terzaghi et al. (1996) and Axelsson & Mattson (2016). However, to be quite safe, the applied shear stress $\tau$ must not exceed the residual strength $\sigma_R$ in the triggering phase of a landslide, compare with Figure 2.1 and 2.3.
Figure 3.1. Slope analysis. For $\rho g = 18 \text{kN/m}^3$, $H = 40 \text{ m}$ and $\tan \beta = 0.04$ we obtain $\tau_o = \rho g H \cos \beta \sin \beta = 18 \cdot 40 \cdot 0.0399 = 26.7 \text{ kPa}$ which together with a rising water pressure may occasionally be higher than the maximum shear stress $s$ and for most of the time higher then the residual shear stress $s_R$, compare Figure 2.1, 2.3 and Table 2.1 with values of $s_R$ ($S_{ur}$ in Table 2.1) as low as 2 and 8 kPa.

Slides retrogressing upwards, so called flowslides, have been studied in Canada by e.g. Quinn (2009) and Locat et al. (2011, 2013, 2015). Such an investigation has also been done for the North Spur, Leahy (2015), Ceballos (2016). The results have initiated stabilization work on the slopes of the North Spur, see Figure 3.2, and Cut-Off Walls are constructed to prevent water to flow through the slope. Downwards progressive slides, so called spreads, have only been studied for a horizontal failure surface, Leahy (2015).

Figure 3.2. Stabilization work carried out to mitigate retrogressive upward slides, Leahy (2015), Ceballos (2016).
4. Progressive Failure Analysis

One critical issue is if the shear stresses that are created when the water ratio is raised, are smaller than what the soil can take. The case is illustrated in Figures 4.1 to 4.4. The load increases with $N_w$ when the water level is raised with $\Delta H = 22$ m from +17 m to +39 m:

$$N_w = 0.5 \cdot \gamma_w \cdot \Delta H^2 = 0.5 \cdot 10 \cdot 22^2 \text{ kN/m} = 2420 \text{ kN/m}$$

When the load $N_w$ is gradually increased, additional shear stresses will develop along possible shear slip surfaces. The stresses will initially be highest close to the cut-off wall and will get lower further down the slope. The stresses $\tau$ can be calculated with the progressive failure analysis developed by Stig Bernander (2000, 2011, and 2016). The calculations can be done with different assumptions for values of the material properties. This will be further commented on in section 5.

In Appendices I-III, arguments are given for the need of an up-to-date progressive failure analysis of the proposed dam at Muskrat Falls. There also the influence of water content and the drainage is discussed.

The stabilizing works on the shores that are in progress may help counteract retrogressive **upwards** slides but they still leave the **central core of the ridge** susceptible to a slide failure. The highly varying properties in the slope may easily cause a local failure in a weak part. This will initiate local deformations that will cause also surrounding stronger parts to deform and soften and a global failure may then occur.

Analyses have recently been carried out by Robin Dury (2017) and Stig Bernander, see Appendix IV.

For the highest material properties in Table 2.1 ($s_u = 135$ kPa and $s_u/s_R < 4$) calculations indicate that the ridge may be stable. However, for material properties in the lower range in Table 2.1 and Figure 2.3, the critical load $N_{crit}$ will be lower than the applied load $N_w$ and a failure will occur.

Using material properties suggested by Leahy et al. (2015, 2017), see Table 2.1, Dury obtained that the critical load-carrying capacity $N_{crit}$ is less than 1000 kN/m whereas a rise of the water level with 22 m will, as indicated above, give an increased load of $N_w = 2420 \text{ kN/m}$. This is more than twice of what the ridge may stand with the assumed properties.

Two analyses made with the original spreadsheet of Stig Bernander are enclosed as Appendix IV and they show similar low safety factors as the calculations by Dury (2017).

For a case with an in situ shear stress $\tau_o = 21.1$ kPa and with material properties $s = 60$ kPa, $s_R = 12$ kPa, $s/s_R = 60/12 = 5$ he obtains $N_{cr} = 866 \text{ kN/m}$ and a safety factor $F = N_{cr} / N_w = 866 / 2420 = 0.357 < 1$.

For another case with a higher in situ shear stress $\tau_o = 41.1$ kPa and with slightly better material properties $s = 70$ kPa, $s_R = 14$ kPa, $s/s_R = 70/14 = 5$ he obtains $N_{cr} = 521 \text{ kN/m}$ and a safety factor $F = N_{cr} / N_w = 521 / 2420 = 0.215 < 1$.

More material tests are necessary to establish the real deformation properties of the soil in the ridge and stabilization work may be needed to eliminate the risk for a landslide. These questions are further discussed in Appendix III, Section 5.8 and a procedure is proposed on
how to check the material properties and how to compact the soil to make it less prone to liquefaction.

**Figure 4.1.** Schematic drawing of a section through the dam. When the water level is raised with 22 m, from +17 m to +39 m, a force $N_w$ starts to act on the cut-off wall. The question is if the shear stresses $\tau - \tau_o$ on a possible slip surface are big enough to hold the force $N_w$ in equilibrium. Here $\tau_o$ is the in situ stress before the water level starts to rise.

The lower left figure illustrates the material properties of the soil with a shear stress/strain ($\tau/\gamma$) diagram with a maximum shear stress $s$ and a residual softened shear stress $s_R$. The dotted line indicates the classic ideal plastic assumption of no reduction of the shear strength. When the force $N_w$ starts to act on the cut-off wall, the soil behind the wall starts to deform ($\gamma$) and shear stresses ($\tau$) will be growing according to the stress-strain diagram. When the shear stresses reach the maximum value $s$ they start to soften and are finally reduced to the residual value $s_R$ close to the wall.

The maximum value the wall may carry is $N_{crit} = \int (\tau - \tau_o) \, dx$ and this is illustrated in the lower right figure. In order for the slope to remain stable, $N_{crit}$ must be at least as large as $N_w$. To calculate $N_{crit}$ for varying material properties is the main objective of a stability analysis. Applying the unrealistic assumption of ideal plastic behaviour (green dotted line) there is in many cases no apparent stability problem.

Figure 4.3. Section of the North Spur and location of the assumed failure planes, one horizontal and one inclined in the lower of the two silty clay layers and one curved in the lower clay layer. Dury (2017).
5. Phases in a progressive failure

A method for progressive failure analysis has been developed by Stig Bernander et al. (1978, 1981, 2000, 2008, 2011 and 2016). When an additional load $\Delta N$ is entered in a slope it is kept in equilibrium by additional shear stresses $\Delta \tau$, see Figure 5.1. The shear stresses have their highest values close to the force $\Delta N$ and abate further downslope. After the shear stresses $\tau$ have reached the maximum value $s$, they are reduced, see Figure 2.1, and shear stresses further downslope must be engaged to equilibrate $\Delta N$. The mechanism can be studied with a finite difference method where local downhill deformations $\Delta \delta_N$ caused by
normal forces $\Delta N$ are maintained compatible with deviatory shear deformations $\Delta \delta_r$ above the potential failure surface, see Figure 5.1.

![Figure 5.1](image.png)

**Figure 5.1.** Principle of finite difference method where deformations $\Delta \delta_N$ due to normal forces $\Delta N$ are kept compatible with deformations $\Delta \delta_r$ caused by shear stresses $\Delta \tau$. The in situ pressure $E_o$ may vary along the slope.

The failure process can be divided into five phases and six moments a-e, see a simplified idealised example in Figure 5.2, Bernander et al. (2011, 2016) and Dury (2017):

**Moment a, Phase 1:** In-situ conditions. No additional load $q$ or $N_q$ is present and the in situ stress is in this example $\tau_0 = 20.8$ kPa. The slope has an inclination of 6.5:100 (corresponding to an angle $\beta = 3.287^\circ$) to the left but turns horizontal further to the right,

**Moment b, Phase 2:** A load $q$ is applied giving $\tau = s = c = 30$ kPa. The shear stresses can be integrated to the force $N_q = 189$ kN/m for an influence length $L_b = 85.5$ m.

**Moment c, End of Phase 2 and start of Phase 3:** The shear stress has decreased to $\tau = \tau_0 = 20.8$ kPa at the point of application of $N_q$ (and $q$). The shear stresses can be integrated to $N_{q, crit} = 231$ kN/m for $L_{critical} = 94.3$ m. This is the maximum additional load the slope can take without causing a local failure. The safety factor for a local failure will be $F_I = N_{crit} / N_q$

**Moment d, Phase 3:** An unstable dynamic phase starts. The shear stress $\tau$ has decreased to the residual value $s_R = c_R = 15$ kPa and the load that can be taken is reduced to $N = 215$ kN/m for an influence length of $L_d = 99.7$ mm.

**Moment e, Phase 3:** The negative shear stresses balance the positive so that $N = 0$ at the point of application. The maximum load $N_{q, crit} = 231$ kN/m has travelled downslope and the influence length is $L_e = 139.6$ mm. Moments d and e are hard to observe as they are part of an unstable ongoing dynamic phase.

**Moment f, End of Phase 3, Phase 4 (&5):** The in situ stresses $\tau_0$ decrease from $L = 150$ m where the slope starts to turn horizontal. The pressure $N$ is now caused by the weight of the sliding mass $N = LH\rho g \sin \beta$. The residual shear stress $s_R = c_R$ may be reduced due to dynamic action. The pressure is “permanently” or “temporarily” balanced by passive resistance if $(E_o + N)_{max} < E_o, Rankine$, and in this case only a local displacement occurs and the
masses stop moving. The failure plane develops far into the unsloping ground before equilibrium is reached. However, if \((E_0 + N)_{\text{max}} > E_{\text{p, Rankine}}\) a collapse will occur. This is called Phase 5 with large masses being moved. Please observe that the scale in the diagram for this moment is different from the earlier ones in moments a-e.

**Figure 5.2.** Five phases 1-5 and six momenta a-f in a Progressive Failure Analysis of an idealised slope with an inclined surface, Bernander et al. (2011, 2016), Dury (2017).

The safety factor for a global failure will be \(F_{\text{II}} = E_{\text{p, Rankine}} / (E_0 + N)\). The total earth pressure \(E = E_0 + N\) as function of the deformation \(\delta\) at \(L = 0\) for the different moments are given in Figure 5.3. In the case of North Spur the final failure occurs when Phase 3 is initiated. There is no possibility for the remaining part of the slope to withstand the pressure as the ridge ends in Churchill River downstream the Muskrat Fall. So \(E_{\text{p, Rankine}}\) will decrease more and more along the slope and \(F_{\text{II}}\) will become rather small and irrelevant, so it will be only the safety for a local failure that is important,
Figure 5.3. The total earth pressure $E = E_0 + N$ as function of the deformation $\delta$ at the point of application of $q$ and $N$ during the moments a – e. Bernander et al. (2016).

The principal features of progressive failure analysis is also treated in e.g. Quinn (2009), Gylland (2015), Locat et al. (2011, 2013, 2015) and in the workshop proceedings L’Heureux et al. (2013) and Tharkur et al. (2017).

6. Conclusions

Progressive failure analyses have been performed according to a finite difference method developed by Stig Bernander (1978, 2000, 2008, 2011, and 2016). The development of a simplified spreadsheet by Robin Dury (2017) has allowed getting numerical results for several cases of study and assumptions.

For assumed material properties and geometries of failure, the critical load-carrying capacity for the North Ridge dam at Muskrat Falls is below 1000 kN/m whereas a rise of the water level with 22 m will give an increased load of $N_q = 0.5 \gamma_w H_d^2 = 0.5 \cdot 10 \cdot 22^2 = 2420$ kN/m. This is more than twice of what the ridge may stand with assumed properties.

More material tests are necessary to establish the real deformation properties of the soil in the ridge and stabilization work may be needed to eliminate the risk for a landslide. One method is to compact the soil to make it less prone to liquefaction.
7. References


**Appended Reports**

I. **Lower Churchill River, Riverbank Stability Report, 2015-10-14**  
Prepared for Grand Riverkeeper Labrador, Inc, by Stig Bernander

II. **Further comments, 2016-01-07**  
Further Comments on the Updated Nalcor Report of 21July-2014 by Stig Bernander

III. **Comments on “Progressive failure study”, 2016-09-15**  

IV. **Spreadsheet Analysis, 2017-06-01**  
Stability of the Hydropower Dam at Muskrat Falls studied by Stig Bernander with a finite difference method according to Bernander (2000, 2008, 2011)
On Specific questions regarding the formation of the Churchill River Valley and Comments on stability issues related to the North Spur.

Executive summary.

The intent of this report is to explain the extraordinary features of the Churchill River Valley, and to comment on North Spur stability regarding future 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 – like in the rest of the valley – is an on-going geological process.

This report explains the extraordinary features of the Churchill River Valley and includes comments on the North Spur stability in respect of the future impoundment.

The width of the Churchill River bed, upstream and downstream of Muskrat Falls, differs in an exceptional way from normal riverbed formations. Along a stretch of at least some 30 km, the Churchill River Valley, normally has a width of about 1 km. 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 immediately downstream of Muskrat Falls, of about 70 metres is due to the presence of a ‘whirlpool’ where the water current is so strong that sedimentation of the eroded marine sediments originating from the upper Churchill Valley cannot take place.

The contention of this document does not imply 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 no matter what be shown to be non-existent.

Modern research requires that the stability analysis of long slopes with sensitive clay must carefully take the risk of ‘brittle slope failure’ into consideration. As the impoundment represents a gigantic external force (locally on the cut-off wall), a careful study related to progressive failure is an unavoidably necessary measure.

Friction as such is normally a dependable stability agent but, in the current case, the voids of the loose mixed soil are filled with soft and sensitive clay material, the strength of which is not compatible with currently (or in the 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, with loose granular structure of the coarse-grained portions of the soil, shear deformation will tend to decrease the pore volume containing clay or water. This brings about an inherent propensity to soil liquefaction. The proneness to 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 the analysis is based on the Plastic Equilibrium mode.

Dependable stability analyses must therefore consider the potential risk of progressive failure formation due to the intrinsic tendency to liquefaction, particularly regarding the Upper Clay layers. Such analysis must, of course, be based on rapid un-drained direct shear tests on virtually un-disturbed clay samples, as progressive failures tend to develop at high rates of deformation. The diameter of test samples should not be less than 100 mm.

These direct shear tests should not be deformation-controlled – i.e. being carried out in such a way that the development of failure surfaces is not restrained.

The very fact that the Churchill River valley has developed in the way it actually still does substantiates the validity of the geotechnical conditions mentioned above, and which are dealt with in more detail further on in this report. The soil masses behind the riverside slope have actually exerted their vertical pressures during millennia, and yet even moderate changes of lateral loading conditions – such as e.g. hydraulic pressure change, seismic activity, gradually failing lateral support, creep deformations and the due loss of shear resistance (because of proneness to liquefaction), can release enormous landslides of the kind at Edward Island.

The installation of a watertight membrane, the cut-off wall, is of course advantageous for promoting effective pressure increase on soil layers that are truly abiding by the normal laws of frictional resistance in granular soils. However, the behaviour of a mixed soil with lean clay content may, as will be demonstrated in the following, be totally different. The reduced porosity generated by additional shear deformation may simply result in liquefaction, whereby the loss of shear resistance, and due shear deformation, will in turn generate a tendency to liquefaction further along a potential failure surface, hence resulting in a possible global progressive failure condition.
In fact, considering the type of sensitive behaviour of the lean Upper Clay No.2 layer, the local concentration of hydraulic pressure at the cut-off wall may even create a highly disadvantageous condition. Local (concentrated) loading is namely 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.

In order to illustrate the specific stability conditions along the riverside slopes of the Churchill River Valley, a stability analysis of a typical riverside situation has been carried out in Appendix A, (Cf Figure 4.4.) The result of the analysis is commented in Section 4.23.

As the clay content in 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 demonstrating the decisive effect of varying ground water conditions in the soil mass behind a typical riverside slope in the Churchill River valley.

**Case a.** Ground water level at ground surface, roughly renders a safety factor $= 1.09$
**Case b.** Ground water level at 5 m below ground surface, renders a safety factor $= 1.43$

This analysis also indicates why the steep riverside slopes may, at least transiently, remain stable.

The contention of this document does not imply 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, all stability analyses related to the impoundment must ‘no matter what’ prove that the possibility of such a failure is definitely eliminated.
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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](image1.png)

**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.

Denotations: \(W_p_1\) = Riverbed width (≈ Wet perimeter),

\(D\) = Mean water depth (WL = Water level).

![Figure 1.2](image2.png)

**Figure 1.2** Riverbed formation of the type occurring in the Churchill River valley with a remarkably wide but shallow riverbed. Denotations as above.
The width of the Churchill River bed, 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 1 km. 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 Canada Centre for Mapping, Department of EM&R)](image)

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 immediately downstream of Muskrat Falls of about 70 metres is due to the presence of a ‘whirlpool’ where 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 water current ‘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 carried away by the streaming water.

The intent of the following article is to explain the extraordinary features of the Churchill River Valley, and to comment on North Spur stability regarding future impoundment.
2: On Extreme Sensitivity of Lean Clays

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

2.1 The type of biotite —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 \( w_L \) and the Plasticity Limit \( w_P \). The water contents 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:

\[
I_L = \frac{(w - w_P)}{(w_L - w_P)} \tag{Equation 1}
\]

where the parameter \( w_L \) represents Liquidity Limit – i.e. the water content at which the clayey soil material behaves as a liquid on being heavily remoulded.

\( w_P \) is the limit of plasticity defining the water content at which the clay ceases to be plastic.

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

\[
I_P = w_L - w_P \quad \text{(Figures 3.3 and 3.4)} \tag{Equation 2}
\]

Thus, if the water content \( w \) of a clay layer exceeds the liquid limit \( w_L \), 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 -

$$\frac{\Delta V}{V} = n$$

......... Equation 3a

where V is the total soil volume.

Another related parameter is porosity (e)

$$e = \frac{\Delta V}{V_s}$$

......... 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

$$n = \frac{e}{1+e} \text{ or } e = n/(1-n)$$

......... Equation 4a, 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 (like sand) may momentarily liquefy.

This constitutes the reputable phenomenon named 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) may decrease 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 resistance may in general only be partial.

However, 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 ($V_{\text{Clay}}$) in relation to the concurrent void volume of the coarser material, i.e. the ratio $V_{\text{Clay}}/\Delta V$.

If, for instance, the clay content $V_{\text{Clay}}$ is significantly greater than the void volume $\Delta V$ – i.e. $V_{\text{Clay}} >> \Delta V$ – then obviously the soil matrix will typically behave as clay, the granular soil particles being immersed 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 or smaller than the concurrent void volume of the granular material, i.e. $V_{\text{Clay}} \leq \Delta V$, the properties of the soil matrix become extremely complex and highly dependent of the consolidation process and possible ongoing change of the relationship between the void volume ($\Delta V$) and the coexisting clay volume ($V_{\text{Clay}}$).

Hence, in the early stages of sedimentation, such a soil will feature high porosity $n = \Delta V/V_S$ and the clay filling the voids will remain extremely soft. However, as the normal pressures increase due to accumulating sediments, the stiffer structure of the granular material will gradually tend 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 will end up in a condition, where 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 will not be related to the total effective sedimentary load, as was the case when $V_{\text{Clay}} >> \Delta V$.

As a result, the mixed soil will finally consist 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 relevance to the actual effective vertical load.

Hence, 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 will decrease, generating excess pore pressure change resulting in reduced effective stress conditions 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 – i.e. even to the extent that most of its shear strength is lost – 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 such as 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.

Potential tendency to liquefaction of this nature can make the results of soil investigations of standard type extremely unreliable and hence leads to debatable results of slope stability analyses.

Soil investigations in lean clay soil material require un-drained direct shear laboratory testing rendering the minimum residual resistance for the relevant rates of load application.

The large strain residual shear resistance ($S_R$) is namely a crucial parameter when predicting both the triggering additional disturbance load 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.
3: Relevance of the phenomena described in Section 2 for clays in 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 parts of the Southeast Labrador such as, 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 was later going to shape 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 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 \( k \approx 1\times10^{-7} \) m/sec.

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 different soil layers, being marine deposits, are likely to be similar over wide areas. For instance, the Upper Clay Layer No 1, near the present water level downstream of Muskrat Falls can be widely observed.
Yet, 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 on Permeability basis

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].

**Table 1 Permeability of soils in the North Spur**

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

<table>
<thead>
<tr>
<th>Layer</th>
<th>Type</th>
<th>Permeability $k$</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sand</td>
<td>$\approx 1 \times 10^{-4}$ m/sec, Upper sand</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Silty clay-1</td>
<td>$\approx 1 \times 10^{-7}$ m/sec, Upper clay 1</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Silty sand</td>
<td>$\approx 0.8 \times 10^{-5}$ m/sec, Upper intermediate Silty Sand Drift</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Silty clay-2</td>
<td>$\approx 1 \times 10^{-7}$ m/sec, Upper clay 2</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Silty sand</td>
<td>$\approx 0.8 \times 10^{-5}$ m/sec, Lower intermediate Silty Sand Drift</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Clay-2</td>
<td>$\approx 1 \times 10^{-8}$ m/sec, Lower clay to great depth</td>
<td></td>
</tr>
</tbody>
</table>

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 early made 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 named ‘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 (1925). “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 No. 1 & 2, the permeability $(k)$ is stated to be $1x10^{-7}$ m/sec. Hence, according to 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 author of this article.
Figure 3.2 Diagram showing analysis according to Hazen’s formula regarding 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]

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 No. 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, the mean permeability ($10^{-7}$ m/sec) being 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, being 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 clays 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 the Liquidity and Plastic 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). Yet, this information was also defined in very general terms such as the table below:

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

<table>
<thead>
<tr>
<th>Property</th>
<th>Range</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water content</td>
<td>21 – 41 %</td>
<td></td>
</tr>
<tr>
<td>Liquid Limit</td>
<td>19 – 39 %</td>
<td></td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>13 – 23 %</td>
<td></td>
</tr>
<tr>
<td>Un-drained shear strength</td>
<td>40 – 120 kN/m²</td>
<td></td>
</tr>
<tr>
<td>Initial void ratio $e_0$</td>
<td>0.93 – 1.06</td>
<td></td>
</tr>
<tr>
<td>Index of Liquidity</td>
<td>0.7 – 3.0</td>
<td></td>
</tr>
<tr>
<td>Index of Plasticity (IP) %</td>
<td>8 – 25</td>
<td></td>
</tr>
<tr>
<td>Sensitivity</td>
<td>2 – 28</td>
<td></td>
</tr>
<tr>
<td>Effective cohesion c’</td>
<td>0 – 10 kN/ m³</td>
<td></td>
</tr>
<tr>
<td>Unit weight at natural water content, $\gamma_m$</td>
<td>18.4 – 19.7 kN/ m³</td>
<td></td>
</tr>
</tbody>
</table>

As the values in Table 2 just 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 as such 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 diagnosed in terms 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 the existence of so called ‘Boarder-line materials.)
Figure 3.3 Relations 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)

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 that 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), Reference [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 ranges of soil properties – applies to extremely lean sandy (silty) clays in these geographical regions, featuring very low Liquid Limits ($w_L$) and high Limits of Plasticity ($w_P$) – thus representing values of the Plasticity Index as low as $I_P \approx 8\%$. ($I_P = w_L - w_P$).
Figure 3.4 Relations 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 1 and 2 shown in Figure 3.1. (Plasticity chart according to A. Casagrande 1932, “Research on the Atterberg limits of soils”, Public Roads 13, pp 121-136. [6] (Note. The colouring of specific areas in the figure has been made by the author of this report).

**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.
3.5.1 Clay exposure just North of the recent (2014) slide that is shown on Figure 4.3.

Figure 3.5 shows a large exposure of the type of lean clay discussed in Section 2. Merely the effect of 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 proneness 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 proneness 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 is not compatible with the vertical pressures that existed before the wide riverbed valley was formed.

In other words, the lean clay in the exposure matches the description in Section 2.3, and which is briefly defined in Paragraph 2.4 Conclusion. This means that the clay content of the mixed soil is highly under-consolidated, thus indicating propensity to soil liquefaction.

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 riverside 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$^2$ as the shear resistance to avoid failure is about $c \approx 0.20 \cdot g \cdot H = 0.2 \cdot 19 \cdot 1.5 \approx 6$ kN/m$^2$.

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

For instance, a clay with a normal illitic clay content, actually 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 WL. (Confer expression in Note * 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.

![Image](image.jpg)

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

Note * ($c \approx 0.45 \cdot w_i \cdot g \cdot H > 0.45 \cdot 0.26 \cdot 19.7 \cdot 30 \approx 70$ kN/m$^2$) Formula according to Hansbo S. 1957, where $w_i$ is the Liquidity Limit. (Cf Reference [7]).
4: Implications of the properties of markedly lean clays for the Churchill River Valley

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 from the final phases of the Great Ice Age.

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

a) In coarser marine sediment layers of fluvial or disturbed water origin (sands and silts in rivers and beaches), the grain size distribution is locally very even, making such soils highly sensitive to erosion.

b) The properties of the Upper Clays 1 and 2 (in the North Spur) depicted in Figure 3.1 are dealt with in Sections 3.1 to 3.4 above. The results of these studies indicate that the Upper Clay layers No. 1 and 2 consist of mixed sandy/silty soils with very sparse contents of clay substance.

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 ‘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 as the depth of the wide and shallow riverbed generally seems to be largely restricted by this layer, whereas the riverside slopes rise steeply up to the ground level of the original marine sedimentary structure. (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. Also this process results in earth slips and slides of the character 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 River Valley by massive landslides

As evidenced by the enormous landslide at Edward Island upstream of Muskrat falls, and 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 be so extensive that they, if the area had been more populated, 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 soon.

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 lean sensitive clay layers that can be seen in Figures 3.5 and 3.6. (This clay layer is also exemplified in Figure 4.4). In other words, the gradually failing lateral support at the riverside slope generates a massive shear stress build-up in the clay layer carrying the overlying soil masses further away from the riverside.

Figures 4.2a and 4.2b The Edward Island landslide (200?). The two photos have been taken in different angles and aerial positions. The enormous extent of the landslide can be understood 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 shear stress increase causes a creep movement in the slope direction that in turn will generate 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, very likely causing massive increase of hydraulic pressures in 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 Churchill River Valley

Figure 4.4 shows 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. The section is assumed to have marine sediments similar to those in the North Spur.](image)

The object of the analyses 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 $\phi^\circ$. Two cases are analysed –

**Case a.** Ground water level (GWL) at ground surface.
**Case b.** Ground water level (GWL) at 5 m below ground surface.

4.2.4 Conclusions from the analysis in Appendix A

On the basis of mainly frictional resistance, the safety factor in **Case a** is estimated to be:

$F_s = \frac{6502}{5985} \approx 1.09$, and in **Case B**, $F_s = \frac{7006}{4885} = 1.43$

The mean shear stress along failure surface BC is then: (plastic approach)

$\tau_{BC} \approx \frac{H_z}{L_{BC}} = \frac{30}{50} = 120 \text{ kN/m}^2$

According to the NALCOR Report, Ref. [2], the Liquid Limits ($W_L$) for the mixed soils range between 20 and 39%.
However even if, hypothetically, the clay content in the lean bottom clay layer had significantly exceeded the current void ratio, having for instance a Liquid limit \( W_L = 40 \% \), the mean shear strength along failure surface BC – estimated with Hansbo’s formula – applying to illitic clays – would still only be:

\[
    c \approx 0.45 \cdot \sigma' \cdot W_L \quad \text{where } \sigma' \text{ denotes the pre-consolidation pressure.} \quad \text{(Reference [7])}
\]

- i.e. for \( W_L = 40 \% \), \( c = 0.45 \cdot 9.9 \cdot 30 \cdot (40/100) = 53.5 \text{ kN/m}^2 \), and for \( W_L = 19 \% \), \( c = 0.45 \cdot 9.9 \cdot 30 \cdot (19/100) \approx 25.4 \text{ kN/m}^2 \)

**Point A** - Considering that, in this condition, the required shear resistance to avoid failure is at least \( \tau_{BC} = 120 \text{ kN/m}^2 \) – i.e. more than 2.2 times the available strength of a normally consolidated illitic clay with \( c = 53.5 \text{ kN/m}^2 \) – 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.

The effect of even minor proneness to liquefaction is therefore 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 ground water conditions. For instance, the five metres change of the ground water level (GWL) reduces the safety factor \( F_s \) from 1.43 in Case b) to 1.09 in Case a).

### 4.2.5 The landslide at Rollsbo about 20 km North of Gothenburg, Sweden. (1967)

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, having 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, at least transiently, be stable 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.

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

Friction as such is normally a dependable stability agent but, in the current case, the voids of the loose mixed soil are filled with soft and sensitive clay material, the strength of which is not compatible with currently (or in the past) active vertical effective pressures. Yet, the void clay may in some measure have contributed to the looseness of the granular structure of the mixed soil.

However, 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, possibly resulting in liquefaction, or at least in a drastic loss of shear strength, as explained in Section 2 above. (Confer the Conclusion in Paragraph 2.4.)

The crucial issue in this context is the fact that liquefaction in this case relates to the under-consolidated pore clay as such – i.e. not only to water.

Extreme excess pore water pressures in ‘quick-sand’ normally tend to abate quickly but in clayey soils – depending on layer thickness and low permeability – pore pressure dissipation may be a long lasting process, possibly extending over even years or decades.

The extraordinary development of the Churchill River Valley, as described in Section 1, is due to the lean character of, in particular, the riverbed clay layer, dealt with in Section 3. This clay is highly sensitive and prone to liquefy on being exerted to additional shear deformation and the properties of which – according to the site observations – also conform to those of the Upper Clay 2 in the North Spur.

Importantly, it may be observed that the sensitivity of the clays in the Churchill River Valley is of a specific nature that should not be confused with sensitivity related to highly over-consolidated clays, such as for instance those of over-consolidated clays common in the Québec area.

The correct geotechnical approaches to soil investigation, to type of stability analysis, and to stability criteria, are not identical.

Also, the sensitivity of normally consolidated (and slightly over-consolidated) Scandinavian clays is of a different nature than that of the lean clays in the Churchill River Valley.

Hence, agents triggering landslides, slide progression and the configuration of finished slides in the Churchill River Valley may not be compatible at all with landslides occurring elsewhere.
Slides in the Churchill River Valley are mainly of two kinds:

1) **Smaller**, essentially superficial *slides* along the riverside due to on-going undercutting of the steep riverside slopes by streaming water. These slides are in places recurring to the extent that vegetation does not even 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 riverside. Slides of this kind may be triggered by various agents but in places that are little affected by human activities, the most likely reasons for the large landslides are effects of *seismic activity*, *heavy drawn out precipitation*, *pressure changes in ground water aquifers*.

The *mixed lean clayey* soils possess an inherent *proneness* to *liquefaction*, when exerted to shear deformations – i.e. in accordance with the Paragraphs 2.4 and 4.2.2 above.

The slides, depicted in Figures 4.2 and 4.3, represent good examples of the latter kind of river valley formation such as the one shown in Figure 1.2.

The major problem of the lean clay condition is its **impact** on sensitivity. If the pore volume of the coarse material is *above* the *critical void ratio* ($n_{\text{crit}}$) – i.e. in the state when void volume decreases under shear deformation – the effects of significant shear stress increase (as well as that of vibratory impact loading) will be likely to generate phenomena very similar to hydraulic *‘liquefaction’* in sands. The residual shear resistance may then be reduced to a minor fraction of the initial shear resistance.

It is vital to observe that these landslides **cannot** be **predicted** by means of the conventional so called Plastic Equilibrium Mode that has been a dominant approach to slope stability analysis during most of the 20th century. Modern research has shown that this analytical model *does not apply* to long slopes in sensitive clays and that simply because the 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 related to 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 – like in the rest of the valley – is an on-going process. The problems in this context in the North Spur are primarily connected with Upper Clay layers *(1)* and *(2)*. (See Table 1 in Paragraph 3.3 and Reference [3]).
These issues are of course well known prerequisite conditions that must have been contemplated by NALCOR and SNC-LAVALIN.

![Diagram of Section K–K, NALCOR [3].](image)

The force denoted 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 drawn potential slip surface, 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.)

The way these potential landslide threats seem to have been mainly considered in the design of the North Spur Dam containment are:

1) Pre-consolidation of critical clay layers by lowering the ground water pressures in relevant aquifers.
2) Establishing a water tight membrane (the cut-off wall, COW) in the up-stream part of the North Spur ridge.
3) Establishing erosion protection banks in various places.

5.1 Pre-consolidation of clay layers

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

The problem with the markedly lean clays, with a loose coarse-grained structure, is that although frictional resistance corresponding to essentially symmetrical vertical effective pressures can be mobilized over long time (centuries, millennia), the result of a momentary increase of shear stress and the due lateral shear deformation may generate liquefaction in the mixed soil, whereby all, or most, of the shear resistance may be lost. This is a condition with inherent propensity to progressive failure development.

Potential tendency to liquefaction of this nature makes the results of standard type soil investigations and laboratory testing unreliable. Slope stability analysis, and related safety factors, based on such results may therefore be totally inaccurate.

The very fact that the Churchill River valley has developed in the way it actually does substantiates the validity of the geotechnical conditions dealt with above. The soil masses
behind the riverside slope have exerted their vertical pressures during millennia, and yet even moderate changes of the lateral loading conditions such as water pressure change, seismic activity, gradually failing lateral support, creep deformations and the due loss of shear resistance (because of the proneness to liquefaction), can release enormous landslides of the kind at Edward Island.

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

The corresponding force on the soil mass above level +17, acting on the failure surface indicated in Figure 5.1, (Section 1 000), is 2,420 kN/m, i.e. a horizontal additional force capable of generating significant lateral shear deformation and due loss of shear resistance.

5.2 Establishment of a water-tight membrane (COW = Cut-off wall)

The installation of a watertight membrane (by injecting bentonite) is of course advantageous for promoting effective pressure increase on soil layers that area truly abiding 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 generated by shear deformation may simply result in liquefaction, whereby the loss of shear resistance may in turn generate additional liquefaction further away along the potential failure surface, thus resulting in a global progressive failure condition.

In fact, considering the type of sensitive behaviour of the lean clay (in Upper Clay 2), the local concentration of additional hydraulic pressure at the COW is even likely to create a highly disadvantageous condition, local (concentrated) loading namely being 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 slides in the Churchill River valley are actually precisely due to the presence of the specific type of lean clay formed under marine sedimentary conditions during the Ice Age.

Conclusion: The potential effects of the high local stress build-up along the water tight membrane (COW) should be thoroughly investigated on the basis of the Progressive Failure mode. (e.g. the failure surface indicated in Figure 5.1. should be studied.)

Also, it must be recognized that the Plastic Equilibrium failure mode has no relevance in this context.
5.3 Erosion protection banks

Generally erosion protection is a good measure regarding stabilisation, especially from erosion points of view.

Yet, in respect of the risks related to progressive landslide development, stabilisation of the toes of the slopes is of limited avail.

6: Concluding remarks

The contention of this document does not imply 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 no matter what be eliminated.

Modern research requires that the stability analysis of long slopes with sensitive clay must carefully take the risk of ‘brittle slope failure’ into consideration. As the impoundment represents a gigantic external force (locally on the COW), a careful study related to progressive failure is an unavoidably necessary measure.

Friction as such is normally a dependable stability agent but, in the current case, the voids of the loose mixed soil are filled with soft and sensitive clay material, the strength of which is not compatible with currently (or in the 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, a loose granular structure of the coarse-grained portion of the soil will render a decrease of the pore volume, when exerted to shear deformation. This brings about an inherent propensity to soil liquefaction.

The proneness to 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. Such analysis must, of course, be based on un-drained direct shear tests on virtually un-disturbed 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 restrained, 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 for instance in Reference [9], (11th ICSMFE, San Francisco, 1985).

One way of testing for residual shear resistance could be to retrieve large, virtually undisturbed, samples from apt 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 on-going progressive landslide tends to be fast.

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

Furthermore, the analyses of potentially extensive slides must not be based on the Plastic Equilibrium Concept, 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 no longer applicable, when potential landslides extend more than 70 to 100 metres – the distance largely depending on the depth of the failure surface below the ground level.

Furthermore, the possibility of progressive failure developing in Layer 6 – i.e. ‘the Lower Clay’ extending to 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 $\phi^0$.

The void ratio for loose mixed-grained sands of current type is assumed to be $(n) = 40\%$.

Hence, the porosity $\epsilon = n/(n-1) = 0.40/0.60 = 0.6667 = 66.67\%$

Rock density $\gamma_R = 26.5 \text{ kN/m}^3$, Water density $\gamma_{H2O} = 10 \text{ kN/m}^3$

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

Density (water saturated) $\gamma_w = (w+1)/(w+\gamma_{H2O}/\gamma_R) = (0.2516 +1)/(0.2516 +10/26.5) = 1.2516/(0.2516 +0.3774) = 19.90 \text{ kN/m}^3$

Density (under water $\gamma'_w = \gamma_w - 10 = 19.9 - 10 = 9.90 \text{ kN/m}^3$,

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

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

Internal friction value $\phi = 30^\circ$, Length $B \rightarrow C = 50 \text{ m}$

Effective cohesion $c'$ in the clay layer = $6 \text{ kN/m}^2$. (Cf Section 3.52)

(According to Ref. [2], $c'$ is in the range of $0 \rightarrow 10 \text{ kN/m}^3$).

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

Case a

Ground water level (GWL) at ground surface i.e. $z = 0$, $z_B = 30 \text{ m}$,

Horizontal earth pressure per unit area at a depth of $z = 30 \text{ m}$

$$p_z = \gamma' \cdot z \cdot \tan^2(45 - \phi/2) + \gamma_{H2O} \cdot z = 9.9 \cdot z \cdot 0.33333 + 10 \cdot z$$

$$H_{z=30} \approx 0.33333 \cdot \gamma' \cdot z^2 / 2 + \gamma_{H2O} \cdot z^2 / 2 = 0.33333 \cdot 9.9 \cdot 30^2 / 2 + 1 \cdot 10 \cdot 30^2 / 2$$

$$= 1485 + 4500 = 5985 \text{ kN/m}$$
Mean shear stress along surface BC (plastic approach):

\[ \tau_{BC} \approx Hz = 30 \text{ m} / L_{BC} = 5985/50 \approx 120 \text{ kN/m}^2 \]

Shear resistance along surface B–C

\[ R_{z=30} = o \int_{z_{w}}^{50} c' \cdot dx + \gamma_{w}\cdot(z_{30}-z_{w}) \cdot \tan \phi \]

\[ = 10 \cdot 50 + (19.9-10) \cdot [(30-20 +30-30/2)] \cdot \tan 30\degree = 500 + 9.9 \cdot [600 + 450] \cdot \tan 30\degree = 500 + 10395 \cdot 0.5774 \cdot 6502 \text{ kN/m, roughly rendering a safety factor of only:} \]

\[ F_s(a) = 6502/5985 \approx 1.09 \]

Case b.

Ground water level (GWL) at 5 m below ground surface, i.e. \( z_{w} = 5 \), \( z_{b} = 30 \text{ m} \),

Horizontal earth pressure (\( p \text{ kN/m}^2 \))

\[ p_{z} = \gamma_{d} \cdot z_{w} \cdot \tan^2(45 - \phi/2) + \gamma_{w}\cdot(z_{30}-z_{w}) \cdot \tan^2(45 - \phi/2) + \gamma_{w2} \cdot (z_{30}-z_{w}) \cdot \tan^2(45 - \phi/2) = 0.33333 \]

\[ H_{z=30} = 0.33333 \cdot \gamma_{d} \cdot (z_{w}^2/2 + z_{w}(z_{30}-z_{w})) + 0.33333 \cdot \gamma_{w}\cdot(z_{30}-z_{w}) \cdot 2 + 0.33333 \cdot \gamma_{w2} \cdot (z_{30}-z_{w}) \cdot 2/2 \]

\[ = 0.33333 \cdot 15.9 \cdot (5.5/2 + 5.25) + 9.9 \cdot (30-5) \cdot 2/2 + 1 \cdot 10 \cdot (30-5)^2/2 \]

\[ = 66.3 + 662.4 + 1031.3 + 3125.0 = 4885 \text{ kN/m} \]

The total shear resistance based on friction and effective cohesion (\( c' = 10 \text{ kN/m}^2 \)) with GWL at \( z = 5 \text{ m} \) is:

\[ R_{z=30} = o \int_{z_{w}}^{50} c' \cdot dx = 500 + \{(15.9 \cdot 5 +9.90 \cdot 25) + 9.90 \cdot 25/2 \cdot 25/2\} \cdot \tan 30 \]

\[ = 500 + (79.5 +247.5 +123.75) \cdot \tan 30 = 500 + (327 + 123.75) \cdot 0.5774 \cdot 25 = 7006 \text{ kN/m} \]

This renders a safety factor:

\[ F_s(b) = 7006/4885 \approx 1.43 \]
APPENDIX B

Specific references.


Research Institutes, in which Progressive Failure analyses are recognized methods of procedure and with documented capacity of performing progressive failure analyses are given below.

Luleā Technical University, Luleā, (Sweden),
Norwegian University for Technical & Natural Sciences (NTNU), Trondheim, (Norway),
Norwegian Geotechnical Institute, NGI, Oslo, (Norway),
Laval University, Québec City, (Canada) and
Queen’s University, Kingston, Ontario, (Canada).
APPENDIX C – Comprehensive list of References to publications on the subject of brittle slope failure presented at World Conferences and Symposia.

References

Aas, G., 1966. Special field vane tests for the investigation of shear strength of marine clays. Norwegian Geotechnical Institute, Oslo. Report 68. [In Norwegian.]


II. Further comments, 2016-01-07
Further Comments on the Updated Nalcor Report of 21 July-2014 by Stig Bernander

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Point 2. The Upper Clay layers have liquefied in earlier landslides II-2
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Point 7. Earlier slides in the Churchill Valley may have been progressive II-4
Point 8. The stability analyses seem to be based on Plastic Limit Equilibrium II-5

***********
The following comments can be regarded as complementary to the previous Report, October 14, (2015) by the undersigned in respect of the Nalcor Updated Report 21-JUL-2014 on the Lower Churchill Project.*

The previous report on the subject by this author was by the Grand Riverkeeper Inc. titled: LOWER CHURCHILL RIVER RIVERBANK STABILITY REPORT

*Note: The undersigned has not yet had time and opportunity to scrutinize the new SNC Lavalin report named: “North Spur–Stabilization–Works–Progressive –Failure-Study”. This 128 pages long study was received on January 20, 2016.

As most of the following comments had already been written in December, they will be forwarded to Grand Riverkeeper Labrador, Inc., regardless of possible implications related to the New SNC Lavalin study.

It is considered that they may, in any case, be of importance to GRK – particularly in respect of all the various questions on geotechnical issues raised by persons involved.

***********

The Complementary Comments are denoted Points 1 ➔ 8:
(Note: Minute revisions of linguistic nature made 2016-08-05)

**Point 1.** Every geotechnical engineer knows (or should know) that a loosely compacted sandy water-saturated soil may liquefy if its porosity (usually denoted n) is greater than the critical porosity (n_{crit}). Another way of defining void volumes in soils is by the Void ratio (e), and the Critical Void ratio (e_{crit}).

If, for instance, a loose soil with n > n_{crit} is subjected to deviatory deformation (or vibratory impact), the pore volume decreases generating an excess pore water pressure condition. As a result inter-granular friction decreases – and that even to the extent that the soil may behave as a liquid. This condition is known as ‘soil liquefaction’ and is a well known phenomenon in Soil Mechanics. (See also Point 8).

In the very same way, a loosely layered sandy (or sandy/silty) soil with lean clay content – and with pores containing under-consolidated clay material – may readily liquefy. A difference in this context can be that excess pore pressures in lean clays do not abate as quickly as is likely to be the case in the liquefaction of pure sandy/silty soils.

In the current context, it is apt to refer to a short chapter (Article 17) in the 1967 Edition of the well-known textbook named ‘Soil Mechanics in Engineering Practice’ by Terzaghi and Peck (Reference [3]), where different types of soil liquefaction are dealt with and well explained on five book pages.
**Point 2.** As has been demonstrated in the mentioned RIVERBANK STABILITY REPORT (Ref.[10]), the Upper Clay layers in the Churchill River Valley show clear evidence of being of a highly sensitive nature. (Confer Nalcor soil data, presented on the IWLSC – Conference, (Quebec 2013), Reference [1]).

According to the list of Soil Properties in Ref. [1], the initial void ratios of the lean clayey soils generally range between values of e = 0.93 \(\rightarrow\) 1.06.

As an example, uniform sandy materials are according to Terzaghi & Peck, (Ref.[4]) considered to be loose already at void ratios of 0.85 – indicating that the soils in the North Spur are likely to be extremely loose. However, the critical void ratio is typically much lower (Ref. [3]) but can in any case easily be defined by relevant laboratory testing.

The important requirement according to Equation 1 – in Point 8 below – is not likely to be satisfied for many of the soils in the North Spur.

Furthermore, according to Nalcor, Ref. [1], the range of water contents (w = 21 \(\rightarrow\) 41 %) exceed the liquid limits (w_L or LL) = 19 \(\rightarrow\) 39 %), which is another predicament indicating extreme soil sensitivity.

The many landslides on the North Spur corroborate these data, also indicating very high sensitivity, i.e. of ‘quick clay’ character.

Moreover, the Edward Island landslide, as well as the recent large ‘2014 (or 2013 ?) Landslide’, downstream of Muskrat Falls’, show beyond any shadow of a doubt that the layers of the lean clayey soils have liquefied to the extent that pine trees, which formerly grew in ground close to the riverbank, have been displaced horizontally hundreds of metres during the slide events.

This applies irrespective of whether these landslides are considered to have been triggered by an initial local slip or by instability of retrogressive or progressive nature.

In other words, the lean clayey soils in the layer ‘Upper Cay 2’ have been so sensitive that they have virtually liquefied in the way quick clays tend to do.

**Point 3.** The degree of sensitivity has been discussed by James Gordon, and many others. In Scandinavia, the degree of sensitivity is measured as the ratio between the undisturbed cone shear strength (c_u) and the cone strength (c_r) of the same clay sample in a completely remoulded state. Hence, the sensitivity number is defined as \(S_t = c_u / c_r\).

When the sensitivity (S_t) of clay is \(\geq 50\), the clay is denoted as ‘quick clay’.

However, sensitivity is defined in various ways – e.g. by direct shear tests, compression tests or by tri-axial tests, in which cases notably lower values will be recorded for the same type of clay in comparison with the results from cone tests.

According to Nalcor data in Ref. [1], sensitivities in the North Spur range from 2 to 28. Assuming that these numbers are not related to cone tests, a value of 28 indicates extremely high soil sensitivity. (Yet, even for cone testing, a value of 28 signifies high sensitivity.)

In other words, when evaluating the degree of sensitivity of a soil, i.e. whether it is ‘quick’ or not, it is imperative to define what kind of sensitivity one is referring to.
**Point 4.** Another important feature of the lean clays in the Churchill River valley is that the riverside clayey soil formations, at least in several areas, have remained stable over eons of time (millennia), and yet – when subjected to shear (or deviatory) deformation, due to riverside erosion, the soil material has lost its shear resistance to the effect that gigantic landslides of progressive or retrogressive character have taken place.

The development of the river valley clearly indicates that the originally marine soil layers – although having been exposed to high vertical effective pressures during the postglacial era – have nevertheless a tendency to liquefy when sheared because of lateral loading.

This condition is, as explained in the Report, Ref. [13], related to the scarcity of clay substance in the mixed clayey soil, i.e. so called lean clay. As already stated in **Point 1** above, lean clays with a porous coarse-grained structure are prone to liquefy when subjected to deviatory deformation.

As is also pointed out by Terzaghi & Peck, (Ref. [3]), void clay (as well as silt content) – although being in an under-consolidated state – contributes to preserving a looser granular structure over time than would be possible if the voids had only contained water.

**Point 5.** Although a lot of general information is presented in Reference [1], hardly any data related to specific identifiable soil layers – such as e.g. the Upper Clays 1 & 2 – are presented. Thus, the precise and coherent soil properties applying to samples representative of critical layers are lacking in Ref. 1 ….. a condition that makes it impossible to perform any kind of reliable stability analysis based on soil parameters given in Ref. [1].

There are, for example, no coherent values regarding any of the important soil properties such as: Initial void ratios, Critical void ratios, Water contents, Liquid Limits, Plastic Limits, Un-drained shear strength, Residual shear resistance, Sensitivity, Unit weight etc.

The mean values of many of these parameters have been defined in Ref. [1] but as Nalcor must be aware of, such mean values are of little use for stability analyses in a heterogeneous formation like the North spur.

Lacking parameters are in particular:

- Tri-axial shear (deviatory) tests on undisturbed samples;
- Fast ‘direct shear tests’ on undisturbed soil samples;

Tests of this kind provide the complete stress/strain behaviour related to deviatory deformation. Such stress-deformation relationships are absolutely necessary in progressive failure analysis.

In spite of the fact that the most crucial features of the soil layers in the Churchill River Valley are high porosity and high sensitivity, no test results of this kind have been presented in the Updated Nalcor, 21- July-20 14 Report, (Ref. [2]).

**Point 6.** According to the diagram on Page 33 in the Nalcor report showing “Total head profiles in the spur at U/S “ WL = EL 39 m, the water pressure force acting on the COW above Elevation 25 m is roughly 1500 kN/m after stabilization. This corresponds to an
external, locally concentrated force over a width of 100 m of 150 000 kN/m = 15 000 metric tons/100 m.

Point 7. On Page 9 in the Nalcor Report, under the heading “Safety factor against progressive failure”, the following statements are made:

a) Quotation: “…… calculations are calibrated locally with an existing slope”.

Comment on quotation a):
This may be a feasible approach in a slope formation consisting of uniform soil materials with truly plastic behaviour. However, in a very heterogeneous soil formation with highly sensitive soils, such an approach is not relevant.

b) Quotation: “…… Rotational, flow-slide, spread stability is calculated with a first movement at the toe.

Comment on quotation b):
This would of course be OK, provided the first movement really starts at the toe. This is however by no means certain. Confer next comment.

c) Quotation: “There is no evidence of downhill progressive failure landslide along the Churchill river valley.”

Comments on quotation c)
Firstly, this is a remarkably odd statement considering that there is not likely to exist any other condition along the valley, where a concentrated massive downhill force of 1500 kN/m (= 15 000 metric tons/100m) has been designed to act locally in the soil formation at some distance from the riverside slope …..especially keeping in mind that the soils are very porous with a documented highly sensitive behaviour.

Secondly, Nalcor claims that the two massive recent landslides in the valley are flow-slides. Unless the initial loading and soil conditions are accurately known, it is not possible to define or label the precise character of extensive landslides measuring many hundreds of metres in length. For instance, the Edward Island slide may possibly have been a serial flow-slide but it may just as well have been a forward progressive landslide triggered by an initial local riverside instability or by an ordinary slip-circular slide caused by temporary high water levels further inside the slope formation …. and - may be - with contributing seismic activity. As far as is known to the writer, no detailed soil investigation had been made in the area before the landslide event.

The very circumstance that the riverbank pine trees have been displaced horizontally almost half a kilometre does not indicate that they got there by suction or by flowing downhill. The fact that they must have been pushed there speaks for a slide, which in its main catastrophic phase was forward progressive.

The same reasoning applies to the 2014 Landslide downstream of Muskrat Falls.
Exemplification of landslide complexity:
The famous Rissa slide in Norway is by many geotechnical engineers thought of as a flow-slide – largely because of the famous film documenting the final flow-slide phase of the landslide.
Yet, as was demonstrated in the writers Power Point Presentation in Saint Johns, (Ref. [12]), the Rissa landslide began as a minor ordinary slip-circular slide due to a fill on the bank of the adjacent fjord. The slide then propagated uphill as a serial flow-slide ... that then triggered two consecutive extensive downhill progressive landslides. Then again, as is often the case, sliding continued as a series of flow-slides, whereby the soil debris kept on flowing downhill and disappeared into the depths of the fjord as illustrated by the said film.

**Conclusion:** The Updated Nalcor July 2014 Report contains no evidence actually proving that downhill progressive landslides cannot take place in the Churchill River Valley.

d) Quotation: “Counter measure will be in place to control … “Human triggering”

**Comment on quotation d):**
The Updated Nalcor Report contains no information indicating how, and what sort of “Human triggering” that will be implemented in order to prevent flow-slides, massive retrogressive or progressive landslides in the very heterogeneous and highly sensitive soil formation constituting the North Spur.
Slides in sensitive soils are normally unpredictable and very sudden – especially if the innately sensitive properties of the soil materials are overlooked.

**Point 8.** Stability analysis in the Updated Nalcor Report (21- July- 2014) is – as far as can be concluded – based on frictional resistance and on the hydraulic head profiles shown on pages 32 and 33.
As no shear stress /strain (shear deformation) relationships are presented, it seems evident that the stability analyses are based on the Plastic Limit Equilibrium (PLE or LEM) mode.

As has already been stated in the beginning of this report, frictional resistance is generally a very reliable stabilizing resistance parameter. There are, however, very important conditions that must be fulfilled for this rule to be valid:

**a)** In cases, where the additional lateral load – causing shear deformation – is static, it is imperative that the current porosity value \( n \) is equal (or less) than the critical porosity \( (n_{\text{crit}}) \)
i.e. \( n < n_{\text{crit}} \) or in terms of void ratio \( e \)
\[ e < e_{\text{crit}} \]
where \( e \) and \( n \) relate to each other as \( e = n / (1-n) \) or \( n = e / (1+e) \)

If the condition according to Equation 1a (or 1b) is not fulfilled, the application of even static loading may reduce frictional resistance, even to the extent that liquefaction occurs.

**b)** However, in cases, where the additional stresses change signs as in vibratory impact – i.e. when the axial stresses alternate between \( \pm \Delta \sigma_x \), (or \( \pm \Delta \sigma_y \)) – liquefaction can readily occur even if the condition according to Equations 1 is fulfilled.
The same applies to deviatory deformation, in which case the additional vibratory shear stress alternates between \( \pm \Delta t_{xy} \), i.e. so called stress-strain reversals (Cf Terzaghi & Peck, Ref. [3].)
A vital question in this context: Have the effects of vibratory impact of this kind been considered in the Seismic Analysis by Atkinson? Dynamic impact always calls for a higher degree of compaction – i.e. a condition that should be based on relevant testing procedures.

It is therefore important that all test results related to the porosity of soil layers in the North Spur are made known to GRK. The following data are of particular interest:

1a) The in situ void ratios of loose sand and mixed silty sand layers.
1b) The critical void ratios of these sands and mixed silty sands, obtained by direct shear tests on undisturbed soil samples.

2a) The in situ void ratios of the coarse-grained structure of lean clayey soils.
2b) Shear tests on undisturbed samples of lean clay layers and diagrams showing the complete deviatory stress/deformation) relationship.

It may be noted that, when evaluating the results from the testing of initial void ratios, the difficulty of getting undisturbed soil samples must be recognized. The in situ void volume of materials with high porosity is easily affected by the sampling procedures.

Stig Bernander
**References:**

[1] Geotechnical data related to the Muskrat Falls Dam Project on posters at the IWLSC Conference (Quebec 2013),


*Previous reports or comments (2014 → 2015) on slope stability related to the North Spur or to the Churchill River Valley in general by S. Bernander*

[10] 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


[12] Power Point presentation (Appendix III) in an assembly hall in Saint Johns on October 30, 2014 and at the Memorial University, October 31, 2014

[13] **LOWER CHURCHILL RIVER, Riverbank Stability Report, October 14, 2015 (Revisions of linguistic nature, November 23 and December 5.)**
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COMMENTS ON THE ENGINEERING REPORT
BY NALCOR/SNC-LAVALIN
OF 21 DECEMBER 2015

PREPARED FOR
Grand Riverkeeper Labrador, Inc.

BY
DR. S. BERNANDER

15 September 2016

The third report in the series on:
Churchill River Valley c) Specific issues III, Muskrat Falls

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The following comments should be regarded as complementary to the previous reports by the undersigned dated 14 October 2015 [13] and 7 January 2016 [14] respectively, concerning the *Nalcor Report to the Independent Engineer on the Lower Churchill Project, North Spur Updated, 21-JUL-2014* [2a].

The October 2015 report by this Reviewer was prepared on behalf of the Grand Riverkeeper Inc. and titled *Lower Churchill River Riverbank Stability Report* [13].

The later report, dated 7 January 2016, was titled *Further Comments on the Updated Nalcor 21-JUL-2014 Report* [14].

When the January 7 report was written, the undersigned had not yet had the possibility to review the new Nalcor/SNC-Lavalin report of 21 December 2015 titled *Engineering Report, North Spur Stabilization Works, Progressive Failure Study* [2b].

Having now reviewed the 21 December 2015 *Engineering Report*, 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* [2b]. References will also be made to both of this Reviewer’s earlier comments.
**EXECUTIVE SUMMARY**

The stability of the North Spur as a dam containment structure is a complicated issue.

The criticism of the *Engineering Report* of 21 December 2015, presented below by this Reviewer, does not constitute any prediction of likely or certain North Spur failure due to impoundment, other man-made stress, or seismic action.

However, the accounts of 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.
- The *Report* makes no mention of seismic events, either historical or potential.
- 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.

*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.

Gothenburg, 15 September 2016
Stig Bernander
1. **General Considerations**

The Nalcor/SNC-Lavalin *Engineering Report* is a comprehensive and from many points of view a thorough 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 2b.*

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 (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.

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*Note:* By doing so, the list of references in the current report is consistent with the corresponding list in the author’s 7 January 2016 Comments. [14]
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 [13], 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](image)

- **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.
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.

As has been emphasized in previous reviews [13,14], 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.

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).

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<th>Liquidity Index</th>
<th>Corresponding Liquid Limit</th>
<th>Unit weight</th>
<th>Void ratio</th>
<th>Porosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stratified Drift</td>
<td>w %</td>
<td>PL %</td>
<td>LI</td>
<td>LL %</td>
<td>γw kN/m³</td>
<td>e</td>
<td>n</td>
</tr>
<tr>
<td>Type 1a</td>
<td>43</td>
<td>13</td>
<td>2.8</td>
<td>38.8</td>
<td>17.71</td>
<td>1.14</td>
<td>0.53</td>
</tr>
<tr>
<td>Type 1b</td>
<td>43</td>
<td>15</td>
<td>2.0</td>
<td>36.5</td>
<td>17.71</td>
<td>1.14</td>
<td>0.53</td>
</tr>
<tr>
<td>Type 1c</td>
<td>43</td>
<td>25</td>
<td>1.3</td>
<td>40.6</td>
<td>17.71</td>
<td>1.14</td>
<td>0.53</td>
</tr>
<tr>
<td>Type 2a</td>
<td>35</td>
<td>13</td>
<td>2.8</td>
<td>35.9</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Type 2b</td>
<td>35</td>
<td>15</td>
<td>2.0</td>
<td>32.5</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Type 2c</td>
<td>35</td>
<td>25</td>
<td>1.3</td>
<td>34.4</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Type 3a</td>
<td>30</td>
<td>13</td>
<td>2.8</td>
<td>34.1</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Type 3b</td>
<td>30</td>
<td>17</td>
<td>2.0</td>
<td>32.0</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Type 3c</td>
<td>30</td>
<td>25</td>
<td>1.3</td>
<td>30.6</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Mean values</td>
<td>31</td>
<td>19</td>
<td>1.3</td>
<td>29.5</td>
<td>19.06</td>
<td>0.82</td>
<td>0.45</td>
</tr>
</tbody>
</table>

Relationships

\[ w = n/(1-n) \times \gamma_R \]  \[ n = e/(1+e) \]  \[ e = n/(1-n) \]  \[ w = \text{water content} \]

Density, H₂O-saturated \[ \gamma_w = (w+1)/(w+1 \times \gamma_{H₂O}/\gamma_R) \]  \[ \gamma_{H₂O} = \text{Density of water} = 10 \text{ kN/m}^3 \]

or: \[ \gamma_w = n \times \gamma_{H₂O} + (1-n) \times \gamma_R \text{ kN/m}^3 \]

Dry density \[ \gamma_d = (1-n) \times \gamma_R \text{ kN/m}^3 \]

Assumed density of rock material \[ \gamma_R = 26.5 \text{ kN/m}^3 \]

For comparison, see Terzaghi and Peck [4], 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 [4]

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Porosity</th>
<th>Void ratio</th>
<th>Water content</th>
<th>Water-saturated unit weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniform sand, loose</td>
<td>0.46</td>
<td>0.85</td>
<td>32</td>
<td>18.9</td>
</tr>
<tr>
<td>Uniform sand, dense</td>
<td>0.34</td>
<td>0.51</td>
<td>19</td>
<td>20.9</td>
</tr>
<tr>
<td>Mix-grained sand, loose</td>
<td>0.40</td>
<td>0.67</td>
<td>25</td>
<td>19.9</td>
</tr>
<tr>
<td>Mix-grained sand, dense</td>
<td>0.30</td>
<td>0.43</td>
<td>16</td>
<td>21.6</td>
</tr>
</tbody>
</table>

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 in Soil Mechanics 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 engineers found that the silty clay layer had developed multiple failure surfaces and liquefied over a long lateral distance.

### 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).

<table>
<thead>
<tr>
<th>Type of Lower Marine Clay</th>
<th>Water content w %</th>
<th>Plastic limit PL %</th>
<th>Liquidity index LI</th>
<th>Correspond. Liquid limit LL %</th>
<th>Unit weight kN/m³</th>
<th>Void ratio e</th>
<th>Porosity n</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lower cl Ia</td>
<td>45</td>
<td>13</td>
<td>2.0</td>
<td>35.5</td>
<td>17.53</td>
<td>1.19</td>
<td>0.54</td>
</tr>
<tr>
<td>Lower cl Ib</td>
<td>45</td>
<td>15</td>
<td>1.5</td>
<td>37.5</td>
<td>17.53</td>
<td>1.19</td>
<td>0.54</td>
</tr>
<tr>
<td>Lower cl Ic</td>
<td>45</td>
<td>17</td>
<td>1.0</td>
<td>45.0</td>
<td>17.53</td>
<td>1.19</td>
<td>0.54</td>
</tr>
<tr>
<td>Lower cl IIa</td>
<td>35</td>
<td>11</td>
<td>2.0</td>
<td>28.5</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Lower cl IIb</td>
<td>35</td>
<td>13</td>
<td>1.5</td>
<td>29.8</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Lower cl IIc</td>
<td>35</td>
<td>15</td>
<td>0.9</td>
<td>37.4</td>
<td>18.56</td>
<td>0.93</td>
<td>0.48</td>
</tr>
<tr>
<td>Lower cl IIIa</td>
<td>30</td>
<td>10</td>
<td>2.0</td>
<td>25.0</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Lower cl IIIb</td>
<td>30</td>
<td>13</td>
<td>1.5</td>
<td>26.5</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Lower cl IIIc</td>
<td>30</td>
<td>16</td>
<td>0.9</td>
<td>31.7</td>
<td>19.19</td>
<td>0.80</td>
<td>0.44</td>
</tr>
<tr>
<td>Mean Values</td>
<td>29</td>
<td>21</td>
<td>0.6</td>
<td>39.9</td>
<td>19.33</td>
<td>0.77</td>
<td>0.43</td>
</tr>
</tbody>
</table>

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 \( (n_{\text{crit}}) \);
- 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 [8a]. 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 (Aas, 1966). Legend: brott = failure, Vridning = torsion, dygn = day, vecka = week, grader = degrees.](image)
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 [9].
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 [1] — 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.

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

Figure 5.1. Potential failure planes (I and II) possibly leading to progressive failure development.
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 [1], the lower contour of Upper Clay 2 slopes about 3 metres along the length coordinate $x \approx 200$ m (near the COW) to $x \approx 350$ 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 ($\tau$) due to vertical stress over such an inclination is in the order of

$$\tau = \sigma' \times \sin 0.106.$$  

Considering that $\gamma_w = n + (1-n) \gamma_R$, including the weight of percolating pore water above water level $W_L = +39$, the vertical effective stress ($\sigma'_v$) may be roughly estimated to:

$$\sigma'_v \approx (59-46) \times [0.36 \times 10 + (1-0.36) \times 26.5] + (46-39) \times [0.41 \times 10 + (1-0.41) \times 26.5] + (39-23) \times [0.48 \times 10 + (1-0.48) \times 26.5-1 \times 10] =$$

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

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

Hence the shear stress ($\tau$) at a beginning failure plane of parabolic shape may amount to

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

The impoundment from water level $W_L = +17$ m to $W_L = +39$ m represents a horizontal force above level $+17$ of $H_w \approx 2420 \text{ kN/m}$.  

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

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

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

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

Hence, the total local shear stress could be in the order of $72.6 + 58.6 = 131.4 \text{ kN/m}^2$. Note that this value is higher than almost all the intact undrained shear strength measurements, $s_u = 35$ 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: $s_u = 53–200 \text{ kN/m}^2$ (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).

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. [5,6,7], 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

According to the Nalcor Report to the Independent Engineer, 2014 [2a], the potential effects of earthquakes have been investigated.

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 [13]).

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 REPORT offers no test results showing the impact on residual shear resistance of deviatory deformation and of stress /strain reversals.

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, $n > n_{\text{crit}}$, 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 [3] 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. \( n > n_{\text{crit}} \).**

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, the current REPORT offers no data on the behaviour of these soils when subjected to the types of stress 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 \( n_{\text{crit}} \):

\[
\begin{align*}
\text{Equation 1a} & : & n < n_{\text{crit}} \\
\text{Equation 1b} & : & e < e_{\text{crit}}
\end{align*}
\]

where \( n \) and \( e \) relate to one another as

\[
\begin{align*}
e & = n/(1-n) & \text{or} & & n = e/(1+e)
\end{align*}
\]
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 $\pm \Delta \tau_{x,y}$ or axial stresses alternate between $\pm \Delta \sigma_x$ or $\pm \Delta \sigma_y$ — 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 [3]).

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

### 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 [5,6,7] demonstrate how the risk of a progressive landslide can be estimated from basic geotechnical parameters. In this context it may be noted that SKANSKA Ltd already in 1983–1985 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.**

**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 [8b]. (See Figures 5.2 and 5.3 below). The Bekkelaget landslide was also referred to and commented on by this Reviewer [5,7].
Figure 5.2. The Bekkelaget landslide, Norway. Analysis by Aas (1983). 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. [7] 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.
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 [7].
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 yet reported a true Progressive Failure Analysis, and there is no indication that any such work has 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, and 3.1.3 of these comments, as well as 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 [13].

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. [13]).

Moreover, the permeability values ($k = \text{m/sec}$) in Saint Barnabé-Nord ranged from $1 \times 10^{-9}$ to $5 \times 10^{-9}$ m/s, whereas the $k$-values of the Upper Clays in the North Spur are about $1 \times 10^{-7}$ 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.*

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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 \( n_{\text{crit}} \). Such types of soil may liquefy due to a moderate deviatory deformation or because of minor repetitive stress-strain reversals — and that 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 13 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.
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 large triggering loads 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 ($n_{crit}$) — then it will be necessary to strengthen the affected soil structures. As per Terzaghi and Peck [3], 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 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 [15].

Again, this 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 ($n$) is not too different from the critical soil porosity ($n_{crit}$).
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 [3,4] 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 [5-7] 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 \( n_{\text{crit}} \), 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 follows is explicitly based on elastic-plastic conditions and LEM analysis. Dynamic stress conditions are extrapolated from static ones.

However, as is well-recognised, 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 the Stratified Drift over a long lateral distance. All experts agree that without human 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 despite the proposed stabilization works, then additional stabilization would be required. This Reviewer suggests — tentatively, until the data are better known — that this would be 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 [10].

* [10]
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 on 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 a smaller slide on the north bank just five kilometres downstream of Muskrat Falls* — this Reviewer recommends that a renewed analysis of the risk of 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 liquefaction and flow-sliding.

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 alteration of the current construction program.

* This last landslide has good video documentation, found at https://www.youtube.com/watch?v=L1cL_pN4NIQ.
REFERENCES

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


Further references to previous reports or comments (2014–2015) by S. Bernander on slope stability related to the North Spur or to the Churchill River Valley in general:

[10] 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.

[12] 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.


**Additional references cited in these Comments:**


IV. Spreadsheet Analysis, 2017-06-01

Case 3

\[ \tau_o = 21.1 \text{ kPa} \quad s = 60 \text{ kPa} \quad s_R = 12 \text{ kPa} \quad s/s_R = 5 \quad N_{cr} = 866 \text{ kN/m} \]

Safety factor \( F = N_{cr} / N_w = 866 / 2420 = 0.357 < 1 \)

Case 4

\[ \tau_o = 41.1 \text{ kPa} \quad s = 70 \text{ kPa} \quad s_R = 14 \text{ kPa} \quad s/s_R = 5 \quad N_{cr} = 521 \text{ kN/m} \]

Safety factor \( F = N_{cr} / N_w = 521 / 2420 = 0.215 < 1 \)
Downhill progressive slide - triggering load, basic ex.
Muskrats Fals, Case 3 - Slope 1:25, arctan 0.04 = 2.292dgr
Cr = 12 kN/m2
Input data:
H(x = L) = 15 m
Gradient: Slope 1:25, arctan 0.04 = 2.292dgr
Density = 18.0 kN/m3
C_{peak} = 60 kN/m2
C_{lab} = 50 kN/m2
C_R = 12 kN/m2
C_{R/C_{lab}} = 0.24
C_s = 45 kN/m2
C_{mean} = 52.5 kN/m2
tel = 40 kN/m2
Shear deformation at failure (g_f) = 7.0%
Post-peak deformation at C_R = 0.30 m
Shear deform. at elastic limit (g_{tel}) = 3.50organic chemistry

coconut oil

Stage I
Equ. I:1a \ S:a d_{(x,z)}, t = \int (g_{(x,z), (1)})dz

Stage I
Equ. I:1a \ S:a d_{(x,z)}, t = \int (g_{(x,z), (1)})dz

Stage I
Equ. I:4 \ S:a d_{(x,z)}, t = \int (g_{(x,z), (4)})dz

Stage II
Equ. I:5 \ S:a d_{(x,z)}, t = \int (g_{(x,z), (5)})dz + Sc_{cr} + S_{slip}

Stage II
Equ. I:5 \ S:a d_{(x,z)}, t = \int (g_{(x,z), (5)})dz + Sc_{cr} + S_{slip}

Stage II
Equ. I:5a \ S:a d_{(x,z)}, t = \int (g_{(x,z), (5a)})dz + Sc_{cr} + S_{slip}

Stage II
Equ. I:5a \ S:a d_{(x,z)}, t = \int (g_{(x,z), (5a)})dz + Sc_{cr} + S_{slip}

Stage II
Equ. I:5b \ S:a d_{(x,z)}, t = \int (g_{(x,z), (5b)})dz + Sc_{cr} + S_{slip}
## Downhill progressive slide - triggering load

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* In situ shear stress to be modified by the expression dt = (E_{(n+1)} - E_{(n)}) / dx (E_{(n+1)} and E_{(n)} are in situ earth pressures)
Downhill progressive slide - triggering load

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<th>cu-t el</th>
<th>t el</th>
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Author: Stig Bernander

Calc. S(x) = C_r/ C_{peak} = 0,20

Calc. \( dN = 124,72 \)

\( d(x_n, n+1), N = 0,01617 \)

\( dx, N - dx(t) = 0,00000 \)

Stage I

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<td>Width, b</td>
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Equ 1:a

Author: Stig Bernander

Calc. S(x) = C_r/ C_{peak} = 0,20

Calc. \( dN = 124,72 \)

\( d(x_n, n+1), N = 0,01617 \)

\( dx, N - dx(t) = 0,00000 \)
### Downhill progressive slide - triggering load

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**Stage I**

**Iter. step**

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<th>cu-t el</th>
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**Author: Stig Bernander**

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| S:a d(xn),N | 0,03612 |
| S(x)= | m |

| S:a N x(n+1) | 305,78 |
| S:a dx(x(n+1),N | 0,03876 |
| S:a dx(x(n+1),t) | 0,07487 |
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### Downhill progressive slide - triggering load

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#### Author: Stig Bernander

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Equa.1:4a
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Author: Stig Bernander

Calc. S(x) / CR/C(peak) = 0.20

dx, dN = 78.03

Calc. d(xn+1),N = 0.02458

dx, N-dx(t) = 0.00000

S:a N x(n) = 447.24
S:a d(xn),t = 0.11941
S:a d(xn+1),N = 525.27
S:a dx(n+1),t = 0.014399

m
### Downhill progressive slide - triggering load

<table>
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<th>t el</th>
<th>H =</th>
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<td>In situ</td>
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<td>incre-</td>
<td>Shear</td>
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### Calculations

- S(a) x(n) = 525,27
- S:a dx(n),t = 0,16191, 0,14399
- S:a d(xn),N = 0,14399
- S(a) dx(n+1),N = 0,16769
- S:a d(xn+1),t = 0,16191

### Author: Stig Bernander

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**Equation 1:4a**
### Downhill progressive slide - triggering load

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<td>t(x,z)</td>
<td>dt(x,z)+dt</td>
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### Stage I

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<th>g el</th>
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<th>G mod.</th>
<th>c (peak)</th>
<th>E-mean</th>
<th>t el</th>
<th>H = Width, b</th>
<th>dx (m)</th>
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**Equation 1.4a**

- \( d_{(g)} = \frac{d_{(g) + dz}}{c} \)
- \( G(x) = \int_{x_0}^{x} \rho(x) dx \)

**Author:** Stig Bernander
## Downhill progressive slide - triggering load

**Case, Appendix I - Slope 1:15,337, arctan 0,0652 = 3,73 dgr**

\( Cr = 15 \text{ kN/m}^2 \quad \text{to} = 20.8 \text{ kN/m}^2 \)

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<th>( t_{el} )</th>
<th>( t )</th>
<th>( N )</th>
<th>( S:a \ dn )</th>
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<th>( H+y )</th>
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\( q_{crit} = 57,71 \text{ kN/m}^2 \)
Downhill progressive slide - triggering load

Case, Appendix I - Slope 1:15,337, \( \arctan 0.0652 = 3.73 \text{ dgr} \)

\( C_r = 15 \text{ kN/m}^2 \quad t_o = 20.8 \text{ kN/m}^2 \)

Author: Stig Bernander

---

Case I - Slope 1:20, \( \arctan 0.05 = 2.8624 \text{ dgr} \)
Downhill progressive slide - triggering load
Case, Appendix I - Slope 1:15,337, arctan 0.0652 = 3.73 dgr
Cr = 15 kN/m²  to = 20.8 kN/m²

Author: Stig Bernander

Case I - Slope 1:20, arctan 0.05 = 2.8624 dgr
Downhill progressive slide - triggering load

Case, Appendix I - Slope 1:15,337, arctan 0,0652 = 3,73 dgr

$C_u = 15 \text{ kN/m}^2 \quad t_o = 20.8 \text{ kN/m}^2$

Author: Stig Bernander
\[
\begin{array}{ll}
\text{arctan 0,04} & 0,03997869 \\
\text{sin arctan 0,004} & 0,03996804 \\
18*15*sinarctan & 10,7913704 \\
18,8*28*sinarctan & 21,0391754 \\
\text{delta E} & 9,86895336 \\
\text{delta to} & 7000/35(20) \\
\text{tauo} & 41,9615924 \\
\end{array}
\]
Explanation of denotations

- \( dE/dx \): Shear stress before adding current stress increment
- \( G \): Overload density
- \( \star \): Overload thicknesses
- \( L \): Length of the defined slope
- \( x \): Coordinate from upper boundary condition, where shear stress increment, dN and d(x) = 0
- \( x' \): Coordinate from upper slope limit as defined by \( L \)
- \( x'' \): Coordinate from lower slope limit = \( L - x' \)

\[ H_q \]

\[ \frac{\Delta t}{(q)} = k \gamma \left( \frac{(Hn)^2 - ((Hn+1)^2)}{(2\Delta x)} \right) \]

\[ -dH \cdot dx \cdot \gamma \cdot \sin \arctan(H33) \]

\[ 20,7 \text{ kN/m} \]

\[ \text{Equ.1:4a} \]

\[ \text{Equ.1:1a} \]

\[ 23,16 \quad 23,66 \quad 23,23 \quad 23,64 \quad 0,0000 \]

\[ 15,44 \quad 15,77 \quad 15,483 \quad 19,41 \]

\[ 0,667 \quad 0,666 \quad 0,667 \quad 0,821 \]
\[ \frac{dE}{dx} \quad G = \]

\begin{align*}
9.6 & \\
16.8 & \\
1142.86 & 
\end{align*}

Equ.1:4

Equ.1:4a

Equ.1:1a
\[
\begin{array}{|c|c|}
\hline
dE/dx & G= \\
9.4 & 1142.86 \\
\hline
\end{array}
\]
\[
\frac{dE}{dx} \\
G = \\
9,3 \quad ### \quad 1142,86 \\
13,15
\]

Equ.I:4

Equ.I:4a

Equ.I:1a
G = 9.3

Equ.1:4

Equ.1:4a

Equ.1:1a

1142.86
G = 9,2  
1142,86

Equ.1:4

Equ.1:4a

Equ.1:1a
Equ.I: 4

Equ.I: 4a

Equ.I: 1a
Downhill progressive slide - triggering load, basic ex.

Muskrats Fals, Case 4: Slope 1:25, arctan 0.04 = 2.292 dgr

Cr = 14 kN/m2

Input data:
H(x = L) = 15 m
Gradient: Slope 1:25, arctan 0.04 = 2.292 dgr

Density = 18.0 kN/m³

C_R = 14 kN/m²

C_R/C_lab = 0.255
C_s = 45 kN/m²

Shear deformation at failure (g_f) = 7.0%

C_R/C_peak = 0.20

t_el = 40 kN/m²

Shear deform. at elastic limit (g_el) = 3.50%

t_x,z < t_el

g_x,z(1) = t/v, g_el = v_t/2c - t_el and G = t/2c - t_el

Stage I
Equ. I:1
S:a d(x,z),t = Integral (g_x,z(1))dz

Stage I
Equ. I:4
S:a d(x,z),t = Integral (g_x,z(4))dz

Stage II
Equ. I:5
S:a d(x,z),t = Integral (g_x,z(5))dz + Sc(x - t_x)/c - c_u

Stage IIa
Equ. I:5
S:a d(x,z),t = Integral (g_x,z(5a))dz + + Sc(x - t_x)/c - c_u

Stage IIb
Equ. I:5
S:a d(x,z),t = Integral (g_x,z(5b))dz + Sc(x - t_x)/c - c_u
### Downhill progressive slide - triggering load

<table>
<thead>
<tr>
<th>n</th>
<th>z (m)</th>
<th>In situ* In situ* Shear increment Shear shear</th>
<th>Stress shear Stress Stress Stress Stress</th>
<th>kN/m²</th>
<th>kN/m²</th>
<th>kN/m²</th>
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* In situ shear stress to be modified by the expression \( \Delta t = \frac{(E_{n+1} - E_n)}{dx} \) (\( E_{n+1} \) and \( E_n \) are in situ earth pressures)

Author: Stig Bernander

Calc. \( S(x) \) \( c \in [0,20] \)

Calc. \( \delta N = 53,70 \)

Calc. \( \delta x = 0,01365 \)

Calc. \( \delta x = 0,0001 \)

* In situ shear stress to be modified by the expression \( \Delta t = \frac{(E_{n+1} - E_n)}{dx} \) (\( E_{n+1} \) and \( E_n \) are in situ earth pressures)
**Downhill progressive slide - triggering load**

| Iter. step | No 2 | x1   | x2   | Stress | Mean | \( g f = \) | \( g f = \) | Gradient | \( G \) mod. | \( c \) (peak) | \( E \)-mean | \( t \) el | \( H \) = Width, b | dx (m) |
|------------|------|------|------|--------|------|------------|------------|-----------|-------------|-------------|-------------|----------|-----------|-------------|-------|
| x          | 26,85| x2   | Mean | kN/m2  | kN/m2| 0,070      | 18,000     | 45        | 0,0280      | 0,0400      | 1429        | 70,00    | 3520      | 40,00       | 15,0 |

| z (m) | In situ shear stress | In situ shear stress | Shear increment | Shear stress | \( g r - g e l \) | \( c u - t \) el | \( t \) el | \( d(g) \) | \( d(g)*dz \) | \( c u \) | \( G \) |
|-------|-----------------------|----------------------|-----------------|--------------|------------------|-----------------|--------|---------|-------------|-------|
| 0     | 0,00                  | 41,14                | 45,11           | 6,600        | 51,705           | 48,405          | 0,0420 | 30,00   | 40,00       | 0,00842 | 70,00 | 1429 |
| 1     | 0,75                  | 39,60                | 39,57           | 42,85        | 6,353            | 49,202          | 0,0420 | 29,46   | 39,29       | 0,00759 | 68,75 | 1403 |
| 2     | 1,50                  | 39,06                | 39,03           | 40,59        | 6,105            | 46,700          | 0,0420 | 28,93   | 38,57       | 0,00605 | 67,50 | 1378 |
| 3     | 2,25                  | 38,52                | 38,49           | 38,34        | 5,858            | 44,197          | 0,0420 | 28,39   | 37,86       | 0,00452 | 66,25 | 1352 |
| 4     | 3,00                  | 37,98                | 37,95           | 36,08        | 5,610            | 41,694          | 0,0420 | 27,86   | 37,14       | 0,00297 | 65,00 | 1327 |
| 5     | 3,75                  | 37,44                | 37,41           | 33,83        | 5,363            | 39,191          | 0,0420 | 27,32   | 36,43       | 0,00142 | 63,75 | 1301 |
| 6     | 4,50                  | 36,90                | 36,87           | 31,57        | 5,115            | 36,689          | 0,0420 | 26,79   | 35,71       | -0,00014 | 62,50 | 1276 |
| 6,7   | 5,00                  | 36,54                | 36,51           | 30,06        | 4,949            | 35,012          | 0,0420 | 26,43   | 35,24       | -0,00120 | 61,66 | 1258 |

**Author:** Stig Bernander  
**Dem.:** SCR 0,30  
**Calc.:** S(x)  
**Calc.:** S(a N x(n))  
**Calc.:** S: a dx(n+1),t  
**Calc.:** S: a dx(n+1),t  

Equ 1:4  

\[ S(x) = 0,01968 \]

\[ S: a dx(n+1),t = 0,01968 \]

\[ S: a dx(n+1),t = 0,01968 \]
### Downhill progressive slide - triggering load

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**Author:** Stig Bernander
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**Downhill progressive slide - triggering load**

**Stage I**

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Equation 1:4

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Author: Stig Bernander
### Downhill progressive slide - triggering load

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| Stress |            | Mean | | | | | | |
|--------|------------|------|---|---|---|---|---|
|        |            | kN/m2 | kN/m2 | kN/m2 | 1 | m | kN/m2 |
|        |   Equ 1:4  |       |     |     |   |   |     |

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</table>

<table>
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<tr>
<th>Calc.</th>
<th>S(x)</th>
<th>$c_r/c_{(peak)}$</th>
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</thead>
<tbody>
<tr>
<td></td>
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<td>dN</td>
<td>51,69</td>
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<table>
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<tr>
<th>Calc.</th>
<th>d(xn)</th>
<th>S:a d(xn),t</th>
<th>0,07147</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>dx, N-</td>
<td>S:a dx(N+1)</td>
<td>0,07147</td>
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$dx, N-dx(t) = 0,00000 m$

S:a $N x(n)$: 263,59
S:a $d(xn), t$: 0,07147
S:a $d(xn), t$: 0,07147
<table>
<thead>
<tr>
<th>Iter. step No</th>
<th>x5</th>
<th>x6</th>
<th>Stress</th>
<th>Mean</th>
<th>Gradient</th>
<th>E-mean</th>
<th>H</th>
<th>Width, b</th>
<th>dx (m)</th>
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**Stage I**

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<th>n</th>
<th>z (m)</th>
<th>In situ shear stress</th>
<th>Shear stress increment</th>
<th>Shear stress</th>
<th>gr - gel</th>
<th>cu-t el</th>
<th>t el</th>
<th>d(g)</th>
<th>d(g)*dz</th>
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<td>68,88 67,934 0,0350</td>
<td>30,00 40,00 0,02767</td>
<td>70,00 1143</td>
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<td>65,46 64,549 0,0350</td>
<td>29,46 39,29 0,02322</td>
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<td>58,62 57,779 0,0350</td>
<td>28,39 37,86 0,01654</td>
<td>66,25 1082</td>
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<tr>
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<td>3,00</td>
<td>30,06 37,83</td>
<td>55,20 54,395 0,0350</td>
<td>27,86 37,14 0,01381</td>
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<td>5,00</td>
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<td>44,64 45,357 0,0350</td>
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<td>61,66 1007</td>
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</table>

**Author:** Stig Bernander

| S(a) | 315,28 | S(a) d(xn+1),N | 348,40 | S(a) d(xn+1),t | 0,08113 | m |
| S(a) dxn+1,N | 0,00965 | m |
| dx,N-dx(t) | 0,00000 | m |
## Downhill progressive slide - triggering load

### Stage I

<table>
<thead>
<tr>
<th>Iter. step</th>
<th>No</th>
<th>x6</th>
<th>x7</th>
<th>Stress</th>
<th>Mean</th>
<th>kN/m²</th>
<th>kN/m²</th>
<th>kN/m²</th>
<th>l</th>
<th>m</th>
<th>kN/m²</th>
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<tr>
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### Stage II

| n | z (m) | In situ shear stress | In situ shear stress | Shear increment stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress | Shear stress |
|---|-------|----------------------|----------------------|-----------------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|--------------|
| 20| 0.00  | 40,98                | 40,97                | 68,87                 | 1,126        | 70,00         | 69,435       | 0,0350       | 30,00        | 40,00        | 0,03415      | 70,00         | 1143         | 0,02222      | 68,75        | 1122         |
| 20| 1.75  | 34,52                | 38,89                | 60,56                 | 1,042        | 63,03         | 62,506       | 0,0350       | 28,93        | 38,57        | 0,02104      | 67,50         | 1102         | 0,01452      | 66,25        | 1082         |
| 20| 2.25  | 32,79                | 38,35                | 58,54                 | 0,999        | 59,54         | 59,041       | 0,0350       | 28,39        | 37,86        | 0,01768      | 65,00         | 1061         | 0,01216      | 63,75        | 1041         |
| 20| 3.00  | 30,05                | 37,81                | 55,10                 | 0,957        | 56,05         | 55,576       | 0,0350       | 27,86        | 37,14        | 0,01474      | 65,00         | 1061         | 0,01005      | 63,25        | 1020         |
| 20| 3.75  | 27,32                | 37,27                | 51,65                 | 0,915        | 52,57         | 52,111       | 0,0350       | 27,32        | 36,43        | 0,01206      | 63,75         | 1041         | 0,00811      | 62,50        | 1020         |
| 20| 4.50  | 24,59                | 36,73                | 48,21                 | 0,873        | 49,08         | 48,647       | 0,0350       | 26,79        | 35,71        | 0,00956      | 61,66         | 1007         | 0,00440      | 61,66        | 1007         |

### Parameters

- $g_f = 0.070$
- $c = 18,000$
- $g_{el} = 45$
- $0.0350$
- $0.0400$
- $1143$
- $70,00$
- $2016$
- $40,00$
- $15,0$

### Author

Stig Bernander
Downhill progressive slide - triggering load

Case, Appendix I - Slope 1:25, $\arctan 0.04 = 2.29 \, \text{dgr}$

$C_t = 14 \, \text{kN/m}^2$

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<tr>
<th>Step</th>
<th>$x$</th>
<th>$t_e$</th>
<th>$t$</th>
<th>$N$</th>
<th>S:a dn</th>
<th>$x$</th>
<th>$y$</th>
<th>$H+y$</th>
<th>Gradient</th>
<th>$Gr$</th>
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Water level

$W_L$ =

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<tr>
<td>30</td>
<td>39</td>
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</table>

$N_w =$

| 845.0 | 1805.0 |

Safety factor $(F)$

| i.e. $N_{cr}/N_{w}$ | 531/845 | 531/1805 |

| 0.62 | 0.29 |
Downhill progressive slide - triggering load
Case 4, Appendix I - Slope 1:25, arctan 0.04 = 2.29°
$C_r = 14kN/m^2$

Additional earth pressure, N kN/m
Coordinate (x)

Triggering load
Downhill progressive slide - triggering load
Case 4, Appendix II - Slope 1:25, arctan 0.04 = 2.29 deg
Cr = 14 kN/m²

Shear stress (t), kN/m² vs Coordinate (x)

Triggering load

Author: Stig Bernander
Downhill progressive slide - triggering load
Case 4, Appendix III - Slope 1:25, arctan 0.04 = 2.29 dgr
Cr = 14 kN/m²

![Graph showing downhill progressive slide - triggering load](image_url)
**Explanation of denotations**

- \( \frac{dE}{dx} \) (G) = Shear stress before adding current stress increment
- \( x \) = Coordinate from upper boundary condition, where shear stress increment, \( dN \) and \( d(x) = 0 \)
- \( x' \) = Coordinate from upper slope limit as defined by \( L \)
- \( x'' \) = Coordinate from lower slope limit = \( L - x' \)
- \( L \) = Length of the defined slope
- \( Nq/L \) = Overload thicknesses
- \( Nq \) = Overload density
- \( Hq \) = Ok
- \( Nq/L \) = Length of the defined slope
- \( Nq \) = Overload thicknesses
- \( x \) = Coordinate from upper boundary condition, where shear stress increment, \( dN \) and \( d(x) = 0 \)
- \( x' \) = Coordinate from upper slope limit as defined by \( L \)
- \( x'' \) = Coordinate from lower slope limit = \( L - x' \)

Equation 1:

\[
\Delta t(q) \cdot k \cdot \gamma \cdot \left( \frac{(Hn)^2 - ((Hn+1)^2)}{(2\Delta x)} \right) - dH*dx*\gamma*sin \arctan(H33) = 20,7 \text{ kN/m}
\]

Equation 4a

Equation 1a

Equation 4a

<table>
<thead>
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<th>( \Delta t(q) )</th>
<th>( k \cdot \gamma \cdot \left( \frac{(Hn)^2 - ((Hn+1)^2)}{(2\Delta x)} \right) )</th>
<th>( dH<em>dx</em>\gamma*sin \arctan(H33) )</th>
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<tr>
<td>0,667</td>
<td>0,882</td>
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</tbody>
</table>
Author: Stig Bernander

\[
\begin{array}{c|cc}
\text{dE/dx} & G = \\
9,6 & 16,75 & 1428,57 \\
20 & 7000 & \\
Nq/L & N_q & \\
L & 350 & \text{Equ.1:4}
\end{array}
\]

Equ.1:4

Equ.1:4a

Equ.1:1a
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<th>( \frac{dE}{dx} )</th>
<th>( G )</th>
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\( N_q/L \quad N_q \)

\( L \quad 350 \)

**Equ.1:4**

```
```

```
```

```
```

**Equ.1:4a**

```
```

**Equ.1:1a**

```
```

```
```
Author: Stig Bernander

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Equ.1:4

Equ.1:4a

Equ.1:1a
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Equ.I:4

Equ.I:4a

Equ.I:1a
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<td>Nq/L</td>
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**Equ.1.4**

EQU.I:4

EQU.I:4a

EQU.I:1a
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</tr>
<tr>
<td>L</td>
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Equ.1:4

Equ.1:4a

Equ.1:1a
Riverbank stability in loose layered silty clays

Comments on the North Spur Dam at Muskrat Falls in Churchill River, Labrador, Newfoundland.

Stig Bernander and Lennart Elfgren