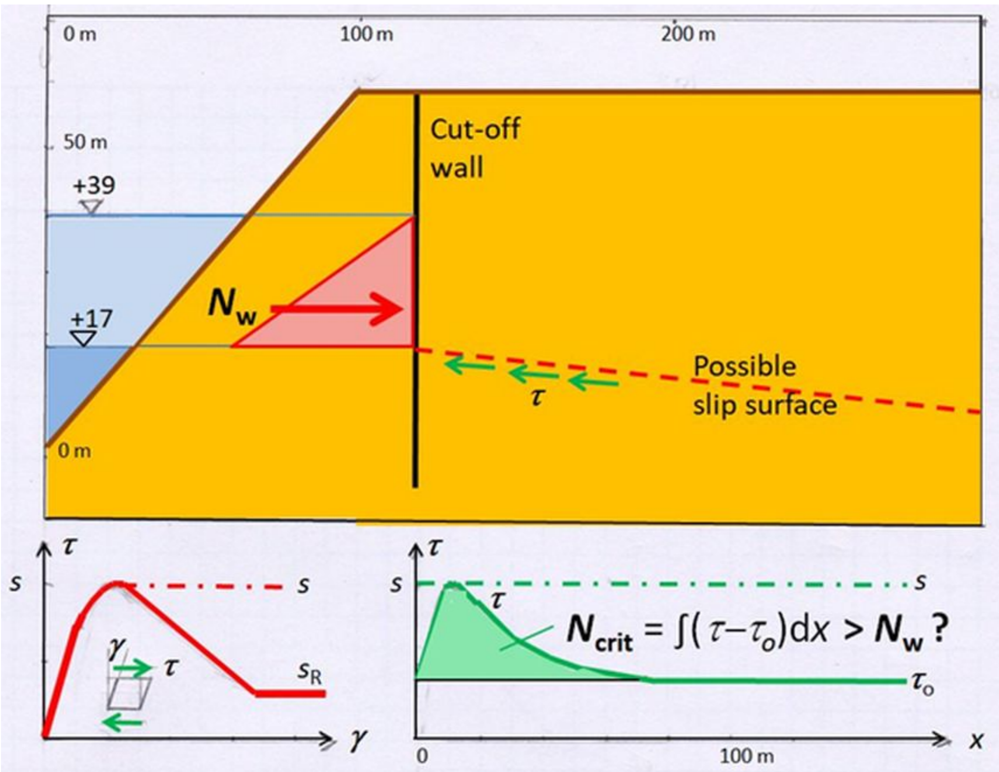
TECHNICAL REPORT

Dam Bank Stability in loosely layered silty sands and lean silty sandy clays

Comments on the risk of failure in the North Spur at Muskrat Falls in the Churchill River Valley, Labrador, Newfoundland



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Cover image:

A schematic view of the North Spur Ridge illustrating one of the stability problems.

When the water level is raised 22 m, from WL= +17 m to WL= +39 m, an immense hydraulic force N_w starts to act on the cut-off wall. The question, among others, is if the shear resistance related to the stress increase $(\tau - \tau_o)$ along a possible slip surface (red dotted line) is big enough to balance the force N_w in the triggering phase of a progressive failure. Here τ_o denotes the in-situ stress before the rise of the water level.

The lower left figure illustrates the material properties of the soil with a shear stress/strain (deformation) (τ/γ) diagram showing a peak shear stress s and a residual shear stress s_R due to strain-softening. The red dotted line indicates the classic plastic Limit Equilibrium Method (LEM) assumption with no deformation-related 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 (γ) and the shear stresses (τ) will rise in accordance with the stress-strain diagram. When the shear stresses approach and pass beyond the maximum value *s,* the soil softens and finally the resistance will be reduced to the residual value s_R close to the wall.

The maximum force increase that the soils behind the wall may resist – applying progressive failure analysis – is $N_{crit} = \int(\tau - \tau_o)dx$, as illustrated in the lower right figure. For the slope to remain stable, N_{crit} must at least balance the force N_w – i.e. the Safety Factor (F_s) being equal to 1,0. Yet, in Soil Mechanics normally a minimum value of $F_s \geq 1.5$ is prescribed. Hence, the main objective of a stability analysis is defining the value of N_{crit} for relevant strain-softening material properties and proper assumptions regarding the initiating failure plane. If unrealistic ideal plastic properties are assumed (green dotted line), there will obviously in many cases, falsely, be no apparent stability problem

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Preface

This report aims to summarize some issues regarding the stability of a dam bank 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 up-to-date analyses based on possible progressive failure formation in the proposed natural dam bank at Muskrat Falls in Churchill River Valley, Labrador/Newfoundland, Canada.

Mölndal and Luleå in July 2017

Stig Bernander   Lennart Elfgren

In this revised version some clarifications and editorial revisions have been made. Moreover Stig Bernander has written a summing up of North Spur stability issues, which has been added as a last appendix.

Mölndal and Luleå in November 2017

Stig Bernander   Lennart Elfgren

Abstract

The differences in landslide analysis between the classic limit equilibrium method (LEM) and a progressive failure procedure is outlined. In LEM the soils are presumed to be fully plastic, whereas in the progressive failure approach the joint effect of strain-softening material properties and deformations in the soil mass are considered.

The risk of failure in the North Spur ridge due to the dam impoundment at Muskrat Falls in the Churchill River Valley (Labrador/Newfoundland) is investigated. An important issue in this context is e.g. that sloping failure surfaces near the cut-off wall (COW) are bound to be much more critical than the horizontal failure planes, which have hitherto been considered according to Nalcor/SNC-Lavalin Engineering Reports.

Results from progressive failure analyses have now been obtained, applying plausible deformation-softening material properties to the soils in the ridge. These results, which are presented at the end of this report, render unsatisfactory safety factors – i.e. lower than 0.5, thus indicating potential risks of failure when the water surface is raised to the proposed levels.

Three reports and a summing up are appended, where Dr Bernander strongly emphasizes the need of stability evaluations based on proper progressive failure analysis – i.e. using soil properties based on tests that are not carried out under fully drained conditions.

Measures to reducing the detrimental effects of high in-situ porosity are also proposed.

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III. Comments on "Progressive Failure Study", 2016-09-15.
Comments by Stig Bernander on the Engineering Report by Nalcor/SNC-Lavalin of December 21, 2015.

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

Case 3: τ_o = 21,1 kPa s = 60 kPa s_R = 12 kPa s/s_R =5 N_{cr} =866 kN/m
Safety factor $F_s = N_{cr} / N_w = 866 / 2420 = 0.357 < 1$

Case 4: τ_o = 41,1 kPa s = 70 kPa s_R = 14 kPa s/s_R = 5 N_{cr} = 521 kN/m
Safety factor $F_s = N_{cr} / N_w = 521 / 2420 = 0.215 < 1$

V. Summing up of North Spur Stability Issues, 2017-10-23
By Stig Bernander, October 23, 2017

Notations

Upper case Roman letters (in alphabetical order)

E = Total earth pressure = E_0 + N (kN/m)

E_0 In-situ earth pressure (kN/m)

$E_{pRankine}$ Critical down-slope earth pressure resistance at passive Rankine failure (kN/m)

F_s^I Safety factor for local failure (N_{cr} / N_q)

F_s^{II} Safety factor for global failure $(E_{pRankine} / E)$

G Secant modulus in shear (GPa)

$H_{x_i \to x_{i+1}}$ Height of element $i \to i + 1$ (m)

K_0 Ratio between minor and major principal stresses

K_p Rankine coefficient for lateral passive earth resistance

L_{cr} Limit length of mobilization of shear stress at N_{cr} (m)

N_{cr} Critical load effect initiating local slope failure (kN/m)

N_q Additional load in the direction of the failure plane (kN/m)

N Earth pressure increment due to additional load (kN/m)

V_p Volume of pores

V_s : Volume of solids

Lower case Roman letters (in alphabetical order)

b Width of element considered (m)

s, s_u Un-drained peak shear strength (also sometimes denoted S, S_u, c, c_u) (kPa)

s_R Residual shear resistance (also sometimes denoted S_R, c_R) (kPa)

e = V_p / V_s = void ratio

g Gravity (9,81 m/s^2)

m mass (kg)

n = $V_p / (V_p + V_s)$ = porosity

q Additional vertical load (kN/m^2)

w water content (= $e \cdot \rho_w / \rho_s$)

Greek letters (in alphabetical order)

β Slope gradient at coordinate x (°)

$\gamma(x,z)$ 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_τ Down-slope displacement in terms of deviator deformation (m)

ρ Soil density (kg/dm^3)

υ Poisson coefficient

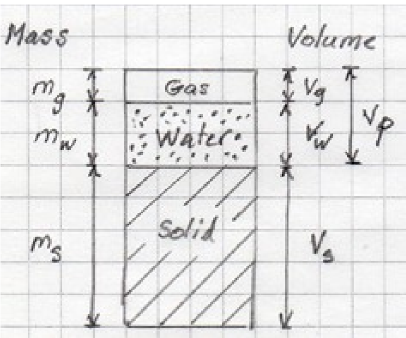
τ_{el} Shear stress at elastic limit (kPa)

$\tau, \tau(x,z)$ Total shear stress in section x at elevation z (kPa)

$\Delta\tau_{x_i \to x_{i+1}}$ Shear stress increment from step i to $i+1$ (kPa)

$\tau_0(x,z)$ In situ shear stress in section x at elevation z (kPa)

Masses, Volumes and Ratios



Void ratio $e = V_p/V_s$

Porosity $n = V_p/(V_p + V_s)$

Water content $w = m_w / m_s = e \cdot \rho_w/\rho_s$

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.

Figure 1.1. Map of the Northern Hemisphere with Churchill River in Canada and Luleå in Sweden marked with red circles,

1. Glacial sediments

The location of the studied riverbank at Muskrat Falls in Churchill River Valley 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 massive granite rock close to the falls, the Rock Knoll. Section A denotes a studied part of the ridge. Dury (2017).

2. Soil properties

Glacial soils (e.g. quick lean clayey sands and porous silty sands) may be extremely sensitive and even liquefy when remoulded. In tests the clays exhibit a peak strength, after which the soil structure may collapse leading to a corresponding reduction of shear resistance.

A typical deviatory stress/shear strain relationship for a sensitive deformation-softening clay is shown in Figure 2.1, Bernander et al. (1981 → 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 and sensitive soils. In the figure, the case with an ideal plastic behavior is indicated with a dotted blue line. Full plasticity along lengthy failure planes is taken for granted in the classic simplified limit equilibrium method (LEM) that is often used for slope stability analysis.

However, there may be considerable deformation-softening – i.e. even liquefaction – not only in silty clays but also in silty sandy soils, where the inter-particle friction plays a greater role than the cohesion, Terzaghi et al (1996).

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

The soil layers in the studied North Spur 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 underneath constitute a heterogeneous mix of clays, porous silts and sands from marine and estuarine deposits named the Stratified Drift. The lower clay layer is located below the stratified drift and is mainly clay of low to medium plasticity. In the studied section A in Figure 1.2, the soil layers are slightly inclined - sloping downwards about 4/100 from the upstream side of the ridge towards the downstream side.

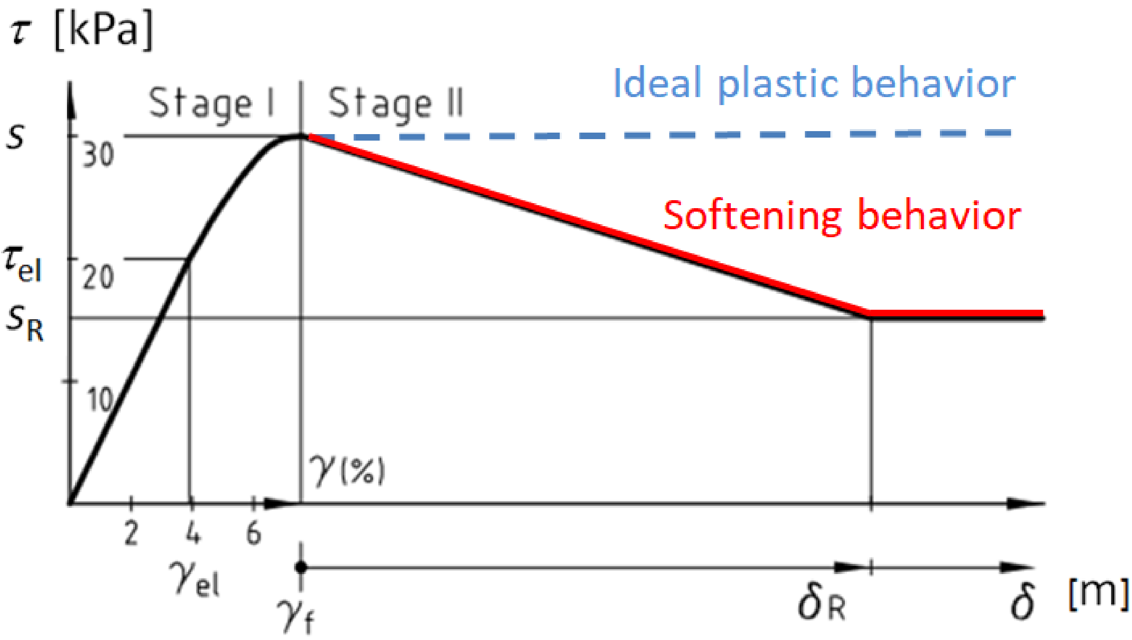


Figure 2.1. Stress-strain (τ/γ) and stress-deformation (τ/δ) relationships in a typical deformation softening clay. The full red line indicates a deformation-softening behavior while the blue dotted line indicates presumed ideal plastic behavior. Stage I is the condition before τ reaches $\tau_{max} = s$. Stage II forms the subsequent deformation-softening development with a final residual shear strength of s_R. The ratio s_R/s is a measure of the sensitivity of the soil.

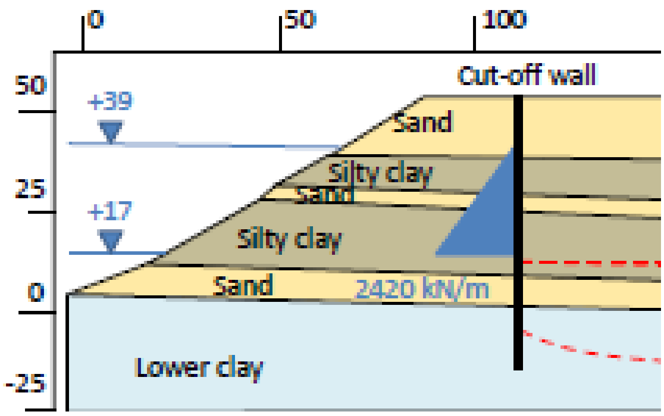


Figure 2.2. Different soil layers in the studied North Spur ridge at Muskrat Falls as shown in Section A of Figure 1.2, Dury (2017).

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 $s_R \approx 17$ kPa 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}, i.e. the sensitivity 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 just assumed.

Table 2.1 shows that there are soil layers – both in the Upper Silty Clay and in the Lower Marine Clay formation - with marked risk of liquefaction or massive loss of residual shear resistance – and in which critical failure surfaces may develop. (Confer Appendix II and III).

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

Property	**Upper Silty Clay**			**Lower Marine Clay**		
	General range	**Average**	**No. of tests**	**General Range**	**Average**	**No. of tests**
Percent finer than 2 microns	35 - 45	-	19	15 - 35	-	-
Water content, *w* %	17 - 43	31	199	17 - 45	29	201
Liquid limit, *LL* %	17 - 43	30	168	22 - 48	37	123
Plastic limit, *PL*%	13 - 22	19	168	13 - 27	21	123
Plasticity index *PI= LL-PL*%	2 - 22	11	168	7 - 25	16	123
Liquidity index. *LI* = (*w-PL*)/(*LL-PL*)	0,6 - 2,8	1,3	168	0,1 - 2	0,6	123
Intact undrained shear strength, S_u, kPa	35 - 135	-	-	53 - 200	-	-
Remolded undrained shear strength, S_{ur}, kPa	2 - 60	-	-	8 - 96	-	-
Sensitivity, in situ, $S_t = S_u/S_{ur}$	1 - 36	10	43	2 - 11	4	35
Large strain friction angle ϕ'_{cv},°	30 - 32	-	-	33	-	-
Effective cohesion, *c'*, kPa	0 - 10	-	-	6	-	-
Salt content, g/l	0,8 - 1,5		-	8 - 22	-	8
Unit weight, γ, kN/m^3	18,4 - 19,7	-	11	19,2 -19,5	-	3
Hydraulic conductivity, *k*, m/s	10^{-7} – 10^{-9}	-	-	10^{-7} – 10^{-9}	-	-

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 modified by Casagrande (1902-1981), see Terzaghi et al. (1996).

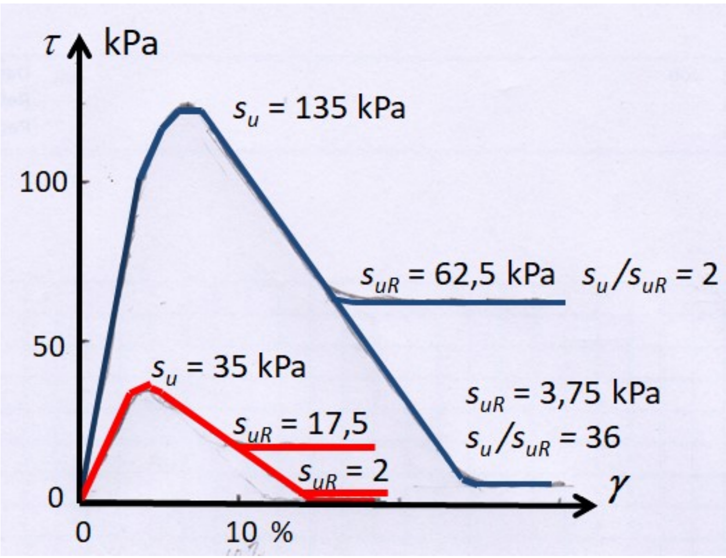


Figure 2.3. Possible varieties in stress-strain relationships for the Upper Silty Clays in the North Spur based on Table 2.1 from Leahy (2015). As no deformation properties are given, the inclinations of the curves are guessed.

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 – based on clearly defined disturbance conditions, Bernander (2011).

A traditional prediction of failure in a long slope is shown in Figure 3.1. As long as the mean shear stress τ in a possible failure surface is smaller than the maximum shear capacity *s* the slope is regarded as being safe. However, to be quite safe, i.e. in terms of progressive failure, the applied total shear stress (i.e. $\Delta\tau(N_q) + \tau_0$) must not exceed the residual strength s_R in the triggering phase of a landslide. Confer Figure 2.1 and 2.3.

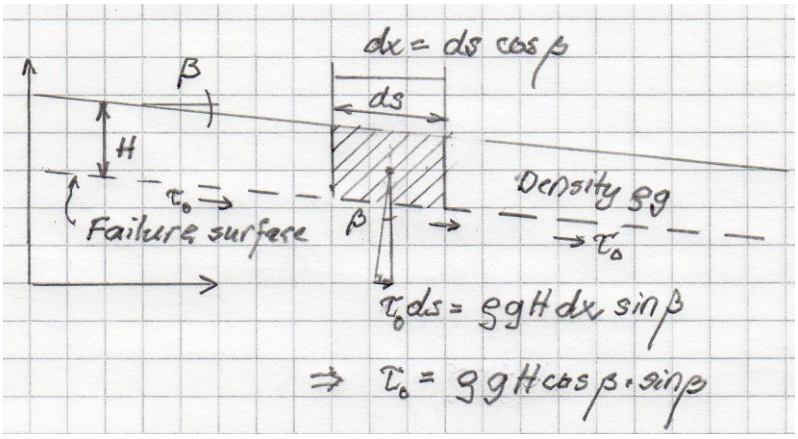


Figure 3.1. Slope analysis. For a density ρg = 18 kN/m^3, a height *H* = 40 m and a slope with tanβ = 0,04 we obtain $\tau_o = \rho g \cdot H \cdot \cos\beta \cdot \sin\beta$ = 18·40·0,0399 = 26,7 kPa, which together with a rising water pressure may occasionally be higher than the maximum shear stress *s* and for most of the time higher than 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, i.e. spreads and flow-slides, 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 (COW) are constructed to prevent water seepage through the slope.

Yet, vitally, forward and downhill progressive landslide development due to the dam impoundment pressure on the soils behind the cut-off-wall, (the COW), have only, as far as is known, been studied presuming horizontal failure surfaces, Leahy (2015).

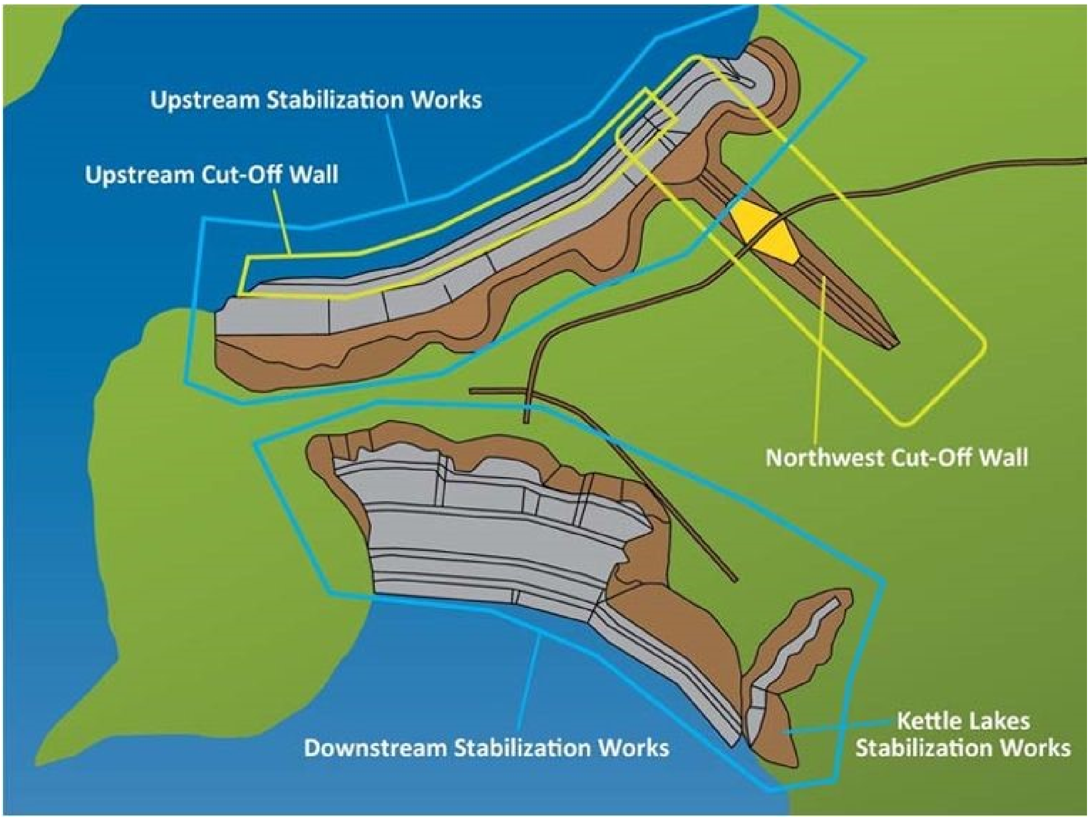


Figure 3.2. Stabilization work carried out to mitigate retrogressive upward slides, Leahy (2015), Caballos (2016).

4. Progressive Failure Analysis

In the progressive failure approach, the joint effect of strain-softening material properties and the simultaneous deformations due to additional loading in the soil mass are considered.

A critical condition in this context arises if the shear stresses generated by the rising water level due to the impoundment exceeds the residual shear resistance of the soils just downstream of the COW.

Another important issue is that a sloping failure surface near the cut-off wall (COW) is bound to be much more critical than the horizontal failure planes, which have been considered according to the Nalcor/SNC-Lavalin Engineering Reports, Leahy (2015), Caballos (2016).

Results from progressive failure analyses have now been obtained, applying plausible deformation-softening material properties to the soils in the ridge. The safety factors, which are presented at the end of this report, are unsatisfactory – i.e. being lower than 0.5 and thus indicating potential risks of failure when the water surface is raised to the intended levels.

The case is illustrated in Figures 4.1 to 4.4. The load increases with N_w when the water level is raised with ΔH = 22 m from +17 m to +39 m:

$$N_w = 0,5 \cdot \gamma_w \cdot \Delta H^2 = 0,5 \cdot 10 \cdot 22^2 \text{ kN/m} = 2420 \text{ kN/m}$$

Hence, when the hydraulic pressure load N_w gradually increases, additional shear stresses will develop along possibly sloping slip surfaces. The stresses will initially be highest close to the cut-off wall but may, approaching and passing the peak stress, fall below the residual resistance, thus entering a virtually dynamic phase. Thus, even presuming a gently sloping failure plane – a total landslide failure can be released.

The shear stresses ($\tau_0+\tau$) can be calculated using the progressive failure analysis developed by S. Bernander (1981 → 2017). The calculations are then based on different assumptions regarding material properties and failure plane geometry. This will be further commented on in section 5.

In Appendices I-III, arguments are given for the need of up-to-date progressive failure analyses of the stability of the proposed dam bank at Muskrat Falls. Importantly, the effects of the crucially decisive relationship between the *current* porosity and the *critical* porosity of a water saturated soil layer is discussed.

The stabilizing works on the shores that are in progress may counteract *retrogressive* spreads and *upwards* slides but, according to the analyses made, the *central core* of *the ridge* may still be susceptible to landslide failure. The highly varying properties of water saturated soil layers in the North Spur constitute a definite risk of potential failure.

Conclusion: the soils behind and near the COW will be subject to an immense additional load. The peak shear strength is here bound to be exceeded, and the related large deviatory deformations may, acting along a sloping failure surface, very likely trigger a progressive failure development resulting in a global landslide disaster.

Analyses by Robin Dury (2017) and Stig Bernander et al. (2017) have recently been carried out showing that the issue ought to be thoroughly investigated. Cf Appendix IV.

Only when using the most favourable material properties (i.e.s_u = 135 kPa & s_u/s_R < 4) in Table 2.1, the calculations indicate that the ridge may stay stable. However, for material properties in the lower range in Table 2.1 and Figure 2.3, the critical load N_{crit} will be significantly lower than the applied load N_w, and a failure will occur under a wide range of circumstances presumed.

Applying the 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 $N_q = N_w$ = 2420 kN/m. This is more than twice of what the ridge may stand under the conditions assumed.

Two analyses using Bernander's original spreadsheet are enclosed as Appendix IV, also showing low safety factors, similar those derived from the calculations by Dury (2017).

For a case with an in situ shear stress τ_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 kN/m and a safety factor $F = N_{\mathrm{cr}} / N_w$ = 866 / 2420 = 0,357 < 1.

In another case with a higher in situ shear stress τ_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 kN/m and a safety factor $F = N_{\mathrm{cr}} / N_w$ = 521 / 2420 = 0,215 < 1.

More material tests are necessary to establish the real deformation properties of the soils in the ridge. Stabilization work (e.g. compaction) may be needed to eliminate landslide risk.

The issue is treated in Appendix III, Section 5.8 and a procedure is proposed on how to check the material properties and how to compact the soil making it less prone to liquefaction.

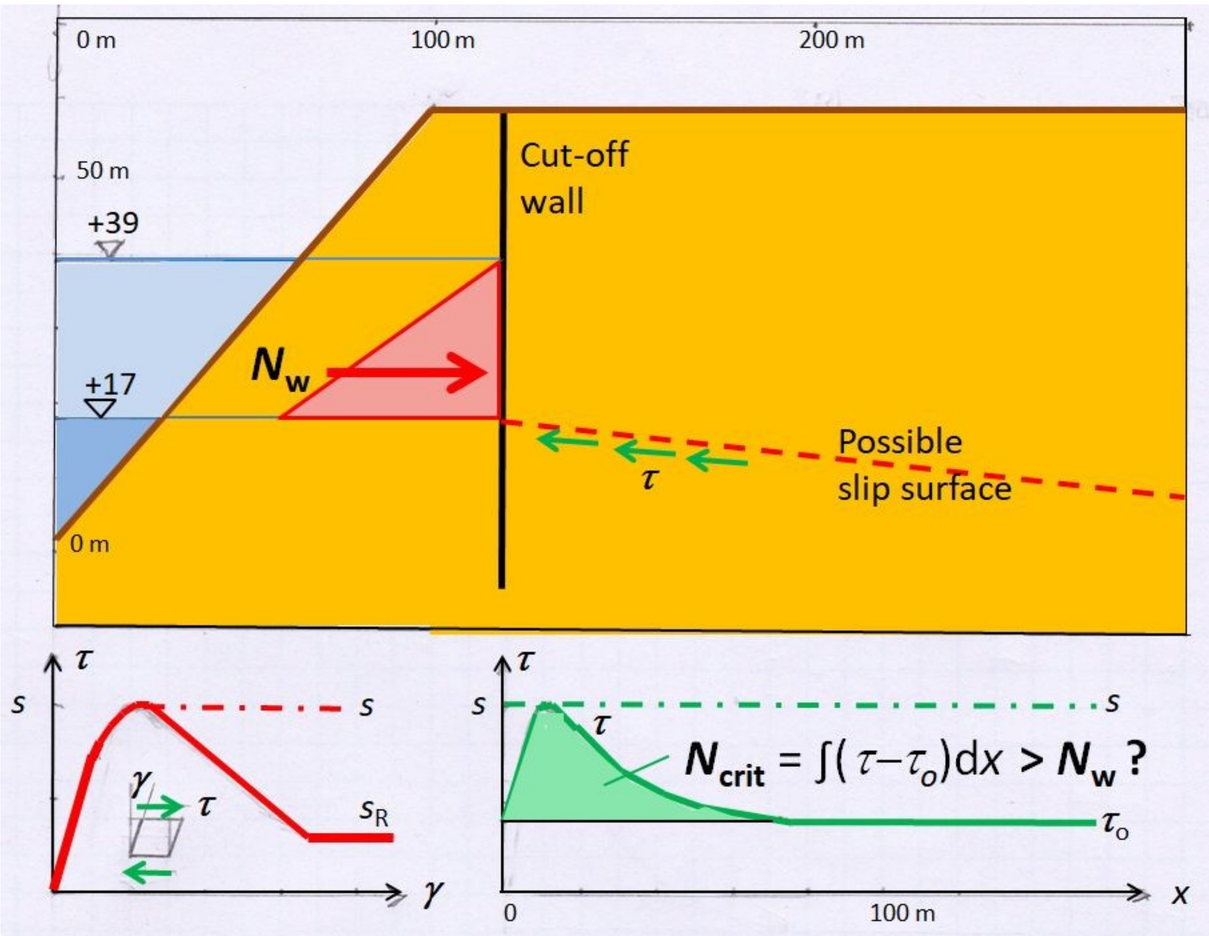


Figure 4.1. Schematic drawing of a section through the dam. When the water level is raised 22 m, from +17 m to +39 m, a force N_w starts to act on the cut-off wall. The question is if the resulting shear resistance τ - τ_o along a possible slip surface suffices to resist the effects of the hydraulic force N_w with an adequate value of the safety factor. Here, τ_o denotes the in-situ prior to impoundment.

The lower left figure illustrates the material properties of the soil based on a shear stress/strain (τ/γ) diagram with a maximum shear stress s and a residual softened shear stress s_R. The dotted line indicates the classic ideal plastic (LEM) assumption of no strain-softening reduction of the shear strength. When the force N_w starts to act on the cut-off wall, the soil behind the wall is deformed (γ) and shear stresses (τ) will be growing according to the stress-strain diagram. When the shear stresses reach and pass the maximum value s the soil material softens, and the resistance is finally being 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. For the slope to remain stable, N_{crit} must be at least as equal to N_w. Calculating N_{crit} for varying material properties is the main objective of the stability analysis.

If unrealistic ideal plastic properties are assumed (green dotted line), there will obviously in many cases, falsely, be no apparent stability problem.

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Figure 4.2. Muskrat Falls Hydro Facilities with the North Spur to the right, SNC Lavalin (2017). http://muskratfalls.nalcorenergy.com/wp-content/uploads/2017/01/North-Spur-Information-Session-Presentation__Jan-2017_Website-posting.pdf

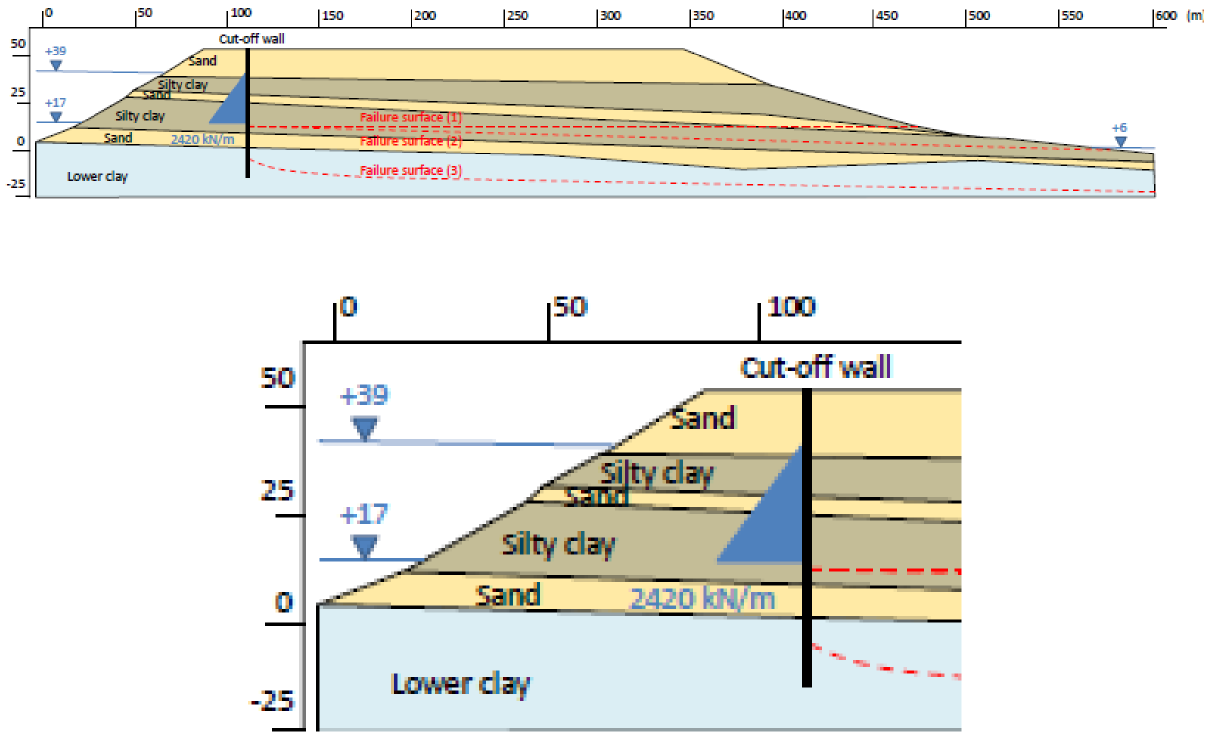

Figure 4.3. Section of the North Spur and location of the assumed failure planes, one horizontal and one inclined in the lower of the two silty clay layers and one curved in the lower clay layer. Dury (2017).

Figure 4.4. The North Ridge during work designed to stabilize the riverbanks, SNC Lavalin (2017).

Figure 4.5. Stabilization of the downstream riverbank, August 2016, https://muskratfalls.nalcorenergy.com/newsroom/photo-video-gallery/muskrat-falls-construction-august-2016/

5. Different Phases in a Progressive Failure

A method for progressive failure analysis has been developed by Stig Bernander et al. (1978 → 2016). When an additional load ΔN 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 location of the force ΔN and abate further downslope. After the shear stresses τ have reached the maximum value *s,* they abate, see Figure 2.1, and the shear resistance further downslope must be engaged to equilibrate ΔN. The mechanism can be studied with a finite difference method, where local downhill deformations $\Delta\delta_N$ caused by normal forces ΔN are maintained compatible with the deviatory shear deformations $\Delta\delta_\tau$ above – and when applicable also below – the potential failure surface, see Figure 5.1.

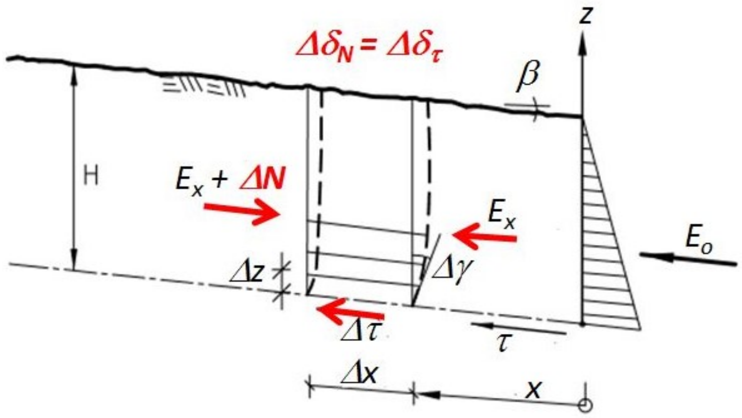

Figure 5.1. Principle of finite difference method (FDM) where deformations $\Delta\delta_N$ due to the normal forces ΔN are kept compatible with deformations $\Delta\delta_\tau$ caused by shear stresses $\Delta\tau$. The in-situ pressure E_o may vary widely along the slope.

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. (1984 → 2016) and Dury (2017).

Phase 1.The in-situ stress in this exemplification is τ_0 = 20,8 kPa. The slope has an inclination of 6,5/100 (corresponding to an angle β = 3,287°) to the left but turns horizontal further to the right, (Moment a)

Phase 2: A load q is applied giving τ = s = c = 30kPa. The shear stresses can be integrated to the force N_q =189 kN/m for an influence length L_b = 85,5m. (Moment b)

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 sustain without a local failure being triggered. The safety factor against a local failure, possibly triggering a progressive landslide, will thus be $F_s = N_{cr} / N_q$. (Moment c)

Phase 3 continued: If N_q exceeds N_{cr} (i.e. for F_s < 1), an unstable dynamic phase is released. In the example the residual value is reduced to $s_R = c_R$ = 15 kPa and the maximum load that can possibly be resisted is reduced to N = 215 kN/m for an influence length of L_d = 99,7 mm. (Moment d).

The negative shear stresses may balance the positive so that N is 0 at the point of application. Unbalanced uphill loads are dynamically transmitted further downslope until a new condition of equilibrium may build up due growing passive earth pressure resistance in less sloping ground. (Moment e).

End of Phase 3, Phase 4 (& 5): The in situ stresses τ_o decrease from L = 150 m where, in this case, the ground becomes horizontal. The additional earth pressure N is now caused by the weight of the totally sliding soil mass, i.e. $N = LH\rho g \cdot \sin\beta$ minus the effects of the residual shear stress $s_R = c_R$, which is likely to be strongly reduced due fast slip in the failure surface. In less sloping ground (which in the current case is horizontal) the downhill active force may, permanently or "temporarily", be balanced by developing passive earth pressure. Thus, if the active force $(E_0 + N)_{max}$ remains less than the maximum passive resistance E_p – as in the

currently studied case − the masses will stop moving, i.e. only resulting in a minor displacements. Yet, the failure plane tends to develop far under the non-sloping ground before equilibrium is reached. (Moment f).
However, if on the other hand (E_0 + N)$_{max}$ exceeds the passive resistance $E_{p,}$ a collapse will occur. This condition is named Phase 5 and constitutes what we actually understand as being a 'landslide'.

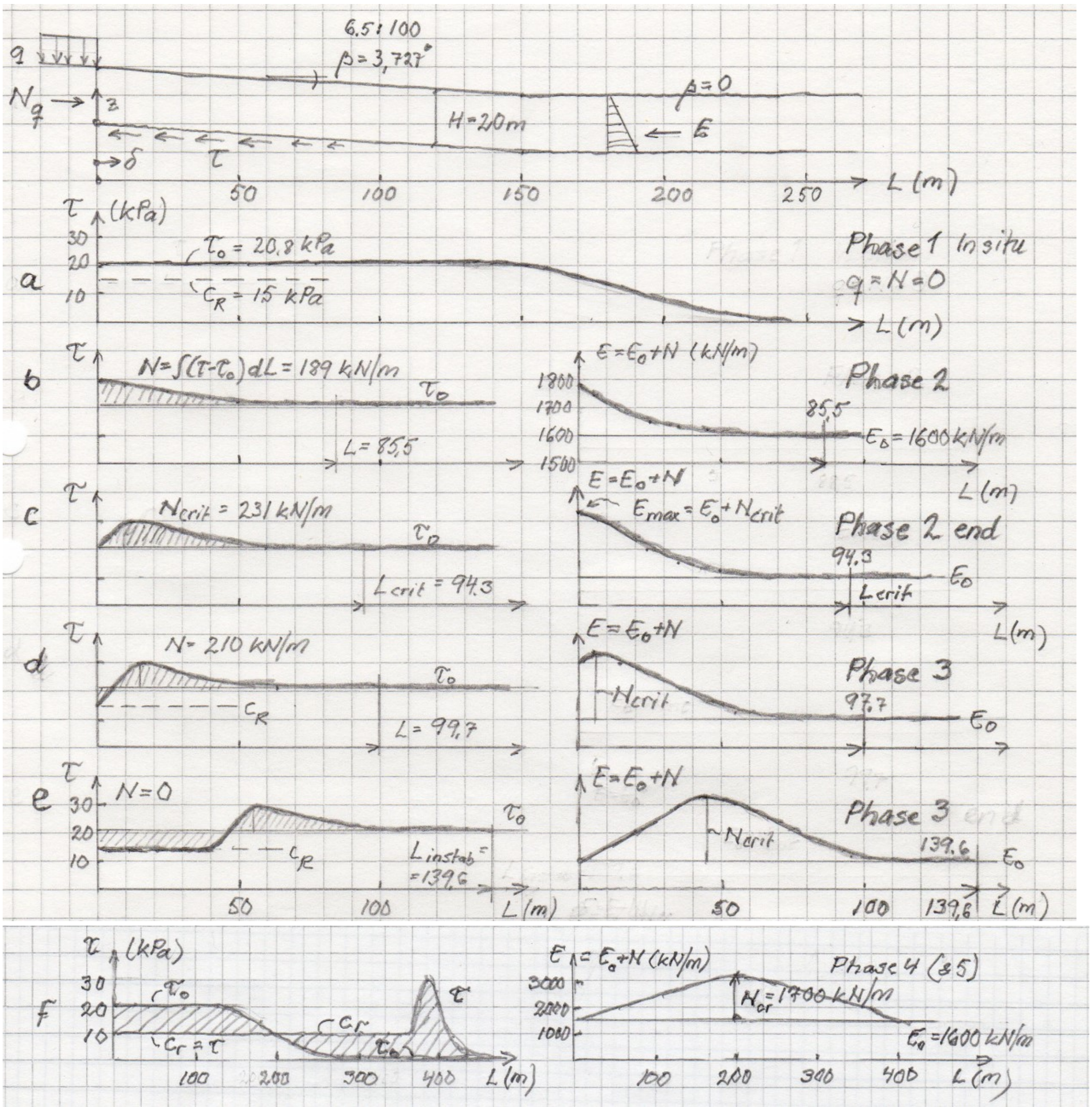

Figure 5.2. Five phases 1-5, and six moments a-f in a Progressive Failure Analysis of an idealised slope with an inclined surface, Bernander et al. (2011, 2016), Dury (2017). Note that the scale in the diagram for moment (f) is different from the scales moments (a-e.)

The safety factor for a **fully developed** global failure will be $F_s = E_p/(E_o + N)$.
The total earth pressure $E = E_0 + N$ for the different moments are given in Figure 5.3.
In the North Spur case, final failure occurs at end of Phase 2, when dynamic Phase 3 is initiated.

In the North Spur, there is namely no possibility for a second stage of equilibrium once progressive failure has been triggered along a sloping failure plane. The eastern slope of the ridge ends in a 70 m deep whirlpool downstream of Muskrat Falls.
The global safety factor F_s is thus not relevant in the North Spur case, where only the safety factor F_s for a triggering local failure near the COW (at the end of Phase 2) is of any importance.

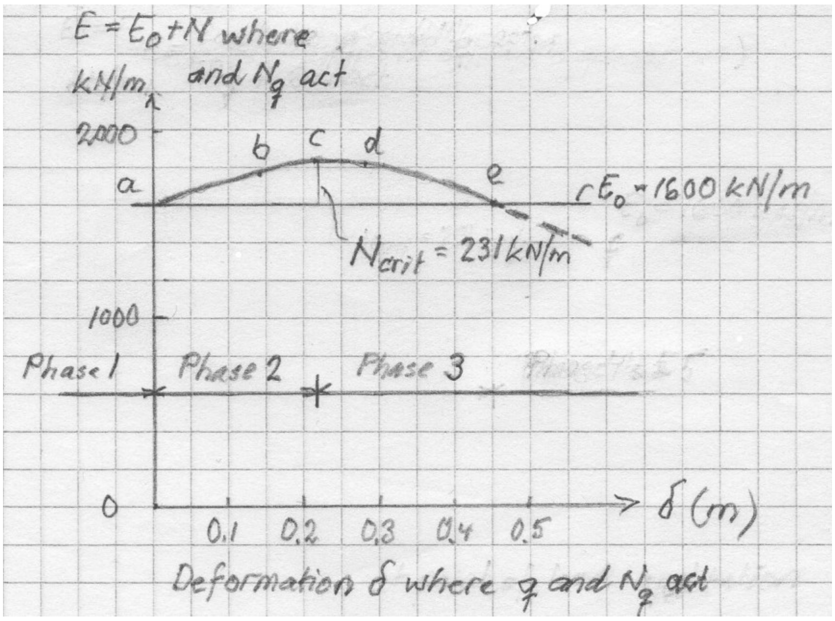

Figure 5.3. The total earth pressure $E = E_0 + N$ as function of the deformation δ at the point of application of q and N during the moments a – e. Bernander et al. (2016).

The principal features of progressive failure analysis are also treated in e.g. Quinn (2009), Gylland (2015), Locat et al. (2011, 2013, 2015), Wang and Hawlader (2017) and in the workshop proceeding L'Heureux et al. (2013) and Thakur et al. (2017).

6. Conclusions

Progressive failure analyses have been performed according to a finite difference method developed by Stig Bernander (1981→ 2017). The development of a simplified spreadsheet by Robin Dury (2017) has allowed getting numerical results for a great number of studies, based on a wide range of data assumptions.

For the 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 the assumed properties.

More material tests are necessary to establish the true 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 sensitive soil layers to making them less prone to liquefaction.

7. References

Axelsson, Kennet & Matsson, Hans (2016). *Geoteknik* (Soil Mechanics and Foundation Engineering. In Swedish). Lund: Studentlitteratur, 464 pp, ISB 978-91-33-08072-7.

Bernander, Stig (1978). Brittle Failures in Normally Consolidated Soils. *Väg- & Vattenbyggaren (Stockholm), No 8- 9, pp 49-52.* Available at http://ltu.diva-portal.org/

Bernander, Stig & Olofsson, Ingvar (1981). On formation of progressive failure in slopes. *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, (ICSMFE),* Stockholm 1981. Vol 3, 11/6, pp 357-362. Available at http://ltu.diva-portal.org/

Bernander, Stig & Gustås, Hasse (1984). Dynamisk studie av ett progressivt brott I en naturlig slant. (A Dynamic Study of Downward Progressive Failure in a Natural Slope. In Swedish), *Proc. Nordic Geotechnical Meeting, Linköping*, Sweden, pp 431- 442. Available at http://ltu.diva-portal.org/

Bernander, Stig; Gustås, Hasse & Olofsson, Jan (1984). Consideration of In-situ Stresses in Clay Slopes with Special Reference to Progressive Failure Analysis. *Proc. IVth International Symposium on Landslides (ISL1984),* Toronto, Canada. 6 pp. Available at http://ltu.diva-portal.org/

Bernander, Stig (1985). On Limit Criteria for Plastic Failure in Strain-rate Softening Soils. *Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE),* San Francisco, Balkema, Vol. 1/A/2, pp 397 – 400. Available at http://ltu.diva-portal.org/

Bernander, Stig; Svensk, Ingvar; Holmberg, Gunnar & Bernander, Jarl (1985). Shear strength and deformation properties of clays in direct shear tests at high strain rates. *Proceedings of the 11th International Conference on Soil Mechanics and , Foundation Engineering (ICSMFE),* San Francisco, Balkema, Vol. 2/B/5, pp 987 – 990. Available at http://ltu.diva-portal.org/

Bernander, Stig; Gustås, Hans & Olofsson, Jan (1989). Improved Model for Progressive Failure Analysis of Slope Stability. *Proceedings of the 12th International Conference on Soil Mechanics and Foundation Engineering, (ICSMFE),* Rio de Janeiro 1989. Vol 3, 21, pp 1539-1542. Available at http://ltu.diva-portal.org/

Bernander, Stig (2000). Progressive Landslides in Long Natural Slopes. Formation, potential extension and configuration of finished slides in strain-softening soils. *Licentiate Thesis 2000:16*, Luleå University of Technology, Available at http://ltu.diva-portal.org/

Bernander, Stig (2008). Down-hill Progressive Landslides in Soft Clays. Triggering Disturbance Agents. Slide Prevention over Horizontal or Gently Sloping Ground. Sensitivity related to Geometry. *Research Report 2008:11*, Luleå University of Technology, ISSN: 1402-1528, 16+101 pp. Available at http://ltu.diva-portal.org/

Bernander, Stig (2011). Progressive landslides in long natural slopes. Formation, potential extension and configuration of finished slides in strain-softening soils. *Doctoral thesis,* Lulea University of Technology, 252 pp, ISBN 978-91-7439-283-8. Available at http://ltu.diva-portal.org/

Bernander, Stig, Kullingsjö, Anders; Gylland, Anders S; Bengtsson, Per Erik; Knutsson, Sven; Pusch, Roland; Olofsson, Jan & Elfgren, Lennart (2016). Downhill progressive landslides in long natural slopes: triggering agents and landslide phases modelled with a finite difference method. *Canadian Geotechnical Journal*, Vol. 53, No. 10, pp. 1565-1582, dx.doi.org/10.1139/cgj-2015-0651

Bernander, Stig; Dury, Robin; Laue, Jan; Knutsson, Sven & Elfgren Lennart (2017): Progressive Landslide Analysis in Canadian Glacial Silty Clay in Churchill River. Poster at the *2nd International Workshop on Landslides in Sensitive Clays, IWLSC* Trondheim, June 2017, 1 p. Available at https://ltu.diva-portal.org/

Ceballos, Alvaro (2016): *North Spur Stabilization Works – Design Report. Lower Churchill Project.* Engineering Report. SLI Document No. 505573-3281-4GER-0601-PS.Nalcor Reference No. MFA-SN-CD-2800-GT-RP-004-01 Rev. B1. Date 30 Jan. 2016. Verified by Regis Bouchard and approved by Greg Snyder, 264 pp. Available at http://muskratfalls.nalcorenergy.com/wp-content/uploads/2013/03/North-Spur-Stabilization-Works-Design-Report-Jan-2016-Final-2.pdf

Dury, Robin (2017). Progressive Landslide Analysis. *MSc Thesis*, Luleå University of Technology, Luleå, Sweden. 65 pp. Available at https://ltu.diva-portal.org/smash/get/diva2:1117330/FULLTEXT02.pdf
The spreadsheet used in the analysis is available at http://ltu.diva-portal.org/smash/get/diva2:1117330/ATTACHMENT01.zip

Dury, Robin; Bernander, Stig; Kullingsjö, Anders; Laue, Jan; Knutsson, Sven; Pusch, Roland & Elfgren, Lennart (2017). Progressive Landslide Analysis with Bernander Finite Difference Method. Poster at the *2nd International Workshop on Landslides in Sensitive Clays, IWLSC* Trondheim, June 2017, 1 p. Available at https://ltu.diva-portal.org/

Gylland, Anders. S. (2012). *Material and slope failure in sensitive clays*. Trondheim: Norwegian University of Science and Technology, Department of Civil and Transport Engineering, *Doctoral Thesis 2012:352*, 238 pp.

Leahy, Denise (2015). *North Spur Stabilization Works. Progressive Failure Study. Lower Churchill Project,* Engineering Report, SNC-Lavalin, Nalcor, SLI Document No. 505573-3281-4GER-0001-01, Nalcor Reference No. MFA-SN-CD-2800-GT-RP-0001 Rev B2, 21 Dec 2015, Verified by Regis Bouchard and approved by Greg Snyder .128 pp. Available at https://muskratfalls.nalcorenergy.com/wp-content/uploads/2016/01/North-Spur-Stabilization-Works-Progressive-Failure-Study.pdf

Leahy, Denise; Bouchard, Regis Leroueil, Serge. (2017). *Potential Landslide at the North Spur, Churchill River Valley*. In “Landslides in Sensitive Clays. From Research to Implementation” Ed. by Thakur, V., L’Heureux, J.-S. & Locat, A., Cham: Springer, pp 213-223. ISBN 978-3-319-56486-9.

Locat, Ariane, Leroueil, S., Bernander, S., Demers, D., Jostad, H.P.& Ouehb, L. (2011). Progressive failures in eastern Canadian and Scandinavian sensitive clays. *Canadian Geotechnical Journal*, 48(11): 1696cal Joudoi:10.1139/t11-059.

Locat, Ariane, Jostad, H.P. & Leroueil, S. (2013). Numerical modelling of progressive failure and its implications for spreads in sensitive clays. Canadian Geotechnical Journal, 50(9), 961-978, doi:10.1139/cgj-2012-0390.

Locat, Ariane, Leroueil, S., Fortin, A., Demers, D. & Jostad, H.P. 2015. The 1994 landslide at Sainte-Monique, Quebec: geotechnical investigation and application of progressive failure analysis. Canadian Geotechnical Journal, 52(9), 490-504, doi:10.1139/cgj-2013-0344

L´Heureux, Jean-Sébastian, Locat, A., Leroueil, S., Demers, D. & Locat, J., Editors (2013). *Landslides in Sensitive Clays. From Geoscience to Risk Management*. Springer, 418 pp, ISBN 978-94-007-7078-2.

Quinn, Peter E. (2009). Large Landslides in Sensitive Clay in Eastern Canada and the Associated Hazard Risk to Linear Infrastructure. *PhD Thesis, Department of Geological Sciences and Geological Engineering, Queen's University, Kingston, Ontario, Canada. April 2009, 465 pp.* Available at: http//qspace.library.queensu.ca/handle/1974/1781

SNC-Lavalin (2017), Lower Churchill Project. North Spur Information Session, January 2017, Slide Presentation available at http://muskratfalls.nalcorenergy.com/wp-content/uploads/2017/01/North-Spur-Information-Session-Presentation Jan-2017_Website-posting.pdf

Terzaghi, Karl; Peck, Ralph B. & Mesri, Gholamreza (1996). *Soil Mechanics in Engineering Practice*, 3rd Edition. New York, Wiley, 592 pages, ISBN: 978-0-471-08658-1. 2nd Edition 1976, 1st Edition 1948..

Thakur, Vikas, L'Heureux, J.-S. & Locat, A., Editors (2017). *Landslides in Sensitive Clays. From Research to Implementation*. Proceedings from the 2nd International Workshop on Landslides in Sensitive Clays, IWLSC Trondheim, 12-14, June 2017, Cham: Springer, 604 pp, ISBN 978-3-319-56486-9.

Wang, Chen and Hawlader, Bipul (2017). Numerical Modeling of Three Types of Sensitive Clay Slope Failures. Proceedings of the 19th International Conference on Soil Mechanics and Geotechnical Engineering, ICSMGE, Soeul, September 17-22, 2017, Technical Committee 103, pp 871-874. Available at: https://www.issmge.org/uploads/publications/1/45/06-technical-committee-03-tc103-44.pdf

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

Comments on the Engineering Report by Nalcor/SNC-Lavalin of December 2015 prepared for Grand Riverkeeper, Labrador, Inc. by Stig Bernander 15 September 2016,.

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)

V. Summing up of North Spur stability issues, 2017-10-23

By Stig Bernander, October 23, 2017

LOWER CHURCHILL RIVER RIVERBANK STABILITY REPORT

PREPARED FOR

Grand Riverkeeper Labrador, Inc.

BY

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October 14, 2015

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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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APPENDICES

REPORT

On Specific questions regarding the formation of the Churchill River Valley and Comments on stability issues related to the North Spur.

1. General

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

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

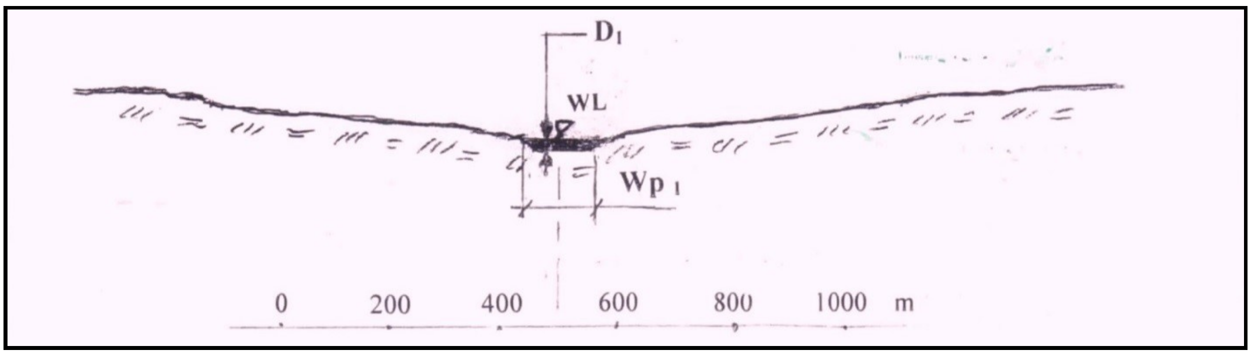


Figure 1.1 Riverbed formation under normal geological conditions, i.e. stable soils with little tendency to liquefaction or weakness in any specific sedimentary layer.
Denotations: Wp_1 = Riverbed width (≈ Wet perimeter),
D = Mean water depth (WL = Water level).

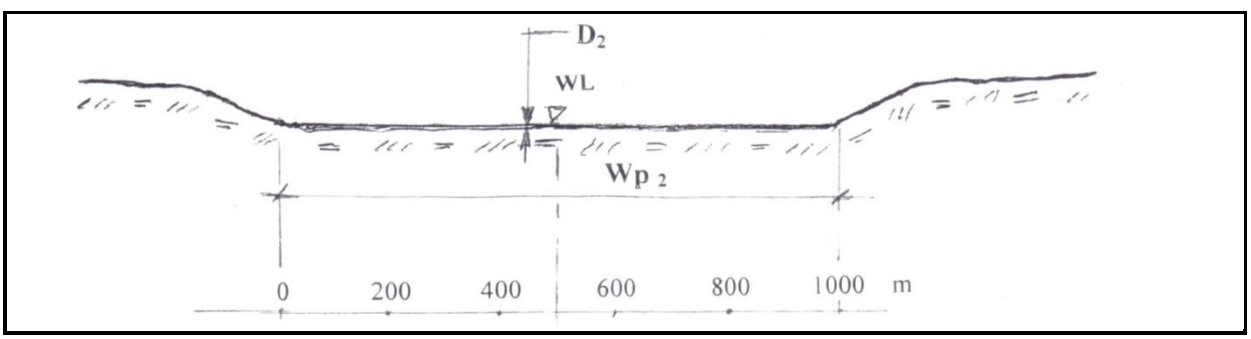


Figure 1.2 Riverbed formation of the type occurring in the Churchill River valley with a remarkably wide but shallow riverbed. Denotations as above.

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.

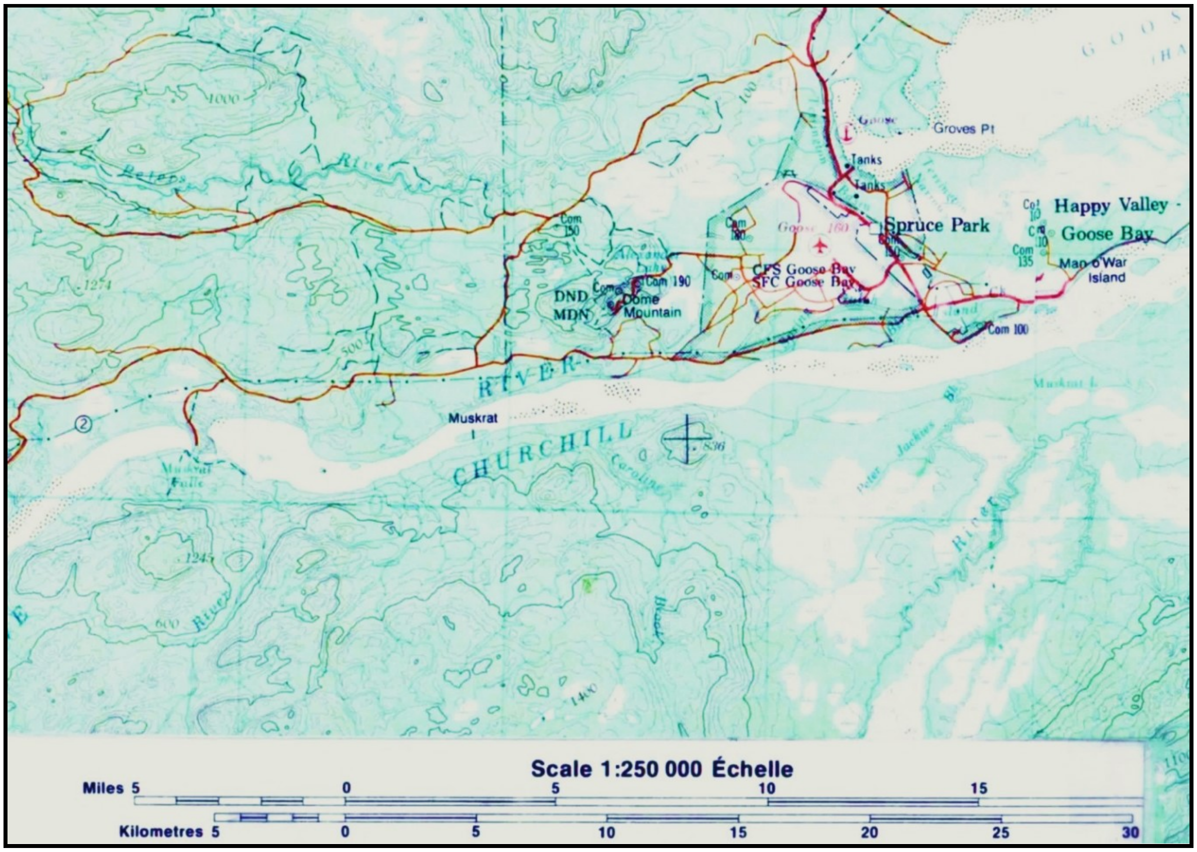


Figure 1.3 Map showing the Churchill Riverbed upstream and downstream of Muskrat Falls.
(Copied from part of map produced by Canada Centre for Mapping, Department of EM&R)

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:

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

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

$$I_P = w_L - w_P \quad \text{(Figures 3.3 and 3.4)} \qquad \text{........ Equation 2}$$

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 -

$\Delta V/V = n$ …….. Equation 3a

where V is the *total soil volume.*

Another related parameter is porosity (e)

$e = \Delta V/V_S$ …….. 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

$n = e/(1+e)$ ***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 $\mathbf{n_{crit}}$ – i.e. a value of n at which the void volume remains constant under further shear deformation. The parameter ($\mathbf{n_{crit}}$) 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_{Clay}) in relation to the concurrent void volume of the coarser material, i.e. the ratio $V_{clay}/\Delta V$.

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

On the other hand, if the volume of clay (i.e. clay particles including adsorbed water) initially is equal or ***smaller*** than the ***concurrent*** void volume of the granular material, i.e. $\mathbf{V_{clay} \leq \Delta V}$, 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 (ΔV) and the coexisting clay volume (V_{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 $\mathbf{V_{clay}} >> \mathbf{\Delta V}$.

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.

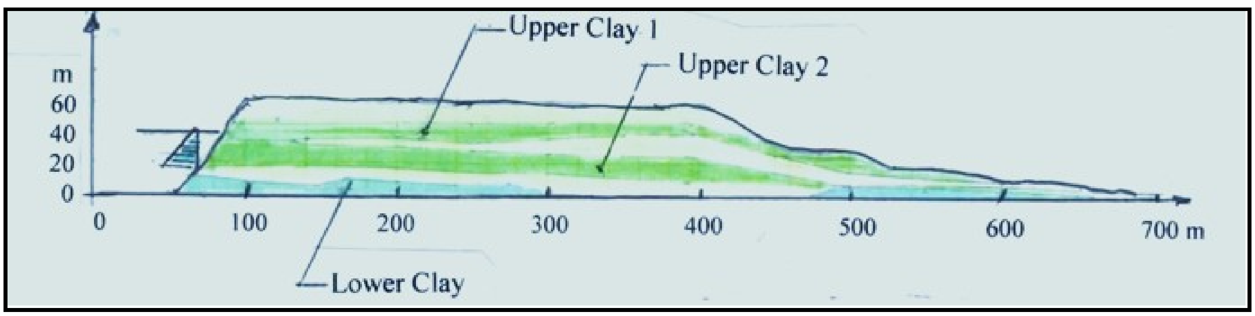


Figure 3.1 Section B-B through the North Spur showing main sedimentary features according to the NALCOR Report, [3]. As indicated in Table 1, the layers of Upper Clay 1 and 2 consist of *lean* silty clays with a permeability k ≈ $1x10^{-7}$ m/sec.

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:

Layer 1, **Sand** – permeability **k ≈ $\mathbf{1 \times 10^{-4}}$** m/sec, Upper sand
Layer 2, **Silty clay-1** – permeability **k ≈ $\mathbf{1 \times 10^{-7}}$** m/sec, Upper clay 1
Layer 3, **Silty sand** – permeability **k ≈ $\mathbf{0.8 \times 10^{-5}}$** m/sec, Upper intermediate Silty Sand Drift
Layer 4, **Silty clay-2** – permeability **k ≈ $\mathbf{1 \times 10^{-7}}$** m/sec, Upper clay 2
Layer 5, **Silty sand** – permeability **k ≈ $\mathbf{0{,}8 \times 10^{-5}}$** m/sec, Lower intermediate Silty Sand Drift
Layer 6, **Clay -2** – permeability **k ≈ $\mathbf{1 \times 10^{-8}}$** m/sec, Lower clay to great depth.

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

A. Hazen was a scientist, who 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 $1x10^{\mathbf{-7}}$ 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.

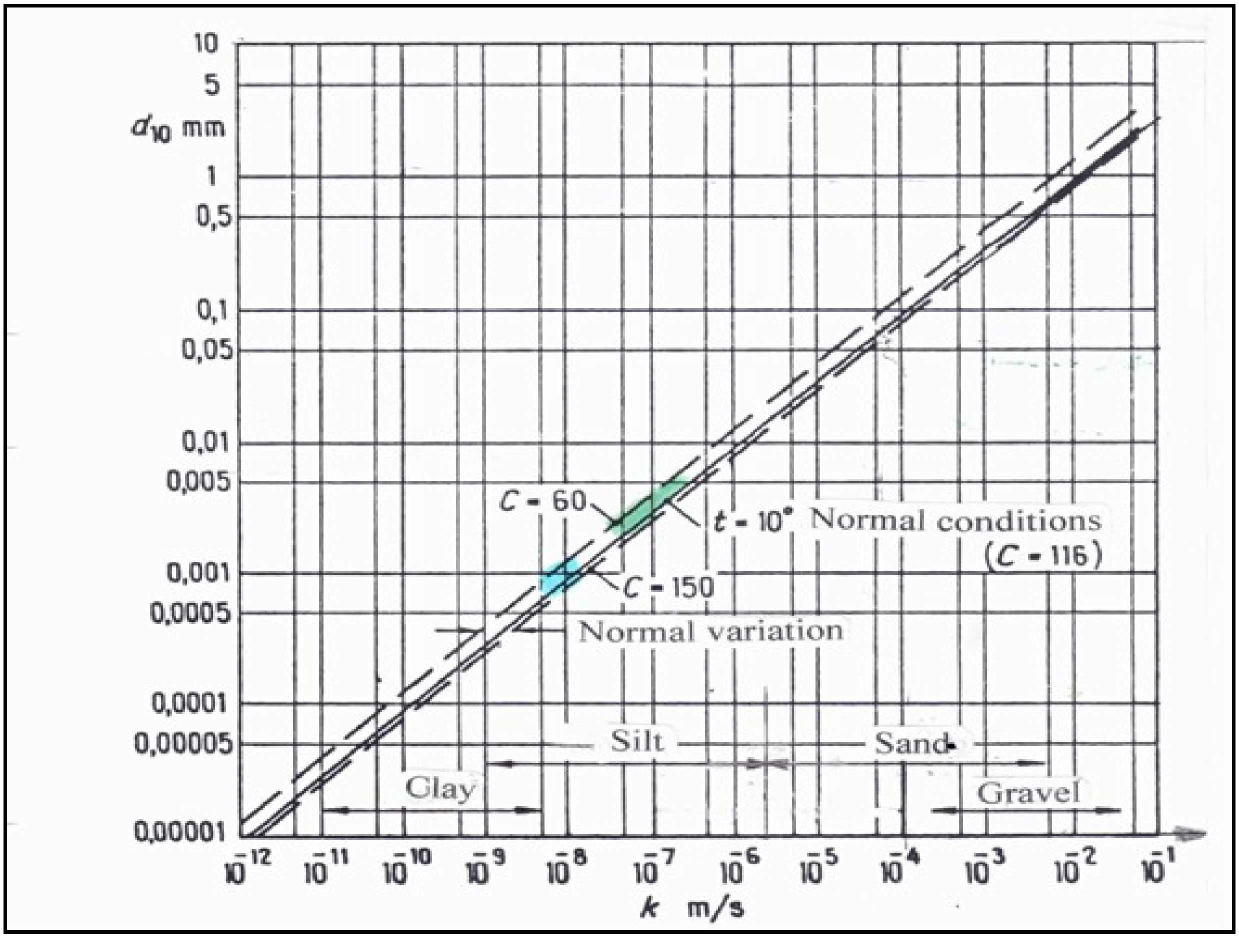


Figure 3.2 Diagram showing analysis according to Hazen's formula regarding the relationship between soil permeability k (m/s) and mean particle size.

Hazen A. (1892), **Physical properties of sands and gravels** with reference to their use in filtration. [4]

Hazen A. (1925), **The filtration of Public Water Supplies**, New York (1925). [5]

Furthermore, the figure shows that the permeability of pure clays ranges between k ≈ 10^{-11} and $10^{-8.5}$ m/sec – the typical permeability being about one hundredth to one thousandth times less than the permeability given by NALCOR for Upper Clays 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 t6able below:

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

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

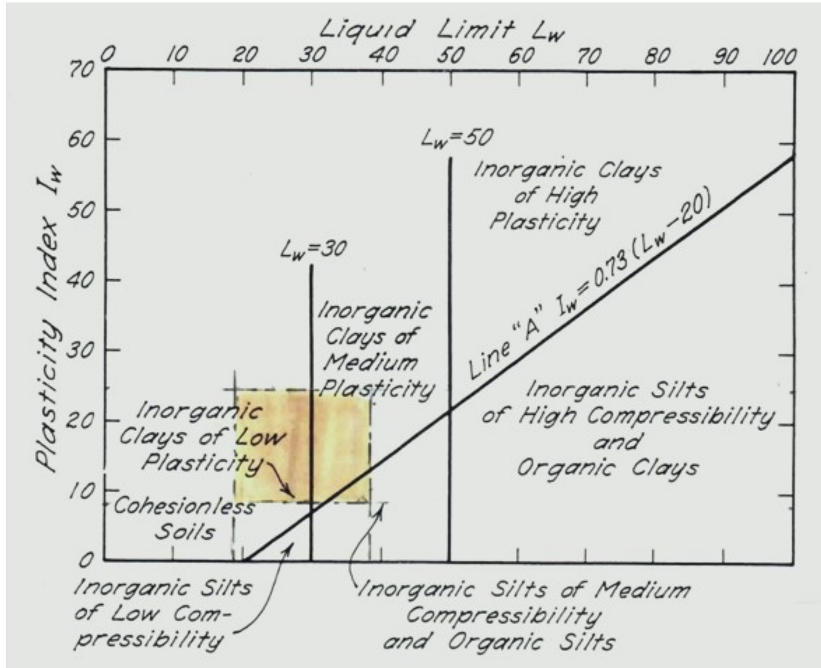
As the values in Table 2 just exhibit wide ranges of soil sample properties, the ***precise*** and ***coherent*** values applicable to *specific soil layers* of *interest* are not presented. The values given in Table 2 are therefore as such of *little* value for geotechnical analysis – e.g. for the assessment 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. (Plasticity chart in accordance with A. Casagrande 1932, *"Research on the Atterberg limits of soils"*, Public Roads 13, pp 121-136.) [6]

(**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 ≈ 8** %. ($I_P = w_L - w_P$).

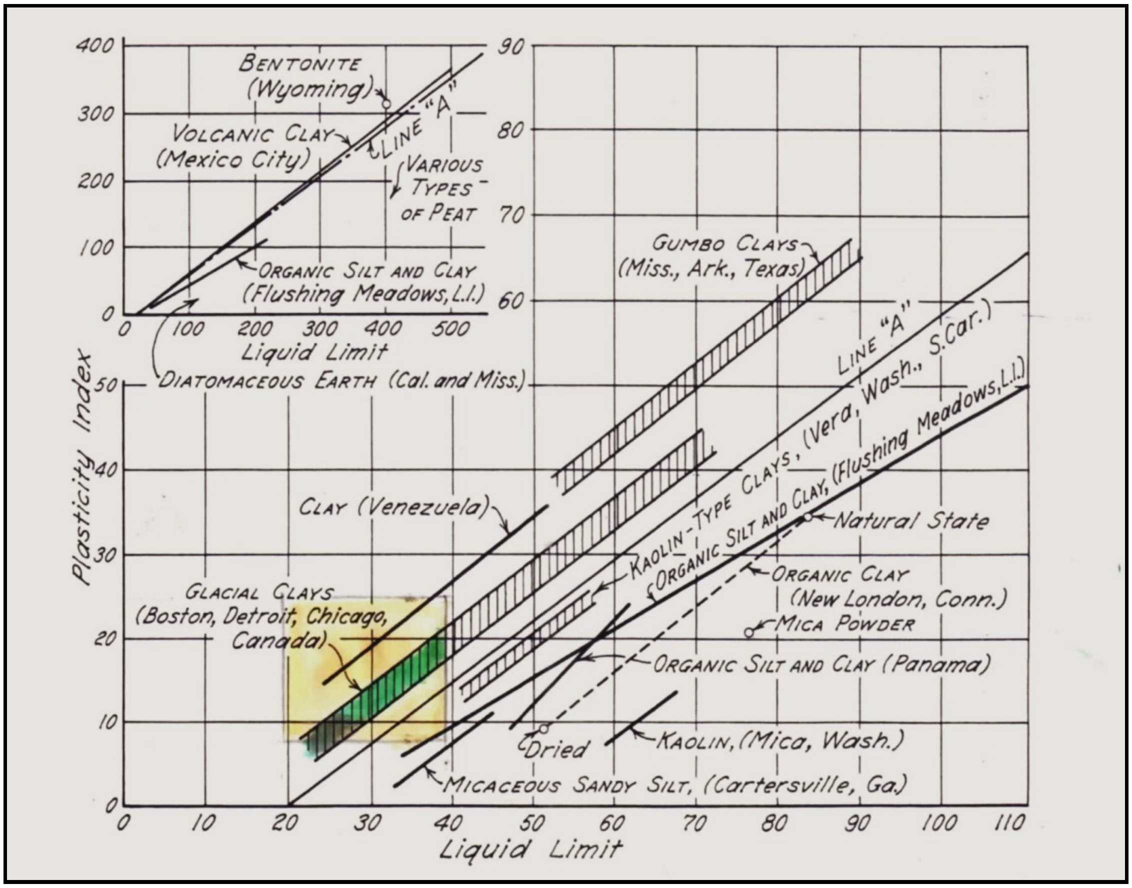


Figure 3.4 Relations between Liquid Limit (W_L) and Plasticity Index (I_P) for a wide range of mixed soils. The yellow area corresponds to the data given by NALCOR [2] in respect of Liquid Limits and the Plasticity Indices applying to the Upper Clays 1 and 2 shown in Figure 3.1.
(Plasticity chart according 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.

Figure 3.5 Clay exposure a few hundred metres upstream of the Large 2014 Landslide seen on Figure 3.6. (Photo: Eldred Davis)

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 $c \approx 0{,}20 \cdot \varrho \cdot g \cdot H = 0.2 \cdot 19 \cdot 1{,}5 \approx 6\ kN/m^2$.

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.

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

Note * ($c \approx 0{,}\ 45 \cdot w_L \cdot \varrho \cdot g \cdot H > 0{,}45 \cdot 0.26 \cdot 19{,}7 \cdot 30 \approx 70\ kN/m^2$) Formula according to Hansbo S. 1957, where w_L 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 smaller 'superficial' slides between Goose Bay and Muskrat Falls. (Photo: Eldred Davis)

4.2 Erosion processes

In the current context, the clay layer spaced above and below the Churchill River water level (i.e. the WL on October 27, 2014) is of particular interest 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.

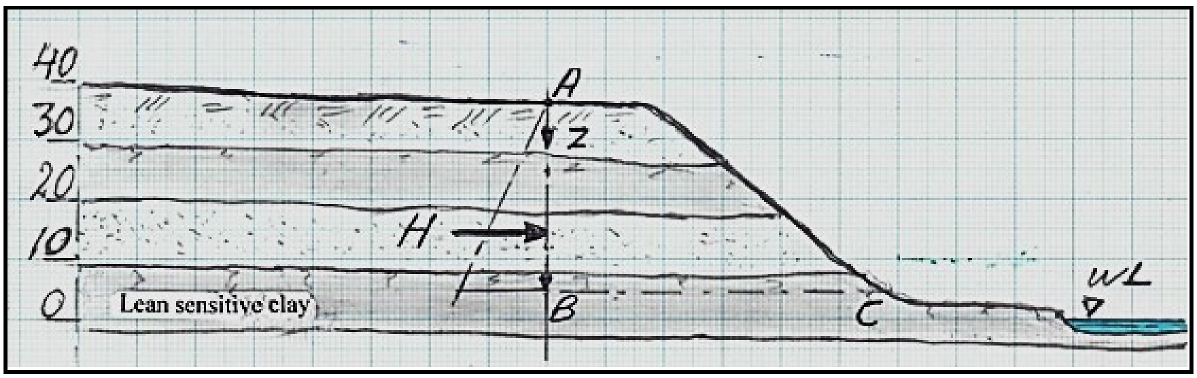


Figure 4.4 Typical slope section close to the 2014 Landslide between Goose Bay and Muskrat Falls. The section is assumed to have marine sediments similar to those in the North Spur.

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 ϕ^{o}. 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:

$\mathbf{F_s}$ = 6502/5985 ≈ **1.09,** and in **Case B,** $\mathbf{F_s}$ **= 7006/4885 = 1.43**

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

$\tau_{BC} \approx H_{z=30m} / L_{BC} = 5985/50 \approx 120$ 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 ≈ $0.45 \cdot \sigma' \cdot W_L$ where σ' denotes the pre-consolidation pressure. (Reference [7])

– i.e. for W_L = 40 %, **c** = 0.45·9.9·30·(40/100) ≈ **53,5** kN/m^2, and for

W_L = 19 %, **c** = 0.45·9.9·30·(19/100) ≈ **25,4** kN/m^2

Point A - Considering that, in this condition, the ***required*** shear resistance to avoid failure is at least τ_{BC} = **120** kN/m^2 – i.e. more than 2.2 times the available strength of a normally consolidated illitic clay with c = 53,5 kN/m^2 – it is thus apparent that the stability of the steep riverside slope, and the soil structure behind it, almost totally depend on the ***frictional*** capacity of the lower clay layer.

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_{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.

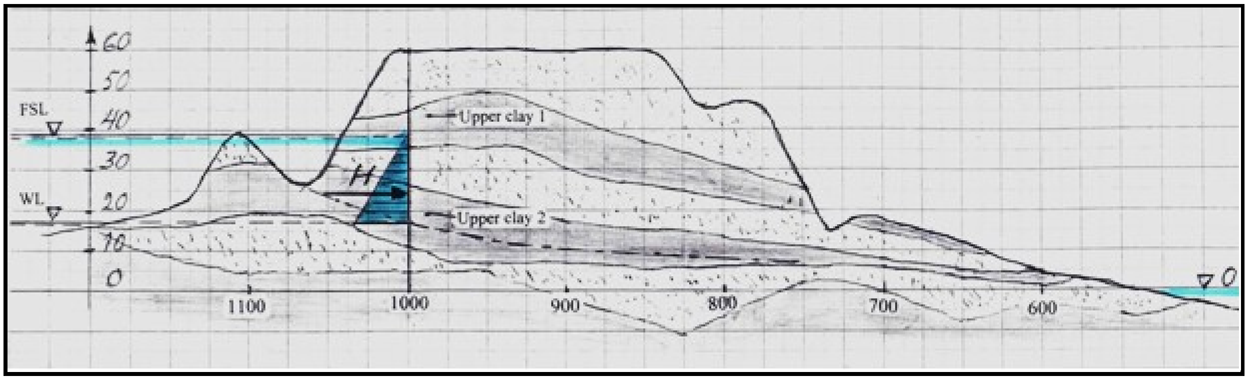


Figure 5.1 Section K–K, NALCOR [3]. The force denoted H represents an additional load due to impoundment from level +17 m to +39 m. At Level +17, the value of H, acting on the soil mass above the drawn potential slip surface, amounts to 2,420 kN/m or 24,200 metric tons over a width of 100 m. (Vertical 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 ***<u>thoroughly investigated</u>*** 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.*

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APPENDIX A

Analysis of the section in Figure 4.4.

The objective of the following analysis, based on the section shown in Figure 4.4, is to demonstrate the propensity for large landslide formation related to the current geological conditions. The soils above level B are taken to consist mainly of uniformly granular friction material (sands and silts) with a mean friction value of ϕ^{o}.
The void ratio for loose mixed-grained sands of current type is assumed to be (n) = 40 %.
(in accordance with Terzaghi & Peck, (Ref. [1] , Article 6, (Table 6.3):

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

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

The water content **w** = $W_{water/}$**/**W_{rock} = e·γ_{H2O}**/**γ_R = 66,67·10/26.5 = 25.16 %

Density (water saturated) $\boldsymbol{\gamma_w}$ = (w+1)**/**(w+γ_{H20}/γ_R) = (0.2516 +1)**/**(0.2516 +10/26.5)
= 1.2516**/**(0,2516 + 0.3774) = **19,90** kN/m^3
Density (under water $\boldsymbol{\gamma_w'}$ = γ_w– 10 = 19,9 – 10 = **9.90** kN/m^3,

Dry density $\boldsymbol{\gamma_d}$ = γ_R**/**(1+e) = 26.5/1.667 = **15,90** kN/m^3
or (γ_d = (1-n)·γ_R **/**1 = (1– 0.40)·26.5 = 15,90 kN/m^3)

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

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

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

Case a

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

Horizontal earth pressure (p kN/m^2) at a depth of z = 30 m
p_Z = $\boldsymbol{\gamma_w'}$·z·$\tan^2(45 - \phi/2)$ + γ_{H20}·z· = 9,9 ·z·0.33333 +10 z
$H_{z=30\,m}$ ≈ 0.33333·γ'· z^2/2 + γ_{H20}· z^2/2 = 0.33333·9.9· 30^2/2 + 1·10· 30^2/2
= 1485+ 4500 = 5985 kN/m

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

Shear resistance along surface B–C

$R_{z=30\,m} = {}_o\int^{50} c'\cdot dx + \gamma_w'\cdot[z_{30}\cdot{}_o\int^{20} dx + {}_{20}\int^{50} z_{30}/2\cdot dx)]\cdot \tan\phi$
$=10\cdot 50+(19{,}9–10)\cdot[(30\cdot 20 + 30\cdot 30/2)]\cdot \tan 30^o = 500 + 9.9\cdot[600 + 450]\cdot \tan 30=$
$= 500 + 10395\cdot 0.5774.= 6502$ 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. $z_w= 5$, $z_B = 30$ m,
Horizontal earth pressure (p kN/m^2)

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

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

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

This renders a safety factor:

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

APPENDIX B

Specific references.

[1] Karl Terzaghi and Ralf B. Peck, **'Soil Mechanics in Engineering Practice'**.
(Reference is made to 2nd version, 1967.)

[2] Information on posters at the IWLSC - Conference in Québec City, (2013).
[3] NALCOR Report to the Independent Engineer (2014-07-21).

[4] Hazen A. (1892), **Physical properties of sands and gravel**s with reference to their use in filtration.

[5] Hazen A. (1925), **The filtration of Public Water Supplies**, New York (1925).

[6] Casagrande A. (1932) – **Research on the Atterberg limits of soils,** Public Roads 13, pp 121-136.
[7] Hansbo S. (1957) – **A new approach to the determination of shear strength of clay by the fall cone test.** Swedish Geotechnical Institute**, (**SGI), Proceedings No 14.

[8] Modified Plasticity Chart, **United Soil Classification System**, USBR 1963.

[9] Bernander & Svensk I, et al, (1985). **Shear strength and deformation properties of clay in direct shear tests at high strain rate**s. Proc. 11 th ICSMFE, San Fransisco, Vol. 2/B/5, pp 987-990.

[10] Aas, G. (1966). **Special Field Vane Tests for the Investigation of Shear Strength of Marine Clays** (in Norwegian), Report, Publication 68, Norwegian Geotechnical Institute, Oslo, Norway.

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

Luleå Technical University, Luleå, (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.]

Bernander, S. 1978. Brittle failures in normally consolidated soils. Väg- & Vattenbyggaren (Stockholm), No. 8–9, pp. 49–52. [Most of Stig Bernander's publications are accessible at http://pure.ltu.se/portal/sv/publications/search.html.]

Bernander, S. 1985. On limit criteria for plastic failure in strain-rate softening soils. *In* Proceedings of the 11th ICSMFE, San Fransisco. Balkema. Vol. 1/A/2, pp. 397–400.

Bernander, S. 2000. Progressive landslides in long natural slopes. Formation, potential extension and configuration of finished slides in strain-softening soils. Licentiate thesis 2000:16, Luleå University of Technology. ISSN: 1402 – 1757. [Available at http://epubl.ltu.se/1402-1757/2000/16/index.html.]

Bernander, S. 2008. Down-hill progressive landslides in soft clays. Triggering disturbance agents. Slide prevention over horizontal or gently sloping ground. Sensitivity related to geometry. Research Report 2008:11, Luleå University of Technology. ISSN: 1402-1528. [Available at http://epubl.ltu.se/1402-1528/2008/11/index.html.]

Bernander, S. 2011. Progressive landslides in long natural slopes. Formation, potential extension and configuration of finished slides in strain-softening soils. Doctoral thesis, Luleå University of Technology, Division of Soil Mechanics and Foundation Engineering in cooperation with the Division of Structural Engineering, 3rd revised version, April 2012. ISBN 978-91-7439-283-8. [Available at http://pure.ltu.se/portal/files/36517492/Stig_Bernander.Rev._April_2012.pdf (accessed 26 November 2015).]

Bernander, S., and Gustås, H.K.G. 1984. Consideration of in situ stresses in clay slopes with special reference to progressive failure analysis. *In* Proceedings of the IVth International Symposium on Landslides, Toronto, Ont. Canadian Geotechnical Society. Vol. 2, pp. 235–240.

Bernander, S., and Olofsson, I. 1981. On formation of progressive failures in slopes. *In* Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, ICSMFE. Stockholm, 1981. Vol. 3, 11/6, pp. 357–362.

Bernander, S., Svensk, I., Holmberg, G., Bernander, J., and Isacsson, K. 1985. Shear strength and deformation properties of clays in direct shear tests at high strain rates. *In* Proceedings of the 11th ICSMFE, San Francisco. Balkema. Vol. 2/B/5, pp. 987–990.

Bernander, S., Gustås, H., and Olofsson, J. 1989. Improved model for progressive failure analysis of slope stability. *In* Proceedings of the 12th ICSMFE, Rio de Janeiro. Balkema. Vol. 21/3, pp. 1539–1542.

Bishop, A.W. 1971. The influence of progressive failure on the choice of the method of stability analysis. [Technical notes.] Géotechnique, 21: 168–172. doi:10.1680/geot.1971.21.2.168.

Bjerrum, L. 1967. Progressive failure in slopes of over-consolidated plastic clay and shales. 3rd Terzaghi Lecture presented in Miami Florida 1966. Journal of the Soil Mechanics and Foundations Division, ASCE, 93(5): 3–49.

Bonadies, F., Nordal, S., Gylland, A.S, Grimstad, G., Jostad, H.P., Cuomo, S., and Cascini, L. 2014. Numerical methods for simulation of downward progressive landslides. *In* Numerical methods in geotechnical engineering. *Edited by* M.A. Hicks, R.B.J. Brinkgreve, and A. Rohe. Taylor & Francis Group, London. pp. 579–584, ISBN 978-1-138-00146-6.

Caldenius, C., and Lundström, R. 1956. The landslide at Surte on the Göta River. Geological Survey of Sweden. SGU Report No. 27. Publ. Stockholm.

Chen, S.Y., Zhang, X.S., and Tang, W.S. 1997. A numerical method for analyzing progressive process of landslide in soil slope. *In* Proceedings of the 9th International Conference on Computer Methods in Geomechanics, Wuhan, China. Balkema. Vol. 13, pp. 1627–1642.

Christian, J.T., and Whitman, R.V. 1969. A one-dimensional model for progressive failure. *In* Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico City. Sociedad Mexicana de Mecanica de Sulelos. Vol. 2, pp. 541–545.

Flodin, N., and Broms, B. 1981. History of civil engineering in soft clay. *In* Soft clay engineering. *Edited by* E.W. Brand and R.P. Brenner. Elsevier, Amsterdam. pp. 25–156. ISBN 0-444-41784-2.

Graham, J., Crooks, J.H.A., and Bell, A.L. 1983. Time effects on the stress-strain behaviour of natural soft clays. Géotechnique, 33(3): 327–340. doi:10.1680/geot.1983.33.3.327.

Gylland, A.S. 2012. Material and slope failure in sensitive clays. Doctoral thesis, Department of Civil and Transport Engineering, Norwegian University of Science and Technology, Trondheim. 2012:352.

Gylland, A.S., and Jostad, H.P. 2010. Effect of updated geometry in analysis of progressive failure. *In* Proceedings of the 7th European Conference on Numerical Methods in Geotechnical Engineering, Trondheim, Norway, 2–4 June 2010. *Edited by* T. Benz and S. Nordal. pp. 497–502.

Gylland, A.S., Jostad, H.P., and Nordal, S. 2014. Experimental study of strain localization in sensitive clays. Acta Geotechnica, 9(2): 227–240. doi:10.1007/s11440-013-0217-8.

Jostad, H.P., Fornes, P., and Thakur, V. 2013. Effect of strain-softening in design of fills on gently inclined areas with soft sensitive clays. *In* Landslides in sensitive clays. From geoscience to risk management. *Edited by* J.-S. L'Heureux, A. Locat, S. Leroueil, D. Demers, and J. Locat. Springer. pp. 305–316. ISBN 978-94-007-7078-2.

Kjellman, W. 1955. Mechanics of large Swedish landslides. Géotechnique, 5(1): 74–78. doi:10.1680/geot.1955.5.1.74.

L'Heureux, J.-S., Locat, A., Leroueil, S., Demers, D., and Locat, J. (*Editors*). 2013. Landslides in sensitive clays. From geoscience to risk management. Springer. ISBN 978-94-007-7078-2.

Larsson, R., and Jansson, M. 1982. The landslide at Tuve, November 30, 1977. Swedish Geotechnical Institute, Linköping. Report No 18.

Leroueil, S. 2001. Natural slopes and cuts: movement and failure mechanisms.

39th Rankine Lecture. Géotechnique, 51(3): 197–243. doi:10.1680/geot.51.3.197.39365.

Locat, A. 2007. Ètude d'un étalement latéral dans les argiles de l'est du Canada et de la rupture progressive. Le cas du glissement de Saint-Barnabé-Nord. M.Sc. thesis, Departement de Génie Civil, Faculté de Sciences et de Genie, Université Laval, Quebec City, Que.

Locat, A. 2012. Rupture progressive et étalements dans les argiles sensibles. [Progressive failure and spreads in sensitive clays.] Ph.D. thesis, Département de génie civil et de génie des eaux, Université Laval, Quebec City, Que.

Locat, A., Leroueil, S., Bernander, S., Demers, D., Locat, J., and Ouehb, L. 2008. Study of a lateral spread failure in an eastern Canada clay deposit in relation with progressive failure: The Saint-Barnabé-Nord Slide. *In* Proceedings of the 4th Canadian Conference on Geohazards. From Causes to Management. *Edited by* J. Locat, D. Perret, D. Turmel, D. Demers, and S. Leroueil. Presses de l'Université Laval, Quebec City, Que. pp. 89–96.

Locat, A., Leroueil, S., Bernander, S., Demers, D., Jostad, H.P., and Ouehb, L. 2011. Progressive failures in eastern Canadian and Scandinavian sensitive clays. Canadian Geotechnical Journal, 48(11): 1696–1712. doi:10.1139/t11-059.

Locat, A., Jostad, H.P., and Leroueil, S. 2013. Numerical modeling of progressive failure and its implications for spreads in sensitive clays. Canadian Geotechnical Journal, 50(9): 961–978. doi:10.1139/cgj-2012-0390.

Locat, A., Leroueil, S., Fortin, A., Demers, D., and Jostad, H.P. 2015. The 1994 landslide at Sainte-Monique, Quebec: geotechnical investigation and application of progressive failure analysis. Canadian Geotechnical Journal, 52(4): 490–504. doi:10.1139/cgj-2013-0344.

Löfquist, B. 1973. Lerskred genom vattenupptryck. [Slides in clays caused by hydraulic uplift.] Väg- och vattenbyggaren, Stockholm, 19(2): 187–190. [In Swedish.]

Pusch, R., Knutsson, S., Liu, X., and Yang, T. 2016. Creep can strengthen clay: a matter of long-term slope stability. Journal of Earth Sciences and Geotechnical Engineering, 6(1): 1–18. Scienpress Ltd. ISSN: 1792-9040 (print), 1792-9660 (online).

Puzrin, A.M. 2016. Editorial: Modern trends in landslide mechanics. Géotechnique, 66(3): 173–174. doi:10.1680/jgeot.2016.66.3.173.

Quinn, P. 2009. Large landslides in sensitive clay in eastern Canada and the associated hazard risk to linear infrastructure. Ph.D. thesis, Department of Geological Sciences and Geological Engineering, Queen's University, Kingston, Ont. Available at: http://qspace.library.queensu.ca/handle/1974/1781 [accessed 21 April 2011].

Rehnström, L. 2013. Analysis of progressive landslides. A review of the simplified calculation model. M.Sc. thesis, Division of Geo Engineering, Chalmers University of Technology, Göteborg, 2013:5. Available at http://publications.lib.chalmers.se/records/fulltext/185611/185611.pdf.

Saurer, E. 2009. Shear band propagation in soil and dynamics of tsunamigenic landslides. Ph.D. thesis, ETH, Zürich. doi:10.3929/ethz-a-005951122. Available at http//www.civil.iitb.ac.in/e.library.queensu.

Skempton, A.W. 1964. Fourth Rankine Lecture: Long term stability of clay slopes. Géotechnique, 14(2): 77–102. doi:10.1680/geot.1964.14.2.77.

Terzaghi, K., and Peck, R.B. 1948. Soil mechanics in engineering practice. New York, John Wiley & Sons, Inc.
Thakur, V. 2007. Strain localization in sensitive soft clays. Ph.D. thesis, Geotechnical Division, Department of Civil and Transport Engineering, Norwegian Institute of Science and Technology, NTNU. International Centre for Geohazards, Trondheim and Norwegian Centre of Excellence. ISBN 9788247139097.
Thakur, V., and Septanika, E.G. 2008. Extended finite element modeling (X-FEM) of progressive sliding in geomaterials, In Proceedings of the Nordic Geotechnical Meeting, No. 15. *Edited by* K. Flaate, T.-E. Frydenlund, J. Prestegarden, and K. Senneset. pp. 447–469.
Troncone, A. 2005. Numerical analysis of a landslide in soils with strain-softening behaviour. Géotechnique, 55(8): 585–596. doi:10.1680/geot.2005.55.8.585.
Zabala, F., and Alonso, E.E. 2011. Progressive failure of Aznalcollar dam using the material point method. Géotechnique, 61(9): 795–808. doi:10.1680/geot.9.P.134.
Zhang, W., Wang, D., Randolph, M.F., and Puzrin, A.M. 2015. Catastrophic failure in planar landslides with a fully softened weak zone. Géotechnique, 65(9): 755–769. doi:10.1680/geot14.P.218.

II. Further comments, 2016-01-07

Further Comments on the Updated Nalcor Report.of 21July-2014 by Stig Bernander

Contents

2016-01-07

Further Comments on the **Updated Nalcor 21-July-2014 Report.**

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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 → 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 → 41 %) exceed the liquid limits (w_L or LL) = 19 →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 ≥ 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 sensitivit***y 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 <u>progressive</u> failure",** the following statements are made:

a) Quotation: "…… ***calculations are calibrated locally with an existing slope".***

<u>Comment on quotation **a)**:</u>
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.***

<u>Comment on quotation **b)**:</u>
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 <u>no</u> evidence of downhill progressive failure landslide along the Churchill river valley."**

<u>Comments on quotation **c)**</u>
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 <u>ordinary</u> 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, (2013), (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* ($\mathbf{n_{crit}}$)
i.e. $\mathbf{n < n_{crit}}$ or in terms of *void ratio* (e) ………… Equ. 1a
$\mathbf{e < e_{crit}}$ ………… Equ. 1b
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 $\pm \Delta\tau_{x,y}$.- i,.e. so called stress-strain reversals
(Cf Terzaghi & Peck, Ref. [3].)

<u>A vital question in this context:</u> 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.

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References:

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

[2] **Lower Churchill Project – NORTH SPUR UPDATED, Independent Engineer – 21 JUL – 2014, Nalcor Report**

[3] **Article 17**, ***Shearing Resistance of Cohesionless Soils***, Pp 107 – 111.
Terzaghi K. & Peck R.B. Soil Mechanics in Engineering Practice ,
Second Edition 1967. (Printed by John Wiley & Sons Inc. New York , London, Sidney.)

[4] **Article 6**, ***Index Properties of Soils,*** Page28,
Terzaghi K. & Peck R.B. Soil Mechanics in Engineering Practice ,
Second Edition 1967. (Printed by John Wiley & Sons Inc. New York , London, Sidney.)

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

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

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

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

[11] Comments on the Nalcor's Report to the Independent Engineer of July 21, dated September 14, 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.)

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

Comments on the Engineering Report by Nalcor/SNC-Lavalin of December 2015 prepared for Grand Riverkeeper, Labrador, Inc. by Stig Bernander 15 September 2016,

Contents

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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FOREWORD

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* ($\mathbf{n_{crit}}$). This relationship, and how it applies to the types of soil in the North Spur, is treated in some detail.

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

* **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.

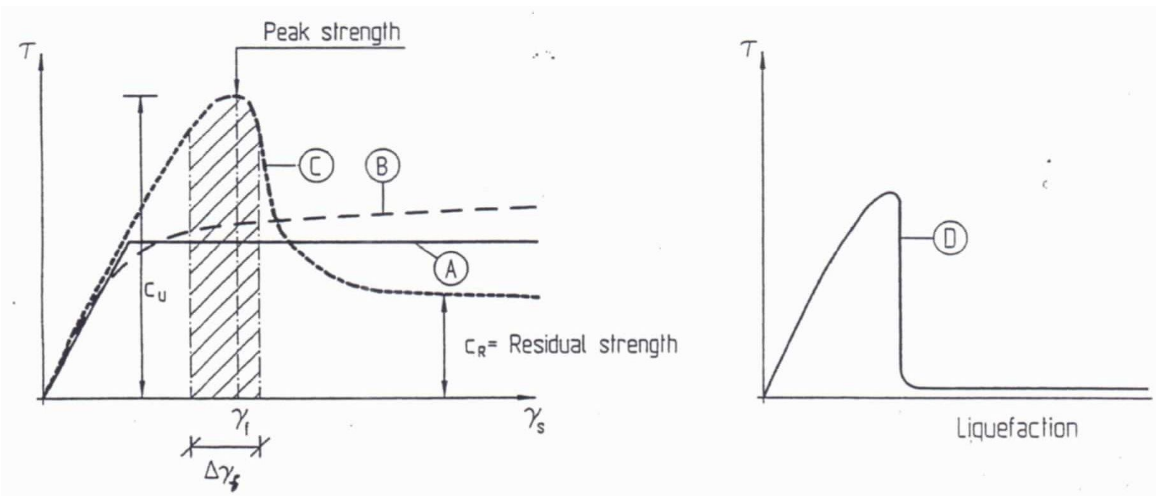

Figure 2.1. Deviatory stress-strain (deformation) relationships of different kinds

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.

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[*] AMEC Earth & Environmental. (2011) "Geotechnical Investigation: Edwards Island Landslide, Churchill River, Labrador". Contract #LC-EV-007. [15]

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).

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

Relationships

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

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

or: $\boldsymbol{\gamma_w} = n\cdot\boldsymbol{\gamma_{H20}} + (1-n)\times\boldsymbol{\gamma_R}$ kN/m^3

Dry density $\quad \boldsymbol{\gamma_d} = (1-n)\ \boldsymbol{\gamma_R}$ kN/m^3

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

For comparison, see Terzaghi and Peck [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]

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

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

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

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

[*] SNC-Lavalin. (30 January 2016) "Lower Churchill Project. North Spur Stabilization Works – Design Report". Nalcor Doc. No. MFA-SN-CD-2800-GT-RP-0004-01. Pages 145–147. [16]

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

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

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

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

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

4. Shear strength — dependence on diverse effects

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

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

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

Other important factors include:

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

According to the **Report**, the peak shear strengths of the North Spur soils have largely been measured by vane tests. In this context, it is of interest to refer to the diagram in **Figure 4.1** published by Aas, 1966 [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.

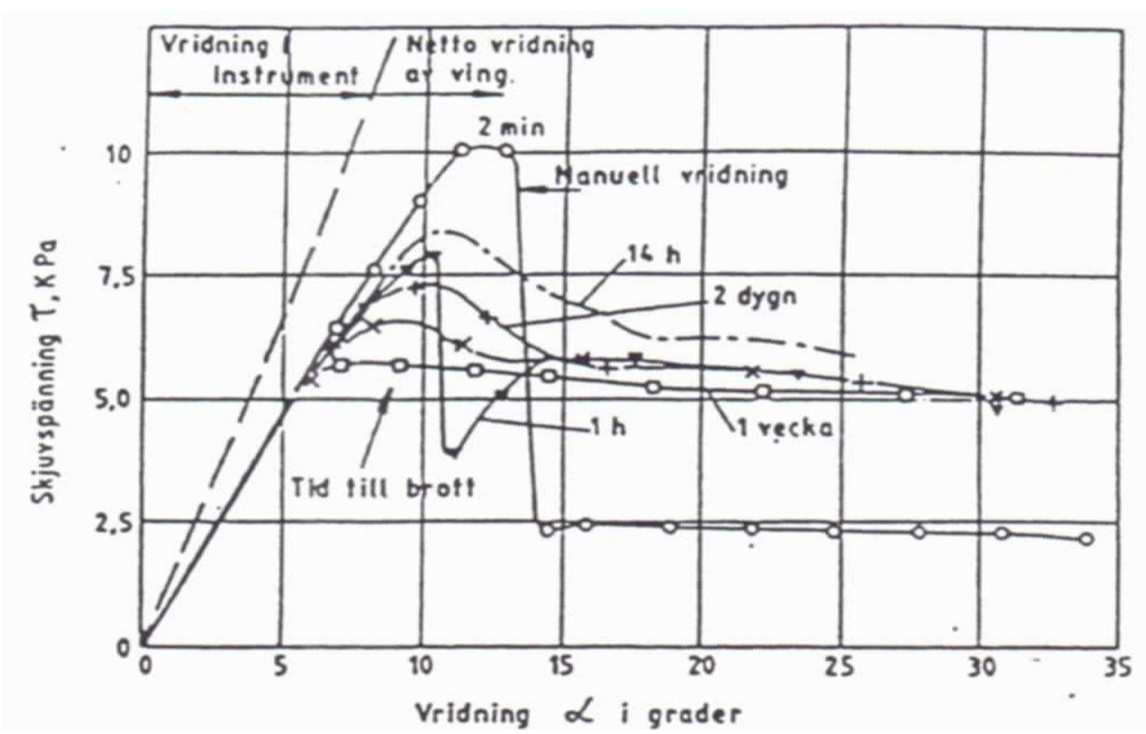

Figure 4.1. Stress-strain (deformation) curves for consolidated, undrained vane tests at different strain rates (Aas, 1966). Legend: brott = failure, Vridning = torsion, dygn = day, vecka = week, grader = degrees.

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).

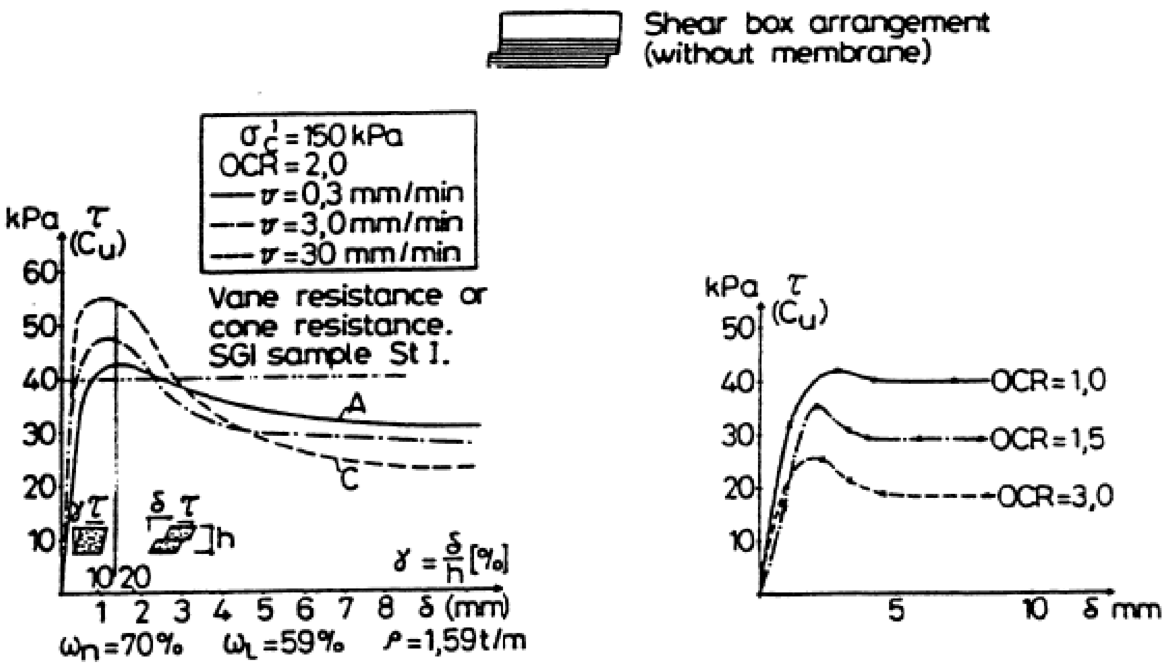

Figure 4.2. Typical test results from consolidated undrained direct shear tests on a normally consolidated Swedish clay. Note that deformation on the horizontal axis is represented both in terms of angular strain and slip displacement in millimetres.*

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**.

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* 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.

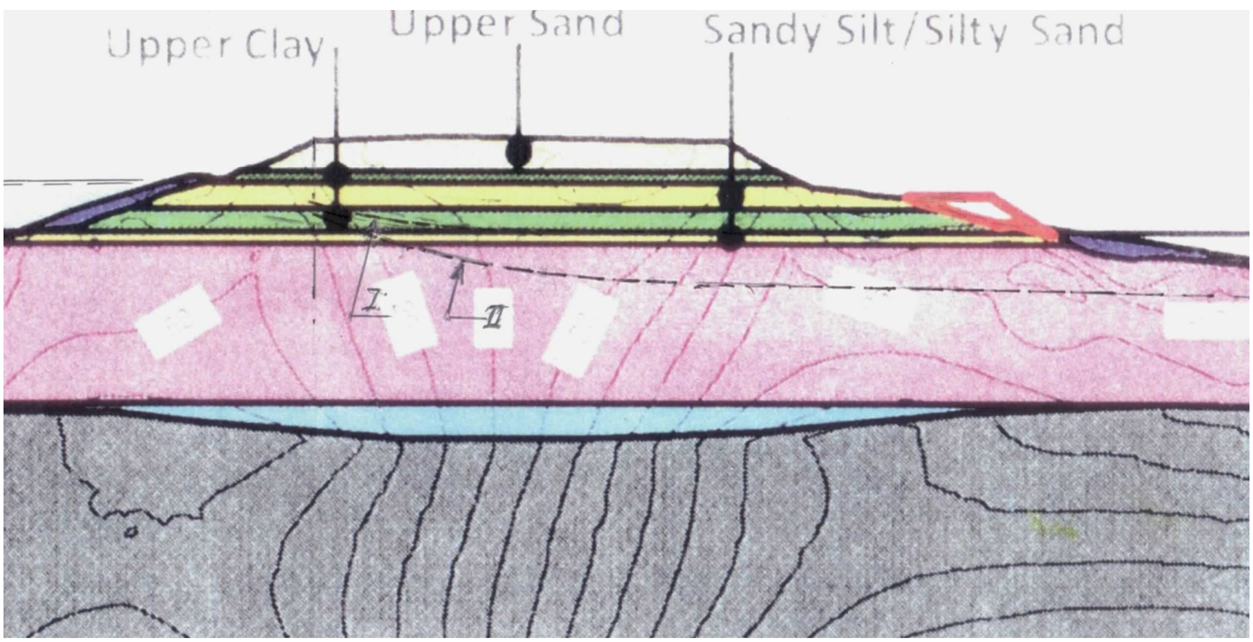

Figure 5.1. Potential failure planes (I and II) possibly leading to progressive failure development.

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

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

According to **Figure 8** in posters at the IWLSE Conference 2013 [1], the lower contour of Upper Clay 2 slopes about 3 metres along the length coordinate x ≈ 200 m (near the COW) to x ≈ 350 m. This is an inclination of about 2%.

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

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

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

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

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

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

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

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

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

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

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

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

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

Hence, the total local shear stress could be in the order of 72.6 + 58.6 = 131.4 kN/m^2. Note that this value is higher than almost all the intact undrained shear strength measurements, $\mathbf{s_u}$ = 35 to 135 kN/m^2, shown in **Table 2-1** of the **REPORT**.

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

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

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

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

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

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:

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[*] Locat A, Jostad HP, Leroueil S. (2013) "Numerical modeling of progressive failure and its implication to spreads in sensitive clays". *Canadian Geotechnical Journal*, 50(9), pp. 961–978. [17]

- 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, $\mathbf{n} > \mathbf{n_{crit}}$, then the soil is prone to massive loss of shear resistance or to liquefaction when sheared or exposed to stress-strain reversals related to vibration, pile driving, seismic activity, etc. (See Terzaghi-Peck, Article 17 [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. $n > n_{crit}$.**

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 ($\mathbf{n_{crit}}$):

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

or in terms of void ratio (e)

$e \leq e_{crit}$ Equation 1b

where **n** and e relate to one another as

$e = n/(1-n)$ or $n = e/(1+e)$

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

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

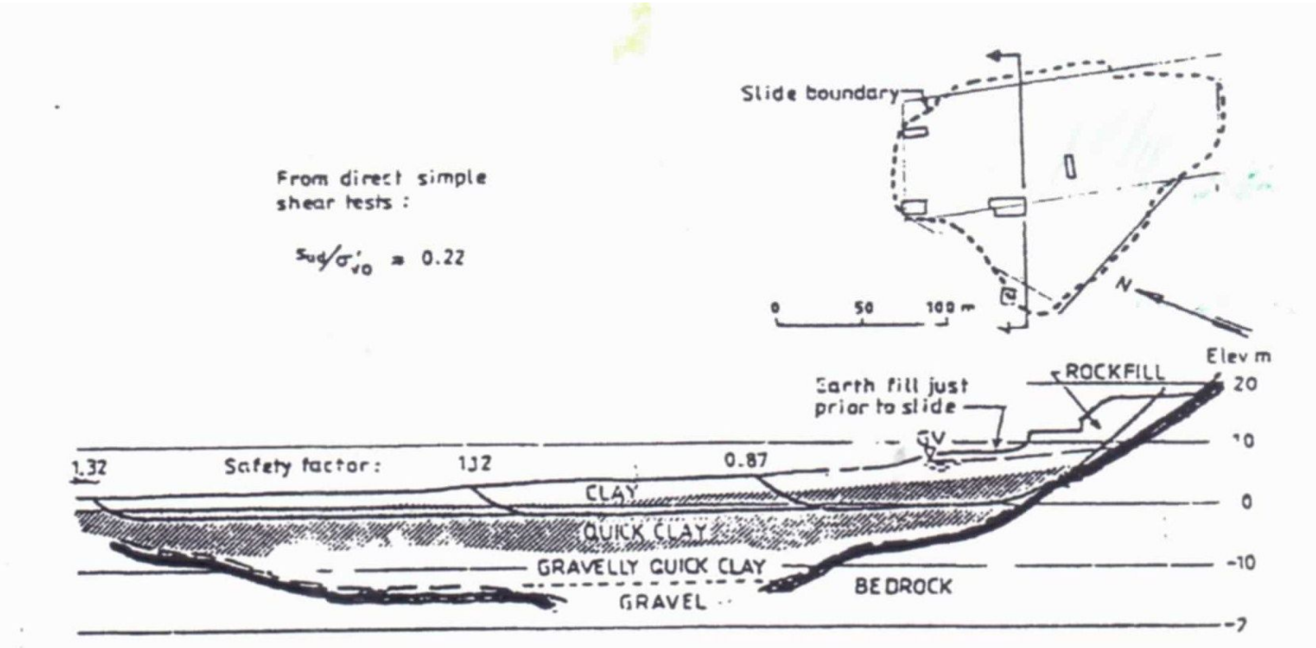

Figure 5.2. The Bekkelaget landslide, Norway. Analysis by Aas (1983). The odd circumstance to be noted here is that the slide actually developed along the 200-metre-long failure surface with the highest safety factor, F_S =1.32, and not along the short failure surface with an insufficient safety factor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

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

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

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

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

dn = Down-slope displacement at x = L
L = Influencial length of force Ni
qcrit = Ncrit/H
t(el) – Elastic limit

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

Case reference number	Input data – Depth to failure surface H = 20 m: Gradient (Gr) tan(Gr)	In situ shear stress to kPa	Peak shear resistance kPa	Lab shear resistance kPa	Assoc. shear deformation %	Elastic limit t(el) kPa	Recid. shear resist. CR/Clab 1	Recid. shear resist. CR kPa	Density of clay g kN/m3	IPFA analysis: Ep-Ea =4H c kN/m	H m	sin Gr 1.00	Results from IPFA analysis #: qf kN/m2	Max. height of fill m	Fpr = qcrit/q q=18 kN/m2	q kN/m2	*Conventional safety factor Fc q=18	Progressve f. PrFA analysis *: qcrit kN/m2	x = L (crit) at Ncrit m	Fpr/Fc
Case 1	0.050	15.98	30	25	3.0	16	0.400	10.0	16.0	2000	20.0	0.0499	150.43	8.36	1.081	18.0	2.23	19.5	111.8	0.484
Case 2	0.060	19.17	30	25	3.0	16	0.400	10.0	16.0	2000	20.0	0.0599	132.88	7.38	0.887	18.0	1.91	16.0	112.7	0.464
Case 3	0.070	22.35	30	25	3.0	16	0.400	10.0	16.0	2000	20.0	0.0698	114.98	6.39	0.645	18.0	1.67	11.6	112.9	0.385
Case 4	0.080	25.52	30	25	3.0	16	0.400	10.0	16.0	2000	20.0	0.0797	97.30	5.41	0.391	18.0	1.53	7.0	104.0	0.256
Case 4a	0.080	25.52	30	25	3.0	16	~~0.400~~	5.0	16.0	2000	20.0	0.0797	97.41	5.41	0.363	18.0	1.55	6.5	100.0	0.235
Case 5	0.050	15.98	30	25	3.0	16	0.600	15.0	16.0	2000	20.0	0.0499	153.46	8.53	1.226	18.0	2.20	22.1	118.5	0.557
Case 6	0.060	19.17	30	25	3.0	16	0.600	15.0	16.0	2000	20.0	0.0599	134.77	7.49	0.994	18.0	1.88	17.9	119.2	0.528
Case 7	0.070	22.35	30	25	3.0	16	0.600	15.0	16.0	2000	20.0	0.0698	115.82	6.43	0.713	18.0	1.65	12.8	119.2	0.433
Case 8	0.080	25.52	30	25	3.0	16	0.600	15.0	16.0	2000	20.0	0.0797	97.14	5.40	0.432	18.0	1.50	7.8	110.3	0.288
Case 9	0.050	15.98	30	25	3.0	16	0.800	20.0	16.0	2000	20.0	0.0499	154.84	8.60	1.425	18.0	2.19	25.6	121.6	0.651
Case 10	0.060	19.17	30	25	3.0	16	0.800	20.0	16.0	2000	20.0	0.0599	137.31	7.63	1.176	18.0	1.85	21.2	127.9	0.636
Case 11	0.070	22.35	30	25	3.0	16	0.800	20.0	16.0	2000	20.0	0.0698	117.26	6.51	0.845	18.0	1.61	15.2	130.1	0.526
Case 12	0.080	25.52	30	25	3.0	16	0.800	20.0	16.0	2000	20.0	0.0797	96.87	5.38	0.505	18.0	1.46	9.1	120.8	0.346
Case 13	0.050	15.98	30	25	3.0	16	0.900	22.5	16.0	2000	20.0	0.0499	100.00	5.56		18.0		25.6		
Case 14	0.060	19.17	30	25	3.0	16	0.900	22.5	16.0	2000	20.0	0.0599	100.00	5.56		18.0		21.1		
Case 15	0.070	22.35	30	25	3.0	16	0.900	22.5	16.0	2000	20.0	0.0698	118.50	6.58	0.956	18.0	1.58	17.2	139.4	0.605
Case 16	0.080	25.52	30	25	3.0	16	0.900	22.5	16.0	2000	20.0	0.0797	96.64	5.37	0.568	18.0	1.43	10.2	129.7	0.398
Mean values							0.2								**0.87**		**1.87**			**0.49**
Deviations - max/min		%																	**34.2**	**-51.5**

Figure 5.3. Relationships between safety factors determined by Progressive Failure Analysis and elastic-plastic LEM analysis. Note especially the column with a red heading [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** = m/sec) in Saint Barnabé-Nord ranged from $1 \times 10^{\mathbf{-9}}$ to $5 \times 10^{\mathbf{-9}}$ m/s, whereas the **k**-values of the Upper Clays in the North Spur are about 1×10^{-7} m/s. This implies that the mixed Upper Clays in the Stratified Drift are from 20 to 100 times more permeable than the clays in Saint-Barnabé-Nord.

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

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[*] Singh A and Mitchell JK. (1968) "General stress-strain-time function for soils". *J Soil Mech Found* Div 94 (SM 1), ASCE, pp 21–46. [18]

In clear contrast, the sensitivity of soils in the North Spur is related to the in-situ soil porosity (**n**) being markedly greater than the value of the critical porosity ($\mathbf{n_{crit}}$). Such types of soil may liquefy 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."*

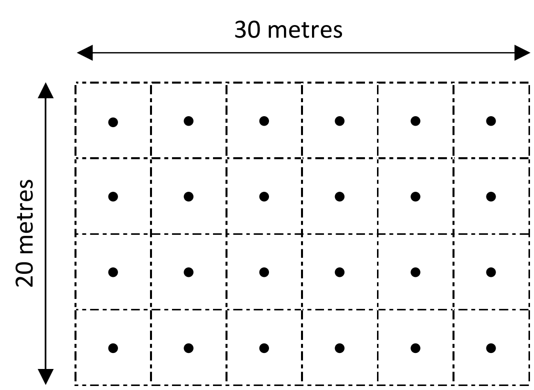
As shown in previous Sections, both the data presented in the **REPORT** and the general character and development of the Churchill River Valley strongly indicate that the in-situ porosities (or void ratios) of some soils of the North Spur are probably critically high. If this is the case, then the safety 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.



Suggested pattern of test piles

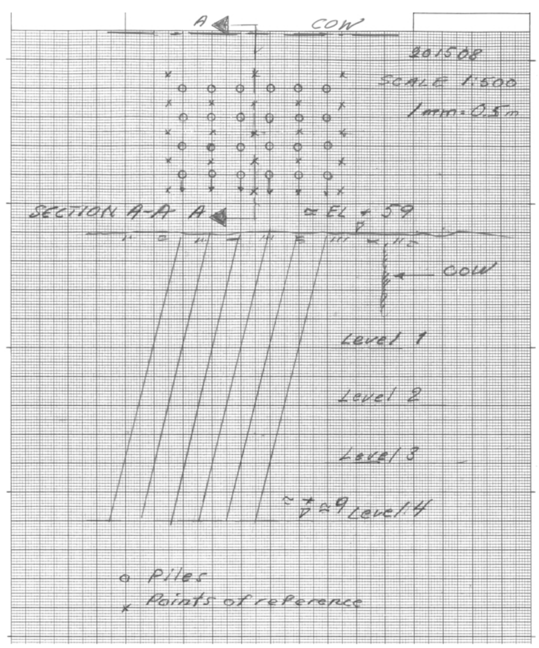
2) The piles may consist of 0.3 m diameter steel pipes fitted with splices every 10 metres. The pile tips should be flat and closed by a perforated steel plate to allow dissipation of water. Alternatively, other methods of drainage may be employed. A point of reference for each pile and its precise level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

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

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

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

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



Impact tests of this kind are the best way to get a realistic notion of the true in situ porosity of such soil layers. The above procedures yield a reliable indication of the effects on soil porosity of heavy vibratory impact. From the measurements of soil settlement it is possible to evaluate the inherent sensitivity of the soil profile, i.e. how sensitive the layers are to deviatory deformation and to stress/strain reversals such as those caused by 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 ($\mathbf{n_{crit}}$) — **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 ($\mathbf{n_{crit}}$).

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 ($\mathbf{n}_{crit}$), soil liquefaction may take place whether or not finger drains are present.

6.6 Investigation of in-situ porosity conditions in soil layers

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

6.7 Required testing

The soil investigations presented in the **REPORT** comprise mostly laboratory testing carried out in 1979 and 2013. Relatively few dynamic tests were done in-situ. The detailed computer model that 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].

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=LIcL_pN4NlQ.

References

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

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

[2b] Nalcor–SNC/Lavalin. (21 December 2015) ***Engineering Report***, North Spur-Stabilization-Works, Progressive-Failure-Study. 128 pages.

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

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

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

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

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

[8a] Aas G. (1966). "Special Field Vane Tests for the Investigation of Shear Strength of Marine Clays". *Report*, Norwegian Geotechnical Institute, Oslo.

[8b] Aas G. (1983). "A Method of Stability Analysis applicable to Natural Slopes in Sensitive and Quick Clays". *Proc. Symposium on Landslides*, Linköping 1982, Swedish Geotechnical Institute, Report no 17, pp 7–25.

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

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.

[11] Bernander S. "Comments on Nalcor's Report to the Independent Engineer of 21 July 2014". Dated 14 September 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.

[13] Bernander S. *Lower Churchill River, Riverbank Stability Report*, 14 October 2015. (Revisions of linguistic nature, 23 November and 5 December 2015).

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

Additional references cited in these Comments:

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

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

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

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

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).

Case 3

τ_o = 21,1 kPa s= 60 kPa s_R= 12 kPa s/s_R = 5 N_{cr} = 866 kN/m

Safety factor $F = N_{cr} / N_w$ = 866 / 2420 = 0,357 < 1

Case 4

τ_o = 41,1 kPa s= 70 kPa s_R= 14 kPa s/s_R = 5 N_{cr} = 521 kN/m

Safety factor $F = N_{cr} / N_w$ = 521 / 2420 = 0,215 < 1

Downhill progressive slide - triggering load, basic ex.

Author: Stig Bernander **OK** **Page 1**

17 05 05

Muskrats Fals , Case 3- Slope 1:25, arctan 0,04 = 2,292dgr

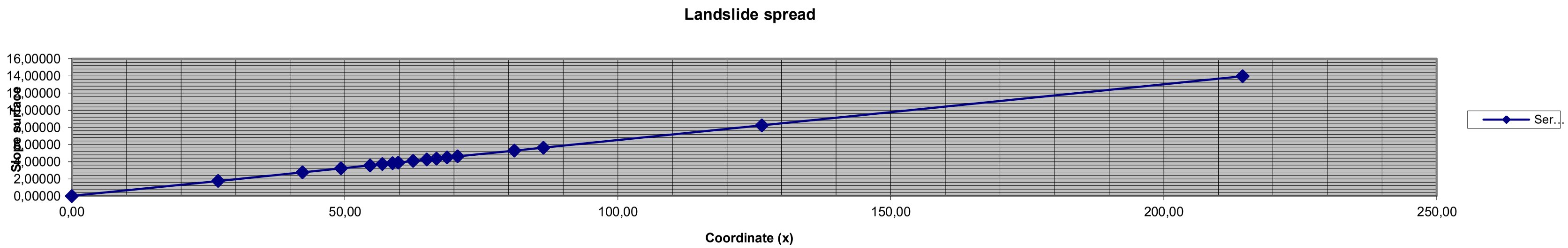
C_R = 12 kN/m2

Input data:

$H_{(x=L)}$ = 15 m **Gradient: Slope 1:25, arctan 0,04 = 2,292dgr**

Density = 18,0 kN/m3

		(Residual shear strength)		**(Surface strength)**	**(Mean shear strength)**
C_{peak} = 60 kN/m2	C_{lab} = 50 kN/m2	C_R = 12 kN/m2	C_R/C_{lab} = 0,24	C_s = 45kN/m2	C_{mean} = 52,5 kN/m2
Shear deformation at failure (g_f) = 7,0%			C_R/C_{peak} = 0,20		t_{el} = 40 kN/m2
Post-peak deformation at C_R = 0,30 m		Shear deform. at elastic limit (g_{el}) = 3.50 %			



$\underline{t_{x,z} < t_{el}}$	**$t_{x,z} < t_{el}$**
$g_{x,z(1)} = t_x/G$, $g_{el} = g_f{*}t_{el}/(2c - t_{el})$ and **$G = t_{el}/g_{el}$**	**Stage I** **Equ. I:1a** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(1)})dz$**
$\underline{t_o < t_{xz} < c,\ t_o > t_{el}}$	**$t_o < t_{xz} < c$** $t_o > t_{el}$
$g_{x,z(4)} = (g_f - g_{el}){*}(1 - ROT(1 - (t_{o(x,z)} - t_{el})/(c - t_{el}))$	**Stage I** **Equ. I:4** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(4)})dz$**
$\underline{t_o < t_{el}}$ **$- ROT(1 - (t_{(x,z)} - t_{el})/(c - t_{el})))$**	$t_o < t_{el}$
$g_{x,z(4a)} = (t_{el} - t_{o,(x,z)})/G + (g_f - g_{el}){*}(1 - ROT(1 - (t_{(x,z)} - t_{el})/(c - t_{el})))$	**Equ. I:4a** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(4a)})dz$**
$\underline{c < t_{xz} < c_R,\ t_o > t_{el}}$	**$c < t_{xz} < c_R$** $t_o > t_{el}$ $t_{xz} > c_R$
$g_{x,z(5)} = (g_f - g_{el}){*}(ROT(1 - (t_{o(x,z)} - t_{el})/(c - t_{el})) -$	**Stage II** **Equ. I:5** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(5)})dz$ +**
$\underline{t_o < t_{el}}$ **$- ROT(1 - (t_{(x,z,max)} - t_{el})/(c - t_{el}))) - (t_{(x,z,max)} - t_{(x,z)})/G$**	$t_o < t_{el}$ $t_{xz} < c_R$ **$+S_{CR}(c - t_{x,z})/(c - c_R)$**
$g_{x,z(5a)} = (t_{el} - t_{o,(x,z)})/G + (g_f - g_{el})(1 - ROT(1 - (t_{(x,z,max)} - t_{el})/(c - t_{el})))-$	**Stage IIa** **Equ. I:5a** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(5a)})dz$ +**
$\underline{t_{x,o} = c_R}$ **$- (t_{(x,z,max)} - t_{(x,z)})/G$**	**$t_{x,o} = c_R$** **$+S_{CR}(c - t_{x,z})/(c - c_R)$**
$g_{x,z(5b)} = g_{x,z}(5)$ (or $g_{x,z}(5a)$) **$- (t_{(x,z,max)} - c_R)/G$**	**Stage IIb** **Equ. I:5c** **S:a $d_{(x,z),t}$ = Integral $(g_{x,z,(5c)})dz + S_{CR} + S_{slip}$**

Downhill progressive slide - triggering load **Stage I** (1) **17 04 14** **Page 2**

H_q		28	26,93	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**26,800**
step	**No 1**	**x_o**	x_1	**0,00**	**Stress**	**x1**	**Mean**		kN/m2		kN/m2		**x1**	**26,80**
n	**z (n)**	**In situ***	**In situ***	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20			$t_o(x,z)$	$t(x,z)$	$dt(x,z)$	$t(x,z)+dt$		0,0350	20,00		**Equ 1:a**			
0	0,00	21,94	21,14	**21,14**	**5,000**	**26,138**	23,638	0,0350	20,00	**40,00**	0,00438		60,00	1143
								0,0350				0,00324		
1	0,75	21,40	20,60	20,60	4,813	25,411	23,005	0,0350	19,75	**39,50**	0,00426		59,25	1129
								0,0350				0,00316		
2	1,50	20,86	20,06	20,06	4,625	24,684	22,371	0,0350	19,50	**39,00**	0,00415		58,50	1114
								0,0350				0,00307		
3	2,25	20,32	19,52	19,52	4,438	23,957	21,738	0,0350	19,25	**38,50**	0,00403		57,75	1100
								0,0350				0,00298		
4	3,00	19,79	18,98	18,98	4,250	23,230	21,105	0,0350	19,00	**38,00**	0,00391		57,00	1086
								0,0350				0,00289		
5	3,75	19,25	18,44	18,44	4,063	22,503	20,471	0,0350	18,75	**37,50**	0,00379		56,25	1071
								0,0350				0,00280		
6	4,50	18,71	17,90	17,90	3,875	21,776	19,838	0,0350	18,50	**37,00**	0,00367		55,50	1057
								0,0350				0,00182		
6,67	5,00	18,34	17,54	17,54	3,749	21,288	19,414	0,0350	18,33	**36,67**	0,00358		55,00	1048
	15,00								30,00	15			45,00	

Author: Stig Bernander	SCR	0,30	c_R=	12,000					m
Calc.	S(x)		$c_R/c_{(peak)}$	0,20	S:a N x(n)	0,00	S:a d(xn),t	0,01995	
			dN	67,00	S:a d(xn),N	0,0000	S(x)=		m
Calc.			d(xn,n+1),N	0,01995	S:a Nx(n+1)	**67,00**	Slip d(slip) =	0,00000	m
			dx,N-dx(t)	**0,00000**	S:a dx(n+1),N	**0,01995**	S:a dx(n+1),t	**0,01995**	m

* In situ shear stress to be modified by the expression dt =(E(n+1) -E(n))/d: (E(n+1) and E(n) are in situ earth pressures)

Downhill progressive slide - triggering load **Stage I** (2) **17 04 20** **Page 3**

	Hq	26,93	26,31	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**15,487**
step	**No 2**	**x1**	x2	**26,80**	**Stress**	**x2**	**Mean**		kN/m2		kN/m2		**x2**	**42,29**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20			to(x,z)	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:a**			
0	0,00	21,14	**20,98**	**25,98**	**6,268**	**32,245**	29,111	0,0350	20,00	**40,00**	0,00986		60,00	1143
								0,0350				0,00711		
1	0,75	20,60	**20,44**	24,68	6,033	**30,711**	27,695	0,0350	19,75	**39,50**	0,00910		59,25	1129
								0,0350				0,00654		
2	1,50	20,06	**19,90**	23,38	5,798	**29,177**	26,278	0,0350	19,50	**39,00**	0,00833		58,50	1114
								0,0350				0,00595		
3	2,25	19,52	**19,36**	22,08	5,563	**27,643**	24,862	0,0350	19,25	**38,50**	0,00753		57,75	1100
								0,0350				0,00534		
4	3,00	18,98	**18,82**	20,78	5,328	**26,109**	23,445	0,0350	19,00	**38,00**	0,00672		57,00	1086
								0,0350				0,00472		
5	3,75	18,44	**18,28**	19,48	5,093	**24,575**	22,029	0,0350	18,75	**37,50**	0,00588		56,25	1071
								0,0350				0,00408		
6	4,50	17,90	**17,74**	18,18	4,858	**23,042**	20,613	0,0350	18,50	**37,00**	0,00502		55,50	1057
								0,0350				0,00237		
6,67	5,00	17,54	**17,38**	17,31	4,700	**22,014**	19,664	0,0350	18,33	**36,67**	0,00443		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,000
Calc.	S(x)		cR/C(peak)	0,20
			dN	124,72
Calc.			d(xn,n+1),N	0,01617
			dx,N-dx(t)	**0,00000**

S:a N x(n)	67,00
S:a d(xn),N	0,01995
S:a Nx(n+1)	**191,72**
S:a dx(n+1),N	**0,03612**

			m
	S:a d(xn),t	0,03612	
	S(x)=		m
Slip	d(slip) =	0,00000	m
	S:a dx(n+1),t	**0,03612**	m

Downhill progressive slide - triggering load **Stage I** (3) **17 04 20** **Page 4**

	Hq	26,31	26,03	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**7,011**
step	**No 3**	**x2**	x3	**42,29**	**Stress**	**x2**	**Mean**		kN/m2		kN/m2		**x3**	**49,30**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ.I:4a**			
0	0,00	20,98	**20,93**	**32,20**	**10,000**	**42,196**	37,196	0,0350	20,00	**40,00**	0,01867		60,00	1143
								0,0350				0,01359		
1	0,75	20,44	**20,39**	30,59	9,625	**40,211**	35,399	0,0350	19,75	**39,50**	0,01757		59,25	1129
								0,0350			**Equ 1:a**	0,01277		
2	1,50	19,90	**19,85**	28,98	9,250	**38,227**	33,602	0,0350	19,50	**39,00**	0,01649		58,50	1114
								0,0350				0,01196		
3	2,25	19,36	**19,31**	27,37	8,875	**36,242**	31,804	0,0350	19,25	**38,50**	0,01539		57,75	1100
								0,0350				0,01112		
4	3,00	18,82	**18,77**	25,76	8,500	**34,257**	30,007	0,0350	19,00	**38,00**	0,01426		57,00	1086
								0,0350				0,01026		
5	3,75	18,28	**18,23**	24,15	8,125	**32,272**	28,210	0,0350	18,75	**37,50**	0,01311		56,25	1071
								0,0350				0,00938		
6	4,50	17,74	**17,69**	22,54	7,750	**30,287**	26,412	0,0350	18,50	**37,00**	0,01192		55,50	1057
								0,0350				0,00578		
6,67	5,00	17,38	**17,33**	21,46	7,499	**28,958**	25,208	0,0350	18,33	**36,67**	0,01110		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,000						m
Calc.	S(x)		cR/C(peak)	0,20	S:a N x(n)	191,72		S:a d(xn),t	**0,07487**	
			dN	114,05	S:a d(xn),N	0,03612		S(x)=		m
Calc.			d(xn,n+1),N	0,03876	S:a Nx(n+1)	**305,78**	Slip	d(slip) =		m
			dx,N-dx(t)	**0,00000**	S:a dx(n+1),N	**0,07487**		S:a dx(n+1),t	**0,07487**	m

Downhill progressive slide - triggering load **Stage I** (3) **17 04 20** **Page 5**

	Hq	26,03	25,82	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**5,323**
step	**No 4**	**x3**	x4	**49,30**	**Stress**	**x4**	**Mean**		kN/m2		kN/m2		**x4**	**54,62**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ.I:4a**			
0	0,00	20,93	**20,86**	**42,13**	**10,615**	**52,740**	47,433	0,0350	20,00	**40,00**	0,03066		60,00	1143
								0,0350				0,02213		
1	0,75	20,39	**20,32**	40,02	10,217	**50,236**	45,127	0,0350	19,75	**39,50**	0,02835		59,25	1129
								0,0350				0,02047		
2	1,50	19,85	**19,78**	37,91	9,819	**47,731**	42,822	0,0350	19,50	**39,00**	0,02624		58,50	1114
								0,0350				0,01895		
3	2,25	19,31	**19,24**	35,81	9,421	**45,227**	40,517	0,0350	19,25	**38,50**	0,02428		57,75	1100
								0,0350				0,01752		
4	3,00	18,77	**18,70**	33,70	9,023	**42,723**	38,211	0,0350	19,00	**38,00**	0,02244		57,00	1086
								0,0350				0,01617		
5	3,75	18,23	**18,16**	31,59	8,625	**40,218**	35,906	0,0350	18,75	**37,50**	0,02069		56,25	1071
								0,0350				0,01489		
6	4,50	17,69	**17,62**	29,49	8,227	**37,714**	33,601	0,0350	18,50	**37,00**	0,01901		55,50	1057
								0,0350				0,00928		
6,67	5,00	17,33	**17,26**	28,08	7,960	36,036	32,056	0,0350	18,33	**36,67**	0,01793		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,000
Calc.	S(x)		cR/c(peak)	0,20
			dN	141,46
Calc.			d(xn,n+1),N	0,04454
			dx,N-dx(t)	**0,00000**

					m
S:a N x(n)	305,78		S:a d(xn),t	**0,11941**	
S:a d(xn),N	0,07487		S(x)=		m
S:a Nx(n+1)	**447,24**	Slip	d(slip) =		m
S:a dx(n+1),N	**0,11941**		S:a dx(n+1),t	**0,11941**	m

Downhill progressive slide - triggering load **Stage I** (5) 17 04 20 Page 6

	H_q	25,82	25,72	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**2,275**
step	**No 5**	**x4**	x5	**54,62**	**Stress**	**x5**	**Mean**		kN/m2		kN/m2		**x5**	**56,90**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ.I:4a**			
0	0,00	20,86	20,84	**52,73**	**4,830**	**57,556**	55,141	0,0350	20,00	**40,00**	0,03953		60,00	1143
								0,0350				0,02805		
1	0,75	20,32	20,30	50,09	4,649	**54,739**	52,415	0,0350	19,75	**39,50**	0,03528		59,25	1129
								0,0350				0,02521		
2	1,50	19,78	19,76	47,45	4,468	**51,922**	49,688	0,0350	19,50	**39,00**	0,03193		58,50	1114
								0,0350				0,02288		
3	2,25	19,24	19,22	44,82	4,287	**49,104**	46,961	0,0350	19,25	**38,50**	0,02907		57,75	1100
								0,0350				0,02084		
4	3,00	18,70	18,69	42,18	4,106	**46,287**	44,234	0,0350	19,00	**38,00**	0,02651		57,00	1086
								0,0350				0,01900		
5	3,75	18,16	18,15	39,54	3,924	**43,469**	41,507	0,0350	18,75	**37,50**	0,02417		56,25	1071
								0,0350				0,01731		
6	4,50	17,62	17,61	36,91	3,743	**40,652**	38,780	0,0350	18,50	**37,00**	0,02199		55,50	1057
								0,0350				0,01070		
6,67	5,00	17,26	17,24	35,14	3,622	**38,764**	36,953	0,0350	18,33	**36,67**	0,02060		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,000						m
Calc.	S(x)		cR/C(peak)	0,20	S:a N x(n)	447,24		S:a d(xn),t	**0,14399**	
			dN	78,03	S:a d(xn),N	0,11941		S(x)=		m
Calc.			d(xn,n+1),N	0,02458	S:a Nx(n+1)	**525,27**	Slip	d(slip) =		m
			dx,N-dx(t)	**0,00000**	S:a dx(n+1),N	**0,14399**		S:a dx(n+1),t	**0,14399**	m

Downhill progressive slide - triggering load **Stage I** (6) **17 04 20** **Page 7**

Hq		25,72	25,65	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,00**	3000	**40,00**	15,0	dx (m)	**1,900**
step	**No 6**	**x5**	x6	**56,90**	**Stress**	**x6**	**Mean**		kN/m2		kN/m2		**x6**	**58,80**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ.I:4a**			
0	0,00	20,84	20,82	**57,53**	**2,470**	**60,00**	58,764	0,0350	20,00	**40,00**	0,05159		60,00	1143
								0,0350				0,03446		
1	0,75	19,45	20,28	54,65	2,377	**57,03**	55,842	0,0350	19,75	**39,50**	0,04030		59,25	1129
								0,0350				0,02846		
2	1,50	18,06	19,74	51,78	2,285	**54,06**	52,919	0,0350	19,50	**39,00**	0,03559		58,50	1114
								0,0350				0,02533		
3	2,25	16,67	19,20	48,90	2,192	**51,09**	49,996	0,0350	19,25	**38,50**	0,03196		57,75	1100
								0,0350				0,02282		
4	3,00	15,29	18,66	46,02	2,100	**48,12**	47,073	0,0350	19,00	**38,00**	0,02889		57,00	1086
								0,0350				0,02065		
5	3,75	13,90	18,12	43,15	2,007	**45,15**	44,150	0,0350	18,75	**37,50**	0,02616		56,25	1071
								0,0350				0,01869		
6	4,50	12,51	17,58	40,27	1,914	**42,18**	41,228	0,0350	18,50	**37,00**	0,02368		55,50	1057
								0,0350				0,01151		
6,67	5,00	17,24	17,22	38,34	1,852	**40,20**	39,269	0,0350	18,33	**36,67**	0,02211		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,000						m		
Calc.	S(x)		cR/c(peak)	0,20	S:a N x(n)	525,27		S:a d(xn),t	**0,16191**			**0,14399**
			dN	72,10	S:a d(xn),N	0,14399		S(x)=		m		
Calc.			d(xn,n+1),N	0,02370	S:a Nx(n+1)	**597,37**	Slip	d(slip) =		m		
			dx,N-dx(t)	**0,00578**	S:a dx(n+1),N	**0,16769**		S:a dx(n+1),t	**0,16191**	m		

Downhill progressive slide - triggering load **Stage I** (7) **17 04 20** **Page 8**

H_q		25,65	25,61	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**60,01**	3000	**40,00**	15,0	dx (m)	**1,000**
step	**No 7**	**x6**	x7	**58,80**	**Stress**	**x7**	**Mean**		kN/m2		kN/m2		**x7**	**59,80**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ.I:4a**			
0	0,00	20,82	20,81	**59,99**	**0,010**	**60,00**	59,995	0,0350	20,01	**40,00**	0,05102		60,01	1143
								0,0350				0,03421		
1	0,75	19,43	20,27	56,99	0,010	**57,00**	56,995	0,0350	19,76	**39,50**	0,04021		59,26	1129
								0,0350				0,02838		
2	1,50	18,04	19,73	53,99	0,009	**54,00**	53,996	0,0350	19,51	**39,00**	0,03547		58,51	1114
								0,0350				0,02523		
3	2,25	16,65	19,19	50,99	0,009	**51,00**	50,996	0,0350	19,26	**38,50**	0,03182		57,76	1100
								0,0350				0,02271		
4	3,00	15,27	18,65	47,99	0,009	**48,00**	47,996	0,0350	19,01	**38,00**	0,02873		57,01	1086
								0,0350				0,02052		
5	3,75	13,88	18,11	44,99	0,008	**45,00**	44,997	0,0350	18,76	**37,50**	0,02598		56,26	1071
								0,0350				0,01855		
6	4,50	12,49	17,57	41,99	0,008	**42,00**	41,997	0,0350	18,51	**37,00**	0,02348		55,51	1057
								0,0350				0,01140		
6,67	5,00	17,22	17,21	39,98	0,007	**39,99**	39,987	0,0350	18,34	**36,66**	0,02191		55,00	1048

Author: Stig Bernander	SCR	0,30	cR=	12,002
Calc.	S(x)		cR/c(peak)	0,20
			dN	39,19
Calc.			d(xn,n+1),N	0,01371
			dx,N-dx(t)	**0,02040**

					m
S:a N x(n)	597,37		S:a d(xn),t	**0,16100**	
S:a d(xn),N	0,16769		S(x)=		m
S:a Nx(n+1)	**636,56**	Slip	d(slip) =		m
S:a dx(n+1),N	**0,18140**		S:a dx(n+1),t	**0,16100**	m

Transfer to Stage II

Downhill progressive slide - triggering load

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

C_R = 15 kN/m2 **t_o = 20.8 kN/m2**

Step	x		t_{el}	t	N	S:a dn		x	y	H+y	Gradient		
	m		kN/m2	kN/m2	kN/m	m		m			Gr		
0	0,00		**40,00**	**21,14**	0	0,000	**$t = t_o$**	0,00	0,00000	20,000	0,0652	**Equ. I:4a**	**Stage I**
1	26,80			26,14	67,0	0,020		26,80	1,74741	21,747	0,0652		"
2	42,29			32,24	191,7	0,036		42,29	2,75719	22,757	0,0652		"
3	49,30			42,20	305,8	0,075		49,30	3,21432	23,214	0,0652		"
4	54,62			52,74	447,2	0,119		54,62	3,56139	23,561	0,0652		"
5	56,90			57,56	525,3	0,144		56,90	3,70972	23,710	0,0652		"
6	58,80			60,00	597,4	0,168		58,80	3,83361	23,834	0,0652		"
7	**59,80**			**60,00**	**636,6**	**0,181**	**$t=c_{peak}$**	59,80	3,89881	23,899	0,0652	**$t < c_{peak}$**	"
8	62,53			54,94	736,7	0,223		62,53	4,07686	24,077	0,0652	**Equ. I:5a**	**Stage IIa**
9	65,05			44,86	810,3	0,266		65,05	4,24133	24,241	0,0652		"
11	66,83	L_{cr}		34,81	844,2	0,299		66,83	4,35716	24,357	0,0652		"
12	68,76			24,75	861,8	0,336		68,76	4,48300	24,483	0,0652		"
13	70,69	L_{instab}		20,63	**865,7**	**0,373**	**N_{cr}**	70,69	4,60884	24,609	0,0652		"
14	81,09			20,98	867,5	0,748	$t_o = c_{R(x)}$	81,09	5,28694	25,287	0,0652		"
15	86,43			20,98	868,5	0,941	**L_{instab}**	86,43	5,63512	25,635	0,0652		"
16	126,43			20,98	875,5	2,395		126,43	8,24319	28,243	0,0652	**Equ. I:5c**	**Stage IIb**
17	214,48			20,98	891,0	5,635		214,48	13,98421	33,984	0,0652		"
18	385,07			20,98	920,9	12,075		385,07	25,10698	45,107	0,0652		"

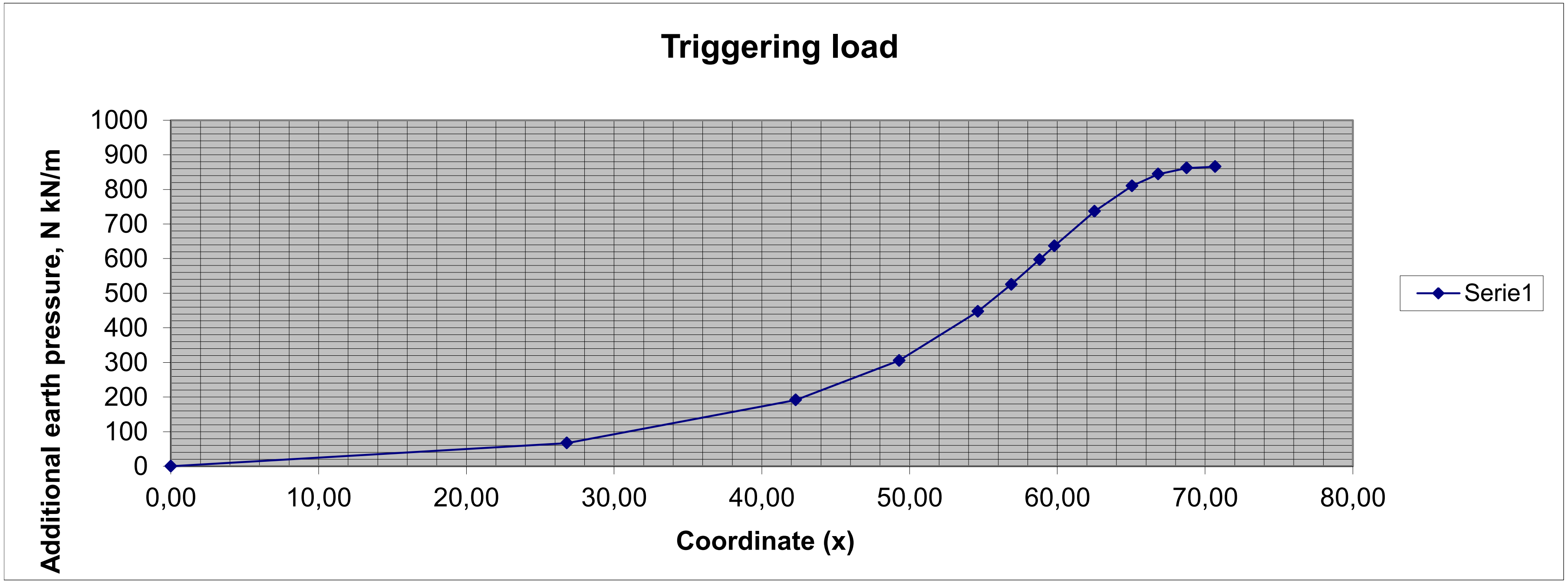
q_{crit} **57,71** kN/m2

Downhill progressive slide - triggering load

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

C_R = 15 kN/m2 t_o = 20.8 kN/m2

Author: Stig Bernander



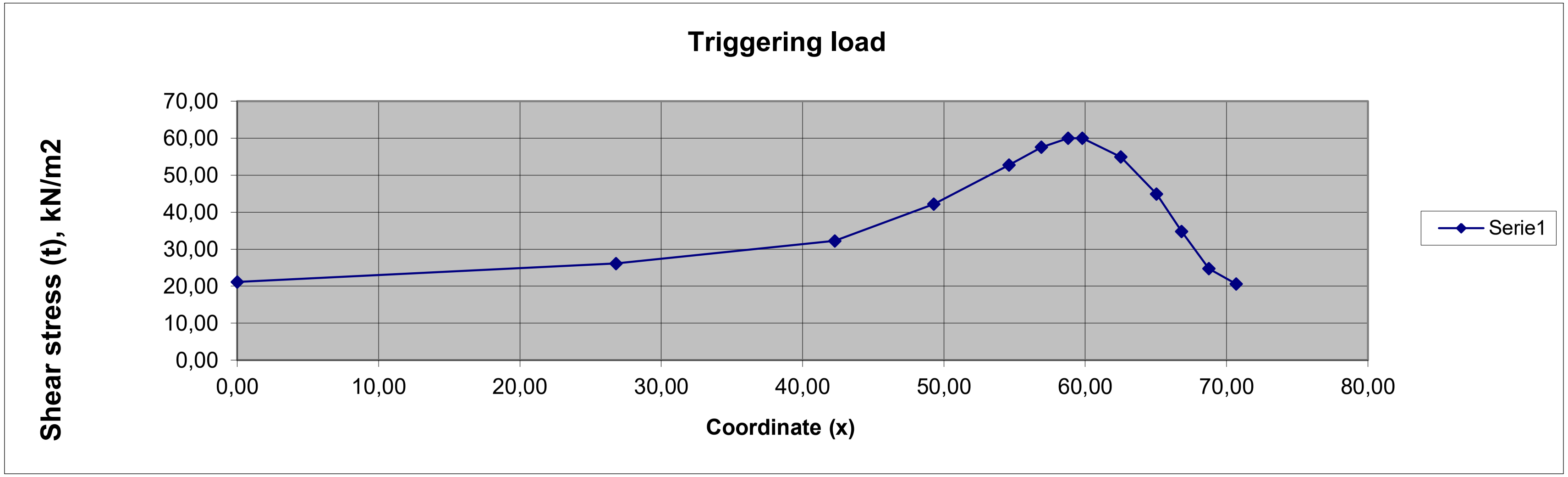
Case I - Slope 1:20, arctan 0,05 = 2,8624 dgr

Downhill progressive slide - triggering load

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

C_R = 15 kN/m2 t_o = 20.8 kN/m2

Author: Stig Bernander



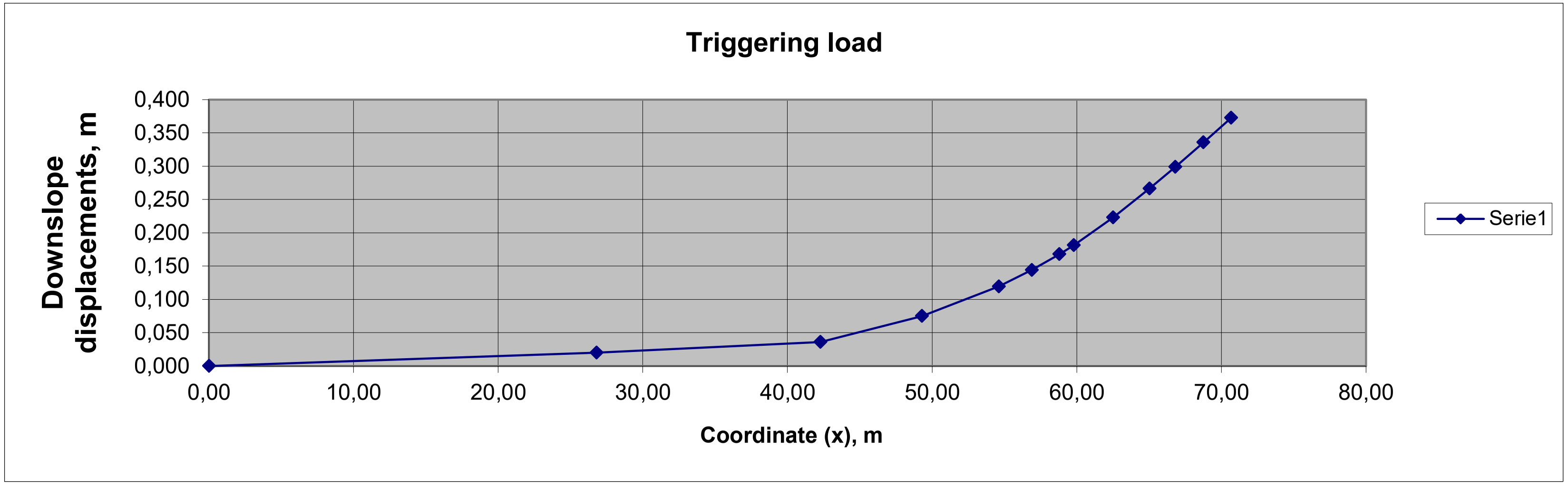
Case I - Slope 1:20, arctan 0,05 = 2,8624 dgr

Downhill progressive slide - triggering load

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

C_R = 15 kN/m2 τ_o = 20.8 kN/m2

Author: Stig Bernander



arctan 0,04		0,03997869
sin arctan 0,004		0,03996804
18*15*sinarctan		10,7913704
18,8*28*sinarctan		21,0391754
delta E		9,86895336
delta to	7000/35(	20
tauo		41,9615924

Explanation of denotations

dE/dx		G=	* Shear stress before adding current stress increment	* Overload density	19,3 kN/m3
9,9	Ok	1142,86	* L = Length of the defined slope	* Overload thicknesses	30,00 31,00
			* x = Coordinate from upper boundary conditon, where shear stress increment, dN and d(x) = 0		26,80 0,00
		OK	* x' = Coordinate from upper slope limit as defined by L		
		Equ.I:4	* x" = Coordinate from lower slope limit = L - x'		

H_q

###########		$\Delta t_{(q)}$	**k* γ*((Hn)2-((Hn+1)2)/(2Δx)**		-5,9 kN/m	
			- dH*dx*γ*sin arctan(H33)		20,7 kN/m	
###########						
###########					20,7	
###########						
Equ.I:4a						
###########						
Equ.I:1a						
###########						
###########						
	23,16	23,66	23,23	23,64		0,0000
###########	15,44	15,77	15,483	19,41		
	0,667	0,666	0,667	0,821		

-

Downhill progressive slide - triggering load, basic ex. **Author: Stig Bernander** **OK** **Page 1**

Muskrats Fals , Case 4- Slope 1:25, arctan 0,04 = 2,292dgr **17 06 01**

C_R = 14 kN/m2

Input data:

H(x = L) = 15 m **Gradient: Slope 1:25, arctan 0,04 = 2,292dgr**

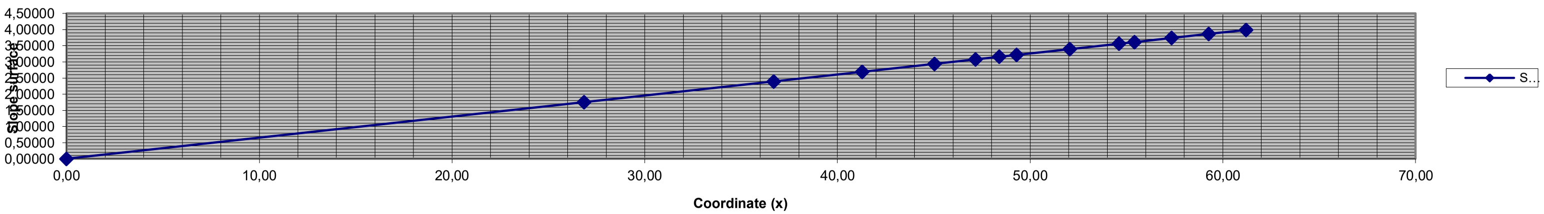
Density = 18,0 kN/n **(Residual shear strength)** **(Surface strength)** **(Mean shear strength)**

C_{peak} = 70 kN/m2 C_{lab} = 55 kN/m2 C_R = 14 kN/m2 C_R/C_{lab} = 0,255 C_s = 45kN/m2 C_{mean} = 50 kN/m2

Shear deformation at failure (g_f) = 7,0% C_R/C_{peak} = 0,20 t_{el} = 40 kN/m2

Post-peak deformation at C_R = 0,30 r Shear deform. at elastic limit (g_{el}) = 3.50 %

Landslide spread



$t_{x,z} < t_{el}$	**$t_{x,z} < t_{el}$**
$g_{x,z}(1) = t_x/G$, $g_{el} = g_f \cdot t_{el}/(2c - t_{el})$ and $G = t_{el}/g_{el}$	**Stage I** **Equ. I:1: S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(1)$)dz**
$t_o < t_{xz} < c$, $t_o > t_{el}$	**$t_o < t_{xz} < c$** $t_o > t_{el}$
$g_{x,z}(4) = (g_f - g_{el}) \cdot (1 - ROT(1 - (t_{o(x,z)} - t_{el})/(c - t_{el}))$	**Stage I** **Equ. I:4 S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(4)$)dz**
$t_o < t_{el}$ **$- ROT(1 - (t_{(x,z)} - t_{el})/(c - t_{el})))$**	$t_o < t_{el}$
$g_{x,z}(4a) = (t_{el} - t_{o,(x,z)})/G + (g_f - g_{el}) \cdot (1 - ROT(1 - (t_{(x,z)} - t_{el})/(c - t_{el})))$	**Equ. I:4: S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(4a)$)dz**
$c < t_{xz} < c_R$, $t_o > t_{el}$	**$c < t_{xz} < c_R$** $t_o > t_{el}$ $t_{xz} > c_R$
$g_{x,z}(5) = (g_f - g_{el}) \cdot (ROT(1 - (t_{o(x,z)} - t_{el})/(c - t_{el})) -$	**Stage II** **Equ. I:5 S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(5)$)dz +**
$t_o < t_{el}$ **$- ROT(1 - (t_{(x,z,max)} - t_{el})/(c - t_{el}))) - (t_{(x,z,max)} - t_{(x,z)})/G$**	$t_o < t_{el}$ $t_{xz} < c_R$ **$+S_{CR}(c - t_{x,z})/(c - c_R)$**
$g_{x,z}(5a) = (t_{el} - t_{o,(x,z)})/G + (g_f - g_{el})(1 - ROT(1 - (t_{(x,z,max)} - t_{el})/(c - t_{el})))-$	**Stage IIa** **Equ. I:5: S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(5a)$)dz +**
$t_{x,o} = c_R$ **$- (t_{(x,z,max)} - t_{(x,z)})/G$**	**$t_{x,o} = c_R$** **$+S_{CR}(c - t_{x,z})/(c - c_R)$**
$g_{x,z}(5b) = g_{x,z}(5)$ (or $g_{x,z}(5a)$) **$- (t_{(x,z,max)} - c_R)/G$**	**Stage IIb** **Equ. I:5 S:a $d_{(x,z),t}$ = Integral ($g_{x,z,}(5c)$)dz+S_{CR}+S_{slip}**

Downhill progressive slide - triggering load **Stage I** (1) **17 06 01** **Page 2**

Hq		28	26,93	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1
Iter.		g f =	0,070	18,000	**45**	0,0280	0,0400	1429	**70,00**	3520	**40,00**	15,0	dx (m)	**26,850**
step	**No 1**	**xo**	x1	**0,00**	**Stress**	**x1**	**Mean**		kN/m2		kN/m2		**x1**	**26,85**
n	**z (n)**	**In situ***	**In situ***	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20			to(x,z)	t(x,z)	dt(x,z)	t(x,z)+dt		0,0420	30,00		**Equ 1:4**			
0	0,00	41,94	41,14	**41,14**	**4,000**	45,137	43,137	0,0420	30,00	**40,00**	0,00296		70,00	1429
								0,0420				0,00218		
1	0,75	40,40	39,60	39,60	3,850	43,447	41,522	0,0420	29,46	**39,29**	0,00286		68,75	1403
								0,0420				0,00212		
2	1,50	39,40	39,06	39,06	3,700	42,758	40,908	0,0420	28,93	**38,57**	0,00280		67,50	1378
								0,0420				0,00208		
3	2,25	39,33	38,52	38,52	3,550	42,068	40,293	0,0420	28,39	**37,86**	0,00275		66,25	1352
								0,0420				0,00204		
4	3,00	38,79	37,98	37,98	3,400	41,379	39,679	0,0420	27,86	**37,14**	0,00269		65,00	1327
								0,0420				0,00199		
5	3,75	38,25	37,44	37,44	3,250	40,689	39,064	0,0420	27,32	**36,43**	0,00263		63,75	1301
								0,0420				0,00195		
6	4,50	37,71	36,90	36,90	3,100	39,999	38,449	0,0420	26,79	**35,71**	0,00257		62,50	1276
								0,0420				0,00128		
6,7	5,00	37,34	36,54	36,54	3,000	39,537	38,038	0,0420	26,43	**35,24**	0,00252		61,66	1258
	15,00								32,14	12,857			45,00	

Author: Stig Bernander	SCR	0,30	cR=	14,000						m
Calc.	S(x)		cR/c(peak)	0,20	S:a N x(n)	0,00		S:a d(xn),t	0,01365	
			dN	53,70	S:a d(xn),N	0,0000		S(x)=		m
Calc.			d(xn,n+1),N	0,01365	S:a Nx(n+1)	**53,70**	Slip	d(slip) =	0,00000	m
			dx,N-dx(t)	**0,00001**	S:a dx(n+1),N	**0,01365**		S:a dx(n+1),t	**0,01365**	m

* In situ shear stress to be modified by the expression dt =(E(n+1) - (E(n+1) and E(n) are in situ earth pressures)

Downhill progressive slide - triggering load **Stage I** (2) **17 06 01** **Page 3**

	Hq	26,93	26,53	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1
Iter.		g f=	0,070	18,000	**45**	0,0280	0,0400	1429	**70,00**	3520	**40,00**	15,0	dx (m)	**9,845**
step	**No 2**	**x1**	x2	**26,85**	**Stress**	**x2**	**Mean**		kN/m2		kN/m2		**x2**	**36,70**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20			to(x,z)	t(x,z)	dt(x,z)	t(x,z)+dt		0,0420			**Equ 1:4**			
0	0,00	41,14	41,11	**45,11**	**6,600**	**51,705**	48,405	0,0420	30,00	**40,00**	0,00842		70,00	1429
								0,0420				0,00600		
1	0,75	39,60	39,57	42,85	6,353	**49,202**	46,026	0,0420	29,46	**39,29**	0,00759		68,75	1403
								0,0420				0,00512		
2	1,50	39,06	39,03	40,59	6,105	**46,700**	43,647	0,0420	28,93	**38,57**	0,00605		67,50	1378
								0,0420				0,00396		
3	2,25	38,52	38,49	38,34	5,858	**44,197**	41,268	0,0420	28,39	**37,86**	0,00452		66,25	1352
								0,0420				0,00281		
4	3,00	37,98	37,95	36,08	5,610	**41,694**	38,889	0,0420	27,86	**37,14**	0,00297		65,00	1327
								0,0420				0,00165		
5	3,75	37,44	37,41	33,83	5,363	**39,191**	36,510	0,0420	27,32	**36,43**	0,00142		63,75	1301
								0,0420				0,00048		
6	4,50	36,90	36,87	31,57	5,115	**36,689**	34,131	0,0420	26,79	**35,71**	-0,00014		62,50	1276
								0,0420				-0,00034		
6,7	5,00	36,54	36,51	30,06	4,949	**35,012**	32,537	0,0420	26,43	**35,24**	-0,00120		61,66	1258

Author: Stig Bernander	SCR	0,30	CR=	14,000
Calc.	S(x)		CR/C(peak)	0,20
			dN	71,71
Calc.			d(xn,n+1),N	0,00603
			dx,N-dx(t)	**0,00000**

S:a N x(n)	53,70
S:a d(xn),N	0,01365
S:a Nx(n+1)	**125,41**
S:a dx(n+1),N	**0,01968**

	S:a d(xn),t	0,01968	m
	S(x)=		m
Slip	d(slip) =	0,00000	m
	S:a dx(n+1),t	**0,01968**	m

Downhill progressive slide - triggering load **Stage I** (3) **17 06 01** **Page 4**

	H_q	26,53	26,35	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**70,00**	2816	**40,00**	15,0	dx (m)	**4,595**
step	**No 3**	**x2**	x3	**36,70**	**Stress**	**x2**	**Mean**		kN/m2		kN/m2		**x3**	**41,29**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**cu**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:4**			
0	0,00	41,11	41,07	**51,67**	**6,500**	**58,171**	54,921	0,0350	30,00	**40,00**	0,01239		70,00	1143
								0,0350				0,00886		
1	0,75	39,57	39,53	49,09	6,256	**55,344**	52,216	0,0350	29,46	**39,29**	0,01124		68,75	1122
								0,0350				0,00780		
2	1,50	39,03	38,99	46,50	6,013	**52,516**	49,510	0,0350	28,93	**38,57**	0,00956		67,50	1102
								0,0350				0,00655		
3	2,25	38,49	38,45	43,92	5,769	**49,689**	46,805	0,0350	28,39	**37,86**	0,00790		66,25	1082
								0,0350				0,00531		
4	3,00	37,95	37,91	41,34	5,525	**46,862**	44,099	0,0350	27,86	**37,14**	0,00627		65,00	1061
								0,0350				0,00410		
5	3,75	37,41	37,37	38,75	5,281	**44,035**	41,394	0,0350	27,32	**36,43**	0,00466		63,75	1041
								0,0350				0,00289		
6	4,50	36,87	36,83	36,17	5,038	**41,207**	38,688	0,0350	26,79	**35,71**	0,00306		62,50	1020
								0,0350				0,00127		
6,7	5,00	36,51	36,47	34,44	4,874	**39,313**	36,876	0,0350	26,43	**35,24**	0,00198		61,66	1007

Author: Stig Bernander	S_{CR}	0,30	c_R=	14,000
Calc.	S(x)		$c_R/c_{(peak)}$	0,20
			dN	63,64
Calc.			$d_{(xn,n+1),N}$	0,01710
			dx,N-dx(t)	**0,00000**

S:a N x(n)	125,41		S:a d(xn),t	**0,03678**	m
S:a d(xn),N	0,01968		S(x)=		m
S:a Nx(n+1)	**189,05**	Slip	d(slip) =		m
S:a dx(n+1),N	**0,03678**		S:a dx(n+1),t	**0,03678**	m

Downhill progressive slide - triggering load

Stage I (3) **17 06 01** **Page 5**

	H_q	26,35	26,20	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f=	0,070	18,000	**45**	0,0350	0,0400	1143	**70,00**	2816	**40,00**	15,0	dx (m)	**3,755**
step	**No 4**	**x3**	x4	**41,29**	**Stress**	**x4**	**Mean**		kN/m2		kN/m2		**x4**	**45,05**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:4**			
0	0,00	41,07	41,02	**58,12**	**5,500**	**63,618**	60,868	0,0350	30,00	**40,00**	0,01826		70,00	1143
								0,0350				0,01299		
1	0,75	39,53	39,48	55,21	5,294	**60,506**	57,859	0,0350	29,46	**39,29**	0,01637		68,75	1122
								0,0350				0,01142		
2	1,50	38,99	38,94	52,31	5,088	**57,394**	54,850	0,0350	28,93	**38,57**	0,01409		67,50	1102
								0,0350				0,00976		
3	2,25	38,45	38,40	49,40	4,881	**54,282**	51,841	0,0350	28,39	**37,86**	0,01194		66,25	1082
								0,0350				0,00818		
4	3,00	37,91	37,86	46,49	4,675	**51,170**	48,832	0,0350	27,86	**37,14**	0,00989		65,00	1061
								0,0350				0,00667		
5	3,75	37,37	37,32	43,59	4,469	**48,057**	45,823	0,0350	27,32	**36,43**	0,00790		63,75	1041
								0,0350				0,00520		
6	4,50	36,83	36,78	40,68	4,263	**44,945**	42,814	0,0350	26,79	**35,71**	0,00596		62,50	1020
								0,0350				0,00267		
6,7	5,00	36,47	36,42	38,74	4,124	42,860	40,798	0,0350	26,43	**35,24**	0,00468		61,66	1007

Author: Stig Bernander

	ScR	0,30	cR=	14,000
Calc.	S(x)		cR/c(peak)	0,20
			dN	74,54
Calc.			d(xn,n+1),N	0,02012
			dx,N-dx(t)	**0,00000**

S:a N x(n)	189,05
S:a d(xn),N	0,03678
S:a Nx(n+1)	**263,59**
S:a dx(n+1),N	**0,05690**

			m
	S:a d(xn),t	**0,05690**	
	S(x)=		m
Slip	d(slip) =		m
	S:a dx(n+1),t	**0,05690**	m

Downhill progressive slide - triggering load **Stage I** (5) **17 06 01** **Page 6**

	Hq	26,20	26,11	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f=	0,070	18,000	**45**	0,0350	0,0400	1143	**70,00**	2816	**40,00**	15,0	dx (m)	**2,127**
step	**No 5**	**x4**	x5	**45,05**	**Stress**	**x5**	**Mean**		kN/m2		kN/m2		**x5**	**47,17**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:4**			
0	0,00	41,02	41,00	**63,60**	**3,400**	**66,997**	65,297	0,0350	30,00	**40,00**	0,02334		70,00	1143
								0,0350				0,01640		
1	0,75	39,48	39,46	60,42	3,273	**63,689**	62,053	0,0350	29,46	**39,29**	0,02039		68,75	1122
								0,0350				0,01418		
2	1,50	38,94	38,92	57,24	3,145	**60,382**	58,810	0,0350	28,93	**38,57**	0,01743		67,50	1102
								0,0350				0,01208		
3	2,25	38,40	38,38	54,06	3,018	**57,075**	55,566	0,0350	28,39	**37,86**	0,01478		66,25	1082
								0,0350				0,01017		
4	3,00	37,86	37,84	50,88	2,890	**53,767**	52,322	0,0350	27,86	**37,14**	0,01234		65,00	1061
								0,0350				0,00839		
5	3,75	37,32	37,30	47,70	2,763	**50,460**	49,079	0,0350	27,32	**36,43**	0,01003		63,75	1041
								0,0350				0,00669		
6	4,50	36,78	36,76	44,52	2,635	**47,153**	45,835	0,0350	26,79	**35,71**	0,00782		62,50	1020
								0,0350				0,00357		
6,7	5,00	36,42	36,40	42,39	2,550	**44,937**	43,662	0,0350	26,43	**35,24**	0,00638		61,66	1007

Author: Stig Bernander	ScR	0,30	cR=	14,000						m
Calc.	S(x)		cR/c(peak)	0,20	S:a N x(n)	263,59		S:a d(xn),t	**0,07147**	
			dN	51,69	S:a d(xn),N	0,05690		S(x)=		m
Calc.			d(xn,n+1),N	0,01457	S:a Nx(n+1)	**315,28**	Slip	d(slip) =		m
			dx,N-dx(t)	**0,00000**	S:a dx(n+1),N	**0,07147**		S:a dx(n+1),t	**0,07147**	m

Downhill progressive slide - triggering load **Stage I** (6) **17 06 01** **Page 7**

	Hq	26,11	26,06	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**70,00**	2816	**40,00**	15,0	dx (m)	**1,229**
step	**No 6**	**x5**	x6	**47,17**	**Stress**	**x6**	**Mean**		kN/m2		kN/m2		**x6**	**48,40**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:4**			
0	0,00	41,00	40,98	**66,98**	**1,900**	**68,88**	67,934	0,0350	30,00	**40,00**	0,02767		70,00	1143
								0,0350				0,01908		
1	0,75	38,26	39,44	63,63	1,829	**65,46**	64,549	0,0350	29,46	**39,29**	0,02322		68,75	1122
								0,0350				0,01605		
2	1,50	35,53	38,90	60,29	1,758	**62,04**	61,164	0,0350	28,93	**38,57**	0,01960		67,50	1102
								0,0350				0,01355		
3	2,25	32,80	38,37	56,94	1,686	**58,62**	57,779	0,0350	28,39	**37,86**	0,01654		66,25	1082
								0,0350				0,01138		
4	3,00	30,06	37,83	53,59	1,615	**55,20**	54,395	0,0350	27,86	**37,14**	0,01381		65,00	1061
								0,0350				0,00941		
5	3,75	27,33	37,29	50,24	1,544	**51,78**	51,010	0,0350	27,32	**36,43**	0,01128		63,75	1041
								0,0350				0,00756		
6	4,50	24,60	36,75	46,89	1,473	**48,36**	47,625	0,0350	26,79	**35,71**	0,00889		62,50	1020
								0,0350				0,00408		
6,7	5,00	36,40	36,39	44,64	1,425	**46,07**	45,357	0,0350	26,43	**35,24**	0,00735		61,66	1007

Author: Stig Bernander	SCR 0,30	cR=	14,000
Calc.	S(x)	cR/c(peak)	0,20
		dN	33,12
Calc.		d(xn,n+1),N	0,00965
		dx,N-dx(t)	**0,00000**

S:a N x(n)	315,28
S:a d(xn),N	0,07147
S:a Nx(n+1)	**348,40**
S:a dx(n+1),N	**0,08113**

	S:a d(xn),t	**0,08113**	m
	S(x)=		m
Slip	d(slip) =		m
	S:a dx(n+1),t	**0,08113**	m

0,07147

Downhill progressive slide - triggering load **Stage I** (7) **17 06 01** **Page 8**

	Hq	26,06	26,03	Density	c(s)=	g el =	Gradient	G mod.	c (peak)	E-mean	t el =	H =	Width, b	1,0
Iter.		g f =	0,070	18,000	**45**	0,0350	0,0400	1143	**70,00**	2816	**40,00**	15,0	dx (m)	**0,906**
step	**No 7**	**x6**	x7	**48,40**	**Stress**	**x7**	**Mean**		kN/m2		kN/m2		**x7**	**49,31**
n	**z (n)**	**In situ**	**In situ**	**Shear**	**incre-**	**Shear**	**shear**	**gf - gel**	cu-t el	t el	**d(g)**	**d(g)*dz**	**c**	**G**
	(m)	**shear**	**shear**	**stress**	**ment**	**stress**	**stress**	kN/m2	kN/m2	kN/m2	1	m	kN/m2	
20		to(x,z)	to(x,z)+dt	t(x,z)	dt(x,z)	t(x,z)+dt		0,0350			**Equ 1:4**			
0	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
								0,0350				0,02228		
1	0,75	38,25	39,43	65,43	1,084	**66,51**	65,970	0,0350	29,46	**39,29**	0,02527		68,75	1122
								0,0350				0,01737		
2	1,50	35,52	38,89	61,98	1,042	**63,03**	62,506	0,0350	28,93	**38,57**	0,02104		67,50	1102
								0,0350				0,01452		
3	2,25	32,79	38,35	58,54	0,999	**59,54**	59,041	0,0350	28,39	**37,86**	0,01768		66,25	1082
								0,0350				0,01216		
4	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
								0,0350				0,01005		
5	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
								0,0350				0,00811		
6	4,50	24,59	36,73	48,21	0,873	**49,08**	48,647	0,0350	26,79	**35,71**	0,00956		62,50	1020
								0,0350				0,00440		
6,7	5,00	36,39	36,37	45,90	0,844	**46,75**	46,325	0,0350	26,43	**35,24**	0,00794		61,66	1007

Author: Stig Bernande	ScR	0,30	cR=	14,000		
Calc.	S(x)		cR/c(peak)	0,20		
			dN	25,79		
Calc.			d(xn,n+1),N	0,00775		
			dx,N-dx(t)	**0,00000**		

					m
S:a N x(n)	348,40		S:a d(xn),t	**0,08888**	
S:a d(xn),N	0,08113		S(x)=		m
S:a Nx(n+1)	**374,19**	Slip	d(slip) =		m
S:a dx(n+1),N	**0,08888**		S:a dx(n+1),t	**0,08888**	m

Transfer to Stage II

Downhill progressive slide - triggering load

Case, Appendix I - Slope 1:25, arctan 0,04 =2,29 dgr

C_R = 14 kN/m2

Author: Stig Bernander

Step	x	t_{el}	t	N	S:a dn		x	y	H+y	Gradient			
	m	kN/m2	kN/m2	kN/m	m		m			Gr			
0	0,00	**40,00**	**41,14**	0	0,000	**t = t_o**	0,00	0,00000	20,000	0,0652	**Equ. I:4**	**Stage I**	**Phase 2**
1	26,85		45,14	53,7	0,014		26,85	1,75067	21,751	0,0652	"	"	"
2	36,70		51,71	125,4	0,020		36,70	2,39258	22,393	0,0652	"	"	"
3	41,29		58,17	189,1	0,037		41,29	2,69218	22,692	0,0652	"	"	"
4	45,05		63,62	263,6	0,057		45,05	2,93702	22,937	0,0652	"	"	"
5	47,17		67,00	315,3	0,071		47,17	3,07570	23,076	0,0652	"	"	"
6	48,40		68,88	348,4	0,081		48,40	3,15583	23,156	0,0652	"	"	"
7	**49,31**	**t=c_{peak}**	**70,00**	**374,2**	**0,089**	**t=c_{peak}**	49,31	3,21491	23,215	0,0652	**t < c_{peak}**	"	"
8	52,06		64,93	447,2	0,116		52,06	3,39434	23,394	0,0652	**Equ. 5**	**Stage IIa**	"
9	54,62		54,85	495,7	0,144		54,62	3,56126	23,561	0,0652	"	"	"
11	55,42		49,85	504,9	0,154		55,42	3,61342	23,613	0,0652	"	"	"
12	57,35		44,78	517,3	0,177		57,35	3,73926	23,739	0,0652	"	"	"
13	**59,28**	**L critical**	40,83	**521,1**	**0,201**	**N_{cr}, d_{cr}**	59,28	3,86510	23,865	0,0652	"	"	"
14	61,21		40,77	521,1	0,225		61,21	3,99094	23,991	0,0652	Equ. I:5c	Stage IIb	**Phase 3 ,**
15												"	"
16												"	"
17												"	"
18												"	"

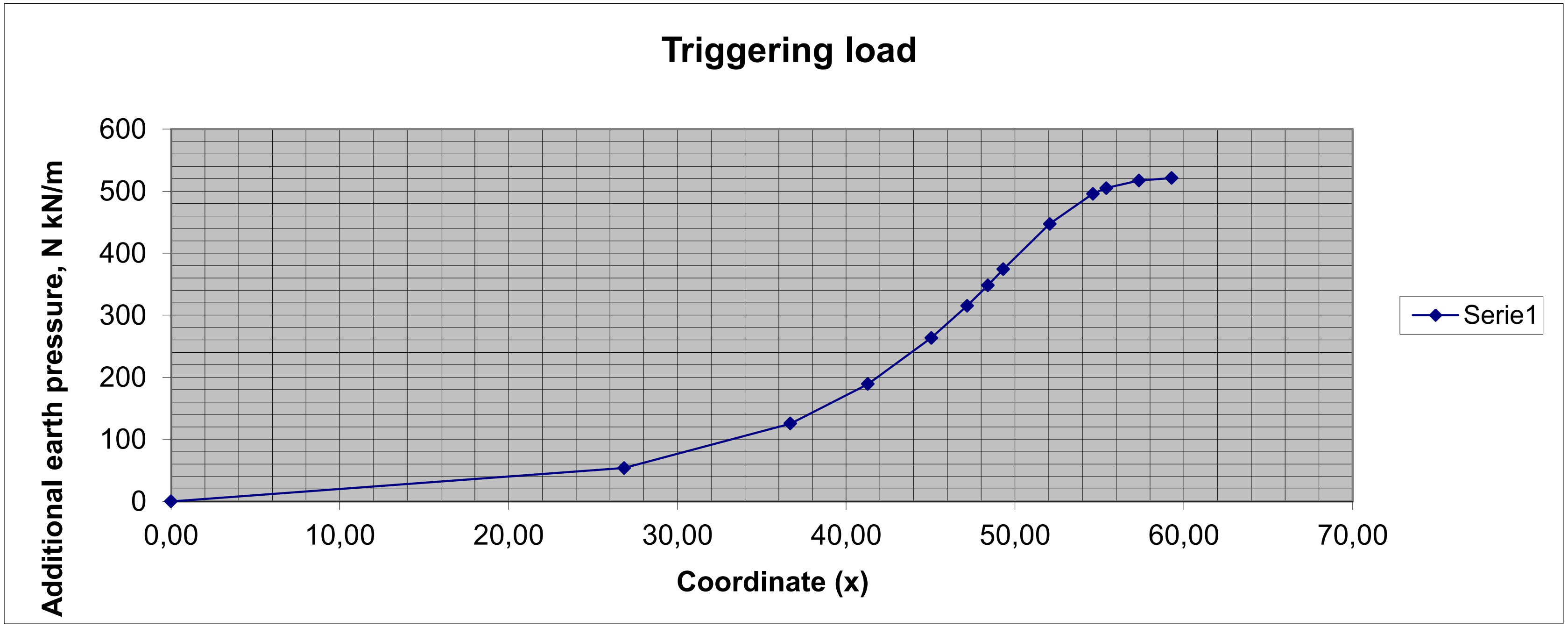
	WL	WL
Water leve	**30**	**39**
Nw =	845,0	1805,0
Safetyfactor (F	**0,62**	**0,29**
(i.e. Ncr/Nw)	531/845	531/1805

Author: Stig Bernander

Downhill progressive slide - triggering load

Case 4, Appendix I - Slope 1:25, arctan 0,04 =2,29 dgr

C_R = 14kN/m2

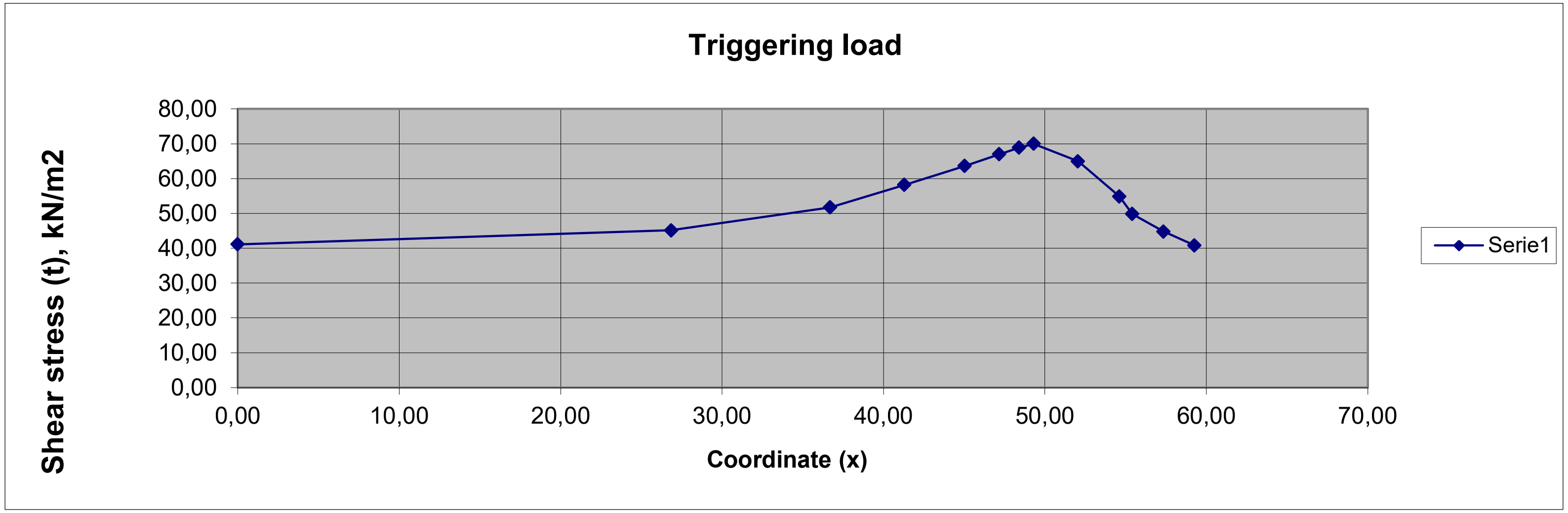


Downhill progressive slide - triggering load

Case 4, Appendix II - Slope 1:25, arctan 0,04 =2,29 dgr

C_R = 14 kN/m2

Author: Stig Bernander

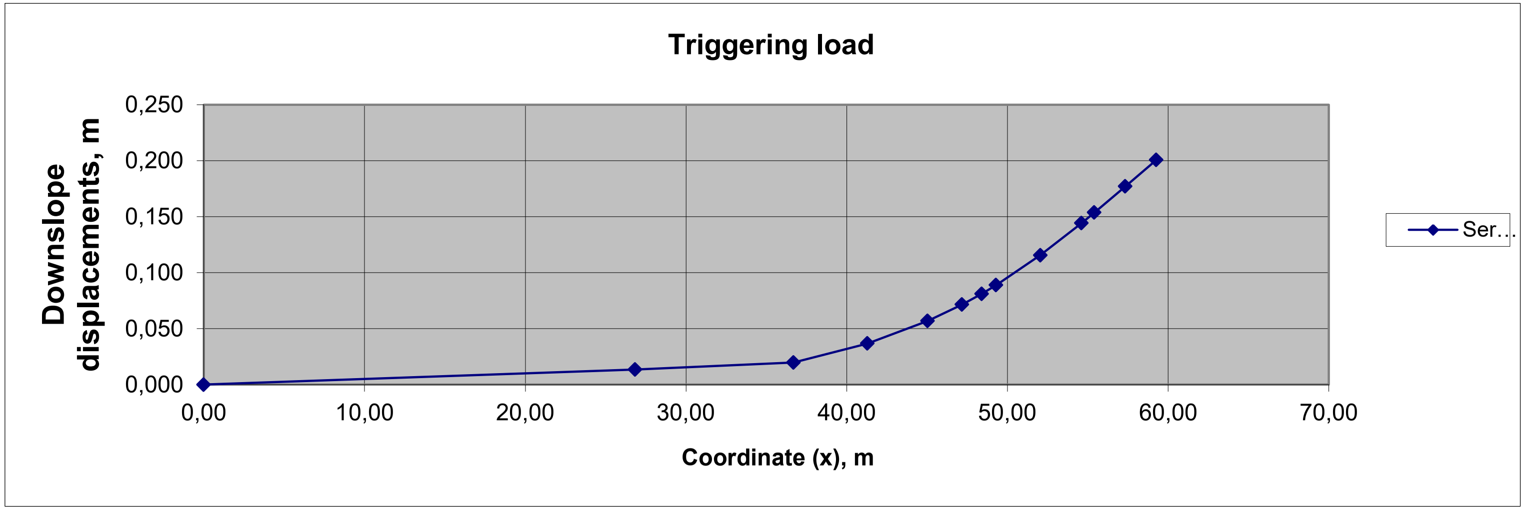


Author: Stig Bernander

Downhill progressive slide - triggering load

Case 4, Appendix III - Slope 1:25, arctan 0,04 =2,29 dgr

C_R = 14 kN/m2



Author: Stig Bernander

Explanation of denotations

dE/dx		G=	* Shear stress before adding current stress increment	* Overload density	19,3
9,9	Ok	1428,57	* L = Length of the defined slope	* Overload thicknesses	30,00
20	7000		* x = Coordinate from upper boundary conditon, where shear stress increment, dN and d(x) = 0		26,85
N_q/L	N_q	**OK**	* x' = Coordinate from upper slope limit as defined by L		
L	350	**Equ.I:4**	* x" = Coordinate from lower slope limit = L - x'		

H_q

########### $\Delta t_{(q)}$ **k* γ*((Hn)2-((Hn+1)2)/(2Δx)** -5,9 kN/m

- dH*dx*γ*sin arctan(H33) 20,7 kN/m

###########

########### 20,7

###########

Equ.I:4a

###########

Equ.I:1a

###########

###########

	23,16	23,66	23,23	43,14	0,0000
###########	15,44	15,77	15,483	38,04	
	0,667	0,666	0,667	0,882	

-

V. Summing up of North Spur stability issues

By Stig Bernander, October 23, 2017

Contents

Summing up of North Spur stability issues

PREPARED FOR

Grand Riverkeeper Labrador, Inc.

by

Dr. Stig Bernander
October 23, 2017

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Table of Contents

Executive Summary

a) Many of the issues dealt with in the current report ***only apply*** to ***fully water-saturated*** highly porous soils.
Note also that in such soils, neither the ***maximum*** *shear strength* nor the ***residual*** *shear resistance* may *comply at all* with the ***laboratory tested*** *drained peak shear strengths* or with *prevailing* piezometric levels. This is because liquefaction (or significant loss of residual shear strength) in ***saturated porous soils*** is mainly related to the ***degree*** of deviatory ***deformation*** involved rather than to concurrent stress levels and piezometric levels. (Confer *Issues No: 2 & 6* and especially Figure 2:1 below.)
Unless it can be *clearly* established that potential failure-surfaces can ***only*** develop in soil layers, in which the prevailing in-situ porosity is of the same magnitude as the critical porosity – i.e. that $\mathbf{n} \approx \mathbf{n_{crit}}$ – calculated safety factors based on the Limit Equilibrium Model (LEM) have ***little relevance*** and the true values will remain ***unknown*** until a ***correct*** approach has been made.

b) One of the ***most questionable*** – and in author's opinion seriously ***erroneous*** – assumptions in the ENGINEERING REPORT, (2015), Ref. [2b], is that the enormous pressure on the COW can be balanced by plastic LEM failure in ***extensive horizontal*** failure planes.

The main point, among other is: How can the geometry of *potential failure surfaces* ever be taken for granted in a mixed porous and sensitive soil mass, the detailed structure of which is not even precisely defined? How can inclining failure planes, in this context, be exempted from general rules in soil mechanics??

c) ***Progressive failure formation behind the COW.***
Water contents greater than the Liquid Limit (i.e. w >> LL) manifest the ***risk of liquefaction*** or of ***high sensitivity*** – in this context indicating the presence of highly sensitive porous layers, not only in the ***Stratified Drift***, but *also* in the ***Lower Clay Formation***.
Such layers may well, as can be concluded from IWLSC (2013)-posters (Ref. [1]), be sloping east-wards towards the deep whirlpool, in accordance with Figures 3:1a & 3:1b (in ***Issue No 3:*** below), and may as already mentioned ***not*** be presumed to being horizontal, as in the Nalcor/SNC-Lavalin ENGINEERING REPORT, Ref. [2b].
Moreover, in general failure planes may of course just as well develop in the *porous sands* and *porous silty sands* as in the lean sandy silty clays of the Stratified Drift. (Confer, for instance, the shape of the 1978 Landslide on the East side of the North Spur.)

Applying the soil properties – based on Tables 2.1 and 2.2 in the ENGINEERING REPORT – to progressive failure along sloping failure planes, analyses by R. Dury and by the author of this report indicate that acceptable safety factors ($F_s > 1.5$) are hard to establish, provided the critical ***water-saturated*** and ***porous*** soil layers have not been duly ***compacted*** or ***de-saturated***.

d) ***About possible progressive failure behind the COW*** in the ***homogeneous Lower Clay?***
Yet, even if the Lower Clay were assumed to be a ***perfectly homogeneous clay*** structure, the effects of progressive failure development along *steeply inclining* failure planes may ***not*** be disregarded but must instead be thoroughly ***investigated***. (Confer Ref [15], Section 6. Concluding Remarks, last sentence.)

Conclusions from Points c) and d): The effects of strain- and deformation-softening in the soils immediately behind the COW – due to the enormous pressure build-up related to impoundment – must be investigated as possible ***progressive failure*** events, and that ***both*** in the ***Upper Clay (2)*** and in the ***Lower Clay*** formations.

e) ***About finger drains and drainage.*** The conditions triggering progressive slope failure are not long extended processes in time. The rate of deformation at the ***end*** of the ***triggering phase*** is normally high, implying that the excess pore-water pressures, liable to build-up at this stage, may not dissipate fast enough through a few *inter- distant* finger drains.
The problem is of course aggravated the greater the difference ($\mathbf{n - n_{crit}}$) happens to be.

Hence, are finger drains a truly ***reliable*** *measure* for the prevention of triggering progressive failure development behind the COW along sloping failure surfaces**??**
This issue must be proven beyond any shadow of a doubt.

f) ***Remedial measures:*** The risk of ***progressive failure development*** may be dealt with by installing efficient *closely spaced* ***drains*** extending through the ***Stratified Drift*** and ***deep down*** into the ***Lower Clay formation***. In these drains, the hydraulic pressure is to be controlled by deep pump wells operating all the time during impoundment.

This drainage should cover an area stretching at least up to some 40 → 50 m East of the COW.

Report: **Regarding Various Issues 1 → 8**

Issue No 1: **About the miss-use** of the **Limit Equilibrium Mode** (LEM) for slope stability analysis.

According to the SNC/Lavalin, reports, (e.g. Ref. [2b]), all predictions of slope stability have – in one way or another – been based on the Limit Equilibrium Mode of analysis (LEM). **Ref. [2b]**, (Chapter 2,'Geology and Soil data' in Sections 2.3.**3** → **4**, Tables 2.1 and 2,2).

This is of course *extremely odd* from a modern R&D point of view, as extensive ***forward progressive*** landslides in Scandinavia and Canada (Saint Fabien, Québec 2004) – as well as extensive ***retrogressive*** spreads in Canada – cannot generally be explained using the LEM mode of failure analysis – and that ***not even in hindsight***. If LEM analysis were always valid, there would be no such thing as *inexplicable landslide failures*.

Maximum potential landslide extension – for ***a reliable*** LEM prediction of landslide failure in sensitive normally consolidated Scandinavian clays – should not exceed about 40 to 70 m. (Confer Issue No 5 below.)

This applies especially to the effect of the enormous water pressure ***change*** on the Cut-off-Wall (the COW) due to the impoundment.
It is in this context crucial to realize that *shear deformations* in the soils beyond the COW are totally related to this ***enormous change*** of ***hydraulic*** pressure – especially in the triggering zone of a possible progressive landslide. (Confer Issues No:s 3 and 4 below).

At the IWLSC 2017 Conference (Trondheim) it was stated (Bouchard, SNC/Lavalin) that progressive failure analyses have been performed regarding the eastern downstream slope. However, as explained in the author's comments on the Nalcor/SNC-Lavalin Engineering Report (Dec. 2015, Ref. [2b]), there is no evidence at all in the Engineering Report of any ***correct*** progressive failure study having been done. (Confer Sections 6.1, 6.2, 6.4 and 6.5 in Ref. [**15**].)

Issue No 2: **Are the soil conditions in the Churchill River Valley identical to those of highly over-consolidated clays in vast areas of Eastern Canada???**

The types of *high sensitivity* of soil layers in the Stratified Drift – as well as those of isolated layers in the Lower Clay formation – should ***not be confused*** with the kind of *sensitivity* that is typical of the ***over-consolidated*** clays common in Eastern Canada.
Such clays can be loaded to peak resistance under relatively small shear strain and the post-peak deformation softening is moderate – at least in the ***triggering phase*** of a downhill progressive or retrogressive landslide. The value of the residual shear resistance (c_R) then tends to be in the range of 0.3 to 0.5 times the peak value.

However, in highly porous soils, where the ***in-situ porosity*** is higher than the *critical porosity* (i.e. when $\mathbf{n} > \mathbf{n_{crit}}$), much greater loss of shear resistance (even liquefaction) is likely to occur due to significant deviatory strain (or shear deformation) – and that may

happen ***irrespective*** of the prevailing ***stress level*** condition (τ/c) or of the current ***piezometric level*** in the critical soil layer.
Furthermore, the *peak shear strength* – established under 'more or less' drained laboratory conditions – is then very unlikely to be attained in the slip surface (or in the shear band) during slide progression. (Confer Figure 2:1.) and **Ref. [3]**, **Terzaghi & Peck**.)

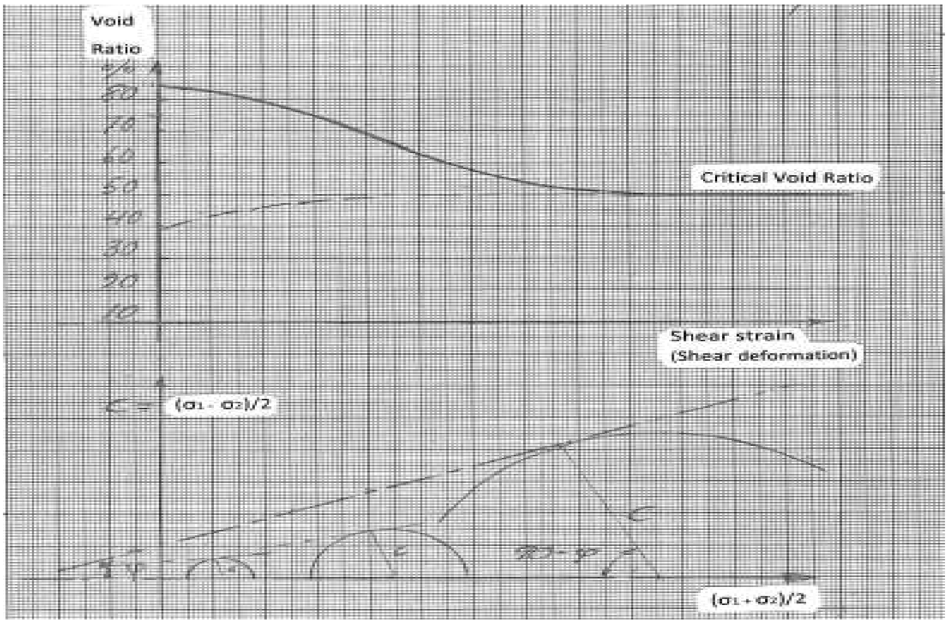

Figure 2:1 Failure modes in porous soils likely to liquefy, i.e. when **n** >> $\mathbf{n}_{crit.}$ (Cf Ref. [13]).

Porous soil layers in the ***Lower Clay*** formation are also likely to be susceptible to liquefaction and radical loss of residual shear resistance. Failure surfaces may therefore just as well develop through porous sands and porous silty sands as in the lean clayey silty sands of the Stratified Drift.

Conclusion: In other words, the assessment of the risk of progressive failure in *long slopes*, and the corresponding safety factors (F_s) cannot reliably be based on LEM analysis.

Issue No 3: **Possible Progressive Failure development in the soil deposits East of the Cut-off Wall (the COW).** Formation of failure surfaces exemplifying Safety Factors based on progressive failure analysis by the author. (Bernander).

3.1 About the sensitivity of Upper Clay 2 in the Stratified Drift
This issue is dealt with in more detail in **Ref. [13]**, (Sections 2. and 3.), where a *Liquid Limit/Plasticity Index* chart (Figure 3:3 in Ref. [13]) shows how the soil deposits in the Stratified Drift fit into a Casagrande diagram. (i.e. with the soil properties as presented in the **Nalcor Report** of 21 July 2014, **Ref. [2a]**).
Moreover, the general soil properties are presented in the Nalcor – SNC/Lavalin ENGINEERING REPORT, **Ref. [2b]**, although, importantly, the ***precise soil structure*** is still not defined or visually documented.

Yet, in the Engineering Report, there are – as already mentioned – no actual studies of ***progressive failure*** formation.

Furthermore, potential failure surfaces related to the massive hydraulic pressure against the COW are presumed to be *perfectly* ***horizontal*** for hundreds of metres and hence, possible failure in the soils behind the COW are taken to be of no importance for the stability of the North Spur.
Now, if horizontal failure planes were the ***only*** option, the author of this report would not either be very worried, as obviously the stresses along such a *long horizontal* failure plane would involve *minor* risk of slope failure – i.e. considering the *horizontal* nature of the ground surface – and that even if the soil layers were to be highly sensitive (but ***not*** actually ***liquefying***) under undrained conditions.

3.2 Effects of a sloping failure surface
However, the stability conditions along a ***sloping*** failure surface in the same type of soil structure would be ***essentially*** *different*. As can be shown by apt progressive analysis, the conditions triggering slope failure change ***radically*** as soon as the possible failure plane is sloping.
For the purpose of underlining this problem, the author has made a few **'check-up'** progressive failure studies yielding the safety factors for the section shown in the figures below. Figure 3:1a, – showing a cross-section through the 1978 Landslide in the North Spur – was presented as Figure 8 on a IWLSC (2013) Poster, (Ref. [1]).
It may be pointed out that neither Figure 7 nor Figure 8 on the Poster (2013) indicated any *perfectly horizontal* layering of the varying types of sediments. According to Section B-B, the mean slope of the lower boundary of the Upper Clay (2) is about 4 % beyond the COW for about 200 m (and > 5% over some 80 m).

Considering the *thickness* of the Upper Clay layer – and the *failure surface* related to a triggering load near the COW – one *may well* add another 5 % or more to the assumed local slope inclination.

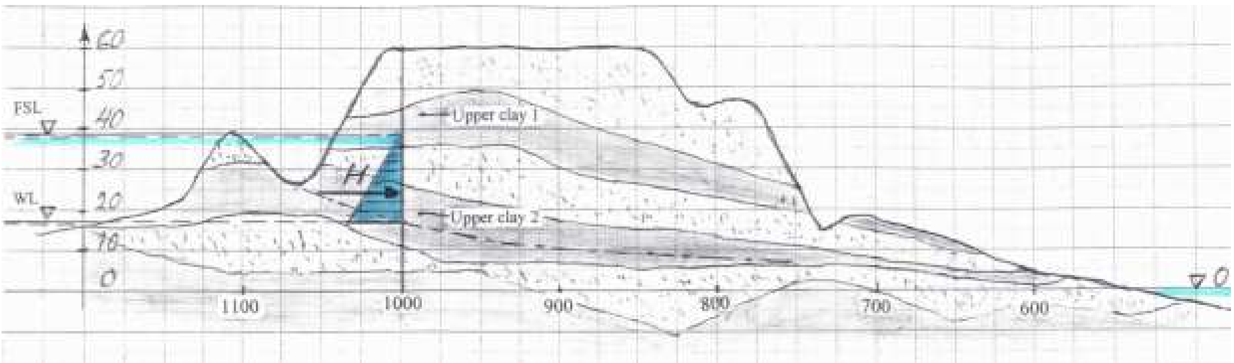

Figure 3:1a Section through the North Spur subject to progressive failure analysis by the author. The soil layers in the section are identical to those in Figure 8 (Section B-B) on a Poster at IWLSC (2013) in Québec City, showing a cross section through the 1978 North Spur Landslide. (Ref. [1]). (The total *deformation-generating* ***change*** of hydraulic pressure related to impoundment has been added to the diagram).

(**Note**: The soil conditions over about 200 m from the COW have ***not*** been affected by the 1978 landslide. Neither this section, nor the other one on the Poster (Figure 7), is indicative of any dominantly horizontal stratification of the soil layers.)

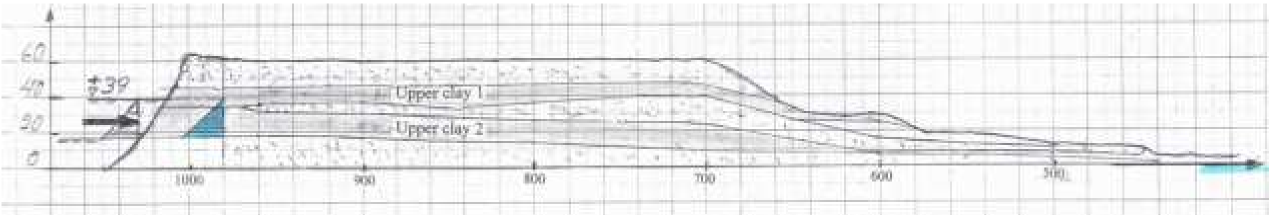

Figure 3:1b showing the section in Figure 3:1a with *Hor./Vert. scales = 1:1*.

Furthermore, it must be emphasized that there is ***no geotechnical law*** stating that *failure planes* are bound to follow the orientation of a sedimentary structure. They will instead adopt the geometrical shape along which failure is most likely to be *initiated* and *developed,* thus rendering the lowest safety factor (F_S) – and that *irrespective* of whether parts of the soil mass consist of *sensitive clays* or of *saturated porous silts or sands.*

The mentioned ***'check-up'*** progressive failure analysis – based on a *moderately* inclining failure plane (i.e. 5 %) over a distance of 100 m from the COW – rendered for instance the safety factors shown in Table 3:1.

Table 3:1 ***Calculated safety factors based on progressive failure formation – Case no 5***

Water level WL	Hydraulic force at the COW kN/m	Slope of potential failure plane %	Peak shear stress c kN/m^2	Residual shear resistance c_R kN/m^2	Critical Load $\mathbf{N_{crit}}$ kN/m	Safety Factor $\mathbf{F_s}$
+32	605	5	70	17.5	≈ 560	**0.92**
+34	781	5	70	17.5	≈ 560	**0.72**
+39	1711	5	70	17.5	≈ 560	**0.33**

Note: Even if the pressure 'build-up' may be a slow process, abiding with plastic LEM conditions, the ***strain rate*** at failure *near* and *beyond peak shear stress* will nevertheless be rapid, in which case the soil resistance may adopt the critical value (N_{crit}). This condition constitutes a normal factor ***initiating*** progressive landslide failures.
If the inclination of the failure surface close to the COW had been assumed to be e.g. 6 % or 7 %, the calculated safety factors (F_s) would have been ***considerably*** *lower* than those shown in Table 3:1.

Issue No 4: Analysis by Robin Dury, 2017. MSc Thesis.

Luleå University of Technology, Sweden, **Reference [9b**]
Case Study of the North Spur at Muskrat Falls, Newfoundland/Labrador, Canada.

4.1 About possible progressive failure in the Upper Clay formation.

Robin Dury's Thesis presents studies focused on the stability of the North Spur based on ***progressive failure*** analyses in accordance with the Bernander Finite Difference Model (FDM).
The studied issue is related to the enormous effects of the impoundment, i.e. raising the upstream water level from WL +17 to WL 39. This implies ***changing*** the ***externally active hydraulic pressure*** (and the related ***shear deformations***) on the soils immediately behind the

COW over a width of say 250 m by some 600 000 kN (= 60 000 metric tons) – i.e. a massive force 'really' capable of triggering progressive failure.

Figure 4:1 shows the safety factors calculated by Dury for *varying sensitivity ratios* and for *different values* of the peak shear strength.

The specific values of the safety factors (F_S) for the water levels ***WL = +34*** and ***+ 39***, pertaining to the *separate* study by the author, mentioned in Section 3 above, have *been added* to Dury's diagram for comparison. The fact that the safety factors shown are not identical is of no import in the current context, as the data presumed are not identical.

(FDM).

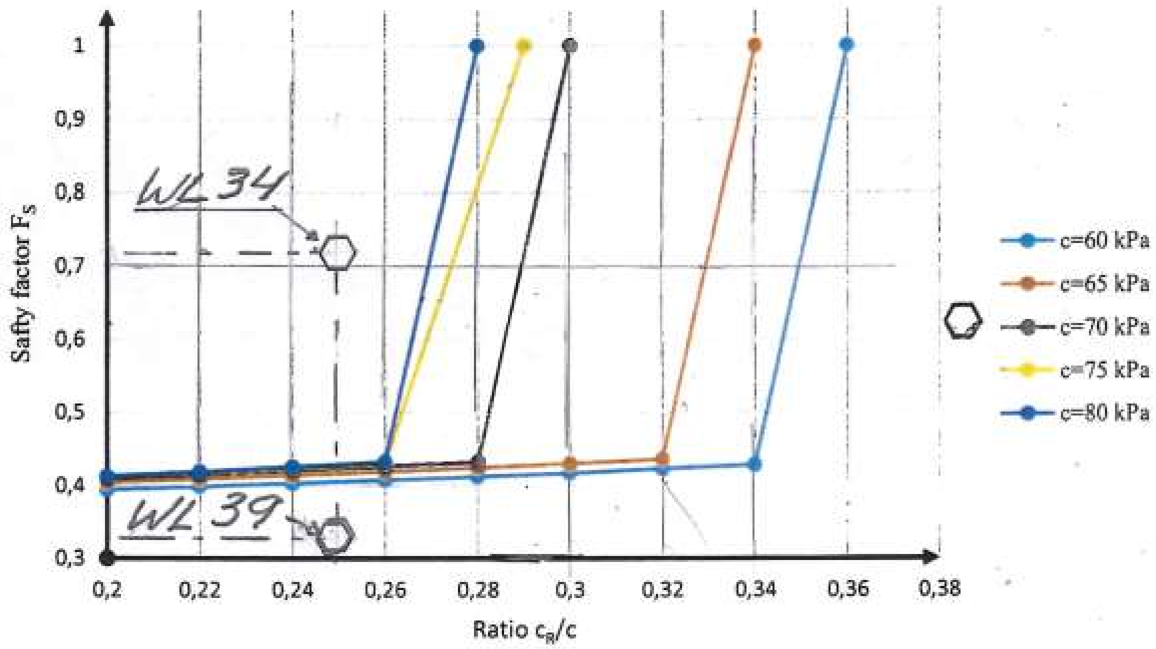

Figure 4:1 Safety factors based on progressive failure analysis (acc. to the Bernander FDM model) as calculated by Dury (2017, Ref.[9b]) for *varying sensitivity* and for *different values* of *the peak shear strength.* In the figure, Dury's analyses are based on the water level WL being + 39 m.
(**Note**: The F_S -values related to Table 3:1 for *WL* = +34 and +39 have been added to Dury's diagram by the author of this report.)

The results of progressive failure analyses depend on various factors such as e.g. the *peak* shear strength (c), the *residual* shear resistance (c_R), the *inclination* and *shape* of the failure surface, the *assessment of in situ conditions, soil porosity* and *earth pressures* in overlying soil layers etc. For the initiation of progressive failure, the geometry of the slip surface is of special importance in the ***triggering zone***, i.e. an area within a distance from the COW (i.e.) of some 30 → 50 m.

As the sedimentary structure, consisting of lean clayey sandy layers and loose silty sands, is not presented in sufficient detail in the Nalcor reports (Ref. [2a] and Ref [2b], it has not been possible to perform precise analyses at this stage. It must also be deliberated that porous silty sandy soil layers with *in-situ* porosity >> *critical* porosity may be ***highly sensitive*** and ***prone*** to ***liquefy*** – (i.e. even without any clay content).

Conclusion: The progressive failure studies clearly indicate that safety factors based on the Limit Equilibrium Mode (LEM) cannot *by far* render *reliable* ***factors of safety*** (F_s), and that the direct effects of the impoundment on the risk of progressive failure initiation in the soils behind the COW should be ***carefully investigated*** considering the *actual in-situ soil porosity*. The normal value for Safety Factors (F_s) in Soil Mechanic's practice is **1.50**.

Regarding the effects of ***finger-drains***, confer ***Issue No 8***

4.2 About possible progressive failure in the Lower Clay formation.
Although the ***mean value*** of the Liquidity Indices in the Lower Clay formation is generally *significantly* lower (i.e. 0.6) than that of the Stratified Drift, there are nevertheless – according to Table 2.2 in the ENGINEERING REPORT, Ref. [2b] – soil layers in the Lower Clay structure, in which the Liquidity Index (LI) substantially exceeds 1.0, (i.e. $1 < LI < 2$**).** Values of LI between 1 and 2 in soil layers indicate ***high porosity*** and ***sensitivity****,* the water content (**w**) *significantly* exceeding the Liquid Limit, (i.e. w >> LL). '
However, values of LI between 1.5 and 2.0 indicate possible ***potential*** for strain- (or deformation-) induced ***liquefaction.*** (Further information about the Lower Clay Formation is given in Table 2 in Ref. [15], Section 3.2).

According to the cross-section on Figures 3:1a and 1b, porous sensitive layers in the Lower Clay may well be sloping and, as has been emphasized in earlier reports by the author, the possibility of ***forward*** (downhill) *progressive failure* due to the enormous water pressure on the COW should be properly investigated. (Confer e.g. Ref. [14], Section 6, ***Conclusive Remarks***; Ref. [15], Section 5.1, ***About Failure Surfaces;*** and Ref. [16], ***Points*** *7* and ***8***.)

The possibility of progressive failure in the Lower Clay formation has been studied in Dury's MSc Thesis, (Section 5:3), applying the Bernander Finite Difference Method (FDM), **Reference [9b].** This study is also based on the section defined by Fig.8 on the IWLSC (2013) Poster, (Ref. [1]) – i.e. corresponding to Figure 3:1b above.

Table 4:2 Lower Clay formation – Safety Factors
Mutually related values of Peak Shear strength, Sensitivity ratio and Safety Factor.
Values derived from Figure 5:13 in, **Robin Dury's MSc Thesis**, **Ref. [9b].**

c	c_R/c	F_s
110 kN//m^2	0.24	0.59
100 kN//m^2	0.26	0.60
90 kN//m^2	0.30	0.61
80 kN//m^2	0.34	0.60
70 kN//m^2	0.38	0.58
60 kN//m^2	0.46	0.55
50 kN//m^2	0.56	0.49

Robin Dury has evaluated the safety factors for wide ranges of peak shear strengths and sensitivity ratios – i.e. for peak strengths in the order of $c = 50 \rightarrow 110$ kN/m^2, and for residual shear resistance ratios $c_R/c = 0.2 \rightarrow 0.57$.
(**Note:** For the safety factors in Table 4:2 to become greater than 1.0, the values defining the residual shear resistance (c_R/c) must exceed those shown in the table. Calculated values only apply to the specific geometry of the failure planes assumed.)

Conclusions: The results of the computations imply that safety factors based on LEM analysis are unreliable. The true values of the safety factors (F_s) will remain ***unknown*** until a ***correct*** approach has been performed.
The normal value for Safety Factors in Soil Mechanic's practice is **1.50.** However, when progressive failure is an option, even higher values of safety factors should be considered, depending on various basic data. Confer Table B.III in Reference [6].

In porous soils, with a marked tendency to liquefy, the problem defined in **Issue 2** above must also be considered. Progressive failure related to liquefaction (or heavy loss of shear resistance) due to *shear deformation* is – as has repeatedly been emphasized – likely to occur when the current ***in-situ*** porosity (n) exceeds the ***critical*** porosity (i.e. $n > n_{crit}$). (Confer References [3] & [4].)
Failure in isolated highly porous soil layers may then readily develop – and *that* even *irrespective* of the prevailing piezometric level.

Furthermore, in the ***triggering phase*** of progressive failure, the rate of *strain* and *deformation* development at *peak* and *post-peak* conditions is normally a rapid process in sensitive soils. Application of LEM analysis, based on the possible beneficial effects of inter-distant finger-drains, are in this context highly ***disputable.***

Utilizing the soil properties – based on Tables 2.1 & 2.2 in the ENGINEERING REPORT [2b] – for studying progressive failure along sloping failure planes, analyses by R. Dury and by the author of this report indicate that acceptable safety factors (F_s = 1:50) are hard to establish, unless the critical soil layers are duly ***compacted*** or effectively ***de-saturated***.

4.3 About possible failure in homogeneous Lower Clay

Yet, even if the Lower Clay were believed to be a ***perfectly homogeneous clay*** structure, the effects of progressive failure development along ***inclining*** failure planes may not be disregarded, and *should instead* be duly ***investigated***.
(Confer Ref [15], Section 6. Concluding Remarks, last sentence).

Issue No 5: Regarding progressive failure on the Eastern downstream slope.

Generally, progressive failure in long slopes can ***only*** be predicted by relating the deformations in the soil (due to the additional load) to valid stress/strain (deformation) relationships.
As already stated in Issue No 1, maximum potential landslide extension – for ***a reliable LEM prediction*** of landslide failure in sensitive normally consolidated Scandinavian clays – ranges between 40 → 70 m, largely depending on the depth to the studied failure plane.

Applying LEM to potential landslides longer than 200 m – as is done in the ENGINEERING REPORT, (Ref. [2b]) – renders highly ***unreliable*** safety factors in ***sensitive*** and ***water-saturated*** soils. (Cf Ref. [15]).

The work presented by Chen Wang an Bipul Hawlader in Ref. [2d] relates to retrogressive failure conditions on the East slope. However, as such slides are not a key issue in this report, they are not dealt with in this current context.

Issue No 6: Laterally progressive failure

As already stated**:** Raising the upstream water level from say + 17 to + 39 represents a ***change*** of the hydraulic force on the soil structure close the Northern rim by 2420 kN/m. Over a width of say 250 meters, this corresponds to an additional force of some 600 000 kN, most of which will initially – i.e. in the triggering phase of a progressive landslide – act on the COW and the nearby *heterogeneous* ***water-saturated*** *porous* soil masses.
Although the rate of impoundment may be a relatively slow process, the horizontal thrust will in the *triggering progressive* failure phase generate ***large shear deformations*** and raised hydraulic pressures in the saturated lean clayey – and mixed (clayey) silty, sandy – soils immediately East of the COW, possibly leading to liquefaction or massive local loss of shear resistance. This phenomenon, which in the current case, is mostly related to grain instability in porous saturated soils, may as mentioned, take place ***irrespective*** of prevailing *stress levels* and *pore water pressures* – whether they be static or related to percolation. (Cf Issue No 2.)

Furthermore, even if the effects of impoundment were slow, the rate of deformation in different parts of the soil mass will, by far, not be identical over a width of say 250 m
The heterogeneity of the soil volume involved will – both in the vertical and the horizontal directions make the rates of deformation vary both in ***space*** and ***time****.* This constitutes an important fact that must also be deliberated when evaluating the risk of progressive failure development in potentially extensive landslides*.
(* A *similar* phenomenon with enormous impacts:
Continental Drift goes on at the slow rate of a few centimeters per year over long periods of time – ***but nevertheless***, mankind locally experiences the dramatic effects of *volcanic activity*, *massive sudden movements* of the earth crust, *earth quakes* and *gigantic tsunamis,* i.e. all due to locally *varying* inhomogeneous *geological* conditions.**)**

Evaluating the effects of the gigantic pressure due to impoundment by LEM analysis requires that the soil stiffness – while being dependent on *porosity, friction, cohesion, sensitivity, water saturation, acting vertical stresses, degree of pre-consolidation* etc – is ***uniform*** over a massive soil volume of say 700 000 m^3 (e.g. 250 m *70 m * 40 m).

According to the soil investigations, the North Spur is a very hetero-genius structure thus implying that there may be large portions of the soil volume behind the COW that are considerably stiffer than other neighbouring parts. This means in turn that the stiffer parts may temporarily sustain more of the impoundment pressure than softer parts.
However, the overloaded stiffer parts *near the COW* are bound to fail in due course, and when the accumulated energy built-up in the same is released, ***sudden*** high rate ***strains*** and ***deformations*** will affect the neighbouring soil volumes.
Hence, if tendency to liquefaction (or high sensitivity) exists, *wide landslides* tend to progress laterally. Again, such local failures are likely to happen so rapidly that *fingerprint drains* (also in this context) may not ensure that the *drainage* effects exclude the possibility of liquefaction – or critical loss of shear resistance.
As already mentioned, progressive failure – when developing in the triggering phase – is normally not a slow process at all. Again, this is 'actually' what this type of failure is about.

There are numerous examples of ***lateral slide progression*** in Scandinavia – e.g. the landslides at Göta (1957), Rissa (1978), Vagnhärad, (2000), Småröd (2007) (width ≈ 500 m), etc. As soon as the width of a landslide occurred exceeds its length in the triggering phase, the mentioned conditions, favouring lateral slide progression, are likely to have been important factors in the slide development.

Conclusion: Lateral slide progression should always be considered in situations, where the *potential* ***width*** of a landslide exceeds its likely extension in the ***triggering phase*** (i.e. normally about 50 to 100 m). Factors promoting laterally progressive sliding are *inhomogeneous* and *highly sensitive* layers in the soil structure.

In the North Spur, landslide risk is *primarily related to* ***high porosity*** and ***low clay content*** in the Stratified Drift, as well as in specific porous layers in the Lower Clay formation. Confer **Ref. [13]**, Section 2.

Issue No 7: Over-consolidated clays versus porous soils

In the previous reports, (and in Issue No 2 above), the undersigned has **st**ressed the fact that the sensitivity of highl**y** ***over-consolidated*** clays in Eastern Canada, and that of the ***markedly porous*** soils typical of the Churchill River Valley, are ***not related*** to the *same type* of *basic grain structure* and *evolutionary conditions*. The properties of these soils have very little i9n common

Sedimentary history, porosity, grain size structure, grain shape, clay content, chemistry of *soil constituents,* and the *degree* and *nature* of *over-consolidation* ***radically influence*** the stress/strain (deformation) behaviour of soils – thus also *importantly* affecting the failure processes and the different modes of slope failure.

In fact, this is what the problems related to the ***stability*** assessments regarding of the North Spur dam containment is ***mainly about***. In the previous reports, the author has repeatedly emphasized the crucial effect of the basic relationship between ***high*** in-situ porosity (**n**) of a soil layer and the so called ***critical porosity*** ($\mathbf{n_{crit}}$) of the same type of soil.
This relation constitutes a condition, the ***decisive importance*** of which has, among others, been very clearly described by Terzaghi-Peck, e.g. in **Reference [3].** (Confer also Ref**. [16]**).

Issue No 8: About propensity to soil liquefaction – drainage reliability of sparse spread of finger drains – if any such drains at all near the COW.

As already stated, the conditions ***triggering*** *progressive slope failure* are not long time-extended processes. The rate of strain (and deformation) at the ***end*** of the ***triggering phase*** (preceding the virtually ***dynamic*** third phase) is normally high. This implies that the excess pore-water pressures, liable to build-up at this stage, in porous and sensitive soils, may not dissipate fast enough through a ***few inter-distant*** finger drains – i.e. provided there are any such drains at all in the ***Stratified Drift*** and in ***Lower Clay*** layers just East of the COW.
The problem is of course aggravated the greater the difference ($\mathbf{n- n_{crit}}$) happens to be in porous soil layers.
As previously emphasized, there exist *crucially decisive* conditions, affecting the North Spur stability, also in the Lower Clay sedimentary structure, and that irrespective of whether downhill (forward) or laterally progressive failures are anticipated.

Issue No 9: Remedial measures:

The risk of progressive failure development may be counteracted by ***deep drains*** extending ***through*** the ***Stratified Drift*** and ***deep down*** into the ***Lower Clay*** formation, and in which the piezometric levels are controlled by deep pump wells operating all the time during the period of impoundment.
The extension of the area with ***closely spaced deep drains*** should be based on progressive failure analysis – most likely at least some 40 to 50 m East of the COW.

Hence, a ***vital issue*** in this context i**s**:
Are sparse finger drains (***if any*** in this part of the soil mass) a satisfactorily ***reliable*** *measure* for the prevention of progressive failure development being triggered just East of the COW**??**

This issue must be proven beyond any shadow of a doubt.

Mölndal 2017-10-23

Stig Bernander

References

Nalcor/SNC-Lavalin Reports and Geotechnical Literature

[1] Geotechnical data related to the Muskrat Falls Dam Project on posters at the **IWLSC Conference,** (Quebec 2013):
L´Heureux, J.-S., Locat, A., Leroueil, S., Demers, D. and Locat, J., Editors (2013). *Landslides in Sensitive Clays. From Geoscience to Risk Management*. Springer, 418 pp, ISBN 978-94-007-7078-2.

[2a] **Lower Churchill Project – NORTH SPUR UPDATED, Nalcor Report** to the **Independent Engineer** – 21 JUL – 2014.

[2b] The **Nalcor – SNC/Lavalin ENGINEERING REPORT,** North Spur–Stabilization–Works, Progressive – Failure-Study". (21 Dec. 2015). Leahy, Denise (2015). *North Spur Stabilization Works. Progressive Failure Study. Lower Churchill Project,* Engineering Report, SNC-Lavalin, Nalcor, SLI Document No. 505573-3281-4GER-0001-01, Nalcor Reference No. MFA-SN-CD-2800-GT-RP-0001 Rev B2, 21 Dec 2015, Verified by Regis Bouchard and approved by Greg Snyder .128 pp. Available at https://muskratfalls.nalcorenergy.com/wpcontent/uploads/2016/01/North-Spur-Stabilization-Works-Progressive-Failure-Study.pdf

[2c] Ceballos, Alvaro (2016): ***North Spur Stabilization Works – Design Report.*** *Lower Churchill Project.* Engineering Report. SLI Document No. 505573-3281-4GER-0601- PS Nalcor Reference No. MFA-SN-CD-2800-GT-RP-004-01 Rev. B1. Date 30 Jan. 2016. Verified by Regis Bouchard and approved by Greg Snyder, 264 pp. Available at http://muskratfalls.nalcorenergy.com/wp content/uploads/2013/03/North-Spur-Stabilization-Works-Design-Report-Jan-2016-Final-2.pdf

[2d] Chen Wang, Bipul Hawlader, **Numerical Modelling of three Types of Sensitive Clay Slope Failures.** Dpt of Civil Eng., Memorial Univ. of NFL, Saint Johns, Canada.

[3] **Terzaghi K. & Peck R.B.** Soil Mechanics in Engineering Practice.
Article 17, ***Shearing Resistance of Cohesion-less Soils***, Pp 107 – 111.
(Second Edition 1967. (Printed by John Wiley & Sons Inc. New York , London, Sidney.)

[4] **Terzaghi K. & Peck R.B.** Soil Mechanics in Engineering Practice.
Article 6, ***Index Properties of Soils,*** Page 28.
(Second Edition 1967. (Printed by John Wiley & Sons Inc. New York, London, Sidney.)

[5] Bernander S. (2000) Progressive Landslides in Long Natural Slopes.
Licentiate Thesis. 2000:16 *ISSN:1402–1757 * ISRN: LTU–LIC—00/16—SE. Available at http://ltu.diva-portal.org/

[6] Bernander S. (2008) **Downhill Progressive Landslides in Soft Clays**. 2008:11│ISSN:1402–1528│ ISRN: LTU–FR—08/11—SE. Available at http://ltu.diva-portal.org/

[7] Bernander S. (2011) **Progressive Landslides in Long Natural Slopes**. Formation, Potential, Extension and Configuration of Finished Slides in Strain-softening Soils, **Doctoral Thesis**. ISBN: 978-01-7439-283-8 ISSN: 1402-1544 Available at http://ltu.diva-portal.org/

[8a] Aas, G. (1966). Special Field Vane Tests for the Investigation of Shear Strength of Marine Clays. *Report, Norwegian Geotechnical Institute, Oslo.*

[8b] Aas, G. (1983). A Method of Stability Analysis applicable to Natural Slopes in Sensitive and Quick Clays. Proc. Symposium on Landslides, Linköping 1982, Swedish Geotechnical Institute, Report no 17, pp 7 → 25.

[9a] Bernander, S. & Svensk, I. et al. (1985). Shear strength and deformation properties of clays in direct shear tests at high strain rates. Proc. 11th ICSMFE, San Fransisco, Vol. 2/B/5, pp 987 – 990. Available at http://ltu.diva-portal.org/

[9b] Dury, Robin, (2017). ***Case Study of the North Spur at Muskrat Falls*** in Labrador/ Newfoundland. Canada. Progressive Landslide Analysis. *MSc Thesis*, Luleå University of Technology, Luleå, Sweden. 65 pp. Available at https://ltu.diva-portal.org/smash/get/diva2:1117330/FULLTEXT02.pdf

[9c] Thakur, V., L'Heureux, J.-S. & Locat, A., Editors (2017). *Landslides in Sensitive Clays. From Research to Implementation*. Proceedings from the 2nd International Workshop on Landslides in Sensitive Clays, **IWLSC Trondheim**, 12-14, June 2017, Cham: Springer, 604 pp, ISBN 978-3-319-56486-9.

[9d] Bernander, Stig & Elfgren, Lennart (2017). Dam bank stability in loosely layered sandy clays. *Comments on the risk of failure of the North Spur at Muskrat Falls in Churchill River, Labrador, Newfoundland*. Luleå University of Technology, Technical Report, ISBN : 978-91-7583-928-8 (digital). Include copies of [13], [14] and [15] Available at http://ltu.diva-portal.org/

Previous reports or comments (2014→ 2016) 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.

[11] Comments on the Nalcor Report to the Independent Engineer of July 21, dated September 14, 2014.

[12] Power Point presentation 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. (Minor revisions of linguistic nature on November 23 and December 5) Included as **App. I** in Ref. [9d]

[14] Further Comments on the Updated Nalcor 21- JULY-2014 Report Dated 2016-01-07.Included as **App. II** in Ref. [9d]

[15] Comments by Bernander (15 Sept. 2016) on the NALCOR/SNC-LAVALIN **ENGINEERING REPORT** of 21 December 2015 (Included as **Appending Report No III** in Ref. [9d].)

Section 5.1 Page 14, Reference to Terzaghi-Peck, Article 17, Quotation regarding Spontaneous Liquefaction and True Quick-sands.
Section 5.4, … about Critical Porosity
Section 5.7, Regarding soil properties in the North Spur and Over-consolidated Clays in Eastern Canada.
Section 5.8, A proposal for the realistic testing of the porosity of soils in the Stratified drift.
Section 6.3, …possible Failure in Lower Clay Formation should be considered.
Section 6.5, About Finger Drains.
Section 6.6, About investigation of in situ porosity conditions in soil layers.

[16] Letter to Grand Riverkeeper Labrador Inc. (2016-11-16) concerning North Spur stability. Among other about the sensitivity of *East Canadian clays* vs that of soils in the *North Sp*ur and in the Churchill River Valley.
(A copy of this letter was sent directly by Bernander to the **NFL Minister's Office Secretary,** Miss Fanny Hoddinott at the Ministry of Environment.)

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