TECHNICAL REPORT

Response to and Comments on "Geotechnical Peer Review of Dr. S. Bernander's Reports and Analysis of the North Spur"

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The picture on the cover shows an aerial view of Muskrat Falls on September 27, 2004. The North Spur Ridge, susceptible to a possible dam breach, is located in the centre of the picture just above the falls and the Rock Knoll granite cliff. A possible *downwards progressive failure* would start at the upstream Western slope (to the left in the photo) when the water in the dam is raised (light blue dotted line) and a large horizontal pressure (red arrows) - and accompanying deformations - is induced on the Ridge. If the soil locally cannot withstand this it will lose its strength and subside and the load must then be transferred (*progress*) further into the ridge in the downstream direction. New sections may subside and finally the whole ridge may slide into the downstream river (to the right in the figure). Original picture from Google Earth with new water level (light blue dotted line) and water pressure (red arrows) added by the authors. Location 53°15'01.99"N, 60°46'29.03"W, Elevation 1,72 km. Image ©2018 Digital Globe.

Foreword

The stability of the dam at North Spur in the Muskrat Falls hydro power plant in Churchill River in Labrador, Canada, is a geotechnical challenge with strain-softening soils. The risk of getting an initiation of a forward progressive failure cannot be considered as negligible. The concern from Dr. Stig Bernander, that a proper analysis of such a progressive failure ought to be carried out, should be taken seriously. Such an analysis must use the deformation and strength properties of the soils in question and has, as far as we can see, not been undertaken by the Muskrat Falls Corporation nor its contractors.

Luleå in July 2018

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Preface

This report responds to the statements in the “Geotechnical Peer Review of Dr. S. Bernander’s Reports and Analysis of the North Spur” (GPRP, 2018).

The current report has been written by Stig Bernander with editorial assistance from Lennart Elfgren.

Mölndal and Luleå, July, 2018

Stig Bernander and Lennart Elfgren

Abstract

The concerns regarding the stability of the North Spur can be summarized in three points:

(1) **None of the most critical inclined failure surfaces have been studied** by Muskrat Falls Corporation. These failure surfaces may be initiated on the upstream side of the dam containment. Here the effects of the deformations, caused by the pressure of the rising water level, have to be resisted by the metastable soil layers in the North Spur. A local failure may occur progressing downwards towards the downstream side of the Spur. A catastrophic dam breach would follow. The GPRP further categorically overlooks the fact that horizontal failure planes cannot possibly represent the highest risk of instability irrespective of whether the analysis is based on the Limit Equilibrium Mode (LEM) or on the Progressive Failure Mode.

(2) **The stress/strain deformation properties of the porous soils in the North Spur have not been made available**. Only strength properties, related to fully drained conditions, have been given. How stresses relate to simultaneous deformations under undrained (or partially undrained) conditions have not been defined in any way. Such relationships are crucially essential for any *up-to-date* analysis of slope stability.

(3) **A high risk of North Spur instability has been found related to impoundment.** A series of investigatory calculations have been made, based on deformation properties from similar landslides and on a wide variety of assumed input data for possible critical failure surfaces**.** The results of these analyses indicated a safety factor far below 1.

The peer review does not address the above three points. It gives a good view of the general conditions but also contains misconceptions, erroneous considerations and refutable comments indicating that the earlier reports by Bernander have not been fully understood by the panel members.

As no up-to-date analysis of the stability of the North Spur has been provided, our conclusion is that an independent group of experts, appointed by government, should be entrusted with this important task.

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Executive Summary

The Geotechnical Peer Review Panel in its report (GPRP, 2018) does not address:
- Why the most critical failure surfaces have not been studied.
- Why stress/strain deformation properties have not been provided
- Why analysis of a downhill forward progressive failure, initiated upstream, has been excluded.

Below, a brief summary is given of comments on the *seven items* the Geotechnical Peer Review Panel *do* discuss in its report (GPRP, 2018).

(1) *<u>GPRP statement:</u> The clays found in the North Spur are similar to many of the clays found in Eastern Canada and in Norway*.

<u>Brief comment</u>: It is true that similar clays may be found elsewhere, but their deformation properties must be considered in the analysis. This has not been done in an appropriate way by SNC Lavalin Inc. (SLI). Basically, according to tables in the Nalcor/SLI Engineering Report on Progressive Failure (Leahy, 2015a), the variation of soil properties, in many sometimes thin layers and lenses of varying materials, show little similarity to the highly over-consolidated soils with rich clay content, typical of Eastern Canada.

(2) *<u>GPRP statement</u>: The observed landslide features are also comparable to landslides observed in sensitive clays elsewhere.*

<u>Brief comment</u>: There are different kinds of landslides and various kinds of soil. The downhill progressive landslides, which are currently subject to discussion, are not the normal type of slide in the highly over-consolidated East Canadian clays. In these clays retrogressive spreads are more common and the interest of SLI and GPRP has been focussed on them.

(3) *<u>GPRP statement</u>: The methodology used to evaluate the stability of an initial slide on the North Spur slopes corresponds to the current state of practice.*

<u>Brief comment:</u> The Limit Equilibrium Methodology (LEM) has been the normal state of practise in most of the 20th century but it is certainly not the state of the art for *extensive* landslides in the 21st century – and that especially not for such a precarious case as here.

(4) *<u>GPRP statement</u>: The analyses by SLI are conceptually acceptable to take into account the initiation of progressive failure and to ensure a proper design of mitigation measures*.

<u>Brief comment</u>: This statement is *erroneous* and does not apply at all to slides in slopes with highly sensitive soils. Moreover, the SLI analyses only consider the initial stresses on horizontal failure surfaces, while it is *obvious* that failure surfaces near the upstream West rim will tend to *slope* eastwards – both in the *Upper* and the *Lower* Clay formations.

(5) *<u>GPRP statement</u>: State-of-the-Art methodology has been applied to the North Spur to assess its resistance to earthquakes*.

<u>Brief comment</u>: Bernander questions whether the principles of *progressive failure* have been applied in the current context. These methods have been developed and disseminated by Stig Bernander since 1978.

(6) <u>*GPRP statement:*</u> *With respect to the mitigation and remedial measures at the North Spur, the GPRP finds that the analyses of the cut-off walls presented by Dury and Dr. Bernander are based on several incorrect assumptions and that the results are therefore not realistic. The GPRP is strongly against Dr. Bernanders proposal of driving closely spaced piles in the North Spur to investigate if metastable soils are present. Such an investigation could generate excess pore pressure in the sensitive clay and undermine the stability of the slopes and hence of the entire Spur*.

<u>Brief Comments:</u> The wrong assumption of the height and location of the cut-off-wall does not essentially change the results of the analysis. A till blanket has been provided, which also concentrates the load from the rising water. Frictional forces related to seepage will also be transmitted to the sensitive soil structures near the West rim. The stress conditions in the dam bank rim may of course be somewhat different by varying locations of the COW. Yet, in the current context, the *primary issue* is evaluating the effects of *deviatory deformations* related to the enormous impoundment force – i.e. is the loss of resistance in the metastable soil layers sufficient to initiate progressive failure?

The expressed fear of the reviewers regarding minor impacts and vibration due to activity on the ground surface on top of the rim (at level +60***)*** – as well as their fear of testing the in-situ *porosity* of soils in the bank in a controlled way – indicates that they are, *themselves*, not very confident about the basic stability of the bank.

In the current situation, effective compaction of possible metastable soil layers is probably the only practical mitigation measure to be considered in the assessment of the North Spur stability.

(7) <u>*GPRP statement*</u>*: The aspects of dam breach and consequences downstream at Muskrat Falls have been investigated by SLI.*

<u>Brief Comment:</u> The aim of Bernander's many reports is preventing a possible dam breach. Whatever the consequences of a breach will be, they are bound to be very serious.

Conclusion: As no up-to-date analysis of the stability of the North Spur has been provided, our conclusion is that an independent group of experts, appointed by government, should be entrusted with this important task.

Introduction

In 2013, Stig Bernander took part in an International Workshop on Landslides in Sensitive Clays in the city of Quebec in Canada (Bouchard et al., 2013, L'Heurux et al., 2014). He was there approached by representatives from the Grand Riverkeeper Labrador Inc. about the possible risks related to the planned North Spur part of the dam wall at the Muskrat Falls hydroelectric generating facilities in the Lower Churchill River in Newfoundland and Labrador, Canada.

Stig Bernander was subsequently invited to visit the Muskrat Falls' site and he extensively studied the area in October 2014 including air-borne travelling by helicopter, ground surveys by car and riverbank landings by boat. He also gave lectures on landslide risks in St John's. He afterwards wrote a report on the possible risks with the project in 2015 (Bernander, 2015). Further comments were made in two additional reports in 2016 (Bernander, 2016 a, b) where he commented on the Nalcor – SNC Lavalin Inc. Engineering Reports on design by (Ceballos, 2016, early version 2014) and on progressive failure by (Leahy, 2015a).

A Master of Science Thesis on the subject was carried out by Robin Dury at Luleå University of Technology in 2017 (Dury, 2017). The results were presented at the 2nd International workshop on Landslides in Sensitive Clays in Trondheim in June 2017 (Bernander et al., 2017, Dury et al., 2017, Thakur et al., 2017). A few results from investigatory analyses by Bernander were also presented on this occasion. At the same workshop a paper on the North Spur stabilization works (Leahy et al., 2015) was presented by Regis Bouchard, SNC Lavalin Inc. (SLI), a company delivering engineering procurement and construction management service for the Muskrat Fall Project.

During these years the response given was that everything is OK regarding the North Spur stability during impoundment. In order to mitigate the risks for a dam breach, the facts at hand were then further summarized and disseminated in 2017 and 2018 (Bernander, October 23, 2017) and (Bernander & Elfgren, 2017, 2018). The reports were sent to Regis Bouchard at SLI and to some of the members of the Geotechnical Peer Review Panel *to be*. They were also published on the web. However, no response was obtained.

In February 2018, the Muskrat Falls Corporation finally published a report by a Geotechnical Peer Review Panel (GPRP, 2018) – treating the concerns regarding the North Spur raised by Bernander.

Yet, the Geotechnical Peer Review-Report does *not* address - or even refers to - the Bernander report to the GRK Inc. (of October 23, 2017) titled: "Summing up of North Spur stability issues" or to Bernander & Elfgren (2017). This is of course *rather odd*, as these documents actually *deal with* and explain many of the issues brought up in the more than *three months* later dated report by the Geotechnical Peer Review Panel (GPRP, 2018).

The following *discussion* and *comments* relate to the seven issues discussed by the Geotechnical Peer Review Panel (GPRP, 2018) concentrating on Section 10. Conclusions (pp 26-27).

1. Geology and Geomorphology (Issue 1)

GPRP Statement**:** *Based on the information provided by SLI, the GPRP considers that* ***most*** *of the landslides along the Churchill River valley and at the North Spur are either rotational slides or retrogressive flowslides, and that they are comparable to many of the landslides observed in sensitive clays elsewhere in Eastern Canada and Norway.* (GPRP, 2018, p 9).

Comment: It is correct that many landslides along the Churchill River and elsewhere in Canada are rotational slides or retrogressive flowslides. However, here it is the question of the risk for another type of landslide, not so well-known in Canada, namely a forward progressive landslide. Nowhere else in the valley, or the in the North Spur ridge, has, since the ice age, such a gigantic external load corresponding to the effect of the impoundment ever been applied.

The external hydraulic force acting on the soils near the western upstream slope may well trigger a forward progressive landslide. A dependable, correctly established, *safety factor* in respect of such an event must be established.

As is explained in Bernander (2016a), the Edward Island Landslide (2010) and the large landslide on the downstream North Bank, South of Muskrat Falls (2014) are not rotational or likely to be of retrogressive nature. In fact, the use of the word 'most' in the conclusion made by the GPRP above indicates that even the Peer Panel *itself* is not convinced that their analysis regarding types of slides in the Churchill River Valley *really holds true*.

Conclusion: The GPRP conclusion proves nothing about the risk for a forward progressive failure and is of *limited value* in the current context.

2. Extreme sensitivity and particular structure (Issue 2)

GPRP Statement: The *GPRP concludes that the clayey soils found at the North Spur are comparable to those found in Eastern Canada and in Norway. The material described by Dr. Bernander is not representative of the Upper Clay within the stratified drift un*it. (GPRP, 2018, p 26).

Comment: This is a misunderstanding and is totally wrong, see below.

2.1 Soil properties in the Upper Clay II structure – Consolidation

GPRP Statement*: The GPRP does not agree with Dr Bernander's postulate that the Upper clay is under-consolidated. In fact, no excess pore-pressure has been registered in the Upper clay, and this clay is embedded within very dense silty sandy soils. The aspects of critical void ratio discussed by Dr. Bernander are also confusing as this is normally applicable to granular materials and not to clays.* (GPRP, 2018, p 26).

Comment: Bernander has nowhere claimed that the soil layers in the Stratified Drift are *under-consolidated* with *excess pore water pressures*. What he wants to emphasize is merely the fact that in a very lean clayey soil layer, the volume of the clay content may be less than the porosity of the ***coarse*** material in the soil. This implies that the soil layer as such may be normally consolidated, whereas the minute clay content remains under-consolidated. He Is fully aware that the soil investigations state that the soils are basically normally consolidated, i.e. they are without any registered excess pore water pressures.

However, the GPRP must surely be aware of the fact that slopes of *metastable porous sands* and *silty sands* (as well as of *lean clayey sands* and *silts)* can remain stable for hundreds or even thousands of years (*without excess pore water pressures*) and *yet* – when subjected to significant *shear deformation,* e.g. due to disturbance from vibration, blasting, piling or just *additional* shear – landslides of all sorts may take place. See also Section 4 below.

The notations for porosity and void ratio (*n* and *e*) unfortunately happened to be reversed in Bernander (2015) but the meanings of critical porosity and void ratio (e_{crit}) are the same as in standard literature. Its application to granular materials interspersed with clay particles should be easy to understand.

Conclusion: The GPRP has misunderstood what Bernander has written. The relationships between porosity and void ratio are correct and the related analyses are unaffected.

2.2 Soil properties in the Upper Clay II structure – Clay content

GPRP statement*: Dr. Bernander describes the upper clay as a metastable material and as mixed sandy-silty soils with very sparse clay. From the literature this happens only when the clay content is less than 15 to 25%. In fact, the Upper clay has a clay content between 45 and 65 %.* (GPRP, 2018, p 26).

Comment: The main mistake by the GPRP is that it treats the Upper Clay, which has a thickness of up to 15 m, as a homogeneous material. This is by far not the real case. Instead it is composed of layers and lenses of sandy clay, clayey or silty sands, and sand, each with highly varying soil properties, see Figure 2.1.

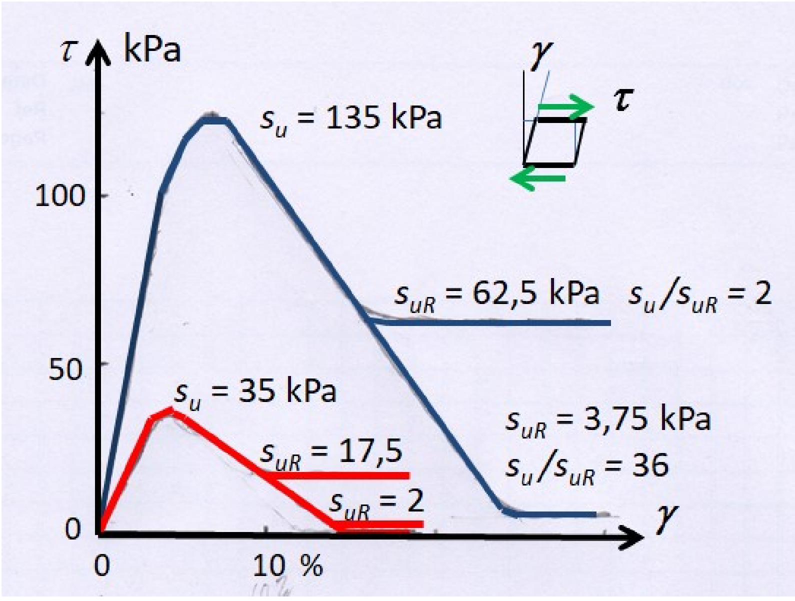

Figure 2.1. Possible variations in stress-strain relationships for the Upper Silty Clays in the North Spur based on Table 2.1 in Leahy (2015a). The undisturbed maximum strength is denoted by s_u and varies between 35 and 135 kPa. The disturbed remoulded strength is denoted by s_{uR} and has values as low as 2 and 3,75 kPa. As no deformation properties have been provided, the values of the deformation angels (γ) and the inclinations of the curves are guessed. (Bernander & Elfgren 2017, 2018).

In October 2013, Bernander visually and tangibly studied large portions of the Churchill River Valley from the downstream Bridge river crossing to areas North of the massive Edward Island landslide.

Numerous soil exposures of the lower Upper Clay in the Sedimentary Drift were scrutinized. In the capacity of being a former chief design engineer (Skanska West Ltd) with marked geotechnical experience, Bernander can assure the GPRP that in none of these exposures were highly over-consolidated layers with rich clay content (i.e. type Eastern Canadian clays) to be seen. Instead just stamping by foot or jumping locally a few times induced wavelike movements in the closely surrounding ground surface.

The character of large portions of the Upper Clay II layer has been explained in Bernander (2015) Sections 2 ”On Extreme Sensitivity of Lean Clays”, as well as in Section 3 “Relevance…”.

However, in the current context, it is quite sufficient to apply the *basic geotechnical relationships* such as the Liquidity Index (*LI*) and the Plasticity Index (*PI*), rendering the relations between:

- the water content w (→ porosity in saturated soils);
- the Liquid Limit (w_L);
- the lower Plastic Limit (w_{pL})

The Liquidity Index (*LI*) is defined as $LI = (w - w_{PL}) / (w_L - w_{PL})$, and by which it is quite possible to identify the character of a soil.

The Plasticity Index, $PI = w_L - w_{PL}$, is a measure of the range of plasticity of a soil.

The following sentence is e.g. a quotation from Terzaghi et al (1996, Section 7.2 Consistency of Remoulded Soils, p. 24):

Quote*: "If the water content of a natural soil stratum is greater than the* ***Liquid Limit*** *(i.e. Liquidity Index greater than 1,0), remoulding transforms the soil into a thick viscous slurry."*

According to SLI's *own data* in the Engineering Report (Leahy 2015a), *Table 2.1*, the *LI* varies between 0.6 and 2.8 with an average value of 1.3.

Admittedly, the value 0.6 indicates that there are local layers of normally consolidated soils with somewhat *richer* clay content *but* the *average value* being 1.3 indicates high sensitivity in most of the Upper Clay II formation. The value 1.3 means that the average water content is 30 % higher than the Liquid Limit (*LL*) – i.e. high sensitivity.

Ironically, it may be concluded from the mentioned *Table 2.1* that, the more layers with rich clay content there are in the Upper Clay II, the more soils with a *low clay content* – of the type identified by Bernander – must be present.

LI = 2.8 represents a layer where the water content is 180 % more than *LL*, indicating a soil layer that is quite conceivable to liquefy.

Considering the Plastic Limit is perhaps even more elucidative. In *Table 2.1*, the general Plastic Index (*PI*) varies between 2 and 22 %, while the average value is 11 %. The following analysis is easily made:

	Thickness of clay with PI = 22 %	*Depth of other soils rendering mean PI =11%*	*Corresponding Plasticity Index PI in the non-clayey soils*
(a)	50 %	50 %	0.0 %
(b)	40 %	60%	3.7 %
(c)	30 %	70 %	6.3 %

This implies that – if in case (a) half of the Upper Clay II had the maximum Plasticity Index of 22 % – the rest of the Upper Clay would consist of sand with no plasticity at all, whereas if for instance in case (b) 40 % of the Upper Clay were clay with PI = 22 %, the range of plasticity of the remaining 60 % of the formation would only be 3.7 %.

Conclusion: The GPRP's statement that the clay content in the Upper Clay II formation is between 45 and 65 % is thus *effectively contradicted* by SLI's *own* list of properties in *Table 2.1* (Leahy 2015a). Instead, the values given in *Table 2.1* thus effectively support the observations and statements made by Bernander regarding the Upper Clay II formation (Bernander, 2015).

In any case, the *LI* and *PI* – values of *Table 2.1* verify that the Upper Clay II can in *no way* be regarded as comparable to the *highly over-consolidated* soils with *rich clay content* typical of Eastern Canada. The properties and the behaviour of soils such as those in the Stratified Drift are therefore totally different from the clays typical of Eastern Canada. (See also the Bernander (2017) report on North Spur Stability Issues.)

Additional comment. Regarding the sensitivity and possible liquefaction of *lean clayey* soils, it may, for instance, be mentioned that the post-slide investigation by the Swedish National Road Administration of the landslide at Rollsbo (1967) – (some 20 km North of Gothenburg) – clearly showed that the failure surface followed a seam of *lean sandy clay* instead of passing through the surrounding very sensitive normally consolidated clay structure.

2.3 Soil properties of the Lower Clay structure

Comment: The *disparate properties* of the Upper Clay are in sharp contrast to the Lower Clay formation, in which the Liquidity Index according to *Table 2.2* (Leahy 2015a) varies between 0,1 and 2 but notably has a *mean value* of *0.6*, implying that the water content is generally lower than the liquid limit and that the sensitivity in most of the soil formation is relatively low.

However, it is of *utmost importance* to observe that, according to *Table 2.2*, there are layers with Liquidity Indices (*LI*) up to 2.0 which means that the water content (*w*) exceeds the Liquid Limit by 100 % thus revealing high risk of local metastable soil layers.

The *average* Liquid Limit being 37 % and that some Plasticity Indices are as low as 7 % point in the same direction.

Conclusion: *Table 2.2* in Leahy (2015a), dealing with the Lower Clay, is indicative of the presence of local metastable soil layers also in this formation. Hence, the possibility of Progressive Failure in the Lower Clay should also be investigated, especially since the slope of an initial failure surface can develop much *more steeply* than for instance in the Upper Clay.

3. Application of Limit Equilibrium Method (LEM) (Issue 3)

GPRP Statement: *Dr. Bernander criticizes SLI for using LEM in stability analysis and mentions that extensive landslides cannot be predicted or explained by using LEM. The GPRP agrees in that LEM cannot be applied to explain the entire event occurring in large retrogressive landslides. However, LEM allows detecting initial failure which may lead to large catastrophic landslides in sensitive materials. SLI's work has focused in using the LEM to prevent an initial failure and ensuing large landslide.*

The GPRP concludes that the LEM methodology applied to evaluate the stability of the North Spur (initial landslide) corresponds to the current state-of-practice. (GPRP, 2018, p 26)

Comment: Although failures on the Eastern Slope of the North Spur have been commented on by Bernander (2015, 2017), the analyses made by both Dury and Bernander *only* relate to *progressive failure* developing on the upstream *West Side*, (Dury, 2017, Dury et al., 2017, Bernander & Elfgren, 2017, 2018)

Firstly, the GPRP statement that… *"LEM allows detecting initial failure….etc."* is *totally wrong* in *strain-softening* soils, and that even more so if there is a potential risk of liquefaction. Surprisingly, the GPRP has apparently not understood the true character of *initial* progressive failure development.

The *main factor triggering* a progressive landslide development is normally a forced shear deformation related to some additional disturbance, and the *strain-softening effect* of this deviatory deformation.

In the current case, a gigantic external force totalling 242,000 kN per 100 m width is applied on the upstream West rim of the North Spur representing an enormous *change* of shear stress and deviatory deformation.

The effects of this enormous force do not just disappear into *nothing*, see Figure 3.1.

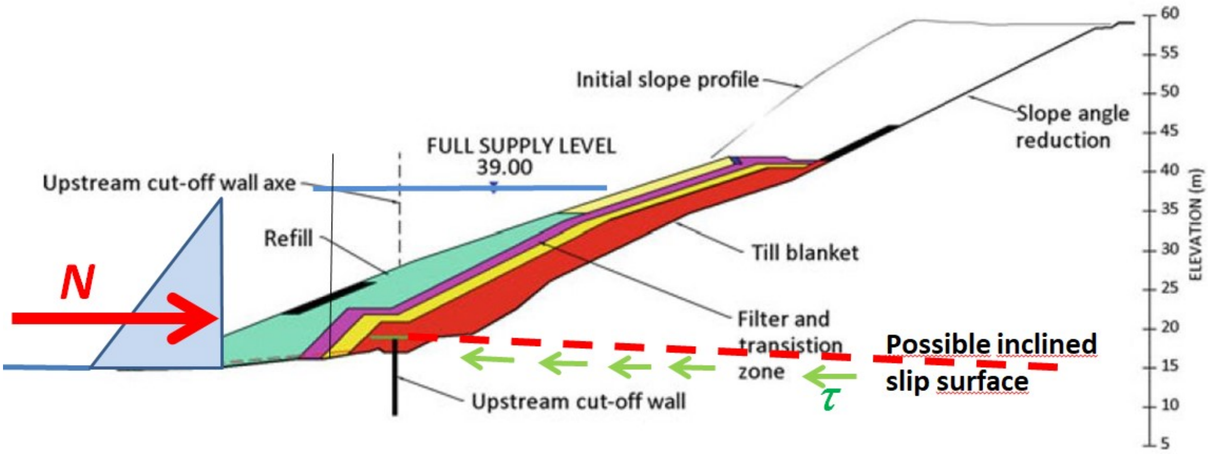

Figure 3.1. The upstream Western slope after stabilization measures according to Leahy et al (2017). To the figure has been added the force ***N*** caused by the raised water level and the resisting shearing stresses τ (green arrows) along a possible inclined slip surface (red dotted line). If the sum of the stresses cannot resist the pressure from the water ***N*** a progressive failure may initiate along the shown inclined slip surface or along another one with less resistance. (Mother Nature will find the weakest link in the chain)

Yet one thing is very *certain*. This force transmission will not, as SLI seems to imagine, adapt to LEM frictional conditions. They will instead be transmitted *by* and *to* very *sensitive soils* that may even have a *potential* to liquefy.

The *undrained* stress/strain (deformation) properties of the extremely sensitive soil layers have not been defined (as shown in Figure 2.1) and the consequences of this lack of knowledge have not been accounted for.

Secondly: a crucially *decisive* factor in this context is of course the *slope* of the potentially developing *failure planes* in the triggering zone.

How can GPRP accept that SLI totally disregards the radical effect of the *inclination* of *failure planes* when accepting SLI's way of.. '*detecting initial failure*'?

Even in the old days, when LEM methodology was still a generally accepted *state of practice*, no practising geotechnical engineer would make any kind of slope stability assessment without duly considering the *inclination* of possible failure planes.

In (Bernander & Elfgren, 2017. 2018), Figure 3.1 exemplifies that the shear stress τ_o may increase with 28 kPa when a small inclination of 4° is considered. This would be more than a 50 % change of the values given for the shear stresses in the upstream bank in Figure 9 in (GPRP, 2018). For greater initial inclinations, e.g. along a failure surface in the Lower Clay, the differences may easily double.

It may also be mentioned that numerous large progressive landslides in Scandinavia have been released by surprisingly small (or seemingly insignificant) effects. One example is the landslide at Bekkelaget, Norway, illustrated in Figure 5.2 in Bernander (2016b). Another example is the Surte landslide (1950), measuring 400 x 600 m^2. It was triggered by driving a few piles for a small family house – thus changing the stability conditions that so far had remained unchanged for some 1,000 years. Minor corrective measures according to Section 8 below would have saved about 40 houses from total destruction (Bernander 2000, 2011, Bernander et al. 2016).

As the SLI has not presented any stress/strain (deformation) relationships likely to be valid for progressive failure analyses, Bernander and Dury have applied 'large *deformation residual shear resistances'* corresponding to values derived from *back-analyses* of large Scandinavian progressive landslides. For instance, in Bernander's analyses, τ_{res}/c = 17.5/70 = 0.25, see Appendix IV in (Bernander & Elfgren 2017, 2018). Dury applied a *wide range* of input parameters

Conclusion: Not only the stresses but also the deformations need to be studied. This has not been done in an appropriate way with the Limit Equilibrium Method.

The fact that SLI has only studied the effect of the impoundment on two horizontal failure surfaces is *astonishing* and has little to do with modern understanding of progressive failure formation. The results by Bernander and Dury indicate low safety factors against progressive failure, and that mainly due to taking the slope of the developing failure surface in the triggering zone into account.

4. Progressive Failure Analysis (Issue 4)

<u>GPRP Statement</u>*: One of Dr. Bernander main criticism is that progressive failure has not been taken into account by SLI. SLI has investigated the shear stress distribution in the North Spur before and after stabilization measures and after impoundment. SLI's strategy is to prevent the initiation of failure downslope or upslope by limiting the shear stress to a value well below the peak shear strength in the North Spur.*

***The GPRP concludes** that the analyses performed by SLI are conceptually acceptable to take into account the initiation of progressive failure. It is however important that SLI have procedures and routines to ensure that the restrictions listed a) to f) are observed at all times, including persons outside of SLI/Nalcor* Energy. (GPRP, 2018, p 27)

<u>Comment:</u> This GPRP conclusion seems primarily to relate to the stability along the Eastern slope. Compare with the previous Section 3.

However, if the GPRP means that the restrictions listed in the items named a) to f) also apply to the prevention of possible progressive failure development at the upstream Western Rim, these limitations are *far from satisfactory*. This is primarily due to the fact that the crucial conditions mentioned in Sections 2 and 3 above have not been dealt with.

As demonstrated in Section 2, even *SLI 's own Tables* of soil properties indicate the presence of highly porous silty *sandy* soils with little *fine grain* content and with a small range of plasticity – i.e. very sensitive *metastable* soils with a potential to liquefy.

In this context, the following passages in Article 17 in Terzaghi & Peck (1976) may be cited:

<u>Quotation 4.1:</u> "*Experience indicates that spontaneous liquefaction most commonly occurs in fine silty sands. This fact, combined with the observed performance of true quick-sands, suggests that the aggregate formed by the sand grains processes a metastable structure, i.e. 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 grains roll down into stable positions. In a sand capable of spontaneous liquefaction, some agent must interfere with this process."*

……………… and *importantly* next paragraph

"The experiments also indicated that the relative density of true quick-sands is <u>very much lower</u> than that corresponding to the critical void ratio".

<u>Quotation 4.2</u>**:***" 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 <u>an earth dam</u>, it is a potential source of danger. Experience suggests that true quick-sands 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 <u>compacted</u> as described in Article 50, because it may contain zones of quick-sands."

For further presentation, analysis and discussion of progressive failure analysis please see Bernander et al. (1978 → 2018). This kind of progressive landslide analysis is not unknown in Canada. Results have been published in the Canadian Geotechnical Journal and elsewhere see e.g. Quinn (2009), Locat et al. (2011), Bernander et al. (2016) and Wang & Hawlader (2017).The method was also discussed in conjunction with the Mount Polley Tailings Storage Facility Breach in British Columbia on August 4, 2014.(Mount Polley, 2015). Here a deformation analysis was undertaken indicating that the collapse occurred when the dam level was risen causing increased shear stresses in a bottom deformation softening layer. The failure occurred when the ratio of the softened shear strength, τ, to the undrained shear strength s_u was 0,29..

Conclusion: Again, the author has no other comments about the Eastern slope stability than those made in Bernander (2016b, 2017)

As already mentioned, the investigatory analyses made by both Dury and Bernander *only* relate to *progressive failure* developing on the upstream West Side. In respect of potential progressive failure triggered in this area, see the conclusions in the previous Section 3, indicating a considerable risk for a breach in the North Spur. The issue should be closely investigated.

5. Dynamic analysis and liquefaction potential (Issue 5)

GPRP Statement: *The GPRP concludes that a State-of-the-Art approach was applied to the North Spur to assess its resistance to earthquakes. SLI's results indicate that the displacements of the crest would be extremely small under a 10 000-year earthquake and that the integrity of the North Spur is assured.*

Comment: Bernander has not studied the dynamic analysis related to earthquake resistance. The question raised by him was only…. whether the dynamic analyses made have been based on LEM type of analysis or if the effects of high sensitivity and possible liquefaction have been contemplated in this context?

Conclusion: No more comment.

6. Mitigation and Remedial Measures (Issue 6)

6.1 COW, till blanket and failure initiation

GPRP Statement: *The GPRP finds that the analyses of the COW presented by Dury and Dr Bernander are based on several incorrect assumptions. The results are therefore not realistic.The incorrect assumptions include the geometry of the problem and of the COW and the initial pore pressure on the downstream side of the COW.* (GPRP, 2018, p 27)

Comment: Here, Bernander *frankly* admits that the location of the COW assumed in the Berrnander/Dury analyses is misplaced.

(The reason for this is that when the Up-stream Typical Cross Section was sent to Bernander it so happened that the printed figure was extremely blurry and that the word *"Up-stream-"* and the *COW* itself were not within the frame of the picture.)

Admittedly, the precise stress situation along the upstream rim is a complicated setting but the *main point in question here* is that the long term previous state of stress is not of *primary* interest in the current context. When considering the initiation of progressive failure in very *sensitive soils*, the *decisive parameter* is the *deviatory deformation* involved.

As already dealt with in Section 3, above, the gigantic *external force* totalling 2 420 kN/m being applied on the upstream West rim of the North Spur, represents an enormous *change* of the *shear stress* conditions and of the related *large deviatory deformations,* see Figures 2.1 and 3.1

In fact, the effective shear stresses in the upstream area are virtually *reverted* – changing direction. Instead of the pore-water seeping westwards down the slope – i.e. shear forces due to seepage acting in a westerly direction – the enormous impoundment pressure generates shear in the *opposite direction*. The shear deformation involved will act on and inside the metastable soils close to the West rim.

If the load is conservatively assumed to be evenly distributed along the first 50 m into the Ridge, we would get a shear stress τ = 2420/50 kN/m^2 = 48,2 kPa. If we to this add the 28 kPa due to inclination (see Section 3), we have a shear stress of 48 + 28 = 76 kPa which is more than double the value of the weakest intact undrained shear strength s_u = 35 kPa in Figure 2.1. Even if we compensate for initial shear stresses of opposite direction, the change in stress and the accompanying change in shear deformation could be very critical in many of the soil layers in the Upper Clay II.

Bernander and Dury made investigatory analyses based on a *wide range* of *input data*, the intention – lacking the appropriate in-put data – obviously not being to establish 'precise and final' factors of safety. The analyses were rather meant to highlight the *intricacy* of the stability conditions on the West rim.

The *deformational effects* of the additional force due to impoundment *cannot* just *vanish.* By blockage, i.e. by the COW, by the up-stream fill and by seepage friction, this force will somehow be transmitted to the *sensitive soil structure* in the vicinity.

How the highly sensitive soils in the Upper Clay II (possibly with a tendency to liquefy) and sensitive layers in the Lower Clay may respond to the shear deformations related to impoundment *has not been addressed* by SLI.

Conclusion: The location of the COW is not the primary issue in the current context. The crucial issue is namely how the shear deformations, related to the impoundment, will affect the highly sensitive soils, possibly initiating progressive failure in the Upper Clay or in steeply sloping sensitive seams in the Lower Clay formation.

The GPRP has not addressed these extremely *vital* issues at all in their review of the SLI investigations.

6.2 Finger drains

GPRP Statement: *Regarding finger drains, the GPRP's opinion is that they are necessary to maintain appropriate drainage on the slope of the downstream face of the spur and to reduce infiltrations and, to some extent, increase the stability of the slope.* (GPRP, 2018, p 27)

Comment: Bernander has nowhere objected to the use of finger drains anywhere in the North Spur. On the contrary he has recommended more efficient drainage of metastable soil layers.

Conclusion: The GPRP statement is pointless.

6.3 Erosion protection measures

GPRP Statement*: The GPRP concludes that erosion protection measures (rip rap) put in place by SLI are necessary along the river to prevent both wave and ice erosion in the future. This will reduce the susceptibility of an initial failure that could occur on both sides of the North Spur.* (GPRP, 2018, p 27)

Comment: Bernander has nowhere objected to the application of erosion protection measures (rip rap) put in place by SLI.

Conclusion: Fine

6.4 Driving closely spaced piles in the North Spur to investigate if metastable soil layers exist

GPRP Statement: *The GPRP is strongly against Dr. Bernander's proposal of driving closely spaced piles in the North Spur to investigate if metastable soils are present. Such investigation would generate excess pore pressures in the sensitive clay and undermine the stability of the slopes and hence the entire Spur.* (GPRP, 2018, p 27).

Comment: Driving of piles in porous soils may certainly be a risky procedure but, with due precautions, such work can be carried out safely.

In engineering practice, Bernander has faced the problem with high porosity innumerable times and therefore – like most practising geotechnical engineers – knows how to deal with it. The routine procedure is of course *checking* the pore water pressures in the ground and *installing* appropriate *drains*.

For instance, when piling with concrete piles, a line of permeable material is often attached to the pile forming a drain, and if too high excess pore water pressures are registered, the piling operation is just *temporarily* moved to another locality on the working site.

The way to deal with liquefaction in metastable soils must be known to the GPRP. See also Bernander (2016b, 2017) Applying such precautions in work, supervised by knowledgeable geotechnical engineers, slides of this kind can easily be *prevented*. Effective measures of avoiding the risk of liquefaction are treated in Section 5.8 in Bernander (2016b), where a six-step procedure is presented. From the measurements of the 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 the analysis in Leahy (2015a) will be generally confirmed.

If, on the other hand, the settlements indicate a high degree of compaction – i.e. the mean in situ porosity (n) is clearly in excess of the critical porosity (n_{crit}) – *then it will be necessary to strengthen the affected soil structures*. As per Terzaghi & Peck (1976), 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. (Bernander, 2016b, p. 22-23). Another possible remedial measure is compaction of metastable soil layers by appropriate grouting procedures. Such a method is presented in (Bernander, 2004).

Furthermore, while firmly dissuading Nalcor/SLI from permitting even all sorts of minor vibratory activity on the *crest* of the North Spur – because of the innate risk of liquefaction in the porous subsoils – the GPRP simultaneously refrains from demanding SLI to correctly investigate the effects on the same porous soil layers related to the enormous external change of *stress* and *deviatory* deformation due to impoundment.

Conclusion: The practical measures to avoiding liquefaction and slides due to piling, blasting or vibration are *well known* in Scandinavia. It would be very surprising if they are not also well known to Canadian geotechnical engineers.
In the current situation, effective compaction of possible metastable soil layers is probably the only practical mitigation measure to be considered.

7. Consequences (Issue 7)

GPRP Statement: *The GPRP concludes that the aspects of dam breach and consequences downstream at Muskrat Falls have been investigated and attended to by SLI and covered by reviews performed by HATCH.*

Comment: Assessing the risk of flooding is a difficult task, and the world record is full of examples of catastrophic failures in this respect – both in developed countries (such as France, Italy, the U.S, Canada) as well as – yearly – in many developing countries.

However, not having studied this issue in much detail, the authors abstain from further comments.

Conclusion: None

8. Summary and Conclusions

The purpose of Bernander's engagement in North Spur stability issues is of course not to argue against the completion of the Muskrat Falls Dam Project. The only aim is to make the predictions of the North Spur stability meet the standards of '*up to date'* Research and Development.

The Geotechnical Peer Review Panel has written a comprehensive report discussing the conditions at the North Spur. They have focussed on well-known types of slides common in Canada as well as along the Churchill River valley. However, they do not seem to fully understand the mechanisms of the – in Canada less frequent – downward progressive failure. For this reason various points of criticism regarding statements by Bernander brought up in the GPRP Report are essentially *wrong.*

The GPRP Report does not either in any way contribute to an improved understanding of the stability of the North Spur related to possible progressive failure development in connection with impoundment.

The fact that the GPRP generally accepts the use *only of Limit Equilibrium analyses* on *horizontal failure surfaces* – thus totally disregarding up-to-date Research & Development – the authors of this report find really astonishing.

Furthermore, while dissuading all sorts of minor vibratory activity on the *crest* of the North Spur – because of the innate risk of liquefaction in soils some 40 to 50 meter below – the GPRP simultaneously refrains from demanding SLI to correctly investigate the effects on the same porous soil layers related to the enormous external change of *stress* and *deviatory* deformation due to impoundment.

In the current situation, compaction of possible metastable soil layers is probably the only effective and practical mitigation measure to be considered. Driving down piles is one approach, another possible compaction method is grouting.

As no up-to-date analysis of the stability of the North Spur has been provided, our conclusion is that an independent group of experts, appointed by government, should be entrusted with this important task.

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About the Authors

Stig Bernander was born on February 5, 1928, in Mnene, Rhodesia, Africa. He attended primary and secondary schools in Mnene and Bulawayo, Rhodesia, and in Gothenburg, Sweden. He studied civil engineering at Chalmers University of Technology and obtained a M.Sc. in 1951.

Having worked for the Swedish Board of Roads in Stockholm 1951-53, he moved to Skanska Contracting Co, which in those days was named Skånska Cementgjuteriet AB. In 1972 he became Head of their Design Department in Gothenburg. He retired in 1991 and started a consulting company of his own, Congeo AB.

Stig Bernander has designed or been engaged in major civil engineering works such as bridges, dams, harbors, tunnels, dry docks, off-shore structures, buildings, underground storages and water supply structures in Sweden, Denmark, Norway, Poland, Monaco, Egypt, Saudi Arabia, India, Sri Lanka and Zimbabwe.

In the years 1980 – 98, Stig Bernander served as an Adjunct Professor at the Division of Structural Engineering at Luleå University of Technology, working primarily with crack prevention and modeling of temperature stresses in hardening concrete - taking various boundary conditions into account.

After the large landslide in Tuve (Gothenburg, 1977), Stig Bernander began developing a finite difference model for slope stability analysis taking the deformation-softening of soft sensitive clays into consideration. In the model, the mean down-slope deformation in each element caused by normal forces is maintained compatible with the deformation generated by shear stresses.

He developed software for the model and presented it at international soil mechanics conferences during the 1980-ies. In 2000 he summarized his findings in a Licentiate thesis. An easy-to-use spread-sheet has also been developed.

In 2011 he further conveyed his experiences of slide modelling in a PhD thesis focusing on the nature of triggering agents and the different phases that a slope may undergo before its stability becomes truly critical.

Lennart Elfgren was born on July 9, 1942, in Gothenburg, Sweden. He studied civil engineering at Chalmers University of Technology and obtained a M.Sc. in 1965 and a PhD in 1971 with a thesis on "Reinforced concrete beams loaded in combined torsion, bending and shear. A study of the ultimate load-carrying capacity"

After a post doc stay at University of California at Berkeley 1972-73 working with curved box-girder bridges he was appointed to a position as Associate Professor in Structural Engineering at the recently started Luleå University of Technology.

In 1981-83 he worked as a Consulting Engineer with Jacobson & Widmark in Gothenburg and in 1982-83 as part time Researcher in the Swedish Research and Testing Institute in Borås.

He returned to Luleå as Full Professor in 1983 and has served as Department Head and Dean of the Faculty of Engineering Sciences. He has studied anchorage of sheet piling in soft clays, anchorage to concrete, fatigue, fracture mechanics and strain-softening materials and, in the last 20 years, assessment and strengthening of existing structures including numerical modelling and full scale testing to failure of bridges. He has been the main supervisor for 14 PhDs and an associate supervisor for another 14. He has published more than 300 papers and reports, see https://ltu.diva-portal.org/

He is a Member of the Royal Swedish Academy of Engineering Sciences (IVA), an Honorary Member of the International Association for Bridges and Structural Engineering (IABSE), a Fellow of the American Concrete Institute (ACI) and of the Swedish Concrete Association, and a long-time Member of the International Federation of Concrete (fib)

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