From: Nikolay Argirov <nikolay.v.argirov@mwhglobal.com>
Sent: Thursday, December 4, 2014 11:56 PM
To: gbennett@nalcorenergy.com; pharrington@lowerchurchillproject.ca
Cc: Krupski, Joseph (Joseph.Krupski@NRCan-RNCan.gc.ca); Manzer, Alison (amanzer@casselsbrock.com)
Subject: FW: Field Trip report
Attachments: Bernander University 1.pptx

Just FYI.

This is what I have to deal with. Looks like the opponents have a long reach. They even try to influence the independent engineer internally. Ingenious!

Good old Jim… they are using him well. My question to you is who funds the Cabot Martin "enterprise" ? Makes me wonder if Hydro Quebec have something to say about it…

Also , you should perhaps give Dr. Bernander a retainer. Hire him on some kind of advisory capacity so instead of criticism he might provide a constructive idea. He might be just fishing for that.

Well Cabot will lose his fire power and probably be upset about it but hey…

Regards,

Nik

From: Jason Hedien

Sent: Thursday, December 04, 2014 12:14 PM

To: Nikolay Argirov; Patrick Corser; Mario Finis; Donald Erpenbeck

Subject: FW: Field Trip report

E-mail with attachment.

From: David Kleiner

Sent: Thursday, December 04, 2014 1:29 PM

To: Jason Hedien; Chris Ottsen; Thomas Andrews

Subject: FW: Field Trip report

To all,

The following email and attached file was sent to me by Jim Gordon. Jim served on the same Hydro Quebec panels with me up to about 2008. He sent the following email to me so that I might alert MWH to the issue of quick clays at "The North Spur", Muskrat Falls Project.

I don't know what our scope of work is as Owner's Engineer or whether it includes review of the stability analyses of the North Spur. I do know that this issue requires careful study and analysis by experts in the specialized field of quick clays.

I am not an expert in quick clays, but I do know that major damaging slides have occurred within this type of material in Quebec, in the Scandinavian countries, and elsewhere. I recall that quick clay outcropped at the downstream end of the tailrace at Hydro Quebec's Eastmain 1 project. A slide at that location would have blocked the tailrace. The tailrace excavation through the quick clays was thoroughly studied. As a result, the excavated slope was supported progressively with filter, rockfill, and riprap as excavation proceeded. No distress occurred during construction or during project operation. Movement and settlement were monitored during construction and during project operation.

I believe that one of you should communicate with NIk to alert him to the issue and ask how it is being dealt with.

I'm interested, so please let me know the outcome. If you wish, you or Nik might communicate directly with Jim Gordon at the address below.

Dave

From: Jim Gordon [mailto:jim-gordon@sympatico.ca]

Sent: Wednesday, December 03, 2014 5:11 PM

To: David Kleiner

Subject: FW: Field Trip report

Dave - trust you are keeping well. Here, recovering from loss of Vera to a massive stroke, health slowly deteriorating. But life continues.

I am communicating with you in the hope that someone further up the ladder at MWH will review the following.

You probably know that MWH Americas is the "Owner's engineer" on the Muskrat Fall project in Labrador. There is considerable controversy over the stability of the north bank of the river, commonly known as "The North Spur", about 1km long and comprising "quick clay".

The project owner, NALCOR has not provided any details on the stability
analysis, and perhaps Nick Argirov, the MWH project manager, does not
appreciate the seriousness of the situation. If the north Spur fails, the
project would become unviable - cost of repair would be just too high. The
resulting lawsuits would involve MWH, since they have approved the
analysis, which, according to Dr. Bernander is based on false information.

Dr. Bernander has visited the site, sponsored by a group of very concerned
citizens, headed by Cabot Martin. Dr. Bernander is the world's foremost
expert on "quick clays". Ed Martin, the CEO of NALCOR has no experience in
hydro work, coming from the oil and gas industry in Alberta, hence bases
decisions on advice from the consultants.

If after looking at the above slide show on the Bernander site visit, and
MWH wished to continue with the investigation, I can forward numerous
files ranging from Dr. Bernander's doctoral thesis to detailed drawings
and reports on the project, all probably also available from Nick.

Regards, Jim.

James L. Gordon

621-15 Place de la Triade

Pointe Claire, Quebec.

Canada H9R 0A3

From: cabot@lucaresources.ca

To: jim-gordon@sympatico.ca

Subject: Field Trip report

Date: Wed, 3 Dec 2014 18:08:55 -0330

Dear Jim

Still working on getting full Bernander presentation online but with
Christmas coming things seem to slip.

I am sending the powerpoint he used to you in three sections in three
emails ; Attached is Part 1.

You may also be interested in the Field Trip report on
www.northspurfieldtrip.com

Shows our trip to Lower Churchill with lots of shots from a helicopter; I
am sure some of this territory will be familiar to you.


Regards

Cabot Martin

Lecture by Dr. Stig Bernander

on "Quick Clay " and
**   The North Spur**

Memorial University
St. John's,NL
October 31, 2014

As this topographic map indicates, the North Spur is seriously scar infested in a way typical of highly sensitive normally consolidated clays or what often referred to as *'Quick clays'*.

Typical Section through the North Spur
Vertical scale <u>twice</u> the horizontal scale
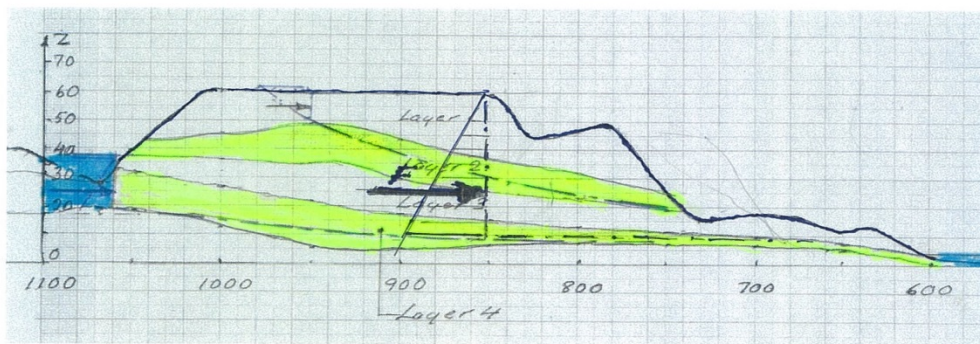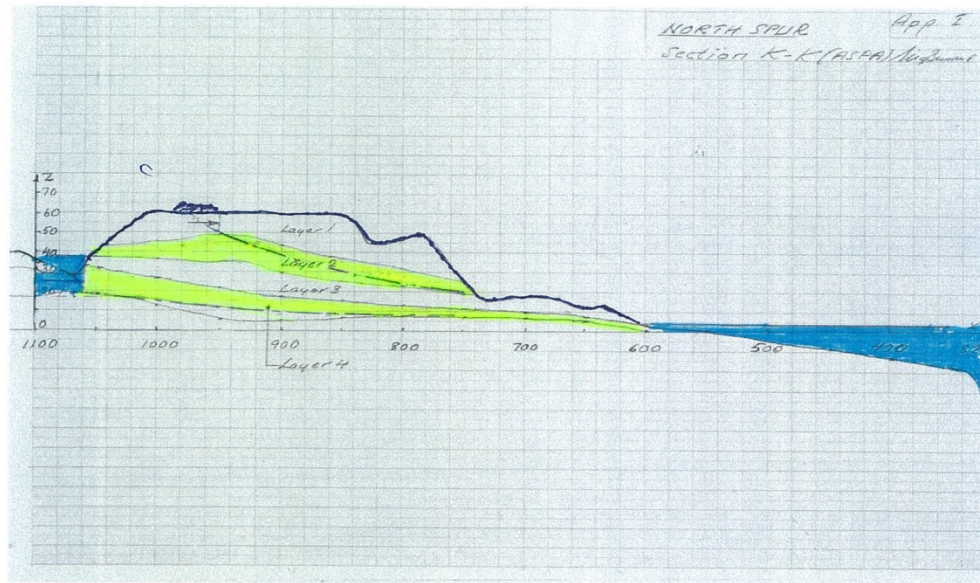
$$F = 6700 \ kN/m$$
$$= 670 \ metric \ Tons/m$$

General

- Concept of "Brittle failure" and
- Liquefaction of Quick Clay
- Equilibrium analysis
- Different kinds of "Progressive Failure"
- Landslide phases

2. General

In the following, it is not my intention to contend that the North Spur containment is an impossible undertaking.

Yet, predictions of slope stability have to be based on *reliable* analyses according to *present know-how* and *R & D*, and that particularly in respect of *possible hazards* related to *brittle failure formation* in the highly sensitive clays of the North Spur.

Generally, the Nalcor report (sent to me 2014-08-27 for review) offers little information to anybody wanting to do a critical evaluation of some of the important geotechnical issues….
especially regarding *possible stability problems* related to the impoundment on the upstream side of the North Spur, as well as to the *possible impact* of various *construction measures* in connection with the dam project.

Little or insufficient information is presented on several subjects related to soil mechanics and to slope stability.

The report indicates little focus on slope stability hazards related to the possibility of *Progressive Slope Failure formation* in the quick clays.

Nalcor/SNC apply the **Limit Equilibrium Stability Analyses methodology** according to a letter from the Department of Natural Resources (Minister Derrick Dalley).

This is on my part a major concern, and that simply because this mode of analysis does *not apply* at all to *long* potential landslides in slopes with *quick clays* , especially if affected by *local additional loading.*

When I refer to **progressive slope failure** as a sort of **brittle** process, I often use the following metaphor:

If you cut a tin sheet or a piece of cardboard with a pair of scissors, the failure (i.e. the cut) is restricted to where you apply the tool. **But**, if you do the same to a window pane, the whole glass sheet will crack in an unpredictable way.

As a student I once turned up at a party, and the host came up to me with a regular water glass half filled with red wine. We raised our glasses, which touched *lightly* in a toast ….. but when tried to drink the wine there was nothing in the glass. The glass bottom was on the floor and the wine had splashed all over. This event resulted from the combined effect of **material brittleness** and **existing high stress levels** in the glass.
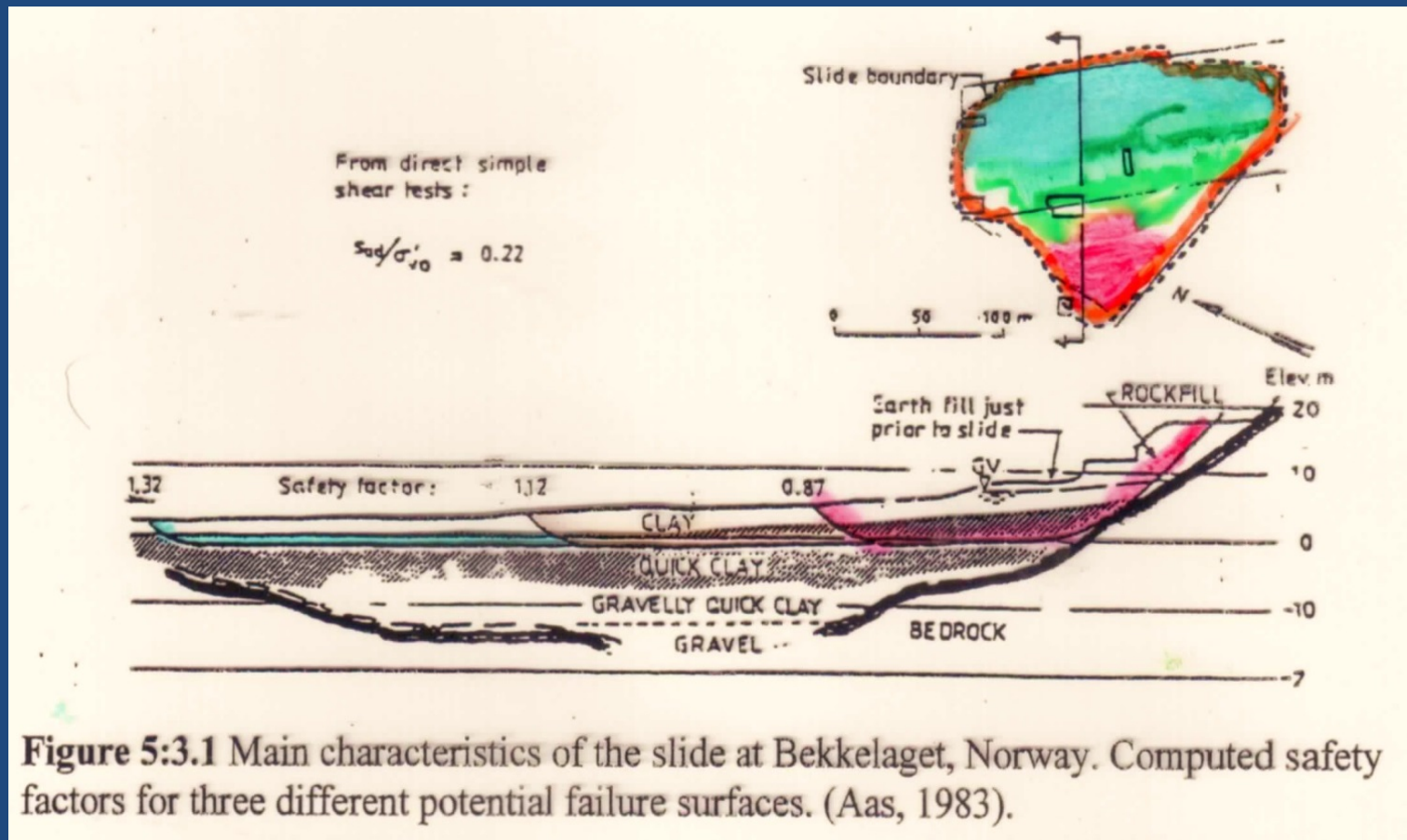
As I intend show later on, slopes of quick clays can fail in a similar way due some small and seemingly insignificant **change** of **load** due to

a) - **high sensitivity** in combination with

b) - **high shear stress levels** in the **in situ** condition.

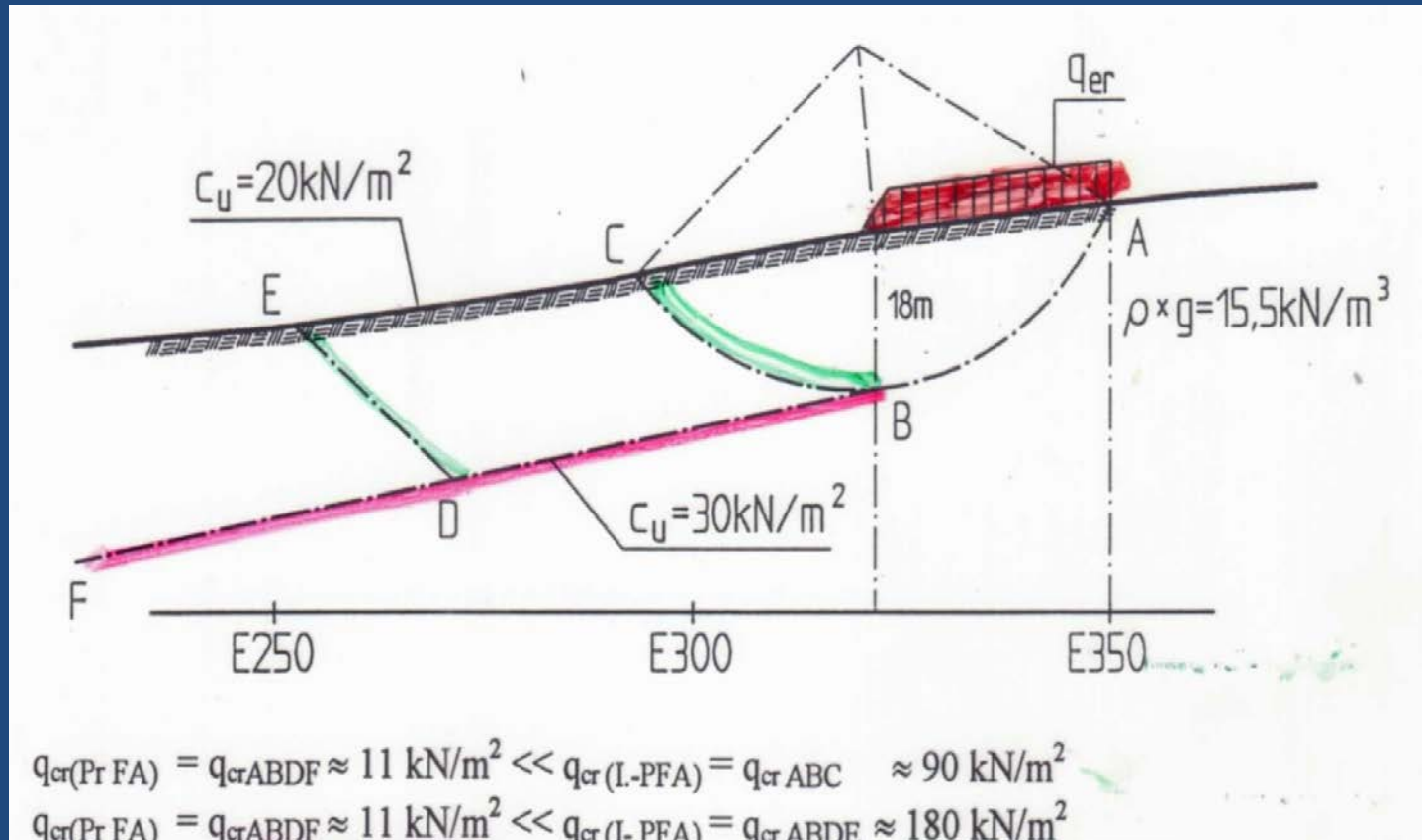_Conclusion:_ **This implies in turn that Plastic Limit Equilibrium analysis** is in general **not applicable** to **quick clay** formations.

However, progressive failure development in soil structures is **predictable** provided correct analysis – considering deformations and the **different phases** of a landslide are considered. Hence, **a progressive landslide cannot be predicted** correctly by just studying **one singular static failure** condition as has generally been practiced all through the last century.

Notable *exemplifications* of the invalidity of PLE and Slip circle analysis

Figure 5:3.1 Main characteristics of the slide at Bekkelaget, Norway. Computed safety factors for three different potential failure surfaces. (Aas, 1983).

However, the apparent oddity of the features of the Bekkelaget landslide is *consistent* with *progressive failure* analysis.

$$q_{cr(Pr\,FA)} = q_{cr\,ABDF} \approx 11\ kN/m^2 << q_{cr\,(I.-PFA)} = q_{cr\,ABC} \approx 90\ kN/m^2$$

$$q_{cr(Pr\,FA)} = q_{cr\,ABDF} \approx 11\ kN/m^2 << q_{cr\,(I.-PFA)} = q_{cr\,ABDE} \approx 180\ kN/m^2$$

In this presentation I therefore hope to spend some time explaining what is meant by

brittle slope failure….

or what in geotechnical terms is known as

progressive slope failure

of which there are three main types:

a) *Retrogressive* slides (in Canada denoted 'spreads', being a recurrent problem in the highly *over-consolidated* of eastern Canada.)

b) *Serial retrogressive* slides (often called earth flows)

c) *Downhill progressive* landslides (Common in *normally consolidated* clays).

3. NALCOR assessments   (Ref Nalcor July 21,2014 North Spur Updated as per Nalcor website)

FOUR POINTS MADE by NALCOR
re Safety factor against progressive failure

1 " • Calculations are based on slope geometry, soil properties, groundwater properties. Calculations are calibrated locally with an existing slope."

2 " • Rotational, flowslide, spread stability is calculated with a first movement at the toe."

3   "• There is no evidence of downhill progressive failure landslide along the Churchill river valley."

4 "• Counter measure will be in place to control "Human triggering."

Comments on NALCOR assessments under the heading

"Safety factor against <u>progressive failure</u>"

Assessment 1

· Calculations are based on slope geometry, soil properties, ground water properties. **Calculations are calibrated locally with an existing slope.**

*Comment: The **first** sentence is self-evident and goes without saying. Yet, the **second** sentence is highly **controversial**. If the calibrations referred to are based on the **Plastic Limit Equilibrium** approach ….. (i.e. **PLE** for short) such calibrations are of little value.*

*Progressive landslide development depends on so **many different factors** that calibrations of this kind have to be based on some kind of Progressive Failure Mode (**PFM** for short) to be of any use. The **deformations** within and outside of the potentially sliding body and the **different phases** of the possible slide must be considered.*

*Impact of **time:*** It may be observed in this context that a specific slope may have existed for **hundreds** (or even thousands) of years but can, depending on the **geometry** of the **potential** failure surface and/or the **rate** of application of the additional **triggering** load, be totally destabilized by seemingly insignificant human activity and other factors.

Progressive failure analysis shows for instance that… if, a fill is placed during a period of say 6 months, a slope may very well remain stable. …..**but** if **exactly** the **same fill** is built up on **exactly** the **same slope location** in say in the time of a week, **a catastrophic landslide can result.**

Assessment 2

" Rotational, flow-slide, spread stability is calculated with **a first movement at the toe.**"

Comment: The analysis must be based on the Progressive Failure Approach.

The toe contributes very little to stability as compared to the active earth pressures acting further uphill.

A small percentage change in the uphill active force is more dangerous than a larger percentage change at the toe.

Assessment 3

"There is **no** evidence of downhill progressive failure along the Churchill river valley."

Comment:

*Large landslides in highly sensitive (quick) clays often develop **all three** of the mentioned **types** of progressive failure.*

*The great **Rissa** landslide in Norway is an **excellent example** of this.*

However, I am sceptical towards this Nalcor assessment …. and that for many reasons.

On our investigation tour, many both ancient and recent landslide scars were seen indicating downhill landslide formation and similar phenomena.

Assessment 4

"Counter measure will be in place to control "Human triggering"

Comment:

Not valid because Nalcor has *not based its analysis on a* ***Progressive Failure*** *Mode.*

The Rissa Landslide - 29th April 1978
A volume of soil of about 5- 6 millions m³ slid into the fjord.
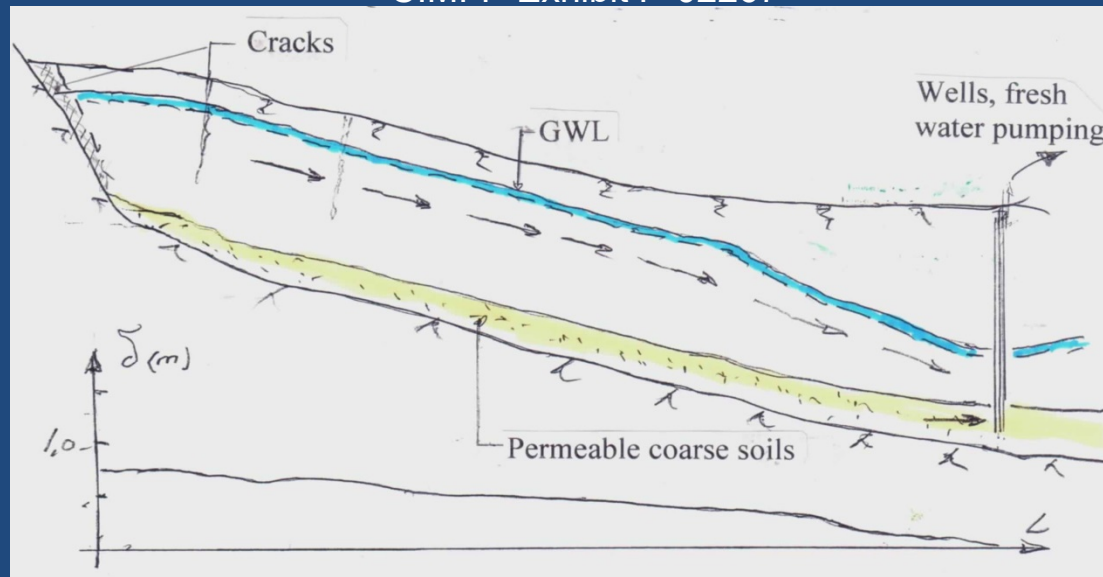
According to the Nalcor report, the intention is to stabilize the downstream slope by *drainage wells*. This is of course an apt measure for preventing failure in coarse materials such as tills and sands.

However, a phenomenon that should at least be *contemplated* in this context are the effects of downhill *creep* movements due to the lowering of the ground water table (GWL) further downhill.

These deformations may not be destabilizing as such …but depending on specific features in the soil structure **deep** cracks often form.

When such cracks get filled with water under continuous raining, they can threat stability… especially in combination with downhill movement causing loss of shear resistance in quick clays.

I have seen many examples of this phenomenon in Sweden.In one instance, I found a 0.5 m wide crack in the clay layers bordering a steep cliff formation. and that about 1 kilometre away from the place where water was being extracted.

4. The progressive FDM-model - Basic principles
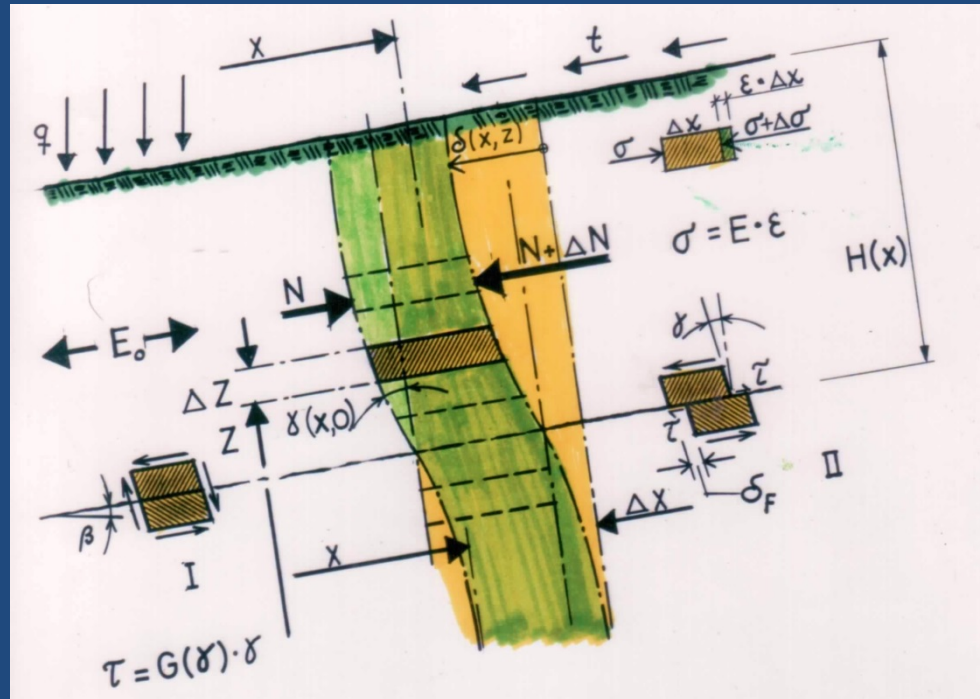
An axial force ΔN generates a displacement $\Delta \delta_N$. The related shear stress increments $\Delta \tau$ generate the deformations $\Delta \gamma$ and $\Delta \delta_\tau$. Compatibility demands that for each vertical element (Δx)

**$\Sigma \Delta \delta_\tau(z) = \Sigma \delta_N(x)$**
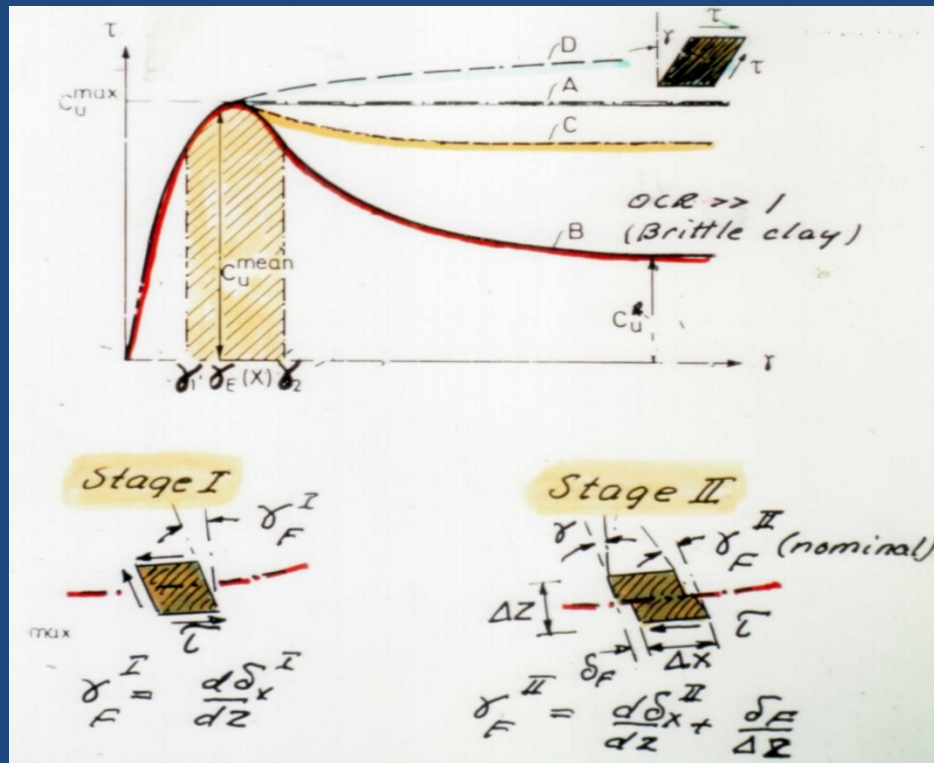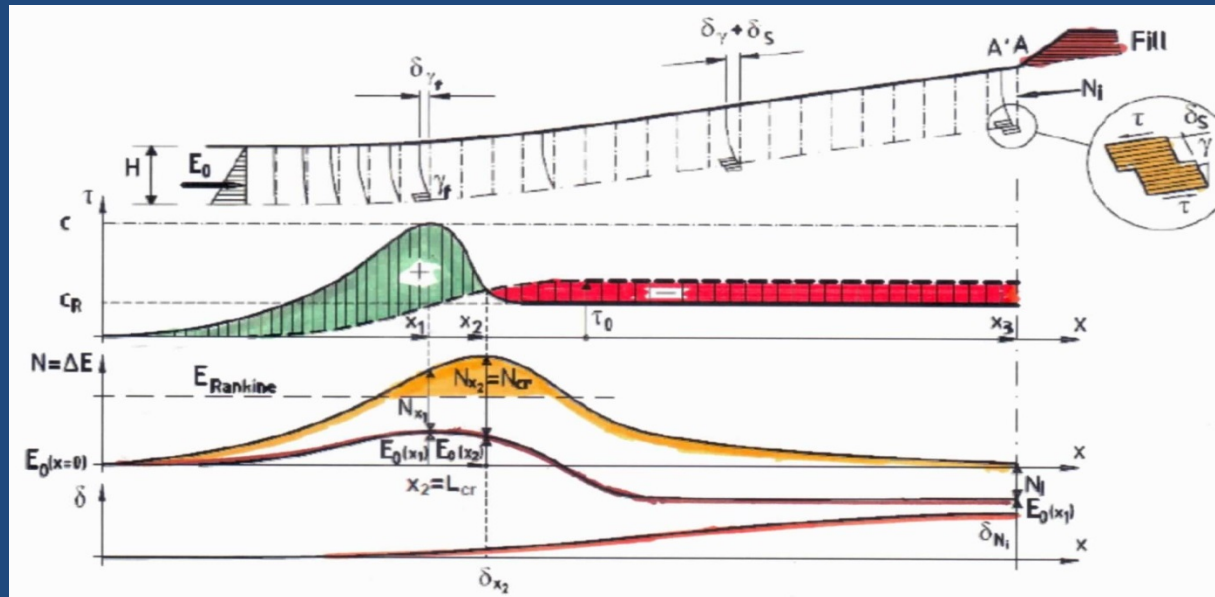
5. Different phases in landslide formation

Progressive failures in natural slopes exhibit several distinct phases that may be defined as follows. The figures on the <u>next slide</u> will illustrate different critical stages in the development of a downhill progressive landslide related to deformation softening.

(In publications since NGM (Linköping, 1984) and ISL (Toronto, 1984), I have distinguished between different stages of downhill progressive landslide formation as follows:

-   the existing (or primordial) *in situ stage, (Phase 1)*

- the *disturbance phase*, subject to conditions relating to the agent triggering the slide. *(Phase 2)*

- an intermediate, *virtually **dynamic stage** of **stress redistribution,*** when unbalanced up-slope forces are transmitted further down-slope to more stable ground. *(Phase 3)*

- a *transitory* (or in some cases *<u>permanent</u>*) *new state of equilibrium* defining the related earth pressure distribution. *(Phase 4)*

- *final breakdown* in passive failure, provided current passive resistance is exceeded in this new state of equilibrium. *(Phase 5)* Phase 5 represents what is **normally understood** as the **actual slide event.**

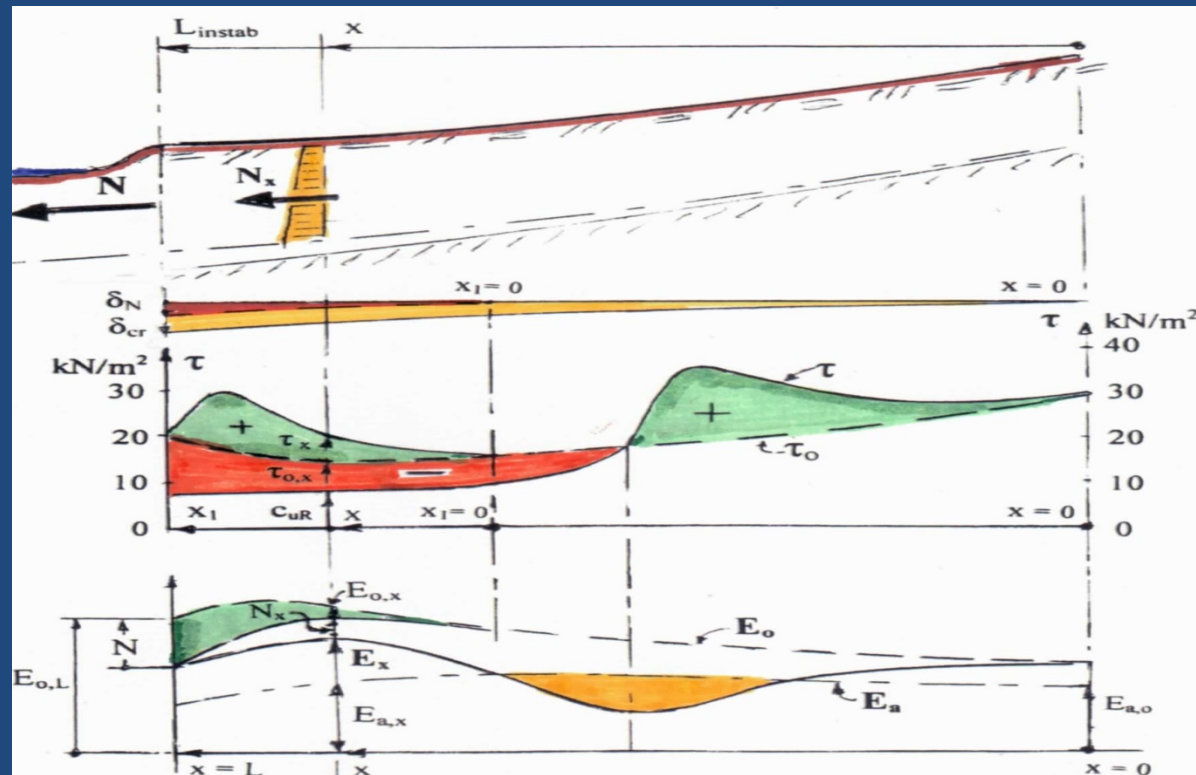- *terminal state* of *equilibrium* - resulting ground configuration. *(Phase 6)*

Phase 2 Triggerig load
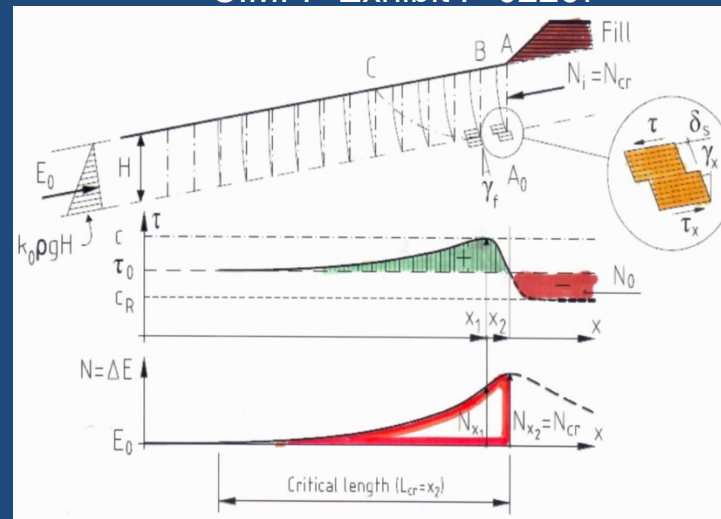
Phase 4 End of the dynamic phase – representing a possible new state of static equilibrium.

Example of *uphill progressive* (retrogressive) slope failures or *'spreads'* as they are usually denominated in Canada.

Retrogressive landslides (Spreads)

Note: Retrogressive landslides (or spreads) are **in no way** less disastrous or preferable to downhill progressive landslides, which are mostly related to foreseeable human activities.
In fact, spreads are actually more **difficult** to **predict** as they are often caused by *long term geological processes* such as e.g. erosion.
Spreads can be calculated using the same *mathematical expressions* as for downhill progressive slides, the only difference being that the signs of certain parameters change from *positive* (**+**) to *negative* (**−**).
The *direction* of the coordinate x-axis is reversed.

6. About Plastic Limit Equilibrium Analysis (PLE analysis)

In this approach, the clay is taken to be a *perfectly plastic material*.

This implies that, at failure, full *shear resistance* is assumed to be mobilized *all along* the potential failure surface.

It also means that, in the limit equilibrium *failure condition*, deformations within the sliding volume of clay are inconsequential,
and hence do not need to be considered in the analysis.

In other words, the volume of sliding soil is assumed to '**infinitely**' stiff
in comparison with the surrounding soil structure. This is of course not really the case at all.
l
The PLE-analysis is reasonably valid in *plastic* clays and *evenly distributed* loading.

However, in the case of soft normally consolidated (or slightly over-consolidated) clays, the validity of *PLE analysis* importantly depends on various factors such as:
- the length of the potential slide.
- the degree of clay sensitivity (*deformation-softening*),
- the *distribution* of the *additional* 'triggering' load,
- the *nature* and the *rate* of application of the additional load, i.e. the
  time factor,
- slope *geometry* and the *structure* of soil layers.

The validity of PLE analysis must *be questioned* in connection with potentially *extensive* landslides, and that particularly if the *additional* load is concentrated *locally*.

Furthermore, if the clay material is *highly* deformation-softening as is the case with the so called 'quick clays', the *Plastic Limit Equilibrium Analysis* is an all together invalid approach.

A crucial point in this context is that the Plastic Limit Equilibrium principle has *wrongly* been applied in engineering practice almost throughout the 20th century, i.e. *even* to extensive landslides measuring several hundreds of metres in length and that also for *local additional* loads.

Using this failure mode, engineers have – *even in hindsight* – *repeatedly failed* to *explain* the real cause, the development and the large spread of   landslides that have actually *occurred*. Nor have these landslides occurred been *predicted* by engineers despite the fact that, in many cases, *regular soil investigations* and *geotechnical analyses* (of that time) were actually duly made.

The references of the Natural Resources Department to slope stability studies carried out in the 1960-ties are therefore in my opinion of very little value in the current context.

The so called *'slip circle'* mode of analysis has always been based on PLE analysis.

Hence, landslide analyses based on large *'slip circles'* (i.e. chord lengths in excess of ≈ 50 meters) cannot be relied upon at all.

(**Note:** I am here referring to landslides having occurred because of *additional local* loading, and the lengths of which have been well in *excess* of 50 metres.
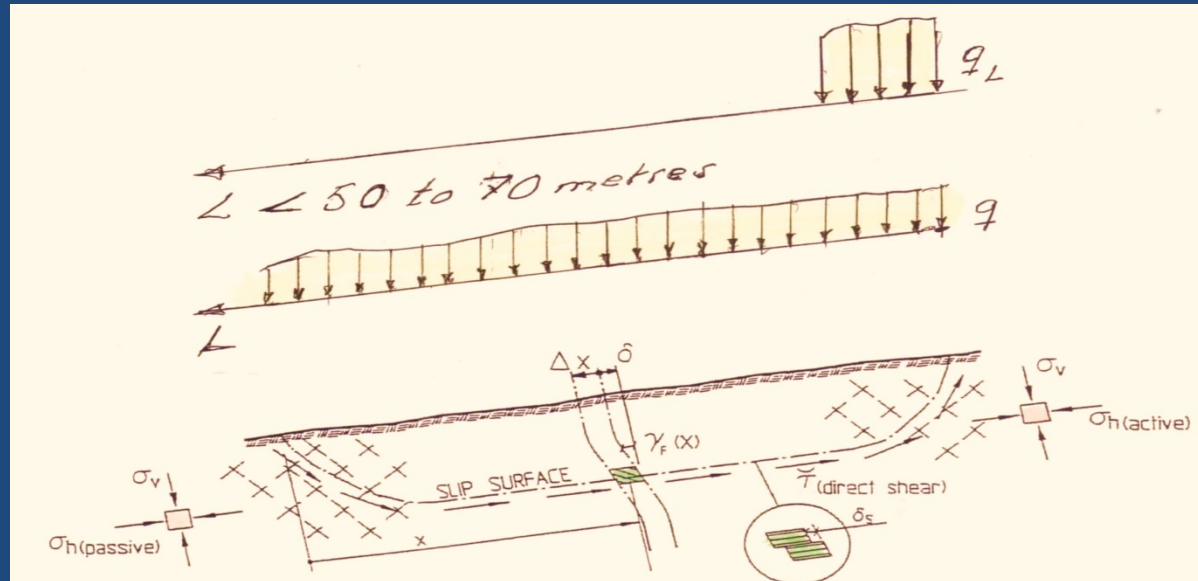
Figure 2:1.1 Shear deformation and shear stresses in a vertical plane of a potentially sliding soil mass. In ideal-plastic failure analysis (I-PlFA) based on unlimited plasticity, the effects of deformations in the soil mass on stress distribution are disregarded.

The *inadequacy*, or in many cases the invalidity, of the Plastic Limit Equilibrium approach (PLE) depends essentially on two factors:
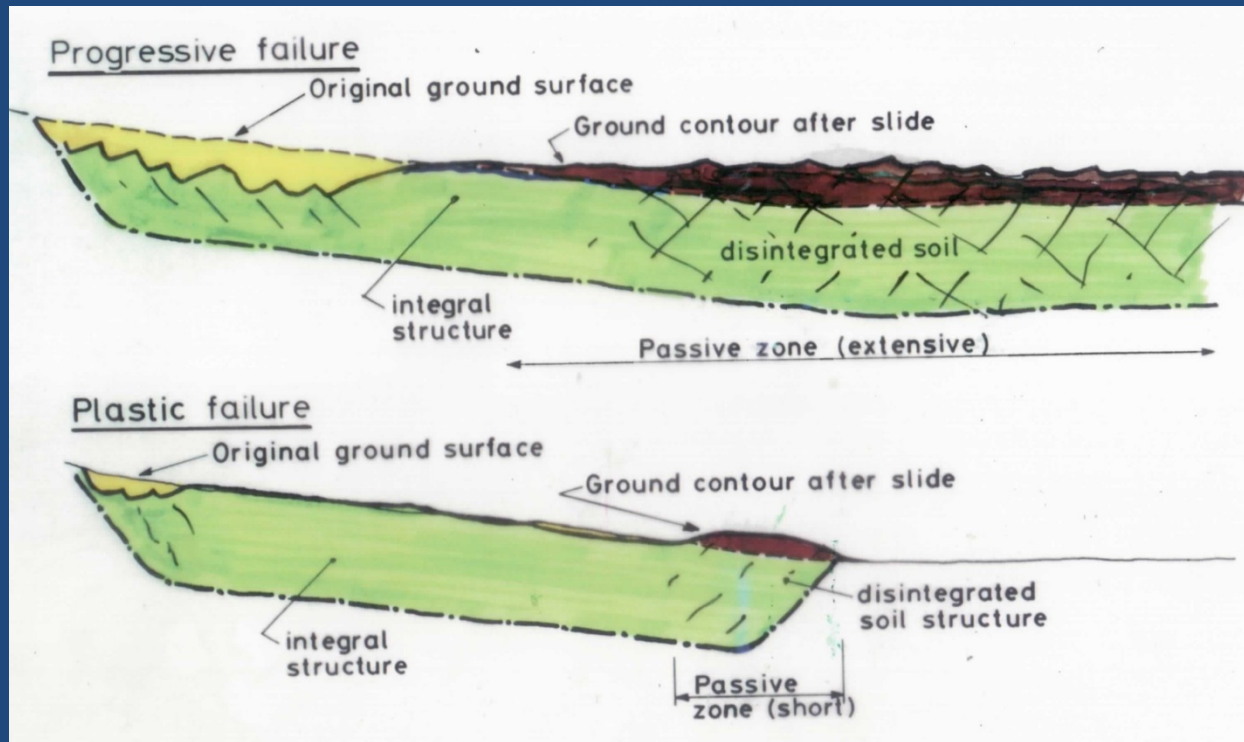
1) The *deformations* in the soil due to additional loading are dis-regarded. In large potential landslides, with lengths in the order of 100 to 600 meters, deformations *have to be considered* in the analysis.

2) In the PLE mode, the safety factor ($f = S_{mean}/\tau_{mean}$) is based on the assumption that the shear resistance (S) is fully mobilized along the potential slip surface in the failure condition, i.e.

$$f = {}_0\Sigma^L (S_x \cdot \Delta x / \tau_x \cdot \Delta x) = S_{mean}/\tau_{mean}$$ **(where L is the length of the slip surface)**

This is of course only true as long as the *distribution* of *in situ* stresses and *additional* triggering stresses are *homogeneous* or at least *roughly similar* in shape along the potential failure surface.

If this condition is *not fulfilled,* as is the case with locally concentrated additional loads, the PLE analysis has *little relevance*.

Progressive failure
Original ground surface
Ground contour after slide
disintegrated soil
integral structure
Passive zone (extensive)

Plastic failure
Original ground surface
Ground contour after slide
disintegrated soil structure
integral structure
Passive zone (short)

Figures featuring typical final appearance of slides in brittle and plastic clays:

Top figure: **Final phase** of progressive (brittle) slope failure with **extensive** passive zone spread.

Figure below: Plastic slope failure with a **local** passive failure zone at the foot of sloping ground. This ground configuration is seldom observed in large landslides (i.e. **>** 50 to 60 meters) in soft clays.

7. Effects of the residual shear resistance in the dynamic Phase 3
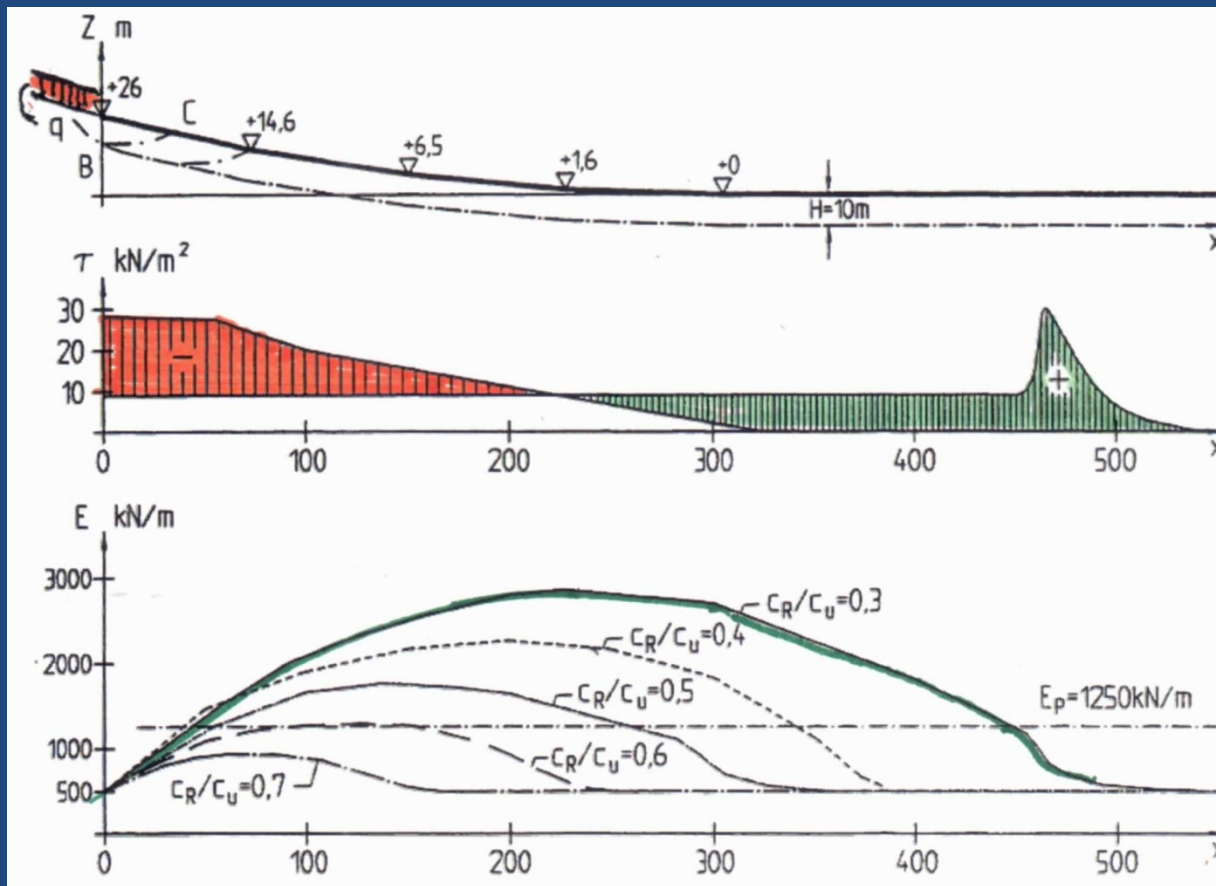
c_R denotes the residual shear resistance in the virtually **dynamic** phase (i.e. Phase 3) in relation to the peak shear stress.

In the current exemplification, there will be no real landslide as long as $c_R/c_u > 0.6$, whereas, if $c_R/c_u = 0.3$, the length of the potential slide will exceed 400 metres.

Hence, when uphill unstable forces are transmitted down to more level ground, the residual shear resistance c_R is a **most decisive parameter** for predicting the degree of catastrophe of a potential landslide.
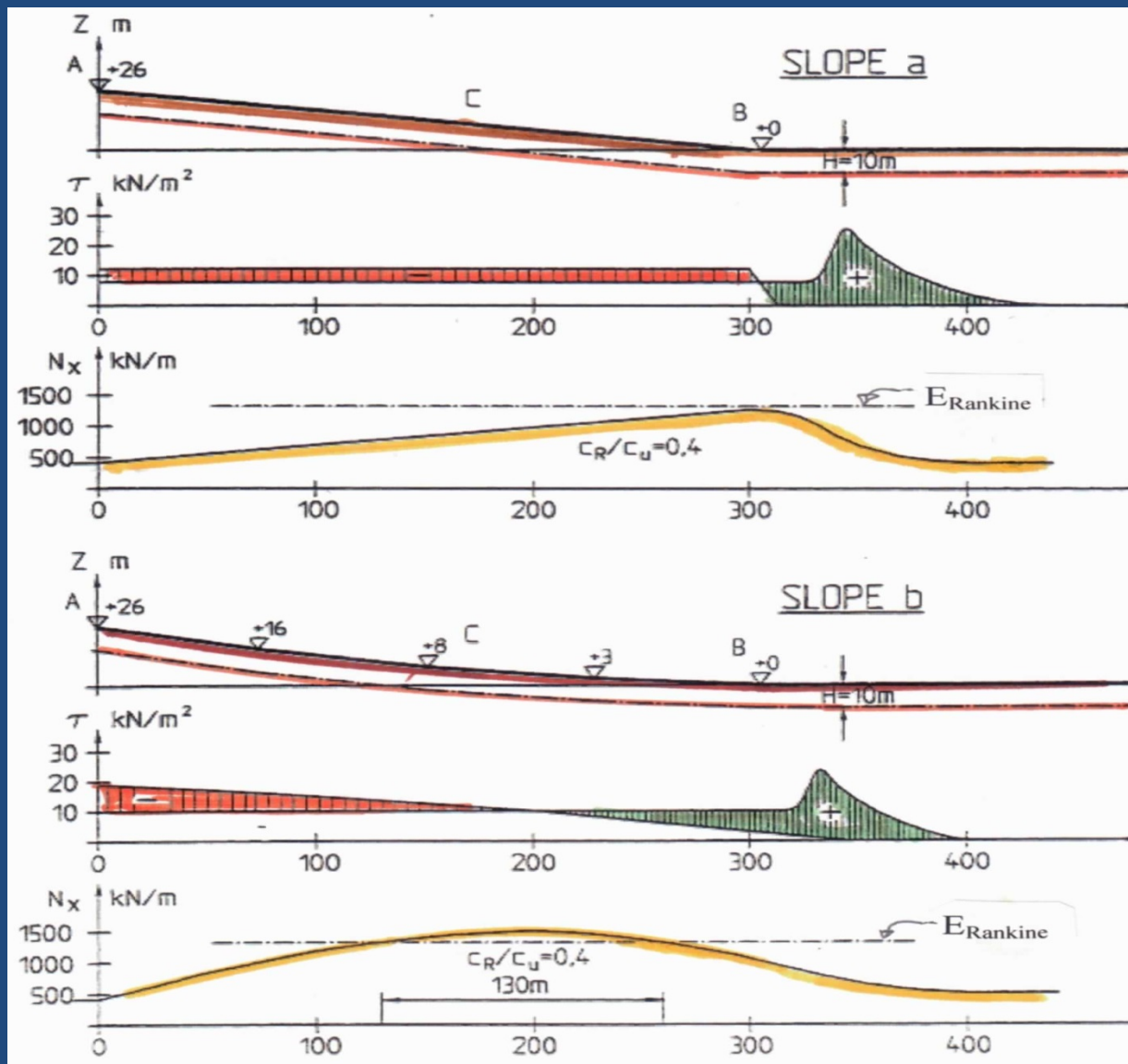
In the case shown, a real landslide will take place *only* in Slope b,

9. Triggering agents

For landslides in soft sensitive clays, it is mostly possible to identify the triggering agent, which is normally directly related to *human activities*. This often applies *even* to slides occurred during intensely wet periods. Water saturation increases in various ways the load setting off a landslide. As indicated in the Table below, the triggering agent is often related to a surprisingly *insignificant* additional load due to construction activities of the following kind:

- Stockpiling of heavy materials, earth fills, construction of supporting road embankments;
- Excavation work, straining the initiation zone in *lateral* direction;
    - Driving of soil displacing steel pipes, prefabricated concrete piles or soil displacing sand drains;
- Soil compaction using heavy vibratory equipment;
- Rock blasting;
- Man-made interference with hydrological conditions, changing the existing ground water regime.

Examples
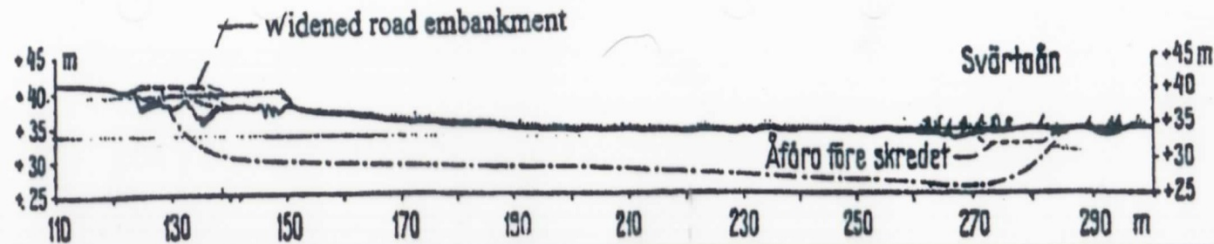
| Locality	Year	Length [m]	Area [100m]²	Triggering agent
The Svärta River	1938	160	2	Local road embankment (Cf figure)
Surte	1950	600	24	Pile driving for a family house
Beckelaget, Norway	1953	160	2	Widening of railway bank up-slope
Rollsbo Kungälv, Sweden	1967		2	Driving of pipes for sand drains
Rödbo, Kungälv	1968		1	Stock piling of blasted rock
Jordbro, Haninge	1972			Local up-slope earth fill
Rävekärr, Mölndal	1971	≈300	15	Pile driving for a family house
Sem, Norway	1974	120		Local earth fill up-slope
Tuve, Göteborg	1977	800	26	Widening of road embankment etc
Rissa, Norway, slide C	1978	800	27	Retrogressive initial slide
Kotmale dam site, Sri Lanka	1981	500	9	Stockpiling of concrete aggregate
Trestyckevattnet, Uddevalla	1990	400	2	Vibration of road embankment
Saint-Fabien, Quebec, CA	2004			Widening of an up-slope embankment
Småröd, Munkedal	2006	230	ca 10	Local up-slope earth fill
Namsos, Norway	2009			Rock blasting

Combined human activity and continuous raining

Analysis considering strain and deformations in a sensitive soil predicts that placing of local additional loads inexorably generates deformations in the downhill direction.

This movement in turn inevitably results in cracking under - and in particular behind – additional load as shown in the figure below.

In spells of continuous precipitation, these cracks may develop a *'jacking action'* process, which is the reason why landslides – even those that are primarily due to *human involvement* often happen during rainy periods.

A specific risky combination is for instance an earth fill getting fully saturated by <u>steady</u> rainfall………like in the case of the 150 000 m^2 Småröd landslide in Southwest Sweden.
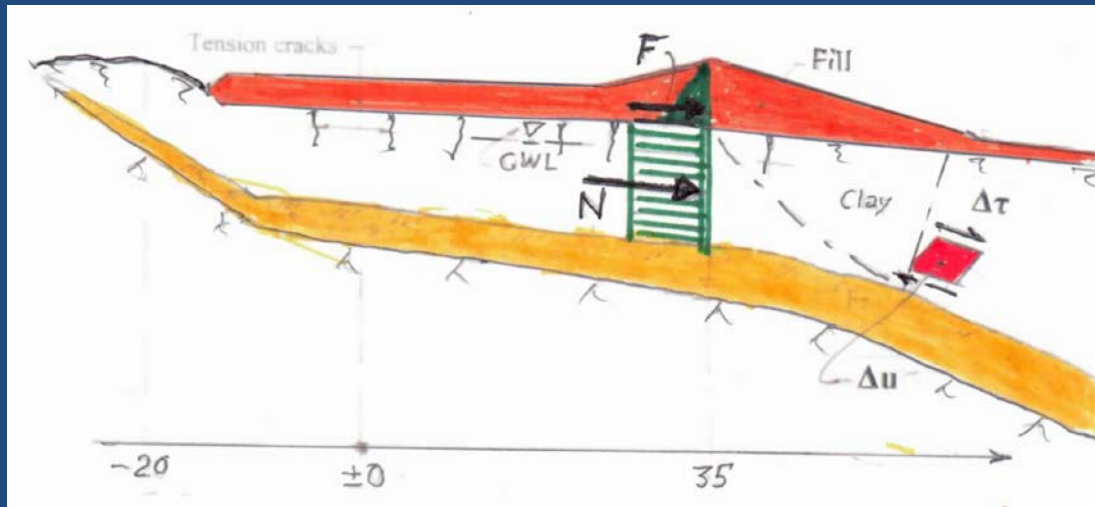
In this landslide, it was estimated that merely the water saturation of the fill increased the downhill acting load by 186,7 kN/m … i.e. by some 750 metric tons over a width of 40 metres, (the width of the fill.)
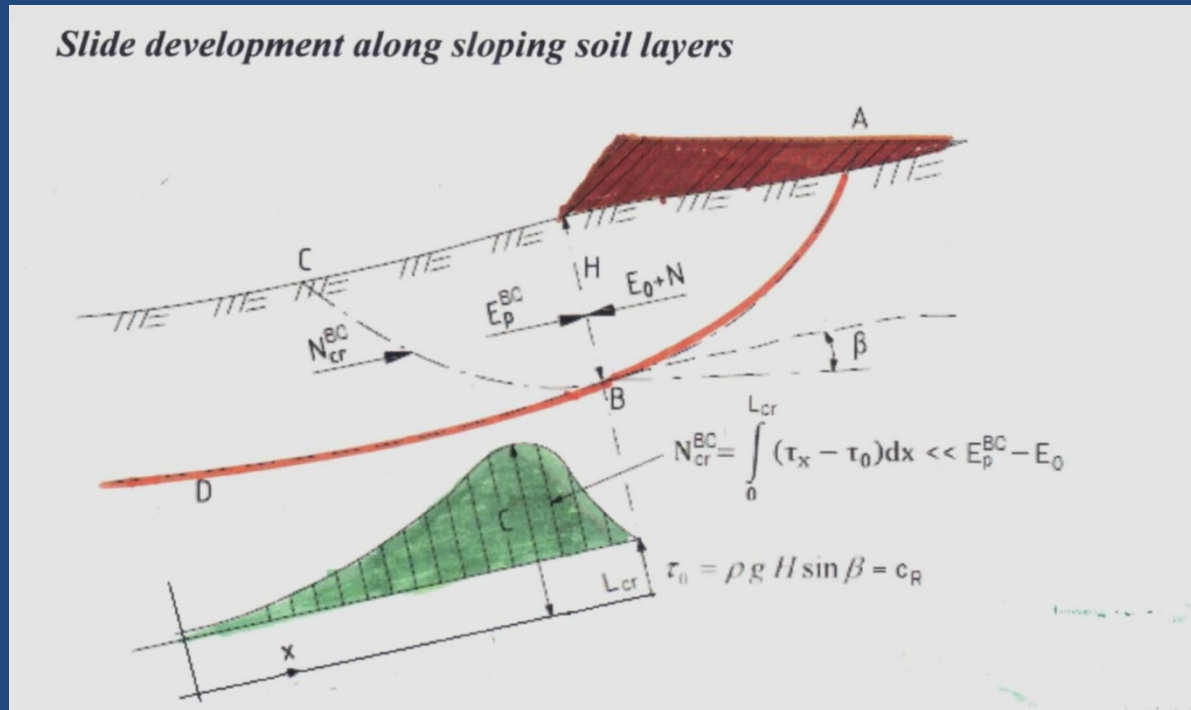
The **immediate cause** of this landslide was water saturation of the provisional storage of sandy material.

This type of additional load is probably the ***most common triggering agent*** in progressive landslides caused by human activities.

Slide development along sloping soil layers

$$N_{cr}^{BC} = \int_0^{L_{cr}} (\tau_x - \tau_0)dx << E_p^{BC} - E_0$$

$$\tau_0 = \rho g H \sin\beta = c_R$$

Maximum additional earth pressure (N) that can be mobilized along failure plane BD. Note that $N_{crit} << E_p^{BC} - E_0$
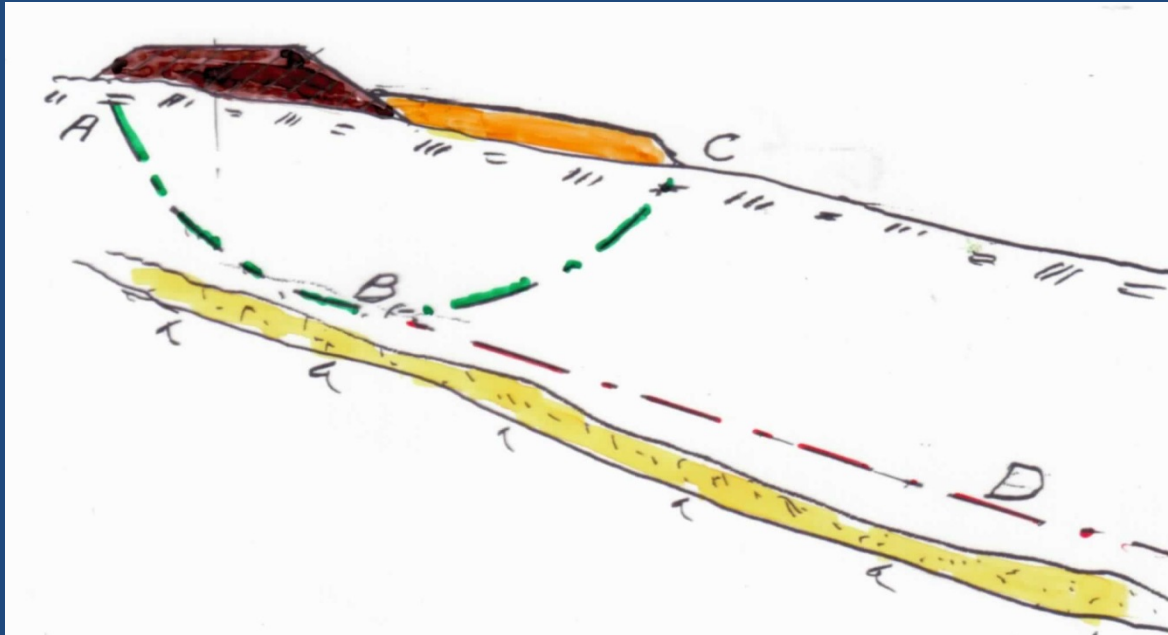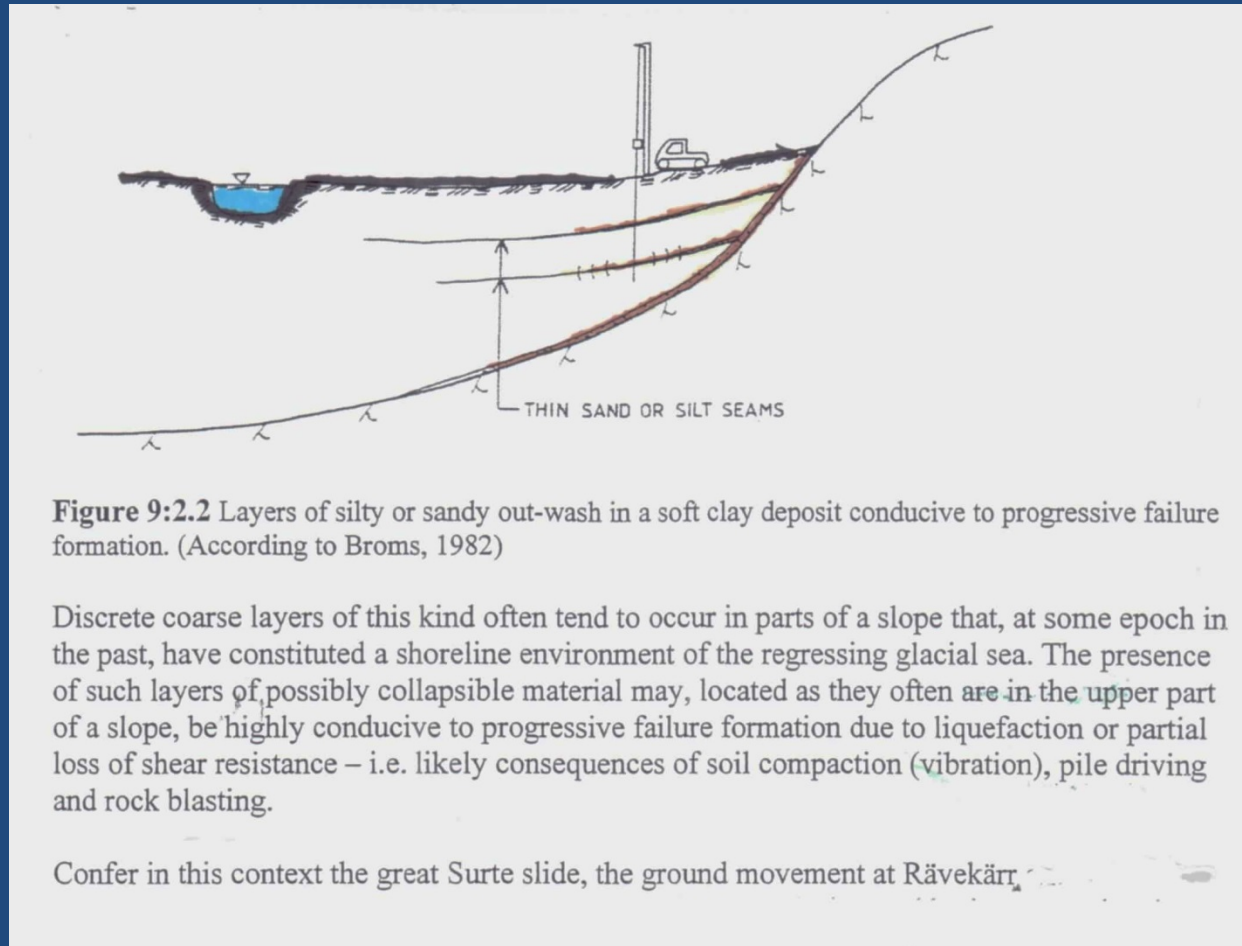
Road embankments are often stabilized by an additional berm which may be an apt measure in perfectly plastic clays.

However, in *sensitive clays,* the resistance along plane ABD is less than along ABC. In such a case, the berm itself is likely to enhance a slope failure that is far more serious than the one intended to be forestalled.

Berms of this kind constitute a very common cause for extensive downhill progressive slope failures.

Pile driving and sand drain installations

Driving soil-displacing piles and sand drains are common triggering events in progressive landslides.

Figure 9:2.2 Layers of silty or sandy out-wash in a soft clay deposit conducive to progressive failure formation. (According to Broms, 1982)

Discrete coarse layers of this kind often tend to occur in parts of a slope that, at some epoch in the past, have constituted a shoreline environment of the regressing glacial sea. The presence of such layers of possibly collapsible material may, located as they often are in the upper part of a slope, be highly conducive to progressive failure formation due to liquefaction or partial loss of shear resistance – i.e. likely consequences of soil compaction (vibration), pile driving and rock blasting.

Confer in this context the great Surte slide, the ground movement at Rävekärr.

Heavy vibratory activity and blasting are other hazardous human construction activities that often set off large landslides.
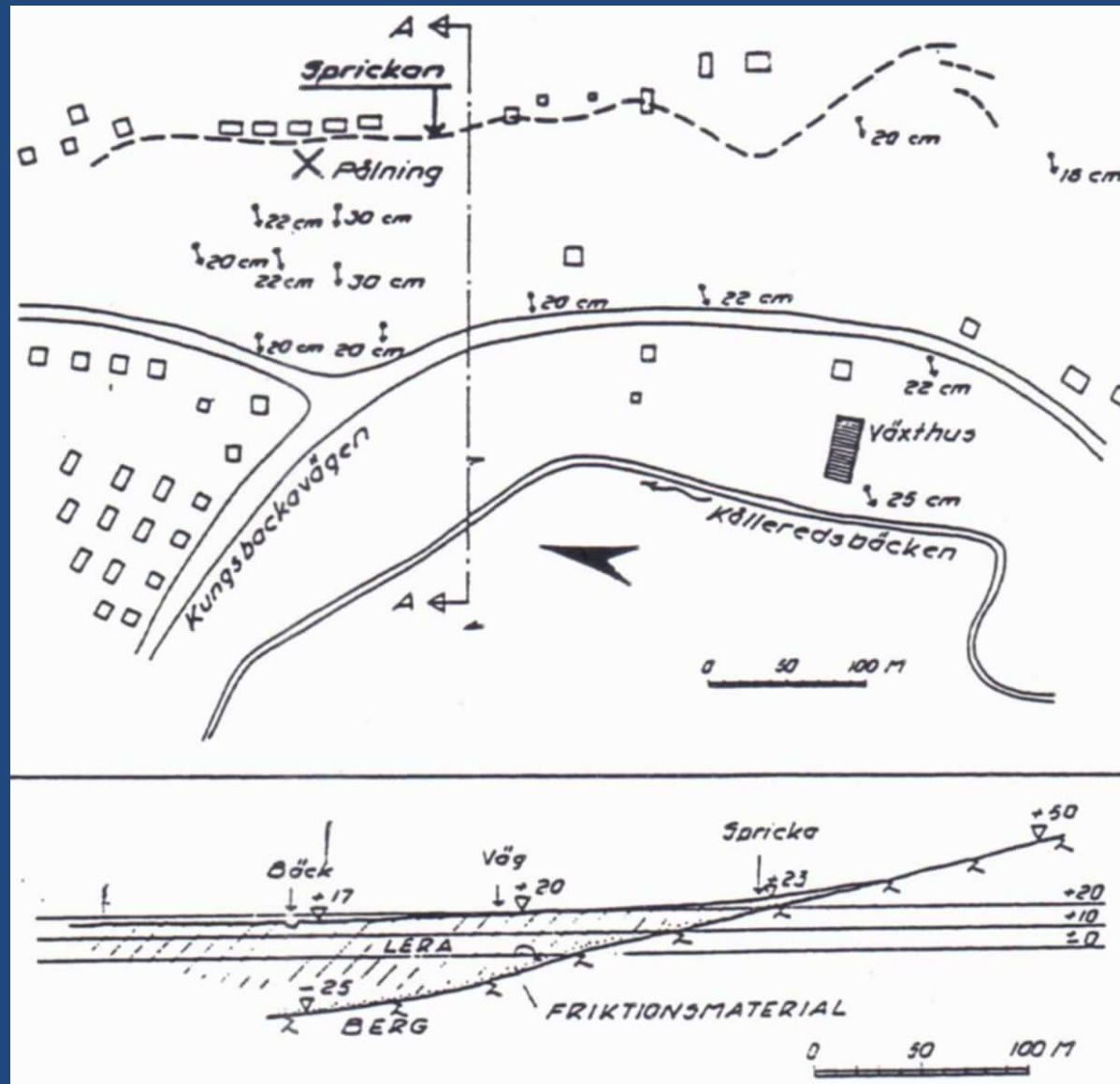
10. Examples of **Progressive Landslides Catastrophes**

:1 The invalidity of Plastic Limit Equilibrium (PLE) approach.
Notable *exemplification* of the invalidity of PLE and Slip circle analysis
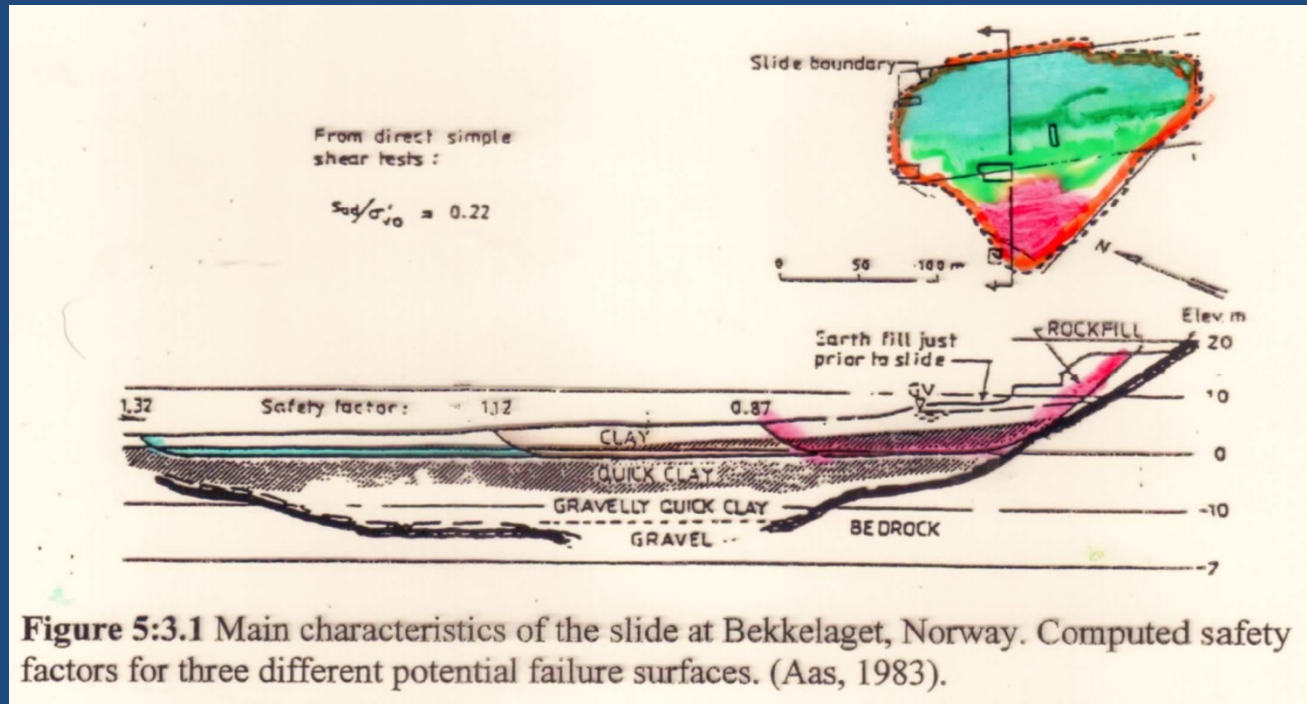
Figure 5:3.1 Main characteristics of the slide at Bekkelaget, Norway. Computed safety factors for three different potential failure surfaces. (Aas, 1983).

However, the apparent oddity of the features of the Bekkelaget landslide is *consistent* with *progressive failure analysis*.

The *inadequacy*, or in many cases the invalidity, of the Plastic Limit Equilibrium approach (PLE) depends essentially on two factors:

a) The *deformations* in the soil due to additional loading are dis-regarded. In large potential landslides, with lengths in the order of 100 to 600 meters, deformations *have to be considered* in the analysis.

b) In the conventional PLE mode, the safety factor ($f = S_{mean}/\tau_{mean}$) is based on the assumption that the shear resistance (S) is fully mobilized along the potential slip surface in the failure condition, i.e.

f $= {}_{0}\Sigma^{L}$ ($S_{x}\cdot\Delta x/ \tau_{x}\cdot\Delta x$) = S_{mean}/τ_{mean} (where **L** is the length of the slip surface)

This is of course only true as long as the *distribution* of *in situ* stresses and *additional* triggering stresses are *homogeneous* or at least *roughly proportional* in shape along the slide.

If this condition is *not fulfilled,* as is the case with locally concentrated additional loads, the PLE analysis has *little relevance*.

1) The Surte Landslide (1950)
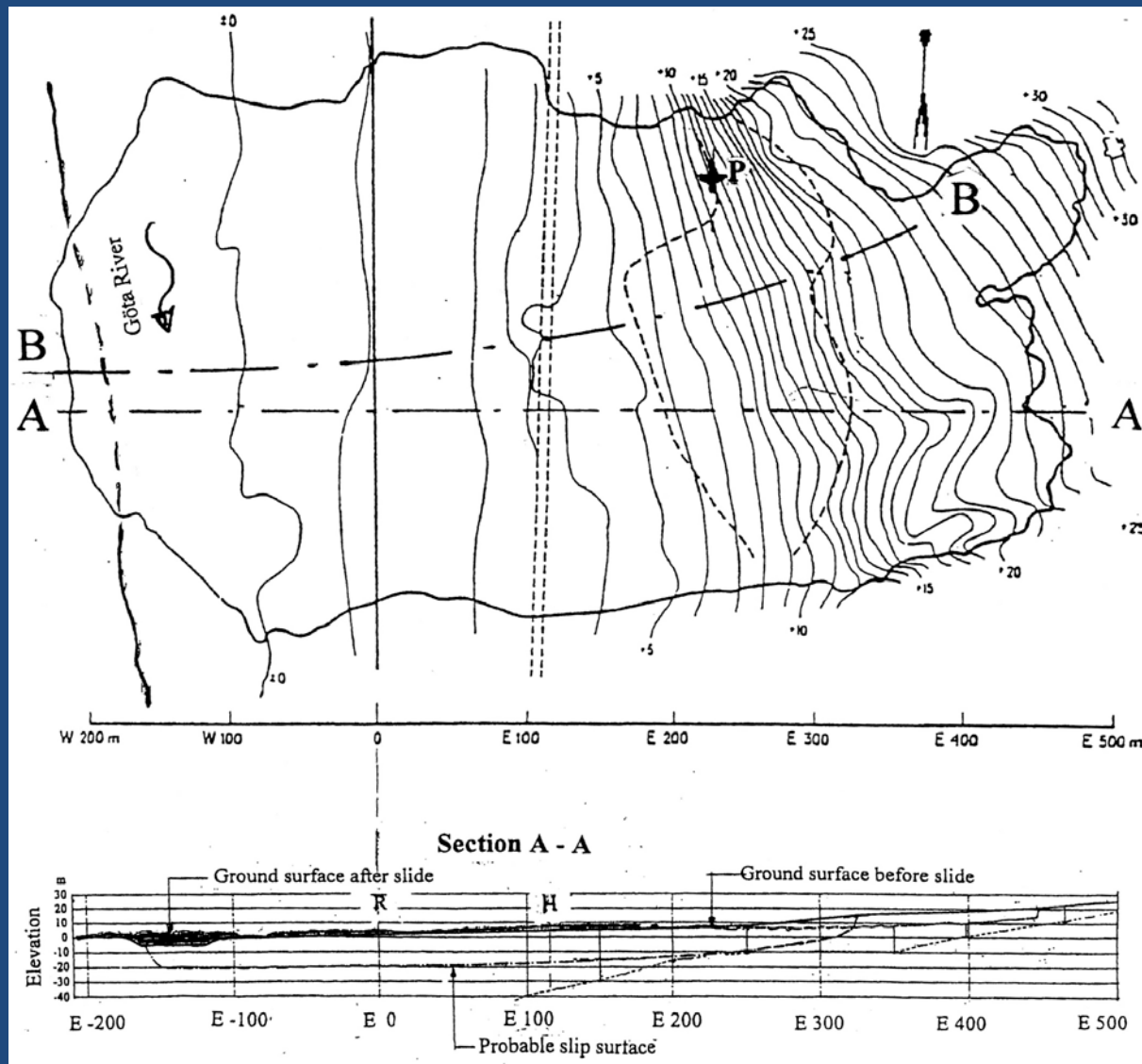
The landslide was triggered by the driving of a few soil displacing concrete piles for a family house in the point marked with +P.
The downhill progressive slide was followed by *serial retrogressive* slides *(so called earth flows).*

The Surte slide brought about a tremendous loss to housing, traffic, transport and shipping. About 40 buildings of different character were destroyed.

As the slide took place at 8 a.m., when many people had already left for work, the death toll was only *one* person. As the houses had been built in the old style, they mostly behaved as *boxes* that did not crumble.

Results of Finite Difference Progressive Failure Analysis (FDM)
Phase 4

The striated area in the top figure indicates the spread caused by *passive earth pressure* resistance being exceeded, and constitutes a measure of the predicted calculated extent of the landslide
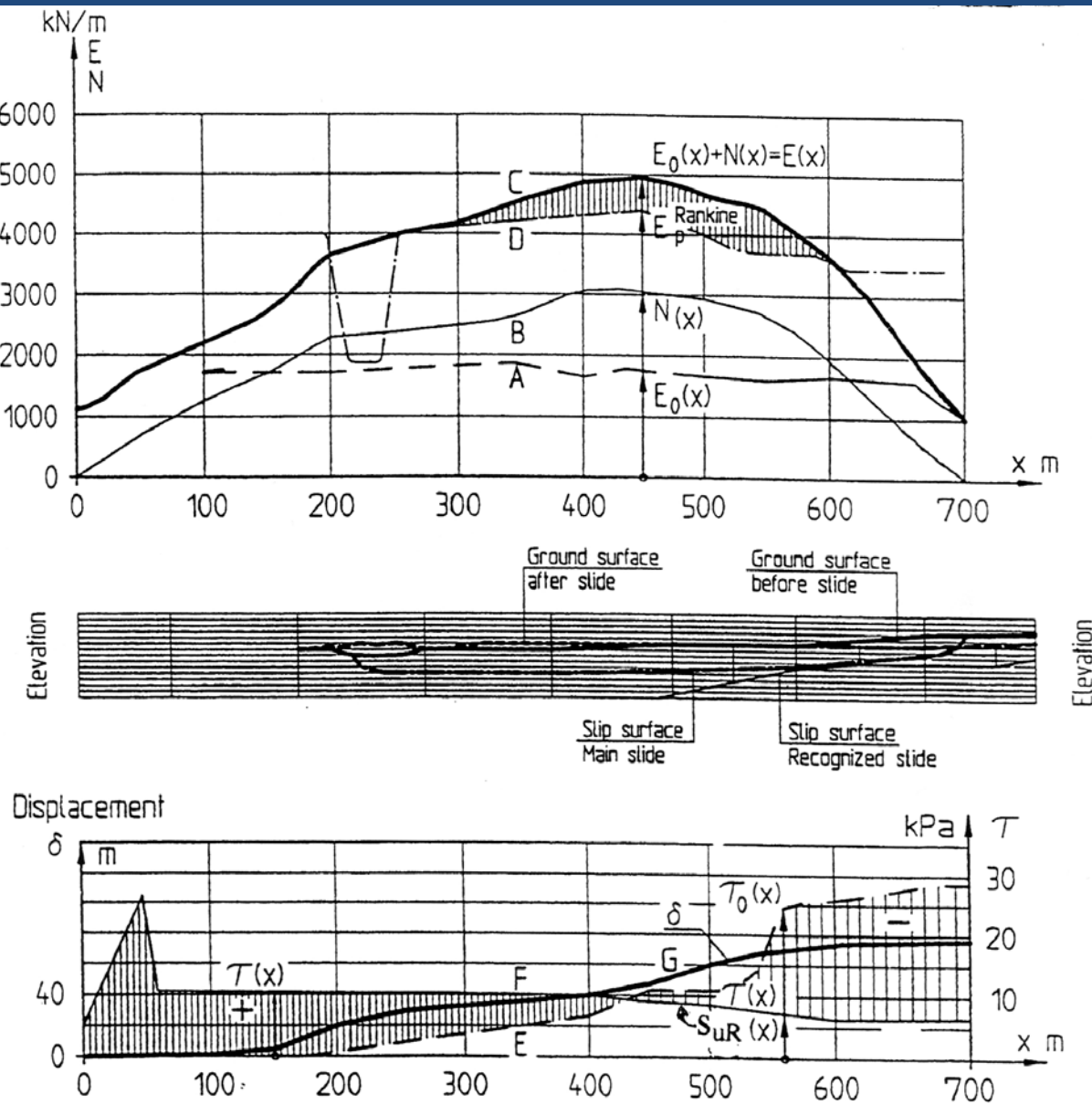
2) The Tuve Landslide (November 1977)

The Tuve landslide was believed by the Swedish Geotechnical Institute to having been triggered by a *combined effect* of
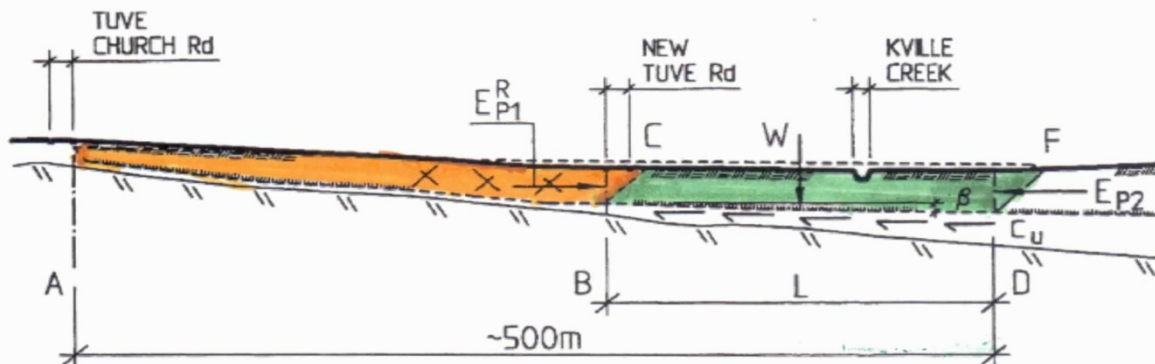**a)** hydrological change due to a recent housing project resulting in higher permeability in surface ground and
b) of a widened road embankment 5 years before the slide event.
Tuve is a community near Gothenburg. The slide took place late in the afternoon of
on November the 30th 1977, just after four o'clock– i.e. at an hour that must have reduced the death toll significantly since people had not yet returned from work or from school.
In all, the slide resulted in nine deaths, the total destruction of *65 family houses* and a drastic change of the topography of some *270 000 m^2* of ground. Settlements in the active zone of about *10 m* and horizontal displacements up to *200 m* were recorded. Upheaval in the passive zone of about 5 m over a distance of about 300 m was noted.

Landslide spread - *Section through the main slide in Tuve*. (Distorted vertical scale).

TUVE
CHURCH Rd

E_{P1}^R

NEW
TUVE Rd

KVILLE
CREEK

A

B

L

D

~500m

The downhill progressive slide was followed by *serial retrogressive* slides *(flows,)* as is usually the case.

Forces required to provoke plastic failure over the valley floor according to I-PlF analysis.
$c_{u(e)} = 0.12 \cdot \sigma_c'$ (Empirical shear strength as assumed in SGI Report No 18)
$E_{p1} << R_{BDF} = \int_0^L c_{ux} \cdot b_x \cdot dx - w \cdot \sin\beta + E_{p2}$ Vert. scale = 2 x hor. scale
E.g. for $\beta = 0$, $b_1 = 400$ m, $b_2 = 600$ m and L = 270 m
For H = 35 m, $E^{Rankine} \approx 4520$ MN $<< R_{BDF} \approx 10655$ MN   **$F_s = 10655/4520 \approx 2.4$**

The FDM-approach *predicts that this condition* already as a *static* condition − i.e. without considering dynamic inertia forces.

A = 16 hectares
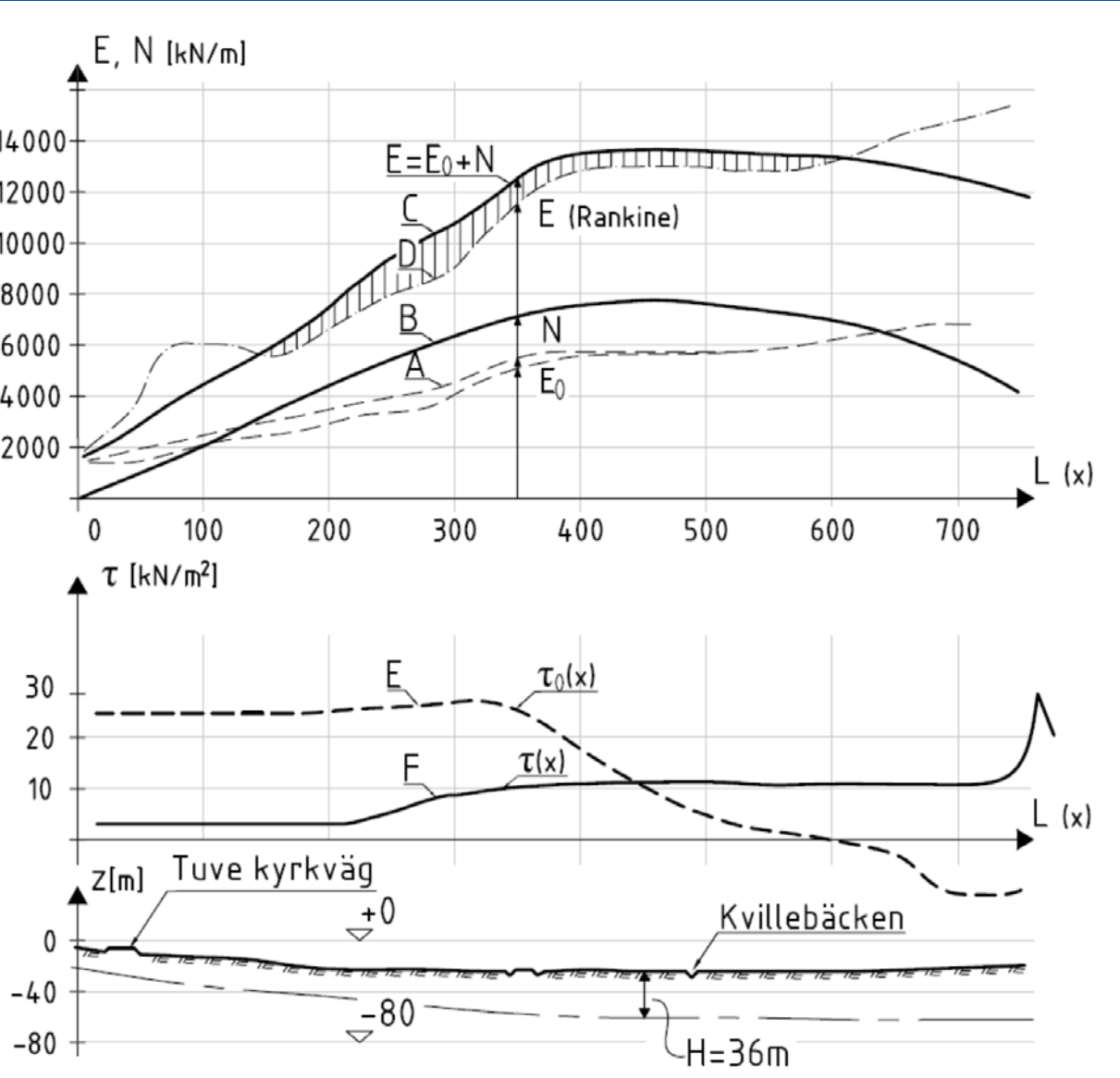$V \approx 5 \cdot 10^6 m^3$

H ≈ 35m

ΔH ≈ 5m

D = 35 m
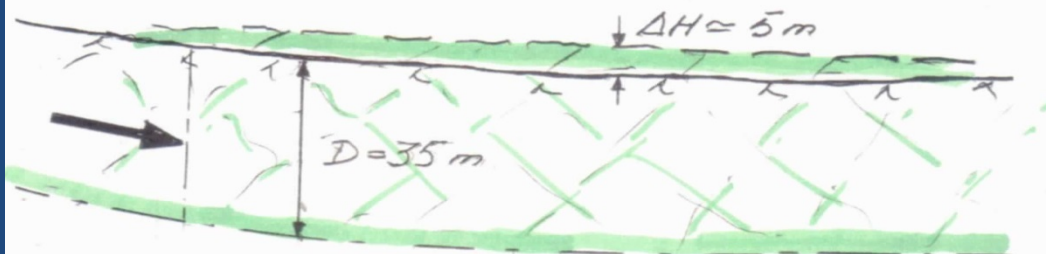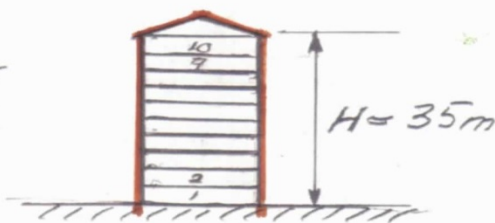
Plasticization of about 16 hectares of level ground down to ca 35 m:s depth. Explainable on the basis of *static conditions*.

The Tuve Landslide could be *explained* using the FDM Progressive Failure Analytical Model.

The calculations revealed that a surprisingly *insignificant additional load* could trigger the landslide.

On the other hand, applying the Plastic Limit Equilibrium approach gave false safety factor slightly *in excess of 2,0* and a predicted landslide configuration very *different* from that of the real landslide.

Yellow colour indicates earth pressure raise in the triggering phase,
Deepening green colour indicates steadily increasing earth pressure
raise.
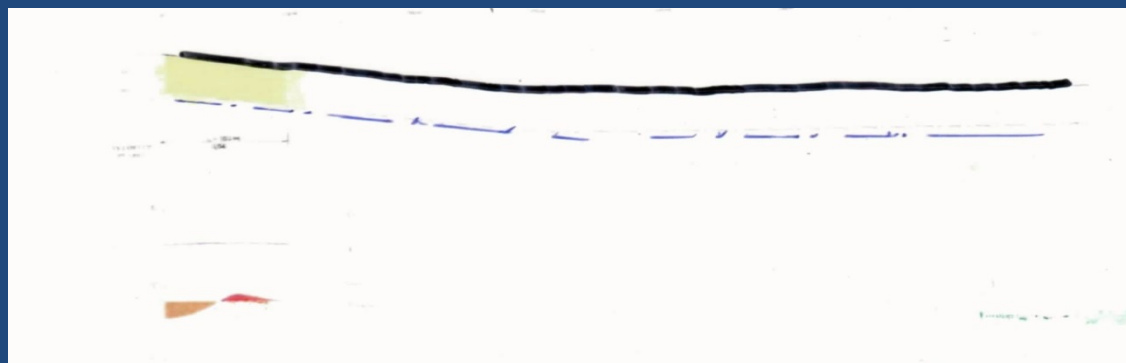
Figure 5:1.9 Progressive failure released by the additional force $N_i = N_{cr}$, dynamic phase I
Time t = 1.3 sec. Deepening colors or shades indicate growth of down slope earth pressures.

|			
Acceleration	= 0.02 m/sec^2	Max. velocity	≈ 0 m/sec
Kinetic energy	≈ 0 kNm	Total length, L	= 100 m
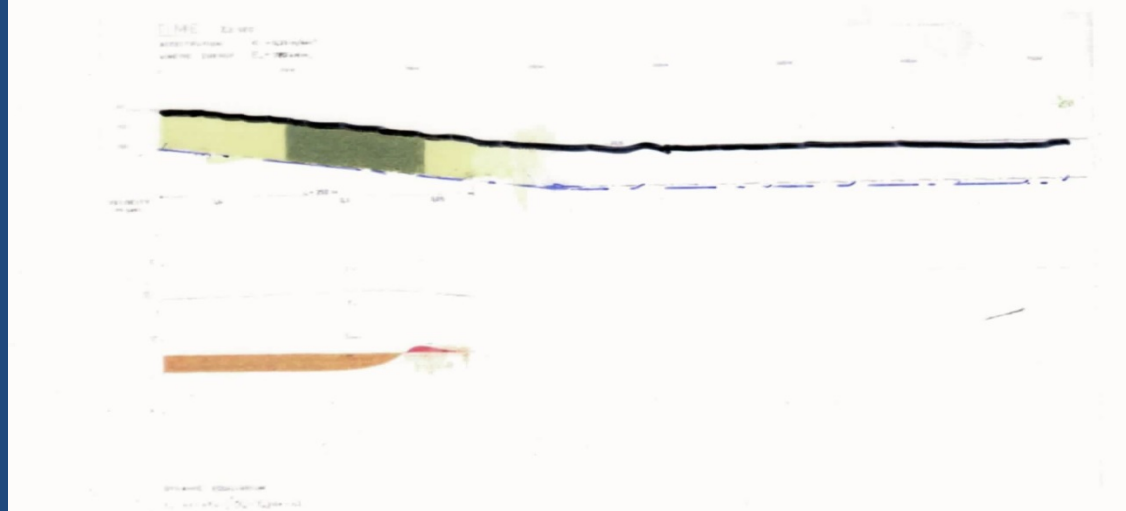
Figure 5:1.10 Progressive failure propagation down-slope, dynamic phase I.
Time t = 3.2 sec
Deepening colors or shades indicate growth of down slope earth pressures.

Acceleration	= 0.20 m/sec^2	Max. velocity	= 1.3 m/sec
Kinetic energy	= 780 kNm	Total length, L	= 250 m

Black colour indicates passive earth pressure being exceeded.
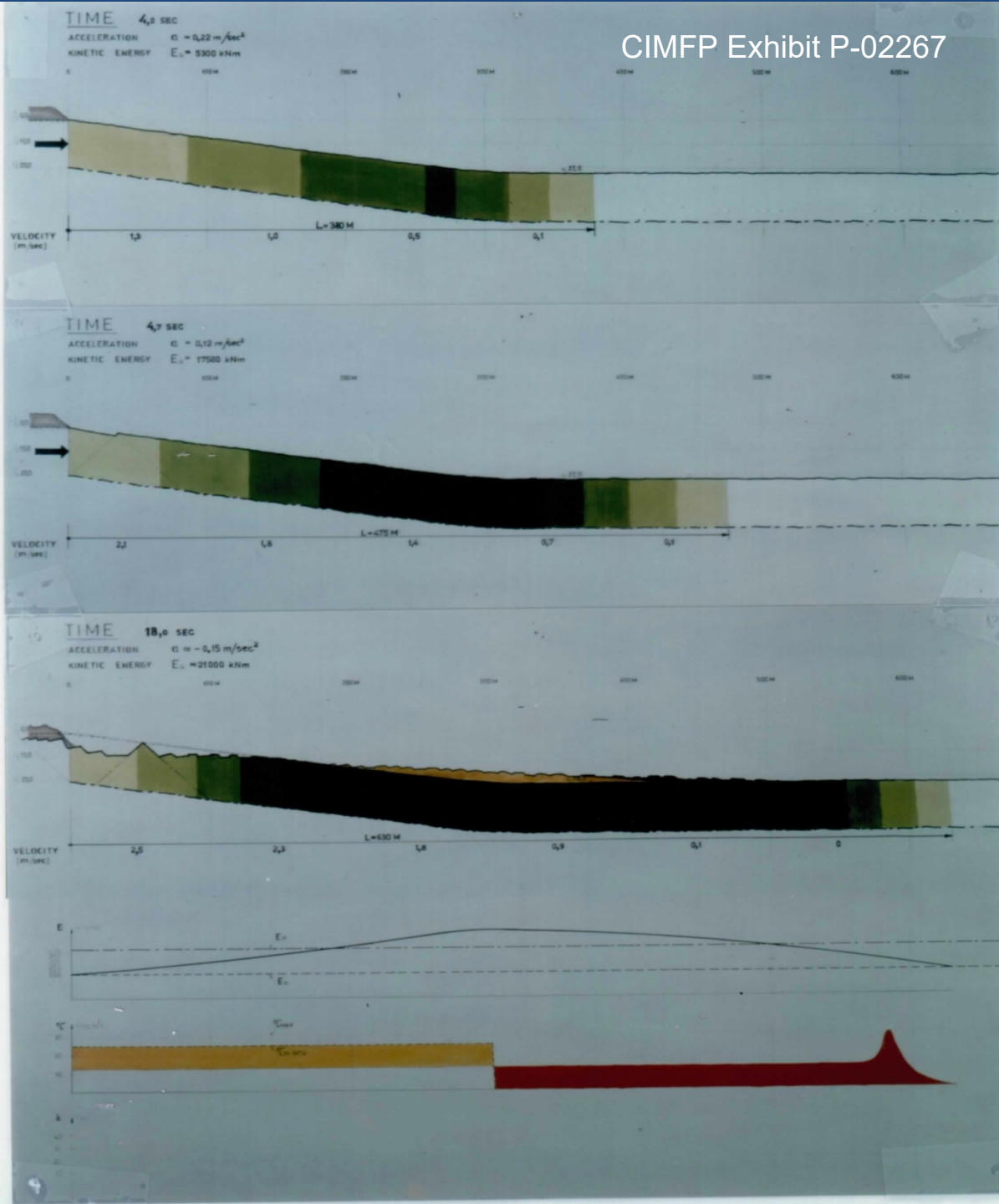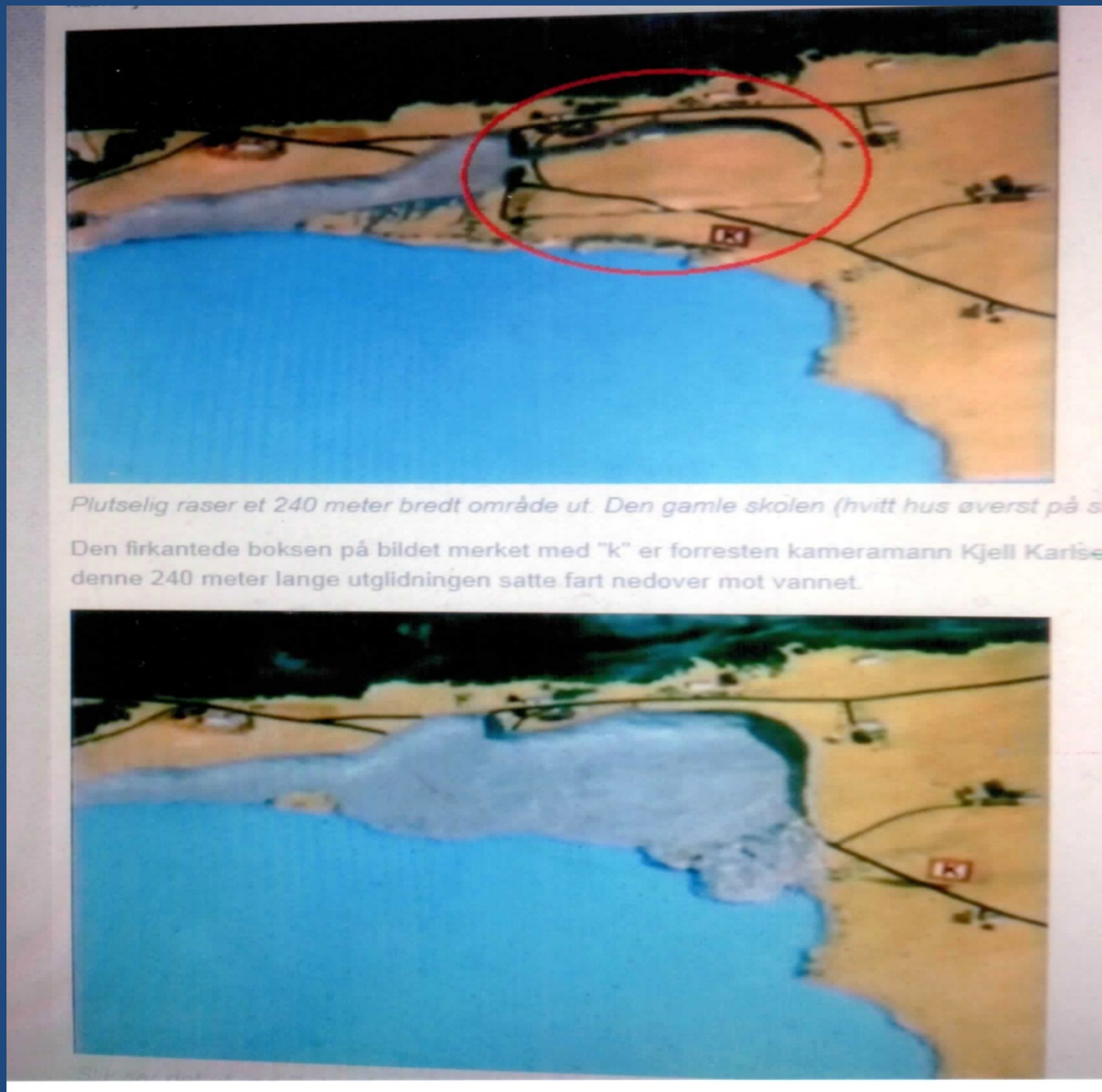
3. The Rissa Landslide, Norway (1978)

The Rissa landslide is a excellent example of the complicated development of slides in 'Quick clays'. (Slide 1)

Rissa (Slide 2) Downhill progressive slide developing
The slide was initially triggered close to the farm house in the
left corner of the figure.

Plutselig raser et 240 meter bredt område ut. Den gamle skolen (hvitt hus øverst på sk

Den firkantede boksen på bildet merket med "k" er forresten kameramann Kjell Karlse
denne 240 meter lange utglidningen satte fart nedover mot vannet.

Rissa **(**Slide 3)

Conclusion:
Like most landslides in quick clays, the Rissa slide features a development comprising both flow slides and downhill progressive slides.

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