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Geological setting and mechanism of Audru landslide

M.Sc. Thesis

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Introduction

Estonian territory, lying at the southern slope of the Fennoscandian Shield, is characterized by tectonic stability and flat topography. In such conditions landslide hazard is usually a rare occurrence. Nevertheless, in densely populated area of Pärnu County in western Estonia, a number of small scale landslides have occurred during the last few years (Kalm *et al*., 2002). In order to explore the landslide preconditions and mechanisms and to map and evaluate the landslide hazard in detail, the lower reaches of the Pärnu, Reiu, Sauga and Audru Rivers were studied (Fig. 1). According to this investigation, three different types of slides exist in the area. The largest and most complex slides investigated took place at glaciolacustrine varved clay slopes covered by less than 3 m of marine sand. Mechanism of such slides remained mostly unexplained (Kalm *et al*., 2002).

Current study is focused on the landslides related to varved clays in order to find out the reasons for their occurrence. One landslide (Audru 1) was chosen for detailed study, mainly because of good accessibility and low slope angle that allowed the use of motorized drilling equipment. Precise mapping of the site was done and 8 boreholes were drilled to get a more detailed picture of local geological setting and to localize slide surface. Varv correlation was carried out to reconstruct events during the slide inside the main sliding body. Two samples were taken to investigate geotechnical properties of the soil. This information was used to generate a computer model, that allowed in depth study of different parameters affecting slope stability. The author participated in all fieldworks, made core descriptions, varv correlation and also carried out computer modeling. Laboratory analysis were made at the geotechnical laboratory of the Estonian Environmental Research Centre and at the Institute of Geology, Tartu University.

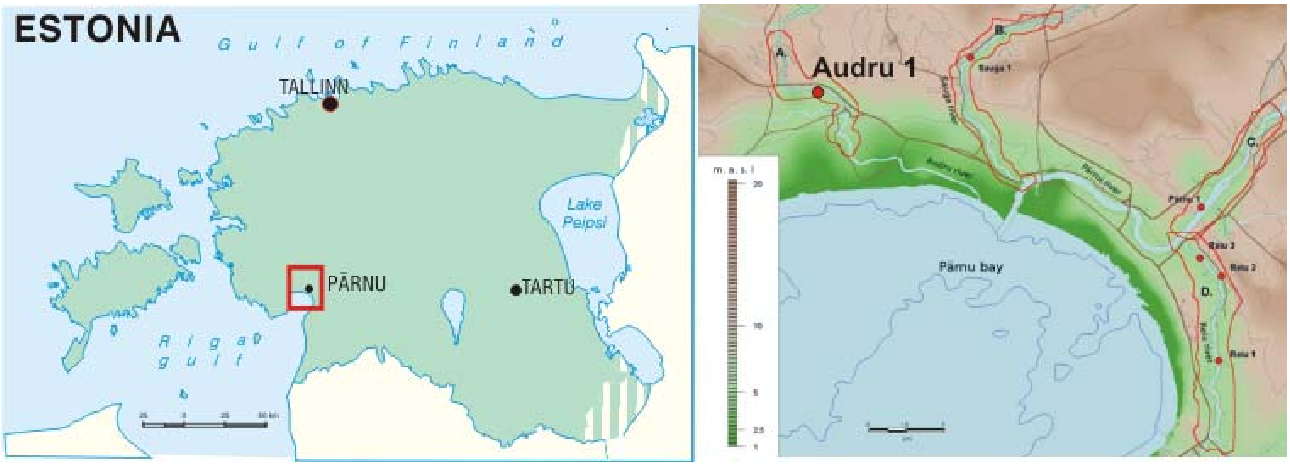


Figure 1. Location map of the studied area. Audru 1 is the currently investigated landslide, red lines mark investigated areas during 2002. (Kalm *et al.,*2002).

1. Geological background

The studied river valleys are eroded into a former marine plain with flat, slightly undulating topography where the altitudes vary between 2...15 m. The steepest slopes (25...27º) and the strongest erosion occur at the bends of river meanders. Outside the valleys the slope angels less than 10^0 are prevailing.

The surface of the Devonian sandstones near Pärnu town lies at an altitude of -10 ...-15 m (Tavast & Raukas, 1982). It is covered by bluish-grey loamy till of the Late-Weichselian age. The latter is followed by up to 20 m thick glaciolacustrine varved clay and silt. The thickness of Holocene marine sands and silts, covering these deposits, reaches in its maximum up to 10 m, being mostly 2...3 m and is only locally absent. Due to alternating transgressions and regressions of the Baltic Sea in the Holocene, buried organic layers from a couple of cm up to 1 m in thickness are common within the marine deposits (Veski *et al.*, 2005 in press). The geotechnical properties of the most common deposits near Pärnu town are presented in Table 1.

Sediment	**Cohesion** *c'* (kPa)	**Internal friction** φ' (º)	**Unit weight** γ (kN/m^3)	**Hydraulic conductivity** K (m/d)
Till	25-100	35-45	20-22	0,003-0,5
Glaciolacustrine clay	0-25	25-35	15-18,5	0,003
Sand (marine)	0-10	30-40	18-20	1-10
Silt (marine)	0-25	25-35	18-21	0,05-0,5

Table 1. Geotechnical properties of the most common deposits in the study area (Kalm *et al.,* 2002).

The geotechnically weakest soil type in the area is rhythmically deposited glaciolacustrine varved clay with alternating clayey winter and silty summer layers. Grain-size distribution is investigated by Reinson (2005), grain-size distribution within a single varve is displayed on Fig. 2 (Reinson, 2005).

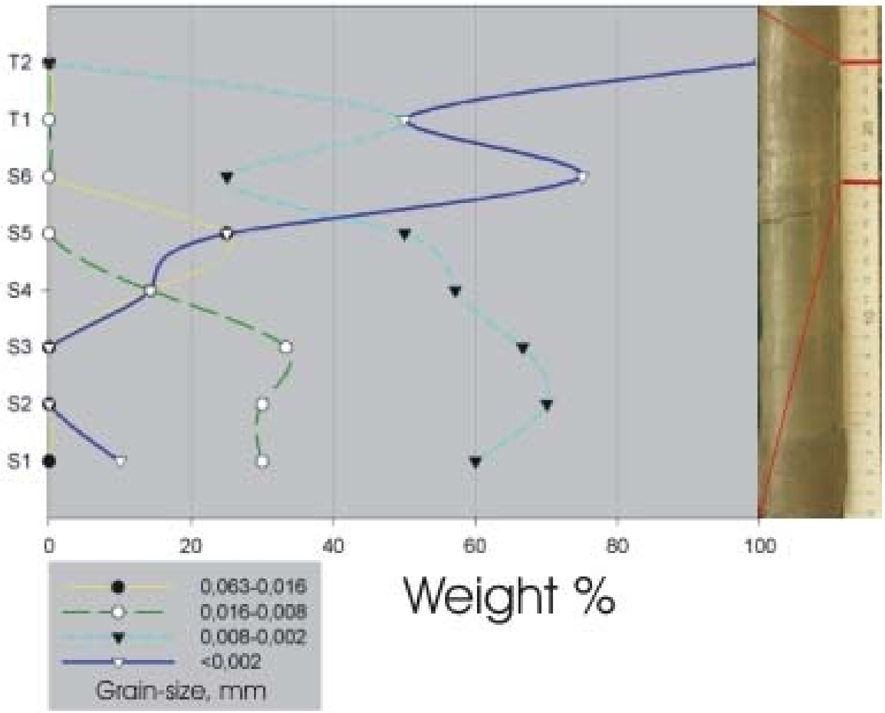


Figure 2. Grain-size distribution within a single varve in the Audru-1 borehole, S1…T2 are sample numbers, S corresponds to summer layer and T to the winter layer (Reinson, 2005).

Varve thickness increases evenly towards depth, the upper ones are around 1 mm thick and the lowest ones around 20…30 cm.

The mineral composition of clay fraction is hydromicas (70...80%), montmorillonite hydromicas (9-19%), kaolinite (6...8%) and chlorite (2...5%), silty summer layers consist of quartz (46...59%), feldspars (14...18%), carbonate (11...21%) and minor amount of other minerals (Kattel, 1989). Clay fraction is dominant, particularly in the upper portion of clays where it constitutes 69...79% (Reinson, 2005). The thickness of silty summer layers increases downwards and the proportion of clay fraction decreases down to 46...51% (Reinson, 2005).

The geotechnical parameters also change together with depth, conversion is even and without sudden changes. Upper layers have average plasticity index 31% and liquid limit 63%, lowest layers 14% and 38% (Kattel, 1989). According to Casagrande plasticity chart upper layer is CL (Fat clay) and lower CH (lean clay) (Coduto, 1998). Water content also decreases towards the depth being in the upper portion of clays 70...90% and decreasing downwards to 30...60% (Kattel, 1989).

Glaciolacustrine clays in Pärnu area are formed in the same proglacial sedimentary basin so their qualities in horizontal scale do not change notably (Kattel, 1989).

Pore water chemistry shows influence of the Litorina stadium of the Baltic Sea, when saltwater with salinity 9…11 ‰ covered former sediments. Influence of the saltwater can be traced 5 meters inside the varved clays (Mets, 2005). High concentrations of sulphate ions (230 mg/l) were found from the lower samples. This is up to ten times more than average 20 mg/l (Karise, 1967) and usually such high concentrations of sulphate is caused by pollution (Karise & Lemberg, 1976).

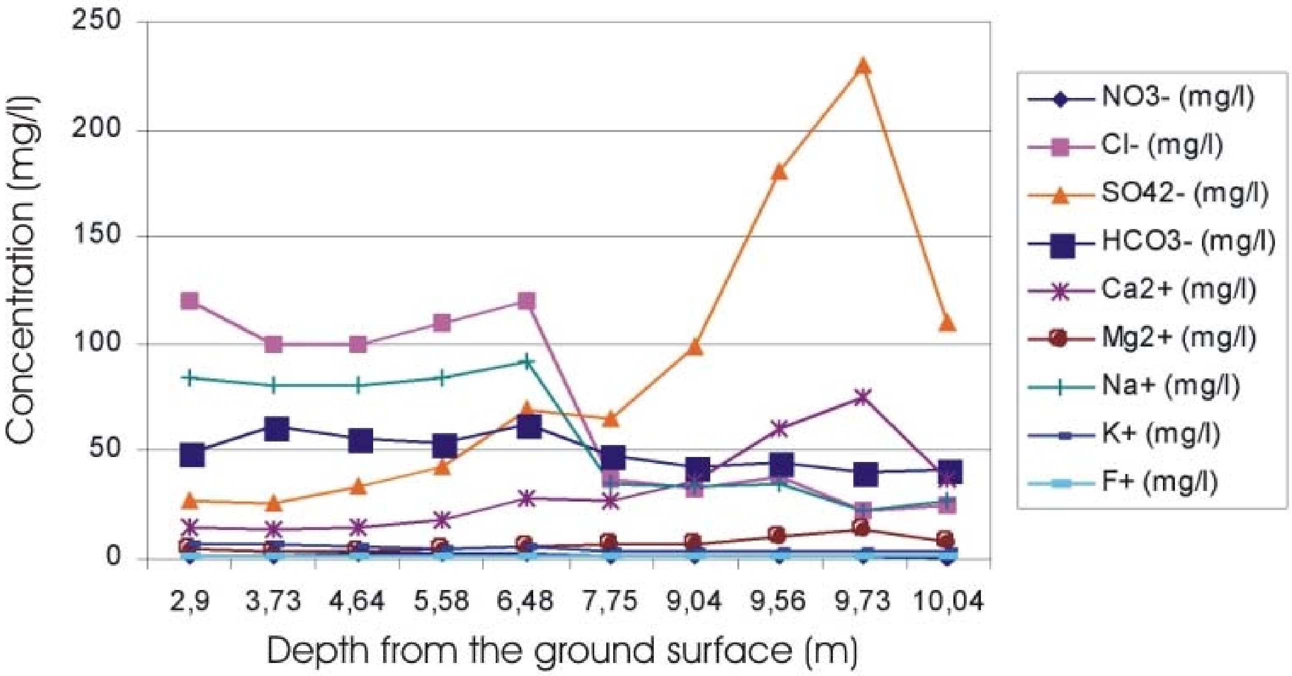


Figure 3. Chemistry of the pore water from the sediment sequence Bh-2 (Mets, 2005).

Two groundwater layers in the Pärnu area are known inside the Quaternary sediments. Upper layer, unconfined, is bound to marine sands and lower one to till. Varved clay acts as a aquiclude between them. Upper groundwater layer is controlled by precipitation, transpiration, evaporation and by the water level of adjoining rivers. Water level in the Audru River seems to be a good indicator reflecting the total influence of aforementioned parameters. Unfortunately there are no measurements about the water table in the Audru River. Closest hydrological station is in the Pärnu town and it measures water level in the Pärnu Bay. Rapid changes of the water level in the Pärnu Bay occur in November to December and March to May (Fig. 4), difference may be up to 1,5 m (Kalm *et al*., 2002). Long term, from 1923 to 1970, water level minimum is - 0,94 m and maximum 2,53 m (Kaljund & Mets, 1976). However, water level in the Audru River is screened from the Pärnu Bay by the water mill dam situated in the Audru village. According to author's experience, the water level in the Audru River (after dam) is probably fluctuating within 1 meter range.

The lower groundwater layer is related to till and is pressurised. Piezometric level of this layer is controlled mainly by infiltration rate at the feeding area. In current case the feeding area is probably north from the Pärnu town where till is outcropping. IPT Projektijuhtimine monitors this aquifer in the Pärnu town and the median values of the data gathered from 14 boreholes over 12 years are presented in Fig. 5.

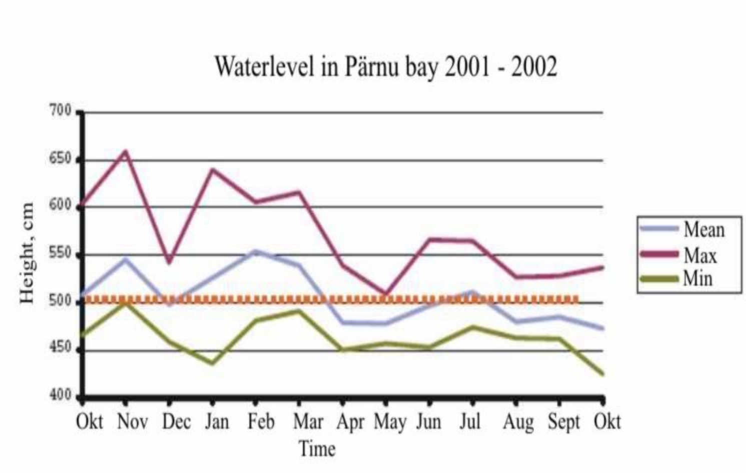


Figure 4. Waterlevel changes in the Pärnu Bay, orange line is a fixed zero (Kalm *et al*, 2002).

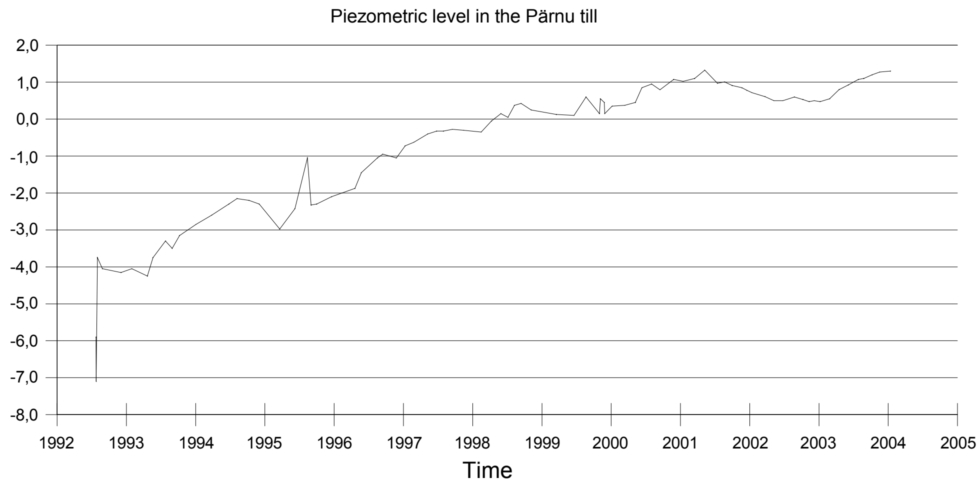


Figure 5. Piezometric levels of the groundwater related to the till in the Pärnu town (Talviste, 2004).

One can see that the piezometric level of this aquifer has been around 0...1,5 m a.s.l. over last 5 years. Although the currently investigated Audru landslide is 6 km away from the Pärnu it's relatively reasonable to assume that the groundwater levels are similar as Audru landslide and Pärnu town belong to the same geotechnical region (Vilo, 1986) and former sedimentary basin (Kattel, 1989).

2. Earlier investigations

First records about landslides in Estonia go back to Middle Ages when part of the Lihula fortification (western Estonia) slid into the river (http://www.sloleht.ee/... (20.05.05)). Earliest modern records date from 1927 when a landslide occurred inside Pärnu (Kaljund, 1967). Almost all landslides are related to varved glaciolacustrine clays or Ordovician and Silurian klints (klint - a local name to up to 56 m high cliffs or escarpments in Ordovician and Silurian sedimentary rocks) (Vilo, 1986). Landslides that occur at klints are usually rockfalls or rock slides, rotational slips are related to Cambrian blue clay (Miidel & Raukas, 2004). Since those slides evolve inside rocks not in the soft sediments, sliding mechanism of which is somewhat different they are out of scope of current study.

Glaciolacustrine varved clays cover a wide area in western Estonia and this is also the area where landslides are most abundant (Vilo, 1986). Due to flat topography the slides occur mainly at the banks of the river valleys. The largest slide ever recorded in Estonia took place near Pärnu town, at Sauga river valley and measured 200x100 m (Vilo, 1986). Numerous landslides took place inside the Pärnu town at 1960-ties, for example in 1960 close to Fishfactory and in 1966 close to the factory “Viisnurk” (Vilo, 1986). The landslide that occurred near “Viisnurk” was intensively investigated by geologists who drilled 60 boreholes, took 106 geotechnical samples and also modelled the landslide (Kaljund, 1967). They found that soil strength properties necessary for a slide to occur were unusually low compared to average strength properties known for varved clay in the area (Kaljund, 1967).

Olli and Martin (1961) mentioned one section of the Sauga River valley, neighbouring the Nurme village, where a number of terraces point to earlier landslides. They suspected a slow creep of the clay from planes towards the valley to be the trigger of slides, because otherwise these low angle slopes would have been stable as the present erosion is almost negligible. They also pointed out similarities between those slides and Norwegian quickclay slides (Olli & Martin, 1961).

A number of landslides occurred in Pärnu area during spring 2002 (Kalm *et al.,* 2002). To investigate the phenomenon and to find out most endangered areas, the Council of Pärnu County asked the Institute of Geology Tartu University for landslide hazard zonation. The author of the current study participated in this study and as it initiated the current master thesis, it will be shortly described below. Four areas were investigated, namely the lower reaches of Pärnu, Reiu, Audru and Sauga rivers (Fig. 1). Morphology and geological settings of recent landslides were studied in detail. Geology was studied through the description of outcrops and through the corings, also earlier geological and geotechnical information was used. Table 2 displays the properties of 8 different slides that were examined and mapped (Kalm *et al.,* 2002).

Slide	Coordinates	Width (m)*	Length (m)**	Height of scarp (m)	Time of occurrence
Audru-1	E:24°20,09\` N:58°25,26\`	75	36	1,2	Feb.2002
Audru-2	E:24°19,89\` N:58°25,28\`	8	4	0,3	Spring 2002?
Audru-3	E:24°19,89\` N:58°25,28\`	16	4	0,4	Spring 2002?
Sauga-1	E:24°26,41\` N:58°25,72\`	13	13	1,4	Spring 2002
Pärnu-1	E:24°36,29\` N:58°22,70\`	80	42	5,4	April 2002
Reiu-1	E:24°36,21\` N:58°21,60\`	8	15	1,2	2000?
Reiu-2	E:24°37,09\` N:58°21,21\`	23	16	1,5	Feb. 2002
Reiu-3	E:24°36.93\` N:58°19,39\`	22	10	2,5	2000

*measured along the river channel
** measured perpendicularly to river channel

Table 2. Location and main morphological characteristics of the investigated landslides. Question mark where the exact time of occurrence is unknown (Kalm *et al*., 2002). Location of landslides see Fig.1.

The evaluation of the slope stability within the study under discussion was provided through the modelling of described landslides with Janbu corrected method, which is one of the limited equilibrium methods, a group of methods that is most often used for landslide modelling (Sjöberg, 1996). Hydrogeological conditions were included to the model as the groundwater level in the slope (1 meter below the surface). Additional shear stress generated by the groundwater flow inside the slope was calculated using finite element method. Varying soil strength parameters and groundwater level within known limits critical slip surfaces were found that visually matched ones observed at field. Using aforementioned parameters two critical slope angels were calculated for different soil types: one for natural conditions and second with human activity involved (Kalm *et al.,* 2002).

In order to map the landslide hazard in the area, the elevation data from topographic maps in scale 1:10 000 (incl. isobasis with 1 m intervals and elevation points) were used to create the digital elevation model (DEM) for the investigated valley sections. The slope angles were derived from obtained DEM using the terrain slope operator. Slopes were classified/mapped on the basis of the aforementioned "critical slope angle criteria" and soil type. As a result of the classification the critical slopes for each soil type were identified. DEM was then evolved to the distribution map of the classes of slope angles derived from the modelling and combined with the geological settings. Thus, the grid cells, where the slopes are steeper and possibly unstable, compared to the critical values for current soil type, were identified (Kalm *et al.,* 2002).

Examined landslides were divided into three groups according to geological settings and failure mechanism involved. Smallest ones occurred directly at the banks of river channel and were caused by river erosion. Medium sized slides occurred in the fine sand above the water level in the river and failures occurred due to additional shear stress generated by groundwater flow. Largest investigated slides took place at glaciolacustrine varved clay slopes and the causes remained largely unknown (Kalm *et al.,* 2002).

Pille Sedman from IPT Projektijuhtimine first modelled Audru landslide in 2002. Janbu corrected method was used for slope stability analysis and a hypothesis about multi-stage landslide, were smaller landslide near to the river channel took place before the main slide, was proposed (Sedman, 2002).

Outside Pärnu County, landslides related to glaciolacustrine clays in Estonia, have occurred at lower reaches of Vääna and Keila Rivers (northern Estonia) and at the basin of the Kassari River in western Estonia (Vilo, 1986). Beside natural slope processes landslides occur also at man made slopes, mostly at road dams and fills. For example a road dam has failed near Obinitsa (SE Estonia) and close to Kärevere bridge at Tartu-Tallinn highway (Uri, personal communication). Fills have failed close to Maardu Pakterminal (Nelke *et al.*, 1997).

3. Methods

The morphology of the Audru landslide was mapped with tachymeter (Dahlta 010B) and channel morphology of the river with measuring stick along the cross profiles. Thickness of the sand covering varved clay was measured with handoperated auger. Six boreholes were cored using motorized drilling equipment (GEOTECH 504) combined with the Russian type peat corer with the sampler 1 m in length and 10 cm in diameter. Two sediment sequences were obtained with handoperated Russian type peat corer (1 m in length and 7 cm in diameter) (Fig. 6).

The Russian type peat corer is a side filling chamber-type sampler operated by metal rods. It has been designed by Byelorussian geologists and was first presented to the international audience by Jowsey (1966). This discrete point sampler enables one to drive the sampler to any interval in the sediment profile in the closed (empty) position. Once the target depth is reached, the handle is turned to initiate the sampling while the pivotal cover plate supports the cutting action of the corer. As the sampler is turned 180 degrees, the sharpened edge of the bore cuts a semi-cylindrical shaped sample until the opposite side of the cover plate is contacted. The contained sample can now be recovered without risk of contamination by overlying sediments. The sample is extruded from the bore by a rotation where sample rests on cover plate. This design gives relatively undisturbed and uncompressed samples that are needed for varve correlation. Whole vertical section of varved clays was sampled. The 1 m long cores were wrapped in plastic film to avoid drying and placed in half-cut PVC tubes for transport.

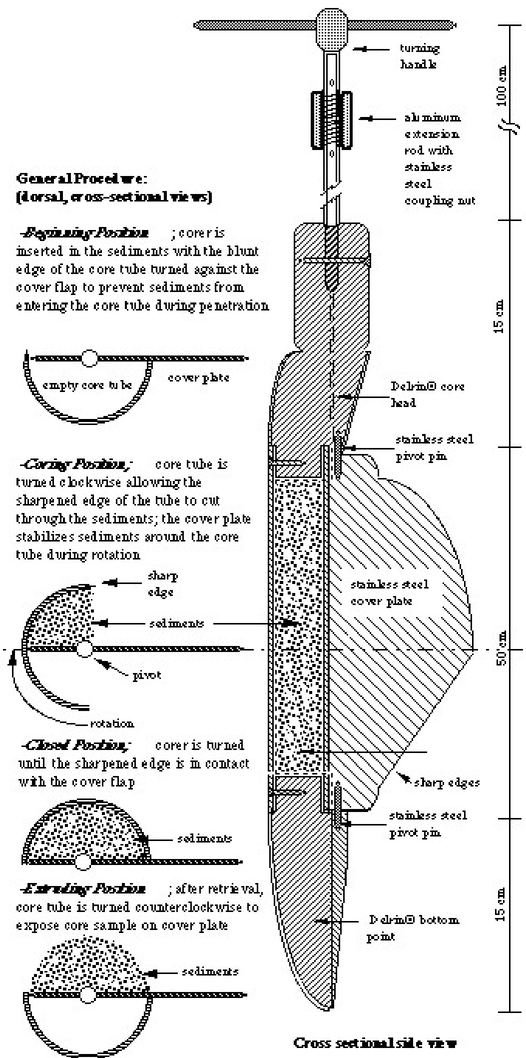


Figure 6. Russian peat corer (http://www.aquaticresearch.com/..., 20.05.05).

Geotechnical samples, recovered with Russian type peat borer, were hermetically sealed into the metal container and transported to geotechnical laboratory of the Estonian Environmental Research Centre for further analysis.

Recovered samples were cleaned three times during three days, after each cleaning destilled water was sprayed over the samples to allow differences between silt and clay drying rate „work out“ so that individual varves could be more easily distinguished. A paper strip was then attached to the surface and the thickness of each summer and winter layer was noted. The individual varve thickness obtained in this way was then measured and computer drawn graphs were created. The second step in the lab was to correlate the sequences. Intervals of homogeneous clay or diffuse varves with an unknown number of varves were left as gaps in the varve graphs. In order to deal with this difficulty – one which complicates arriving at a correlation, the sequences were placed side-by-side on a table and the correlation was made directly from the sediments. Such a comparison provides a varve correlation that is more reliable than that obtained from the comparison of varve graphs.

Geotechnical samples were analysed by Geotechnical Laboratory of Estonian Environmental Research Centre where natural water content, Atterberg limits, grain-size distribution and $CaCO_3$ content were measured using methodology described in Estonian Standards ETC-C1.97, ETC5-C5.97, ETC5-C4.97. Liquid limit for the clays was measured with Swedish cone and also with Vassiljev cone.

Measuring and coring data from fieldwork was used to generate 2D views of the landslide using AutoCad Land Desktop software.

Soil strength parameters for the varved clay, effective cohesion (c’) and effective internal friction (φ’) were calculated by Peeter Talviste from IPT Projektijuhtimine using critical soil state theory. An outline of this theory has earlier been described by P. Talviste (2002).

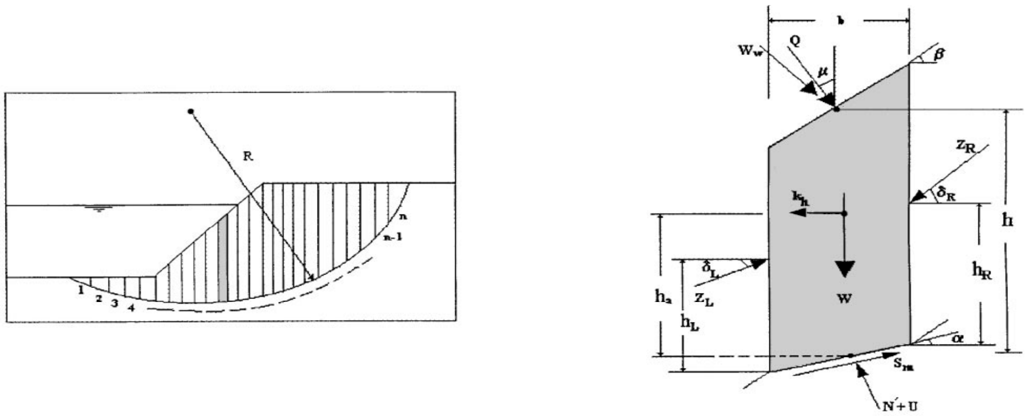
Two different models were used for landslide modelling. One was deterministic model, from limit equilibrium methods group (Sjöberg, 1996). Limit equilibrium methods (LEM) are most often used for slope stability calculations because of relatively simple principles and proven reliability (Coduto, 1998). This method is using concepts from the classical theory of plasticity. Limit analysis is concerned to determine the collapse load for a structure. The collapse load is defined here as a load which causes extensive plastic failure of the slope resulting in displacements, which increase without limit while load is held constant (Sjöberg, 1996).

LEM evaluates the slope as if it were about to fail and determine the resulting shear stresses along failure surface. Then, these stresses are compared to shear strength to determine the factor of safety. Factor of safety varies along slip surface where some sections may have failed, while others remain still stable. Limit equilibrium analysis do not attempt to define this distribution, they only give overall value of factor of safety, which is defined as:

$$F = \frac{\int s\, dl}{\int \tau\, dl}$$

Where **s** is shear strength, **τ** shear stress and **l** length along the shear surface, and both integrals are evaluated along their entire length (Coduto, 1998).

Most LEM's divide the failure mass into number of vertical slices, as shown in Figure 7. These slices are chosen such that the bottom of each one passes through only one type of material, and so that the bottom of each slice is small enough to be considered as a straight line (Coduto, 1998).



F = factor of safety.
S_m = mobilized shear strength.

$$S_m = \frac{c'.b + N' \tan\phi}{F}$$

U = pore water pressure.
W = weight of slice.
W_w = surface water force.
Q = external surcharge.
N' = effective normal force
K_h = horizontal seismic coefficient.
μ = Angle of inclination of external load.
Z_L = left inter-slice force.
Z_R = right inter-slice force.
δ_L = left inter-slice force inclination angle.
δ_R = right inter-slice force inclination angle.
H_L = height of force Z_L.
H_R = height of force Z_R.
α = Inclination of slice base.
β = Inclination of slice top.
b = width of the slice.
h = average height of the slice.
h_a = height to the center of the slice.

Figure 7. Principle of slices used in LEM's and forces acting on each slice (Malkawi, 2000).

A large number of methods have been developed using principles of LEM. They differ in how well conditions of equilibrium are satisfied and how interslice forces are included into solution. They can be divided into simple, complex and rigorous methods. For simple methods, the effects of interslice forces are neglected, whereas in complex methods, the interslice methods are included in the formulation. Methods where all conditions of static equilibrium are satisfied are named rigorous methods (Sjöberg, 1996).

Various authors have demonstrated that choice of the method isn't very important, especially in case of rigorous methods. Variance of calculated factor of safety (Fig. 8) is relatively small when rigorous methods are used (Sjöberg, 1996; Zhu *et al*, 2003).

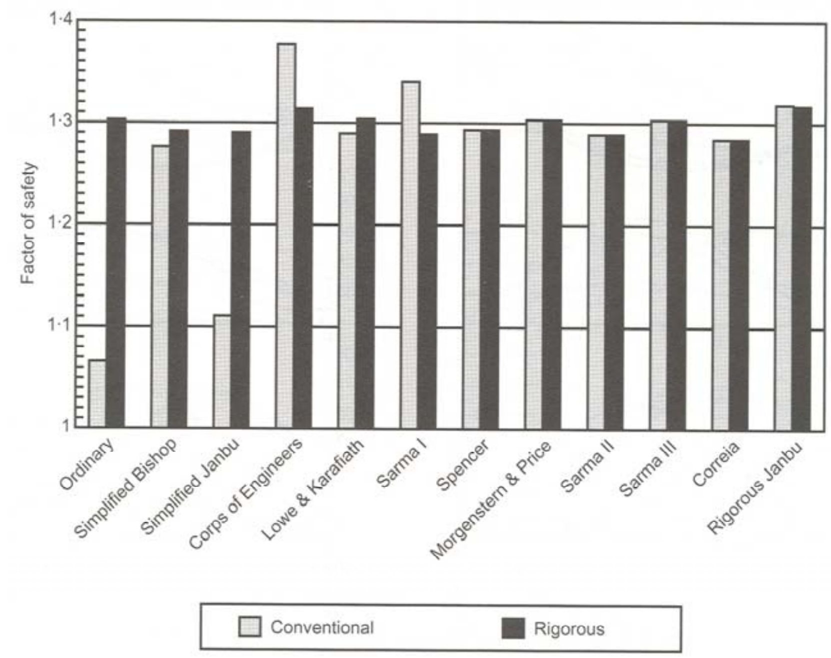


Figure 8. Factor of safety calculated with different methods using circular slip surface (Zhu *et al.*, 2003).

In current study representative of rigorous methods, Janbu corrected method (Rocscience Inc, Slide v5.0 help file), for slope stability modelling was exploited. Janbu corrected method uses Janbu simplified method (which isn't rigorous method) together with an empirical correction factor (Fig. 9) to calculate rigorous factor of safety (Connolly, 1997).

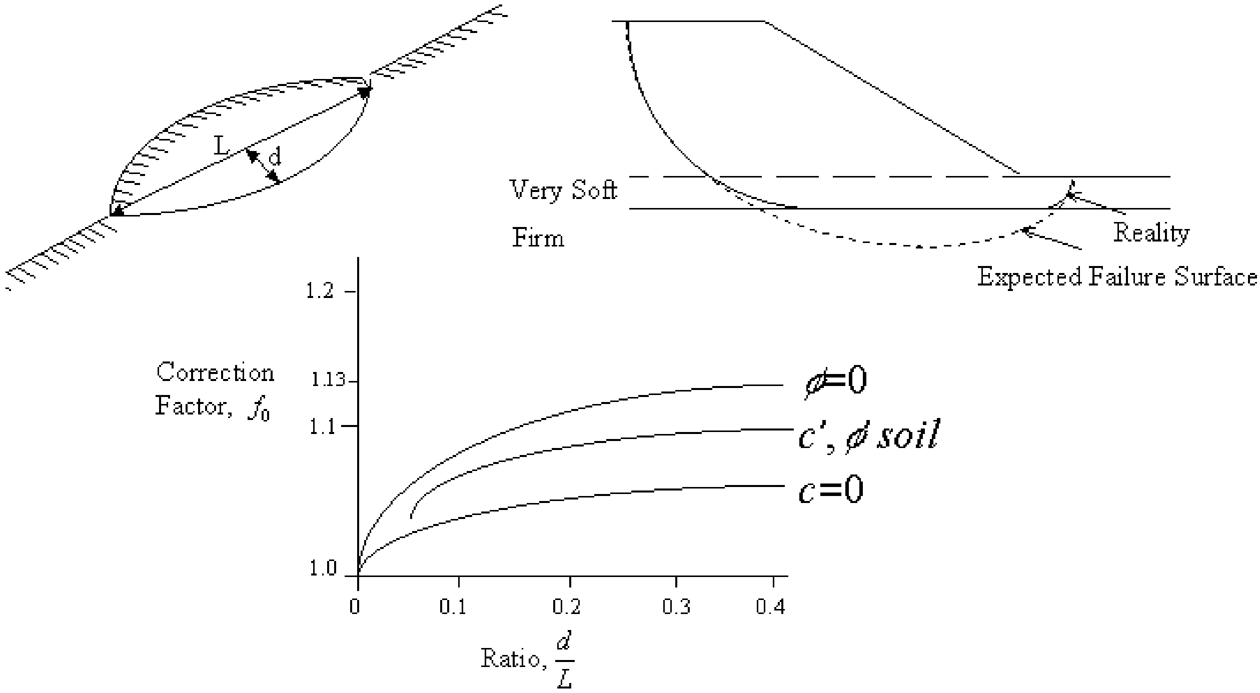


Figure 9. Correction value used by Janbu method (Connolly, 1997).

Although limit equilibrium methods are widely used and in general they are considered to be accurate and reliable, still various authors have pointed out certain short-comings and inaccuracies (Laouafa & Darve, 2002; Sjöberg, 1996; Griffiths & Fenton, 2004). According to Laouafa and Darve (2004) those disadvantages are:

- The factor of safety is a local parameter, which is assumed to be constant along the slip line.
- The sliding mass of soil and the remains of soil are assumed to be rigid.
- The sliding mechanism creates complex stress and strain distributions inside the sliding mass and the remaining soil, which are not taken into consideration.
- Neither stress path dependency nor stress-strain history is considered.
- Velocity and strain (at the material point level or global level) are not calculated or taken into account.
- The sliding mechanism is supposed to affect instantaneously the whole sliding surface (all material points belonging to the slip surface) at the same time. In other terms, these methods exclude the possibility of an initiation at some stages of loading and progressive development of the sliding surface.
- These methods do not involve time evolution of any mechanical or physical quantities.
- The normal stress distribution is not well reproduced and the conventional analyses

consistently ignore the incremental genesis of the slope and thus can never give the true stress distribution.

- The shape of the sliding or the slip surface is in general *a priori* assumed.
- The soil is assumed not to have a stress–strain behaviour with softening.
- The stability of the slope is only checked by the mean of a shear strength criterion, while there exist other modes of failure at lower stress levels.

Due to those reasons the author decided to use finite element method, that is free of those short-comings (Laouafa & Darve, 2002), for comparison modelling. Unfortunately attainable software package, Phase^2 v5.0 (Rocscience Inc.), wasn't suitable for slope stability analysis in this case. This package is more-likely ment for excavation design, although new version, v6.0, offers considerably increased possibilities for slope stability analysis (http://www.rocscience.com).

Nevertheless, the limit equilibrium analysis is relatively simple, reliable and still most used for slope stability analysis (Coduto, 1998; Sjöberg, 1996).

4.Results

Audru landslide occurred in the 11,12 May 2002, near to the old Pärnu-Lihula road, close to the "Rebasefarm" bus station (Fig. 1). Minor cracks occurred 4...5 years earlier, one, 10 cm wide, crack developed during February 2002 (Vahtre, 2002).

The height of the main scarp was 1,4 m and the sliding body moved 1,2 m towards the river (Photo 1).

Photo 1. Fresh scarp of the Audru landslide and skidding tail of the bridge construction. Photos by Peeter Talviste, Pille Sedman.

The river channel was almost totally filled with disturbed, silty clay, this fill was mostly eroded by the Audru river within a year (Photo 2).

Photo 2. The Audru landslide toe, photos are taken in 2002 and 2003 by Peeter Talviste and Volli Kalm.

According to aforementioned classification for the landslides in Pärnu area (Kalm *et al*., 2003) it belongs to the group C, large scale landslides related with the varved clays. This slide was chosen for more detailed investigation because of the good accessibility and geological setting. It was assumed that presence of the varved clay, which is finely laminated sediment, makes sliding zone investigation much easier, also it's possible to use varve correlation for addition information about the interslide events.

Total area of the slide is 75 x 36 meters and volume of the slided soil was around 20 000 m^3.

4.1 Mapping

With information obtained from the soil surface measurements, river depth data and borehole data one can create 3D view of the Audru landslide. Main morphological features of the Audru landslide are presented in Figure 10 and two geological cross-sections, one parallel to the river and other perpendicular (Fig. 10), are presented in Figure 11.

On the map of the Audru landslide (Fig. 10) one major scarp and three or four, if to consider a shallow ditch as a former scarp, minor ones could be followed. Slope angle after sliding event is around 6^o, being around 8^o before the slide. Slope surface is quite even. Main scarp above the sliding body has partly cut the highway, aforementioned minor scarps could be followed at two levels and the slope behind them are little bit turned against the main slope (Photo 3). Fine grained marine sand is outcropping at the surface except at the crest of the sliding body where the artificial fill of sandy gravel was a part of the road dam (Fig. 11).

Photo 3. Questa-like morphology. Photo taken by author.

Clay surface and also thickness is quite uneven (Fig. 11). Clay surface is somewhat elevated in the middle of the main sliding body (reported in the borehole Bh-2) and close to the Audru River (Bh-2x), while there is a deep depression (Bh-5) between those elevated areas (Fig.9). Maximum distance from the ground surface to upper clay surface is 4,5 m (Bh-5) and minimum 1,0 m (Bh-x2; S-20). Transition between minimum and maximum depths occur at 6 m distance. Maximum clay thickness (8,3 m) is reported in the middle of the sliding body and minimum thickness (4 m) in the lower portion of the sliding body (Bh-5). Till surface, underlying the varved clay is almost horizontal laying at an altitude of -4 - -6 m (Fig. 11).

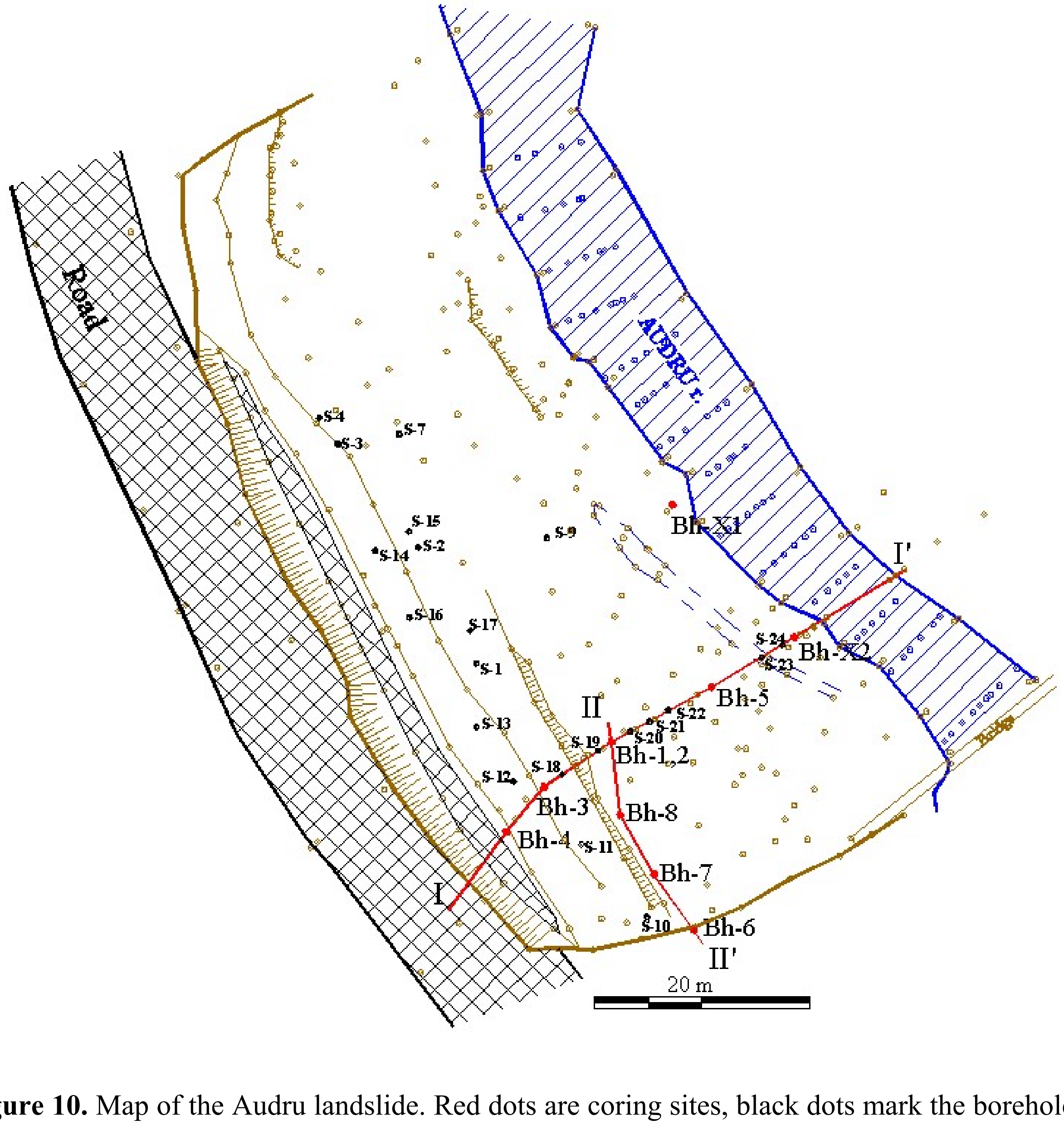


Figure 10. Map of the Audru landslide. Red dots are coring sites, black dots mark the boreholes for sand thickness measurements, brown and blue ones mark the soil surface measurement points. Areas filled with brown, comb-like texture are scarps. Blue dashed line marks shallow natural ditch, possibly a former scarp. Red lines mark the geological cross sections presented in Figure 11.

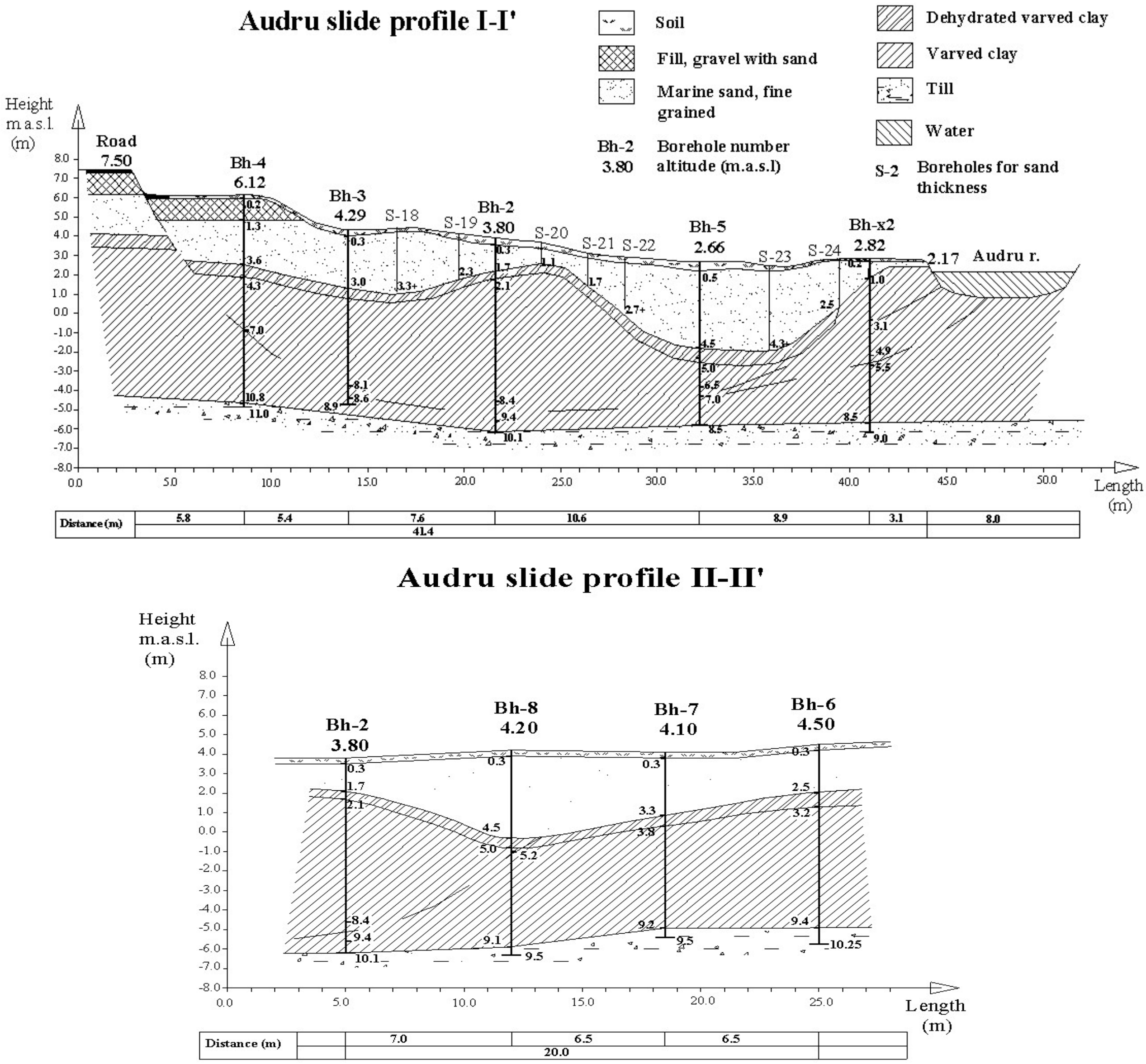


Figure 11. Cross-sections of the Audru landslide. Dashed line marks the slip surface.

Height of the major scarp is 1,2-1,4 m, minor scarps are about 0,3...0,6 m high. Slide surface, recorded from the core description, has an elliptical shape (Fig. 11) with the deepest point at 9,4 m below the ground surface, just above the till. Ruptured zone begins as a thin fault inside the varved clay (Bh-4). Towards the river its thickness increases. Thus, at borehole Bh-4 disturbed zone was 5 cm thick and at borehole Bh-1, 13 m towards the river, the zone was 1 m thick. At the borehole Bh-5 disturbed zone was 0,5 m thick but part of it is missing while the core correlation (see section 4.2) demonstrates the missing interval inside the ruptured zone. At borehole Bh-x2 two sliding surfaces were found, upper one is very sharp, visible as a discontinuity inside the varve interval. Lower one is a 0,5 m thick ruptured zone.

4.2 Core correlation

Glacial varved clays with their characteristic summer (silty) and winter (clayey) layers are interpreted to reflect seasonal variations in the sedimentary environment. Due to their laminated structure varved clays have been used for detail core correlation and chronological purposes. As the sliding body of the Audru landslide is mainly built up of varved clays it was postulated that identification of slip surface/zone and possibly missing intervals in the sediments would be possible through the detailed core correlation. In the visual correlation, made by strait comparison of sediment cores, thickness of varves and seasonal layers, colour and structure of the varves or characteristic varve series was considered. Correlation of varve graphs is demonstrated on Figure 12 and corresponding core correlation is displayed on Figure 13.

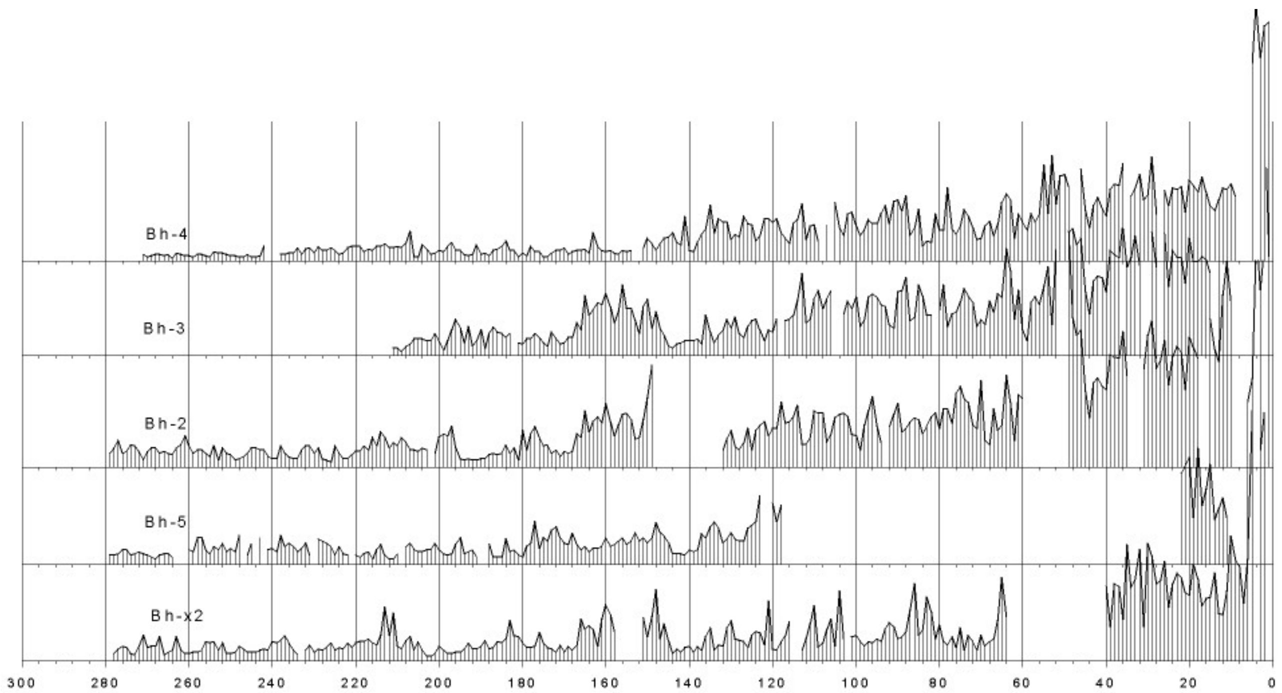


Figure 12. Local varve chronology for the Audru landslide. Varve years are counted from the bottom of the sequences; y-axe demonstrate the thickness of individual varves; x-axe demonstrate the number of varves (years); missing parts of graphs demonstrate either missing sediments or disturbed vares. Location of a site see Fig.1 and location of the boreholes in Fig. 11.

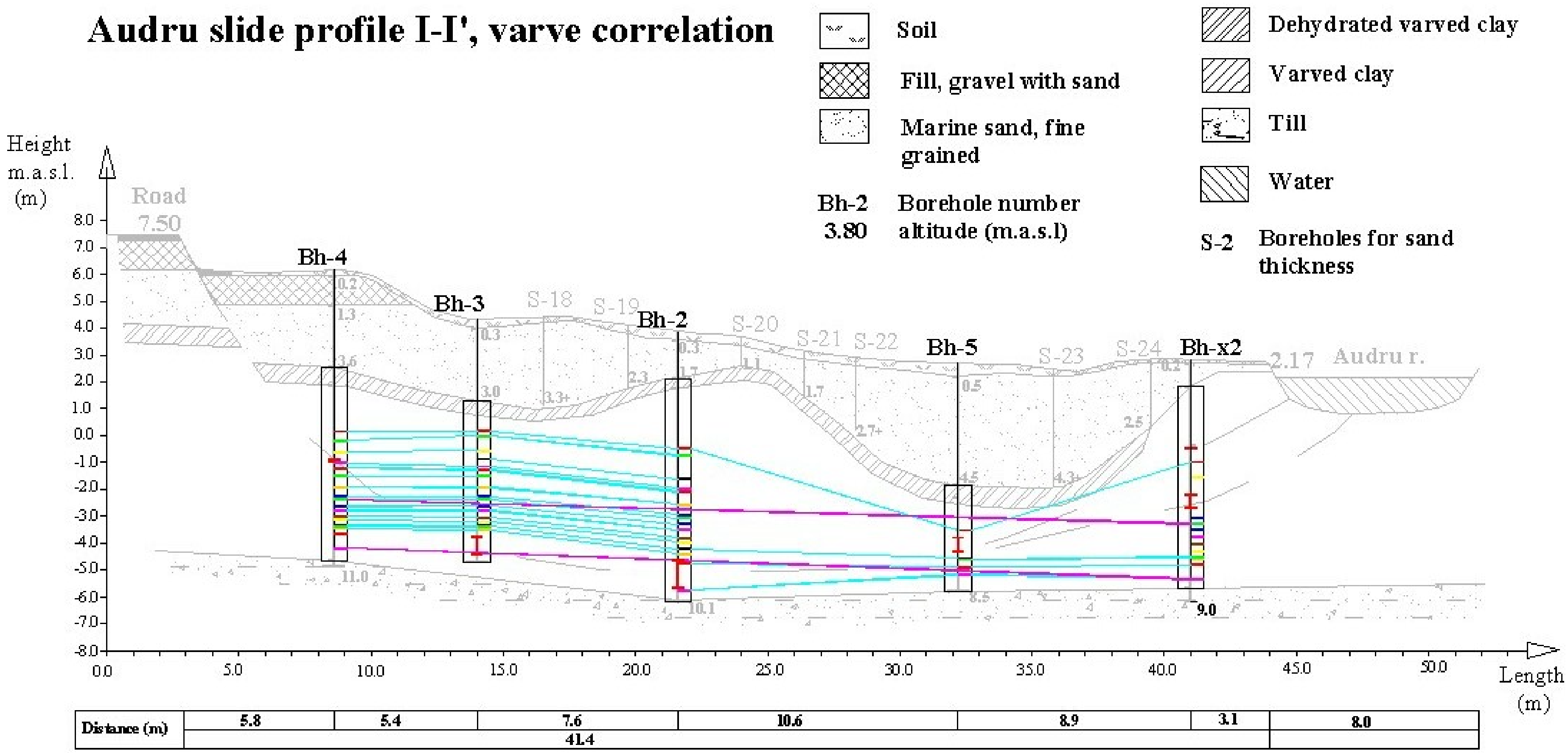


Figure 13. Varve correlation. Coloured lines are markers, magenta lines show how varves were before the slide.

Varve correlation was easy in case of lower portion of sediments where thick proximal varves and varves accumulated in the central part of the proglacial basin are present. Upper, distal varves were more difficult to correlate, as the interval between clear and reliable marker varves was longer. Varve correlation demonstrates that all identified markers were present in the sediment sequences (Bh-4-3-2) at the upper part of the sliding body (Fig. 13). Relatively big interval of clay is missing at the borehole Bh-5. Approximately 2,5...2,8 m of clay is missing between lower black and upper red marker (marker varves) (Fig.13). As the upper marker as well as distal varves can still be found in Bh-5 the missing part of clay isn't eroded away as might be concluded from the changing altitude of the clay surface (Fig. 13), but most likely has been pressed out during the sliding event. Disturbances within the varved intervals are interpreted as slip surface which has elliptical shape and which is displayed on Fig. 13.

4.3 Geotechnical properties of the varved clay

Geotechnical samples were taken for analysis of the Attenberg limits, water content, $CaCO_3$ content and grain size distribution to the geotechnical lab of Estonian Environmental Research Centre.

Complete results of the analysis of the geotechnical properties of discussed varved clay are presented in Appendix 1. In accordance with those results and with experience got from the field and laboratory work whole clay section was divided into four complexes: dehydrated clay, upper, middle and lower portion of clay sequence. Geotechnical parameters of varved clay in Audru site are presented in Table 3. Parameters are calculated by Peeter Talviste.

Section	**Water content** W_n (%)	**Liquid limit** W_l (%)	**Unit weight** γ (kN/m^3)	**E. cohesion** c' (kP)	**E. friction** φ' (°)
Dehydrated	42	62	17,9	-	-
Upper	81	62	15,1	7	13,5
Middle	70	50	15,6	7	16,6
Lower	40,7	42,9	17,8	14	23,8
Lower *	75	42,9	17,8	5	14

Table 3. Geotechnical parameters of the varved clay section.

Strength parameters to asterisked lower section in Table 3, were calculated because two samples (nr. 9897 and 9907) had significantly higher (79,4 % and 75,1 %) natural water content values than other samples (31,2...66,8%). Both those samples were situated inside the aforementioned ruptured zone.

To model material forming the ruptured zone residual strength properties were used. It's known that large shear strain causes soil strength to fall down to the residual level (Fig. 14) (Coduto, 1998). In case of reactivated landslides residual condition is present on the entire slip surface (Mesri & Shahien, 2003). According to Coduto (1998) residual strength is purely frictional (i. e. there's no cohesion), so cohesion, c' was considered zero in case of the ruptured zone material. Frictional component was chosen according to prevailing material. If most of the slide surface passes through middle section of the varved clay, with strength properties c' 7 kN/m^2 and φ' 16,6 ^{0}, the strength of the ruptured zone was chosen as c'_r 0 kN/m^2 and φ'_r 16,6 ^{0}. Sands and gravel have only minor difference between residual and peak strength (Coduto, 1998), so no residual strength criteria was needed for those soils.

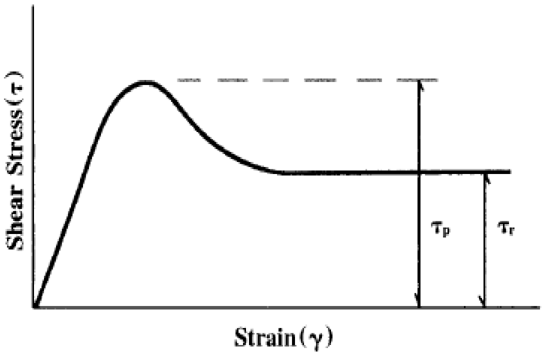


Figure 14. Shear stress and strain relation (Tiande, *et al,* 1999)

As there weren't any samples from the middle complex of the clay sequence, parameters for that part were chosen as mean values between upper and lower complexes of the section. Complex boundaries were identified according to experience gained from the drillings and from the varve correlation. Lower complex includes varves that are more than 50 mm thick, this complex roughly covers half of the total clay section. Lower boundary of the upper complex is a red marker shown in varve correlation (Fig. 11) and the middle complex is between. The upper and middle complexes each cover roughly one quarter of the total clay section. Thickness of the dehydrated clay complex was obtained from borehole logs. It should be taken into consideration that in nature the boundaries between the mentioned clay complexes are transitional and strength parameters of the clay are increasing quite evenly towards depth as varves are getting thicker and so the content of silt and sand fraction in the soil increases and water content decreases.

4.4 Modelling

Limit equilibrium model, exploited in current study, used Janbu corrected method (see section 4.) to calculate factor of safety for the slope. Groundwater conditions were modelled using finite element method that was incorporated to the software. Commercial software, Slide v5.0 (Rocscience Inc.) was used for LEM analysis.

Groundwater conditions are most variable parameters affecting slope stability and rapid changes of them are known as a major cause of slope failures (Wang & Sassa, 2003; Sjöberg, 1996). Therefore for the first step those parameters (water level in the river and piezometric level in the till) were varied to find their influence to the slope stability. Water depth 2 m from the ground surface was considered „normal“, 2,5 m „high“ and 1,5 m „low“. Detailed analysis, with step 20...30 cm was carried out with fixed piezometric level of the lower groundwater layer (see section 1.). Piezometric level was varied around -1...2 m a.s.l. with a step 1 m, one interval was chosen for detailed analysis with the step 20...30 cm.

Results of laboratory analysis demonstrate that natural water content is highly variable, especially in the lower complexes of clay. It's known that geotechnical properties of the varved clay are very well correlated to the natural water content so that higher water content corresponds to weaker strength parameters (Võrk & Vilo, 1977). Therefore we can vary strength parameters effective cohesion (c') and effective friction (φ') of the lower varved clay complex according to water content changes within the limits calculated by P. Talviste.

Geotechnical parameters of marine sand that was used in the slope stability analysis were obtained from the literature. According to Kaljund & Mets (1976) marine sand has following properties: unit weigth γ 18,6 kN/m^3, cohesion c' 10 kP and internal friction φ' 35^0. Till was considered as a bedrock for the model i.e it has infinite strength.

It would be wise to control calculated soil strength parameters, which could be done by analysing investigated slope under “normal” groundwater conditions. If the slope is stable according to the LEM, it means that the calculated parameters aren’t at least too low. Results of analysed slope at Audru landslide (Fig. 11) is displayed on Figure 15.

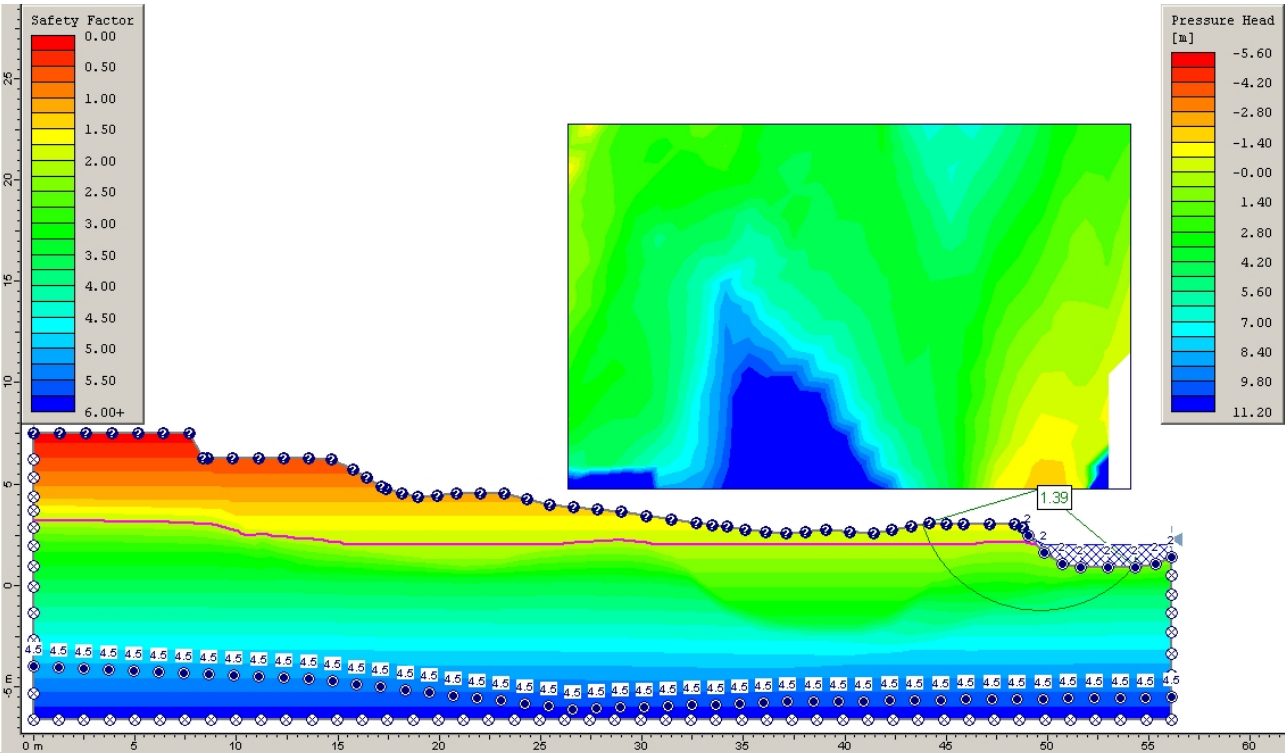


Figure 15. LEM stability analysis of the slope I-I' on Figure 9. Piezometric level is around 0 m a.s.l. and water depth in the river 2 m.

Model demonstrates that current slope is stable under „normal" groundwater conditions which shows that used strength parameters for varved clay aren't too low. One can see that the most endangered area is adjoining the bank of the river channel.

In following a stepwise slope stability analysis in the light of changing slope geometry, hydrological and hydrogeological conditions will be presented

Initial slope (pre sliding event slope) was reconstructed (Fig. 16) and analysed, modelling results are presented on Figure 17.

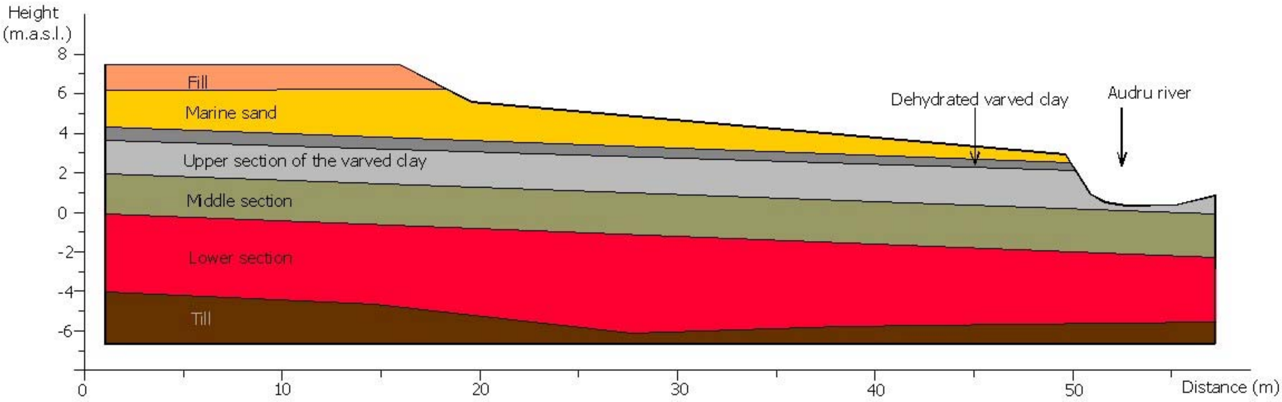


Figure 16. Initial slope model setup and geometry.

According to model the virgin slope i stable when piezometric level of the second groundwater layer is low (-1 m a.s.l.) and water level in the river is high (2,5 m) or normal (2 m). Slope will fail if to increase the piezometric level of the second groundwater layer and to lower the water level in the river (Fig. 17).

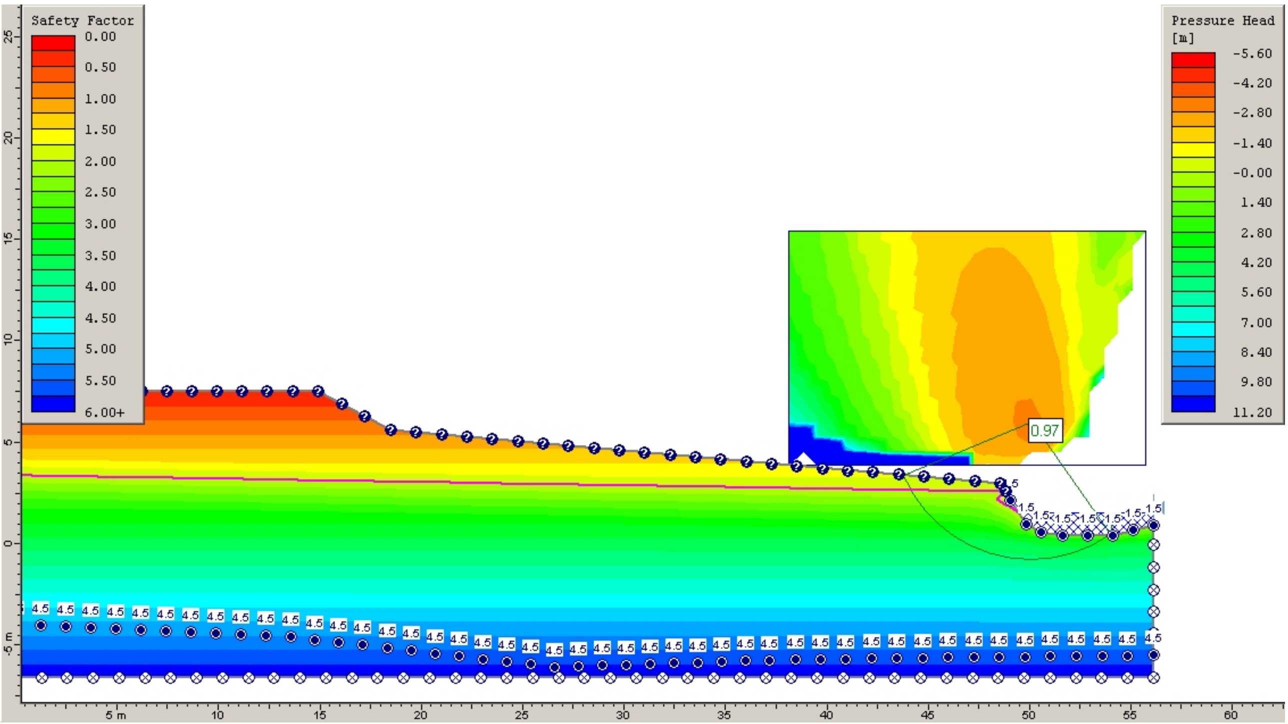


Figure 17. Initial slope, piezometric level around 0 m a.s.l., water depth in the river 1,5 m.

Failure will take place close to riverbank. Besides the water levels the depth of the river channel also controls this failure. It is worth to mention, that in case of 0,5 m deeper channel the slope would fail even without groundwater present. River erosion is the mechanism driving the initial slope towards instability but changes in the water levels are the triggering failure. To investigate the influence of both parameters (the water depth in the river and the piezometric level of the second groundwater layer) to the slope stability, more detailed modelling was carried out (Fig. 18 and Fig. 19),

which demonstrates that the water depth has bigger influence to initial slope stability than the piezometric level of the second groundwater layer. Both relations are almost perfectly linear with correlation factor – 0,99...0,98. Plotted line in Figure 18 can be described with formula Y=0,01+ (0,44*X) there Y (FoS) is a function of X (water depth in the river). Similar formula for Figure 19 is Y=1,19+ (-0,3*X), there X is piezometric level of the second groundwater layer.

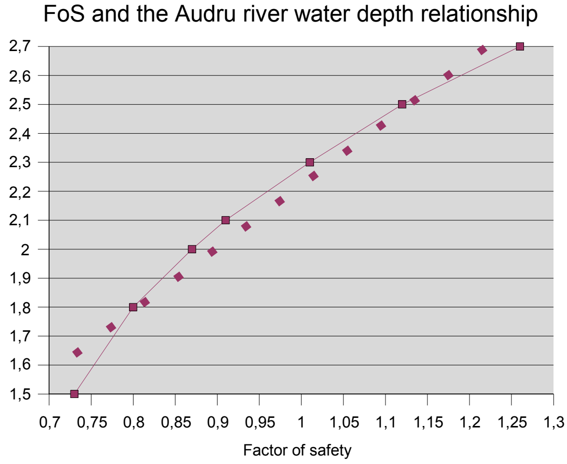


Figure 18. Relation between the water depth in the Audru River and FoS of the initial slope.

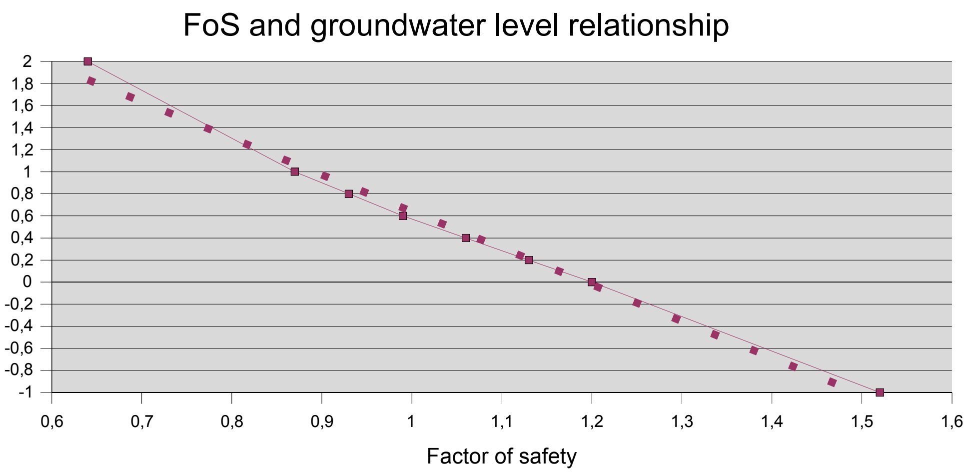


Figure 19. Influence of the piezometric level of the second groundwater layer to the initial slope stability.

The currently used computer program offers also an opportunity to see the influence of the cohesion to FoS (Fig. 20). Cohesion of the upper complex of clay has the biggest influence to the slope stability, increase of cohesion by 20 %, from 7 to 8,4 kN/m^2, increases FoS from 0,9 to 1,3.

Cohesion of the middle complex has a bit smaller influence - increase by 20 %, from 7 to 8,4 kN/m^2 increases FoS from 0,9 to 1,1. As the first slide surface doesn't pass through the lower clay complex, cohesion of the lower complex doesn't affect the initial slope stability.

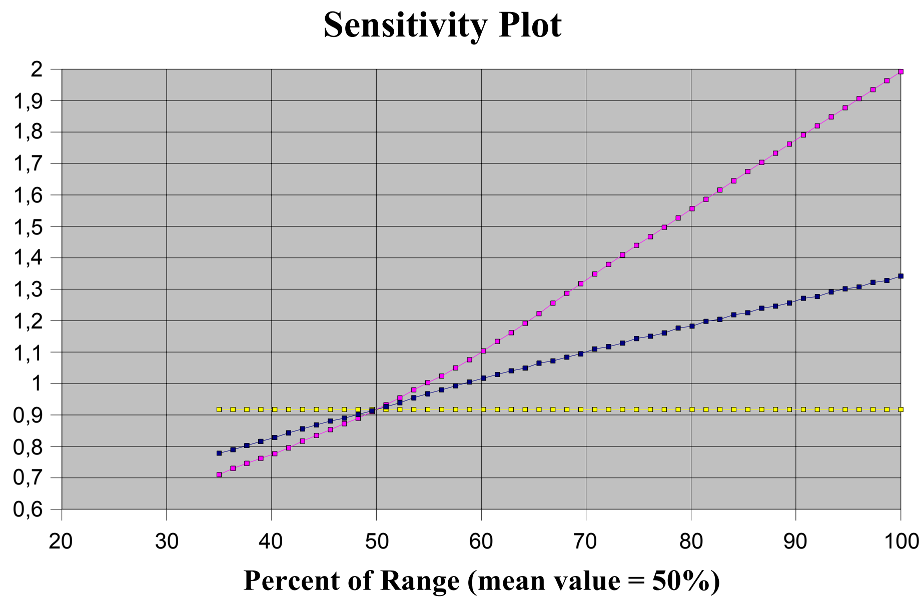


Figure 20. Relationship between cohesions and FoS of the initial slope. Mean value is the value of used cohesion (upper and middle complexes 7 kN/m^2 and lower complex 14 kN/m^2). Magenta line represents the upper complex, blue the middle and yellow the lower complex of clay.

As a result of the first slope failure we get a second slope which is gently sloping and more stable close to channel because some of the material is moved into the river. Reconstructed slope is displayed in Figure 21.

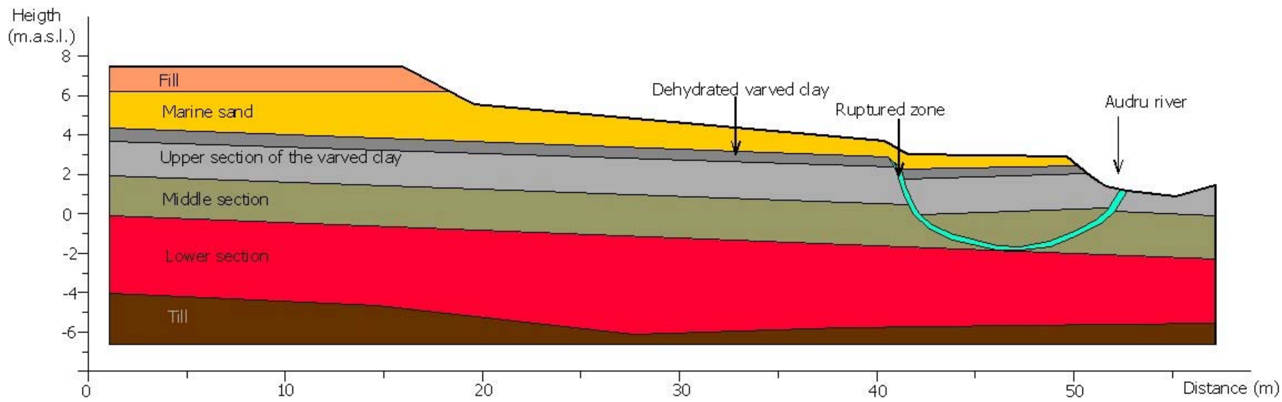


Figure 21. Second slope (after the first failure) model setup and geometry.

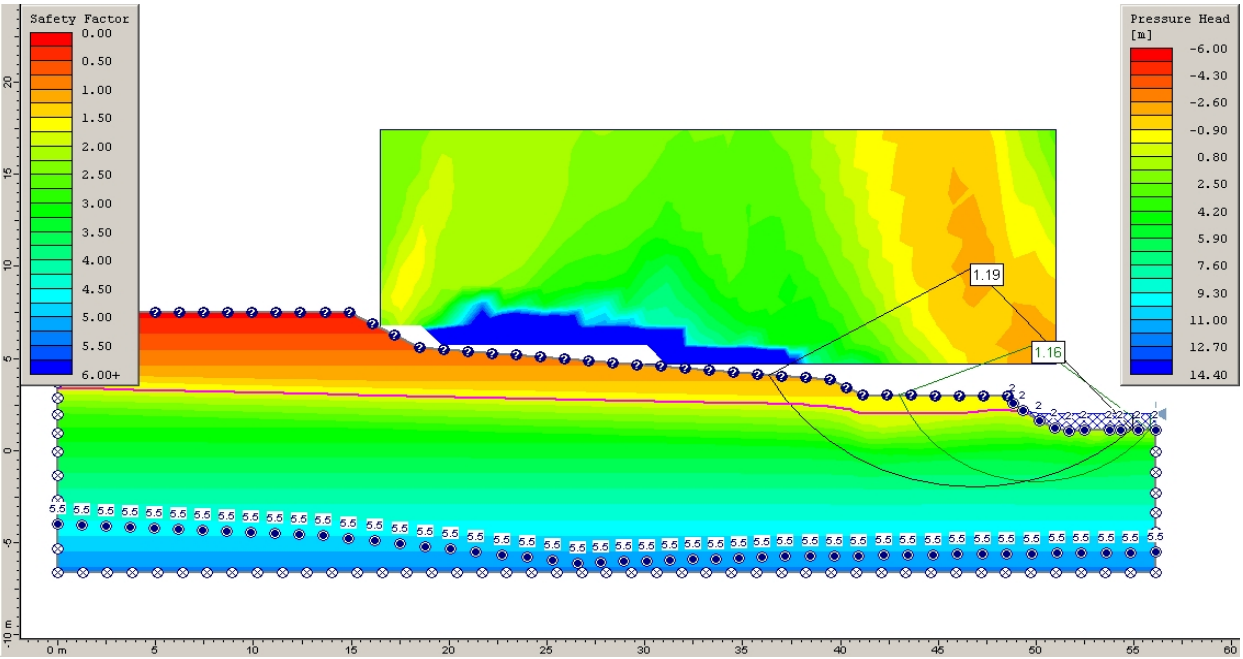


Figure 22. Second slope, piezometric level around 1 m a.s.l., water depth in the river 2 m.

According to model (Fig. 22) the second slope has two areas of instability: one is still close to the riverbank and the other is located somewhat upslope. The modelled slides near to the riverbank are strongly controlled by the water depth in the river as it was demonstrated in case of virgin slope model. Thus, in case of high water level in Audru River, the banks of the river become more stable (Fig. 23).

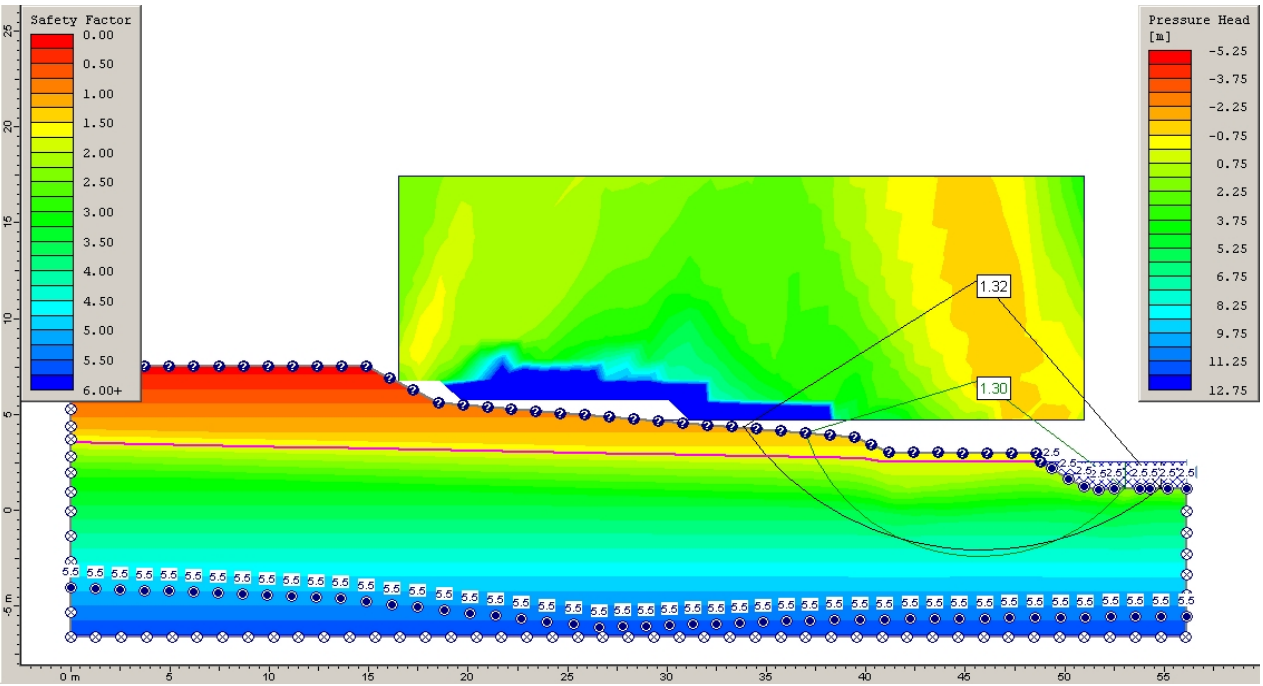


Figure 23. Second slope, piezometric level around 1 m a.s.l., water depth in the river 2,5 m.

As the water level in the Audru River rises, the riverbanks become more stable and the unstable area appears uphill from the first slide scarp. If the piezometric level of the second groundwater layer raises this mentioned slope area becomes unstable e.q. factor of safety decreases (Fig. 24) and finally the slope will fail.

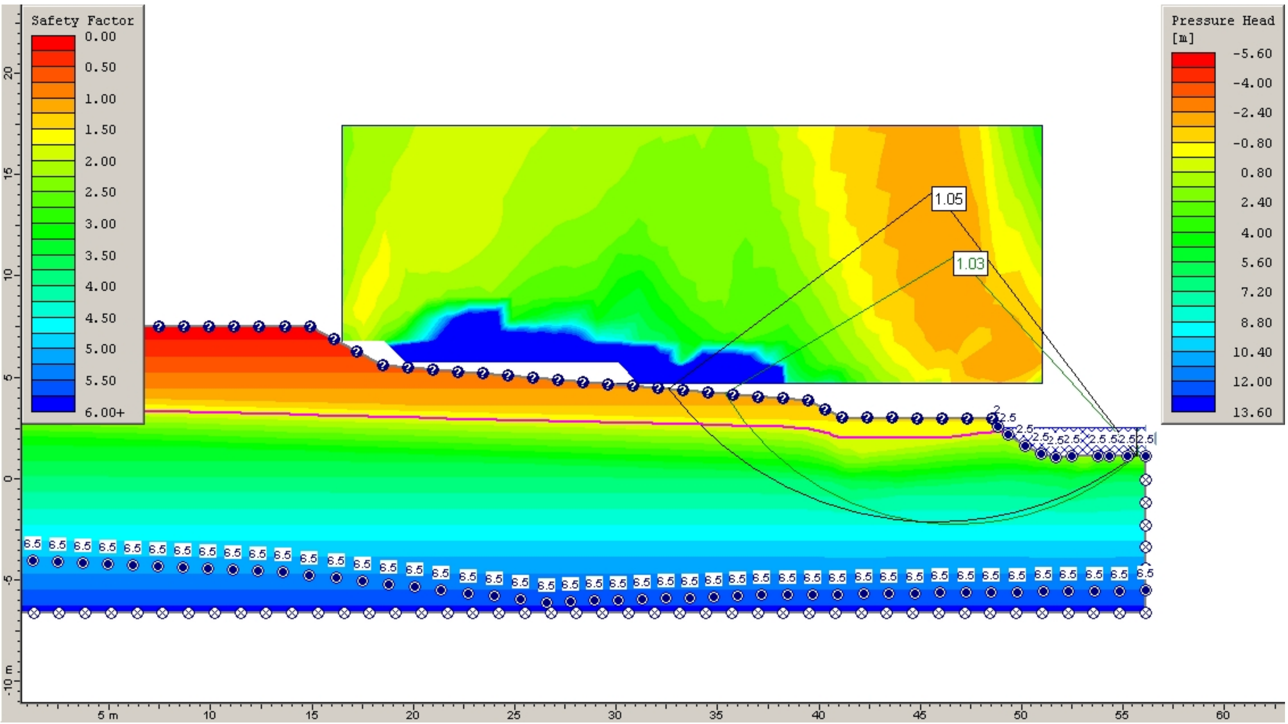


Figure 24. Second slope, piezometric level around 2 m a.s.l., water depth in the river 2,5 m.

Both water levels (second groundwater layer and water level in the river) must be high for this kind of failure. If the water level in the river is „normal“ or „low“ the slope will become unstable only close to the riverbank.

After the second slide, the slope morphology again turns more gentle (Fig. 25). Modelling of this

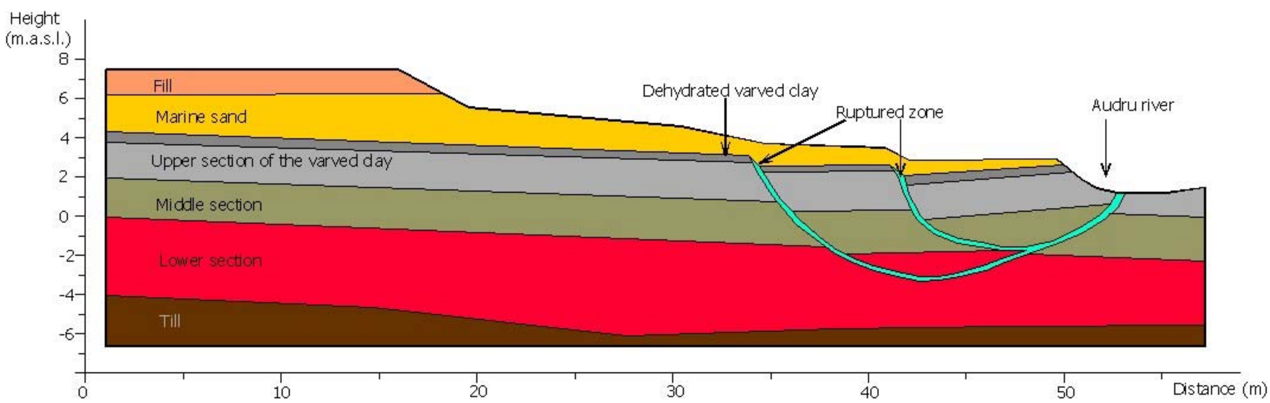


Figure 25. Third slope model, geological setup and geometry.

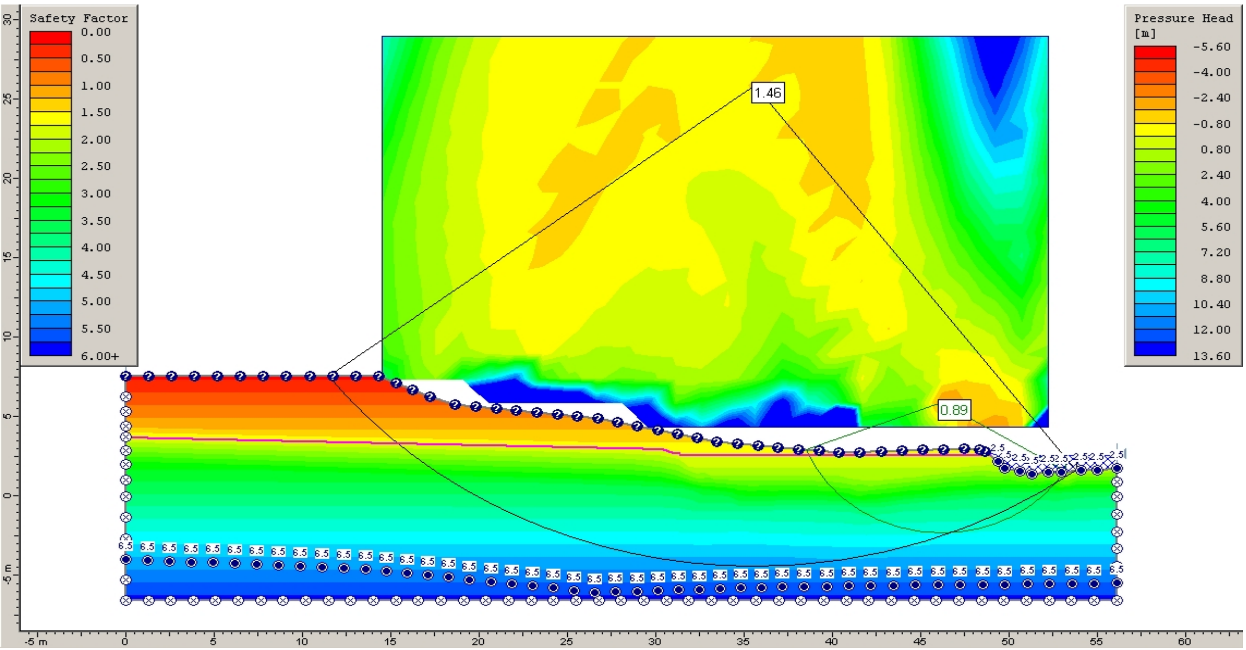


Figure 26. Third slope, piezometric level around 2 m a.s.l., water depth in the river 2,5 m.

slope demonstrates that the area most prone to the failure is again close to the riverbank (Fig. 26), somewhere between the modelled first and the second slide scarps. Within set limits for the water levels and with given strength parameters the model doesn't fail at scale the observed landslide took place. If the lower strength parameters are used, the model becomes unstable at the scale observed in nature (Audru landslide) (Fig. 27).

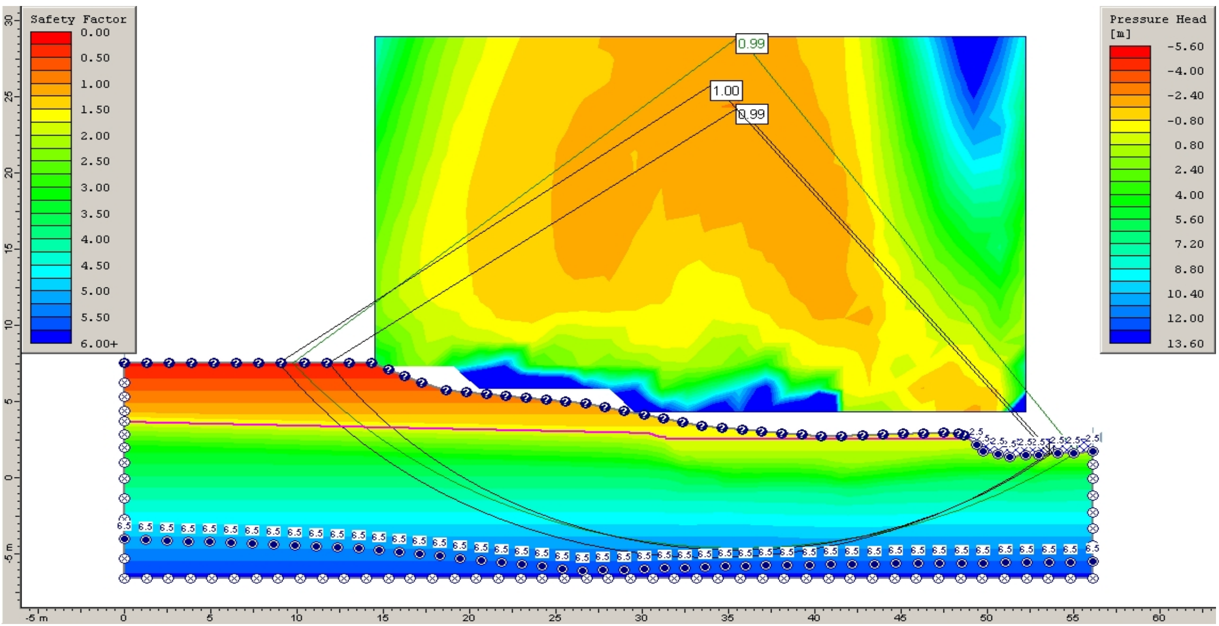


Figure 27. Third slope, piezometric level around 2 m a.s.l., water depth in the river 2,5 m. Strength properties of the lower clay section are c'= 5 kN/m^2 and φ'=14 ^{0} being thus lower than in previous models.

If the strength properties of the lower clay complex are at minimum (calculated by using maximum water content measured in the geotechnical laboratory) the third model becomes unstable. More detailed analysis into the third slope was carried out to show the relationships between FoS and piezometric level of the second groundwater layer (Fig. 28) and between FoS and water depth in the river (Fig. 29).

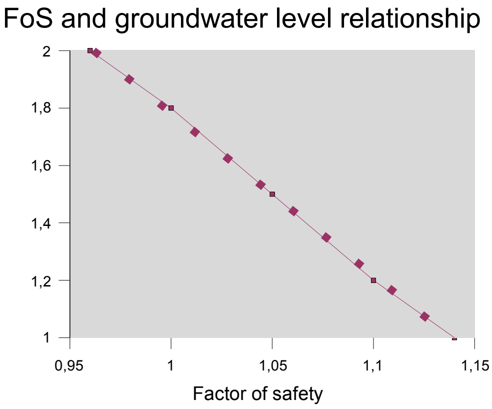


Figure 28. Relationship between piezometric level of the second groundwater layer and factor of safety in case of the third slope.

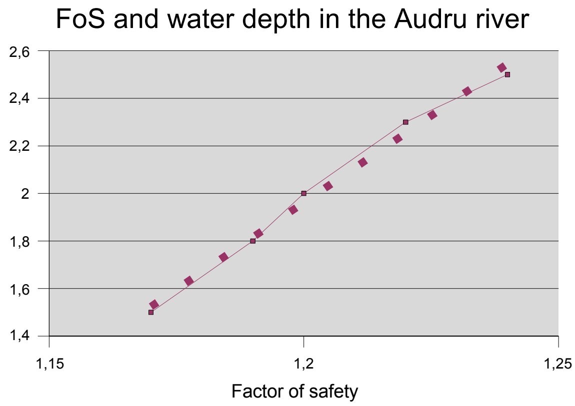


Figure 29. Relationship between water depth in the Audru River and factor of safety in case of the third slope.

Relation between FoS and piezometric level of the second groundwater layer can be described with formula Y=1,31+ (-0,18*X), where Y (FoS) is a linear function of the X (piezometric level). Similar function for FoS and water depth relation is described by formula Y=1,07+ (0,07*X), where X is a water depth in the Audru River. Relation between cohesions of clay layers and FoS of the third slope are in fig. 30.

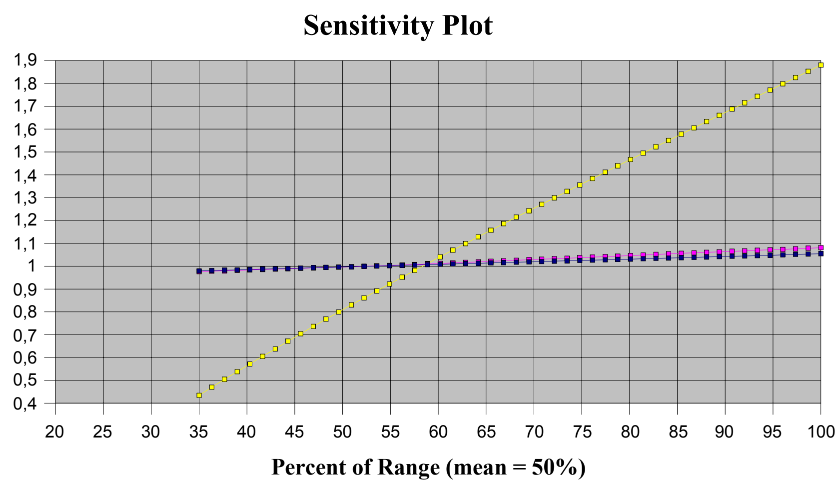


Figure 30. Relation between cohesion of the varved clay complexes and FoS of the third slope. Mean value is the value of used cohesion (upper and middle complex 7 kN/m^2 and lower complex 5

kN/m^2). Magenta line represents the upper complex, blue the middle and yellow the lower complex.

Cohesion of the lower complex has a biggest influence to the factor of safety. Increase of 20 %, from 5 to 6 kN/m^2 increases FoS of the third slope from 0,9 to 1,25. Cohesions of the upper and middle complexes have only minor effect to the third slope stability.

After third slope failure residual slope would look like in Fig. 31.

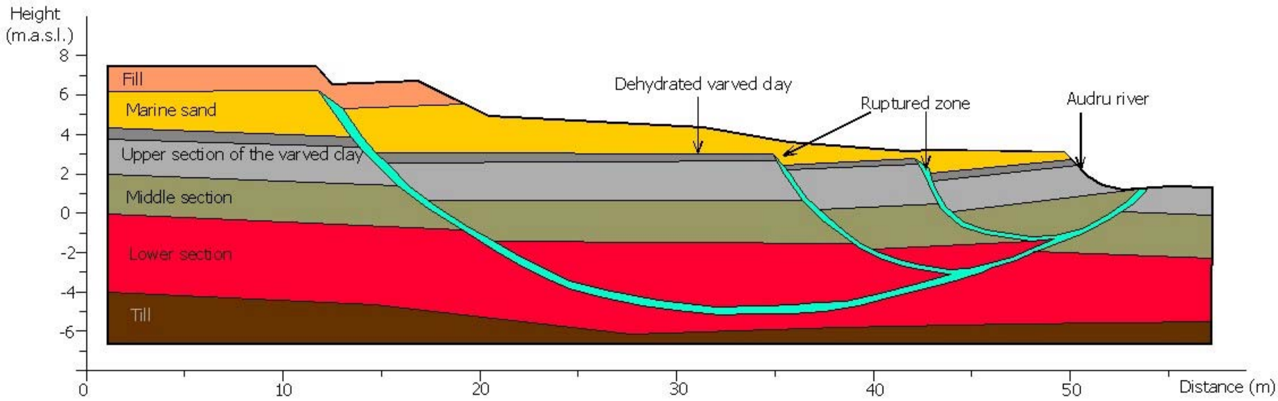


Figure 31. Hypothetical residual slope after third slope failure.

5. Discussion

Minor scarps on the surface of the main sliding body of the Audru landslide, shown in Fig. 10, may indicate reactivation of the previous, smaller sliding surfaces. Reactivation of the old slide is common phenomena reported by various authors (Fletcher *et al.*, 2002; Sjöberg, 1996; Delano & Wilshusen, 2001). The failed slope will be weaker as soil strength in the rupture zone drops down to the residual strength, this happens largely due to particle reorientation and a breakdown of the soil fabric (Coduto, 1998). According to Jaaniso & Oll (1997) the varved clay in Pärnu area is overconsolidated soil, with overconsolidation factor around 1,5 (Talviste, personal communication). Such soils have residual strength significantly lower than the peak strength, this is due to factors just described, plus an increase in void ratio during shear and the resulting increase in water content (Coduto, 1998).

The shear induced increase in void ratio, called dilatancy, may partly explain the high water content measured inside the ruptured zone. Measured values are ca 50 % higher than the average reported by various authors, although large dispersion of the natural water content is specific to the lower complex of the varved clay (Võrk & Vilo, 1977; Kattel, 1989). As the natural water content correlates very well with strength properties of the varved clay (Olli, 1962, Võrk & Vilo, 1977), the strength properties are also considered to be highly variable.

In case of the slope stability analysis one needs to know the strength of „the weakest link“ as the slide surface starts to develop from that point (Muller & Martle, 2000; El-Ramley, 2002). Progressive failures are mostly associated with brittle, stiff soils where the difference of the peak and residual strength is large, but it could also occur in normally consolidated or lightly overconsolidated clays in which the difference between peak and residual strength is smaller (Sjöberg, 1996). The mechanism of the progressive failure in the slope is that the peak shear strength is exceeded at one point of the slope, resulting the stress redistribution due to the lower residual strength of the material. This local failure may occur due to heavy rainfall, snowmelt, creep, etc. even if the whole slope remains stable (Chowdhurry & Flentje, 2003). Stress redistribution around the local failure causes nearby points to yield, which results in further stress redistribution and so the process continues till the whole slope fails (Sjöberg, 1996).

Some authors have suspected large scale creep inside the varved clays (Olli & Martin, 1961; Kaljund & Vilo, 1967). This process starts inside clays when shear stress is 70 % from critical value, in some soils even at 50 % (Coduto, 1998). Movement by several centimetres can drop the shear strength down to the residual level (Burns, 1999). However, after the numerous landslides in Pärnu town during 1960-ties, precise measurements was carried out to find creep behaviour and no movements bigger than measuring error (1...2 mm) were found (Mets & Vilo, 1976)

Therefore the use of the lower, residual strength or fully softened strength parameters for the slope stability analysis should be considered (Muller & Martle, 2000; Burns, 1999). Aforementioned details justify the use of the weaker strength parameters of the lower varved clay complex in the models currently presented.

The modelling results suggest multi-stage sliding. Firstly, small scale slides occur close to the river channel and then the slides are progressively getting bigger. Larger slides are „exploiting" pre-existing weak zone i.e. slip surface of the previous, smaller slide is a part of the bigger slide surface. Similar failure mechanism is achieved in the lab by Wang and Sassa (2003), who named this type as a retrogressive failure. They used fine-grained sand for their experiments, if they increased loess (with grain-size comparable to fine silt and clay) content of the material, the slides become significantly more flow-like (Wang & Sassa, 2003).

Biggest difference between current slope (Fig. 11) and modelled one (Fig. 31) is in the area around Bh-5, where relatively deep, sand filled depression is present. Varve correlation (Fig. 13) shows that up to 2,8 m of the varved clay section is missing between the marker varves. As the upper marker can still be found, the clay isn't eroded away by river. At first glance one may consider the depression in the clay surface at borehole 5 as a former river channel, but this is eventually formed due to absence of the material pressed out by the landslide.

The missing clay is a former part of the middle complex and the upper half of the lower clay complex (Fig. 13). According to Reinson (2005) and Kattel (1989) silt content rises up to 50 % of the total soil weight, as upper layers only have 15...25 % of silt. Silt and fine grained sand is most prone for liquefaction (Coduto, 1998).

In case of the slide induced press-out, a question about the sand cover is coming up. If 2,8 m of the clay is pressed out, a depression at the same scale must be visible on the ground. One can see from the cross-section (Fig. 11) that the depression is filled with sand, so there must have been sufficient time for the smoothing of the surface. This alludes to earlier slides because obviously there wasn't enough time for that, as there was one week between the slope failure and the first investigation done by Talviste (Talviste, personal communication). Proposed explanation for the aforementioned facts is that the part of the varved clay around Bh-5 liquefied and was pressed out into the river channel. This occurred several times, allowing the smoothing of the ground surface in the mean time. Intermediate slide (Fig. 24) probably reactivated several times and each time part of the lower clay complex was pressed into the river channel. Last, large scale slide probably removed ca 0,5 m of clay as there is a shallow depression on the ground surface at this scale around borehole 5.

First small slides make the whole slope weaker by changing slope geometry. A notch locally increases the slope-parallel shear stress, and also generates the slope geometry in which slope-parallel slip plane can intersect the ground surface (Muller & Martel, 2000).

The modelling of Audru landslide shows that there must have been at least one or two smaller slides before the large scale slide. This opinion is supported by previous studies and by varve correlation. During landslide hazard investigation in 2002, two fresh small scale landslides were found on the banks of the Audru river (Kalm *et al.*, 2002). Those slides are very close to the currently described Audru landslide, supporting the hypothesis that river erosion constantly causes instability of the riverbanks at that area. Possibly numerous small scale slides took place before the second, significantly larger slide. According to the modelling relatively high piezometric level of the second groundwater layer is needed for the second, larger slide. It applies also to the third, largest slide. Small scale slides are mainly controlled by the depth of the river channel and the water table in the Audru river, also by the strength of the upper clay complex. Larger slides depend mostly of the piezometric level and of the strength of the lower clay section.

As piezometric level of the second groundwater layer significantly affects slope stability (Fig.19 & 28), it would be interesting to calculate decrease of the FoS of the Pärnu River banks inside the Pärnu town, where piezometric levels have risen since 1990-ties due to decrease of the water consumption (Fig. 5).

Small scale slides are among other factors controlled by the strength of the upper clay complex. This parameter can be affected by vegetation cover, namely by roots. This factor is difficult to estimate as it depends of the vegetation type and density, plant species and age etc. (Ekanayake & Phillips, 2002). According to Roering, *et. al* (2003) root cohesion can be as high as 15,2 kP, this addition to the peak shear strength would significantly increase the slope stability in case of small scale slides (Fig. 20). Therefore forestation of the riverbanks would decrease the risk of failure and it would also slow down the erosion. Opposite process is going on as small trees and bushes were cut down during this spring from the Audru landslide and nearby riverbanks. Vegetation affects only small scale slides as roots probably doesn't penetrate to the lower clay complexes that influence bigger slides (Fig. 30).

From modelling point of view varved clay is very difficult material because of the heterogeneity inside a single varve (Reinson, 2005) and gradually changing soil properties towards depth (Võrk & Vilo, 1976; Kattel, 1988). This difficulty can be solved by using finite element method together with random field theory (Griffiths & Fenton, 2004). As finite element method divides a model into small elements, one can use the random field theory to assign strength properties to a single element according to controlling algorithm. Therefore one can increase soil strength almost gradually (with much smaller step than with currently models) towards depth, simulate variability of the soil strength properties, etc. (Griffiths & Fenton, 2004; El-Ramly *et al.*, 2002).

6. Conclusions

Most important conclusions to emerge from this thesis are:

- Varved clays offer a good opportunity to study geological settings of the landslides due to laminated design;
- For the modelling varved clays are difficult material because of the heterogeneity inside a single varve and gradually changing soil properties towards depth;
- Detailed description of geological setting and morphology of the Audru landslide was presented;
- Audru landslide is a retrogressive, complex slide, consisting of numerous previous, smaller scale slides;
- Minor scarps on the main sliding body of the Audru slide are most likely former slide scarps;
- Varve correlation was useful tool as it gave an opportunity for detailed core correlation and correspondingly additional information about the location of the ruptured zone and interslide events;
- Slip surface in the upper part of the Audru landslide starts as a sharp discontinuity and evolves to up to 1 m thick ruptured zone;
- Part of the ruptured zone in the lower part of the sliding body is missing, material was pressed out into the river channel;
- Strength properties of the lower clay complex in Audru landslide vary significantly from those in upper complexes;
- Small scale slides are mainly controlled by the strength of the upper clay section, by the depth of the river channel and the water table in the Audru River;
- Larger slides depend mostly of the piezometric groundwater level and of the strength of the lower clay complex;
- Large scale failures are „exploiting“ previous slip surfaces, they are probably progressive slides;
- Part of the sliding body showed flow-like behaviour.

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References

Burns, W. J., 1999. Engineering geology and relative stability of the southern half of Newell Creek canyon, Oregon City, Oregon. Portland State University, unpublished MSc thesis, 156 pp.

Chowdhury, R. & Flentje, P., 2003. Role of slope reliability analysis in landslide risk management. Bulletin of Engineering Geology and the Environment, 62, pp. 41-46.

Coduto, D. P., 1998. Geotechnical engineering: principles and practices. Upper Saddle River, New Jersey, 743 pp.

Connolly, H., 1997. World Wide Web Pages for Slope Design, MEng final year project report. School of Engineering: University of Durham, 43 pp.

Delano, H. L. & Wilshusen, J. P., 2001. Landslides in Pennsylvania: Pennsylvania. Geological Survey, 4th ser., Educational Series 9, 34 pp.

Ekanayake, J. C. & Phillips, C. J., 2002. Slope stability thresholds for vegetated hillslopes: a composite model. Canadian Geotechnical Journal, 39, pp. 849–862.

El-Ramly, H., Morgenstein, N. R. & Cruden D. M., 2002. Probabilistic slope stability analysis for practice. Canadian Geotechnical Journal, 39, pp. 665–683.

Fletcher, L., Hungr, O., Evans, S. G., 2002. Contrasting failure behaviour of two large landslides in clay and silt. Canadian Geotechnical Journal, 39, pp. 46-62.

Griffiths, D. V. & Fenton, G. A., 2004. Probabilistic slope stability analysis by finite elements. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 130, No. 5, pp. 507-518.

Jaaniso, V. & Oll, K., 1997. Nõrgalt ületihenenud savi sekundaarsest konsolidatsioonist. Eesti XI geotehnika konverents, artiklite kogumik, Estonian Geotechnical Society, pp. 16-20.

Talviste, 2000. Report No. 00-05-0001. Pärnu ja Sauga jõe kallaste püsivus. Geotechnical report, manuscript at IPT Projektijuhtimine.

Sedman, P, 2002. Report No. 02-05-0162. Audru maalihe. Geotechnical report, manuscript at IPT Projektijuhtimine.

Talviste,P., 2004. Report No. 04-04-0365. Pärnu survelise veehorisondi veetaseme monitooring aastatel 1993-2004. Monitooringu andmed ja vaatlusread. Geotechnical report, manuscript at IPT Projektijuhtimine.

Jowsey, P.C., 1966. An improved peat sampler. New Phytol. 65, pp. 489-493.

Kaljund, E. & Mets, M., 1976. Pärnu sillaäärse kvartali hoonete deformatsioonid. Ehitusgeoloogia kogumik III. Tallinn, pp. 61 – 63

Kaljund, E., 1967. Maalihe Pärnus 1966. aastal. Ehitusgeoloogia kogumik II. Tallinn, pp. 61 – 63.

Kalm, V., Hang, T., Rosentau, A., Talviste, P. & Kohv, M., 2002. Maalihked Pärnu maakonnas. Project report, manuscript at the Institute of geology, Tartu University, 45 pp. (In Estonian)

Karise, V. & Lemberg,U., 1976. Pinnasevee agressiivsus Eesti NSV asulates. Ehitusgeoloogia kogumik III, Tallinn, pp.112-115.

Karise, V.,1967, Eesti pinnasevee looduslik agressiivsus. Geotehnika kogumik II, Tallinn, pp. 47-53.

Kattel, J., 1989. Pärnu viirsavide ruumiline muutlikus. Bac. thesis, manuscript at the Institute of geology, Tartu University, 51 pp.

Laouafa, F. & Darve, F., 2002. Modelling of slope failure by a material instability mechanism. Computers and Geotechnics, 29, pp. 301–325.

Malkawi, A. I. H., Hassan, W. F. & Abdulla, F. A., 2000. Uncertainty and reliability analysis applied to slope stability. Structural Safety, 22, pp. 161-185.

Mesri, G. & Shahien, M., 2003. Residual Shear Strength Mobilized in First-Time Slope Failures. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 1, pp. 12- 31

Mets, M. & Vilo, A., 1976. Nõrga savipinnase nihketugevus. Geotehnika kogumik III, Tallinn, pp. 28-35.

Mets, K., 2005, Merevee keemiline “jälg” viirsavides. Bac. thesis, manuscript at the Institute of geology, Tartu University, 27 pp.

Miidel, A. & Raukas, A., 2004. Mass movements related to the Baltic Klint, Estonia. Risks caused by the geodynamic phenomena in Europe, Abstracts and Field-trip Guideboook. Polish geological institute, pp. 29-31.

Muller, J. R., Martel, S. J., 2000. Numerical Models of Translational Landslide Rupture Surface Growth. Pure and Applied Geophysics, 157, pp. 1009-1038.

Nelke, P., Mets, M., Talviste, P., 1997. Savipinnaste tihenemine ja nõlvapüsivus Pakterminalis. Eesti XI geotehnika konverents, artiklite kogumik. Estonian Geotechnical Society, Tallinn, pp. 66-73.

Olli, V., 1962. Eesti NSV pinnastest. Ehitusgeoloogia kogumik I, Tallinn, pp. 4-14.

Olli, V. ja Martin, L., 1961, Maalihetest Lääne Eesti viirsavides. ENSV TA Geoloogia instituudi uurimused, VI, Tallinn, pp. 227-234.

Reinson, R., 2005.Pärnu viirsavide litoloogiast. Bac. thesis, manuscript at the Institute of geology, Tartu University, 47 pp.

Roering, J. J., Schmidt, K. M., Stock, J. D., Dietrich, W. E.,Montogomery, D. R., 2003. Shallow landsliding, root reinforcement, and the spatial distribution of the trees in the Oregon Coast

Range. Canadian Geotechnical Journal, 40, pp. 237-253.

Rocscience Inc, Slide v5.0 help file

Sjöberg, J., 1996. Large scale slope stability in open pit mining – a review. Technical report, Luleå University of Technology, 229 pp.

Zhu, D. Y., Lee, C. F. & Young, H. D., 2003. Generalized framework of limited equilibrium methods for slope stability. Geotechnique, Vol. 53, 4, pp. 377 – 395.

Talviste, P., 2002. Savipinnaste dreenimata nihketugevus ja kriitilise seisundi teooria. XII Eesti geotehnika konverents, artiklid. Estonian Geotechnical Society, Tallinn, pp. 23-31.

Tavast, E. & Raukas, A. 1982. The bedrock relief of Estonia. Valgus, Tallinn, 194 pp.

Tiande, M., Chongwu, M., Wu, S., 1999. Evolution model of the progressive failure of landslides. Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 10, pp. 822-831.

Vahter, E., 2003. Audru maalihe. Teeleht, No. 3 (31), pp. 11-12.

Wang, G., Sassa, K., 2003. Pore-pressure generation and movement of rainfall-induced landslides: effects of grain size and fine-particle content. Engineering geology, 69, pp. 109-12.

Veski,S., Talviste,P., Hang,T., Rosentau,A. & Heinsalu,A. 2005. Formation of landscape. In David,E. & Lõugas,L. (Eds) Pulli. The oldest Mesolithic site of Estonia (IXth mill.BC) (in print)

Vilo, A., 1986. Ehitusgeoloogia. University of Tartu, 109 pp.

Võrk, E. & Vilo, A., 1977. Lääne-Eesti nõrkade savipinnaste geotehnilised omadused. Geotehnika kogumik IV, Tallinn, pp. 21-35.

Internet webpages:

http://www.aquaticresearch.com/russian_peat_borer_files/Aquatic%20Research%20Instruments%20Russian%20Peat%20Borer.htm (20.05.05)

http://www.rocscience.com (20.05.05)

http://www.sloleht.ee/index.aspx?id=118339 (20.05.05)

Appendix 1. Analysis done by the Geotechnical laboratory of the EERC