FAILURES AT KIMOLA FLOATING CANAL IN SOUTHERN FINLAND

GLISSEMENTS OBSERVES AU CANAL DE FLOTTAGE DE KIMOLA DANS LE SUD DE LA FINLANDE

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SYNOPSIS The paper deals with the failures of the slopes of the Kimola floating canal excavated in soft slightly overconsolidated glacial clay. Most attention is payed to the great failure of November 3rd 1965, because the soil properties and pore pressures in the centre of the failed area were known before the slide. Calculations based on Bishops method give a safety factor close to unity at the time of the failure whereas the conventional ϕ = o method has proved to be unreliable.

INTRODUCTION

On November 3rd 1965, a great failure occurred at Kimola floating canal in southern Finland. In a few seconds about 90.000 m^3 clay slid into the canal, which had been excavated to its final depth in the previous winter, and about 200 m of it became blocked. Many minor slips had occured earlier in the same area. They necessiated investigations to check the design of the canal. One of the most important control sections happened to be near the centre of the failure area. By help of these investigations the instability of the slope in the actual area during wet seasons had preliminary been calculated before the slide. Later on the investigations have been continued in other sections and more minor slips have occurred in other parts of the canal (upper canal).

1.THE SITE AND CONSTRUCTION OF THE CANAL

Kimola floating canal is located at the southern border of the Mid-Finnish lake district about 120 km northeast from Helsinki as shown in figure 1. It connects the lakes of Konnivesi and Pyhäjärvi and its only purpose is to give effect to the floating of timber in the southern part of the Päijänne lake system.

The length of the canal is 5.5 km. The longitudinal section in fig.2 shows schematically the soils which the canal is cutting, some of their properties and also the site of the failure. According to it the canal consists of two parts, the upper canal and the lower one and a timber lift between them.

The excavation of the canal began in the winter of 1962. In the failure area the clay was removed in two stages. The first stage, level 67m, was reached in the winter of 1963 and the second (last) one in the winter of 1965. In the clayey part of the upper canal the excavation was performed in four lifts.

Fig.1 The Site of Kimola Canal

The total amount of soil removed was about 1.5 million m^3, from which about 85% was clay. In the failure area the depth of the cut was 12 m and in the upper canal 14 .. 16 m.

2. DESIGN PRINCIPLES OF THE CANAL

The major part of the cut is made in fat, lightly overconsolidated glacial clay. The stability of the slopes is calculated using the conventional ϕ = o method and the factor of safety $F_{\phi=o} \geq 1.5$ for empty canal. This could be reached only by making large unloading cuts in the upper part of the slopes. The final shape of the cross section thus obtained with unloading cut, upper slope of 1:1.5,terrace (canal way) and lower, partly submerged, slope of 1:2 are shown in figure 3. The undrained shear strength was mainly determined by vane tests and marked τ_f^v in this paper.

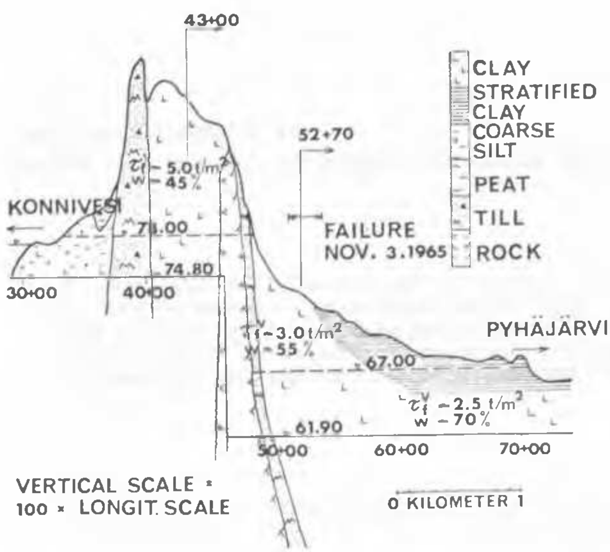

Fig.2 Longitudinal Section of Kimola Canal

3. FAILURES

Immediately after the excavation in the upper part of the lower canal had reached the first stage, the level of 67 m, many minor slips became apparent in the slopes above the terrace. Economically they were insignificant, not exceeding 200...300 m^3, but caused investigations to check the reliability of the $\phi = o$ analysis in this special case. Following investigations, mainly at station 52 + 70, were started:

Pore pressure measurements, started in July 1963 and interrupted by the failure on November 3rd 1965.

Vane tests to determine the effect of vane shape.

Determinations of after-peak values of vane strength.

Laboratory tests mainly in order to determine the effective shear strength parameters, c' and ϕ'.

These tests were carried out in the years 1964 ... 1965.

The large failure, on November 1965, occurred between stations 51 + 80 ... 54 + 20. Its length along the canal was 240 m and its width 80 m. At that time the canal had been in its final depth for nine months.

The slide was preceded by a rainy week, during which the precipitation was about 40 mm. Half of this came during the last twenty-four hours before the slide. In this connection it should be mentioned that all failures in the slopes of this canal in the years 1963...1968 have either been preceded by the thawing of snow or happened at a time during which the weekly precipitation has exeeded 40 mm.

The failure occurred in broad daylight. Apparently the whole mass of clay slid at the same time. Two workers in the vicinity could hear the noise of the moving soil and see a large flood wave rising from the canal.

Simultaneously they could see some trees fall down in the upper border of the failure area. According to them the duration of the slide was "only five seconds". They could not, however, see whether the slide was a single movement, or if there were many rapid failures.

The contures of the failure and schematic cross sections showing the position of the

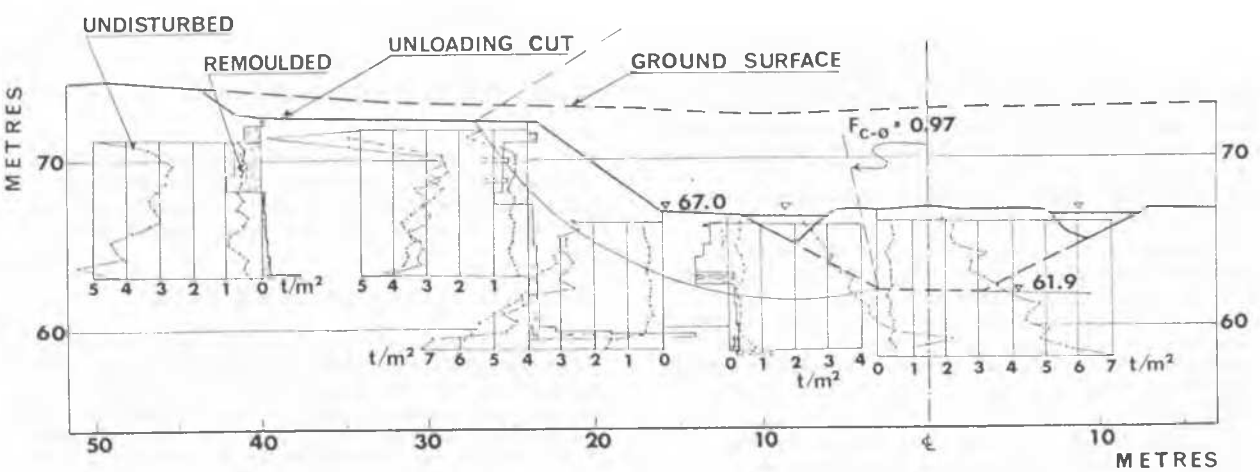

Fig.3 Cross Section with Weight Soundings and Vane Borings Showing the Left Bank of the Canal at Station 52 + 70 (before the Slide)

clay masses before and after the slide, are presented in figure 4. Figure 3 with vane borings and weight soundings illustrates the slope before the failure and figure 5 its condition immediately after the failure. Figure 6 is a photograph taken from the upper border of the slide area.

Geotechnical investigations were continued after the failure. A new control section with pore pressure measurements etc. was started at station 43 + 00 (upper canal).

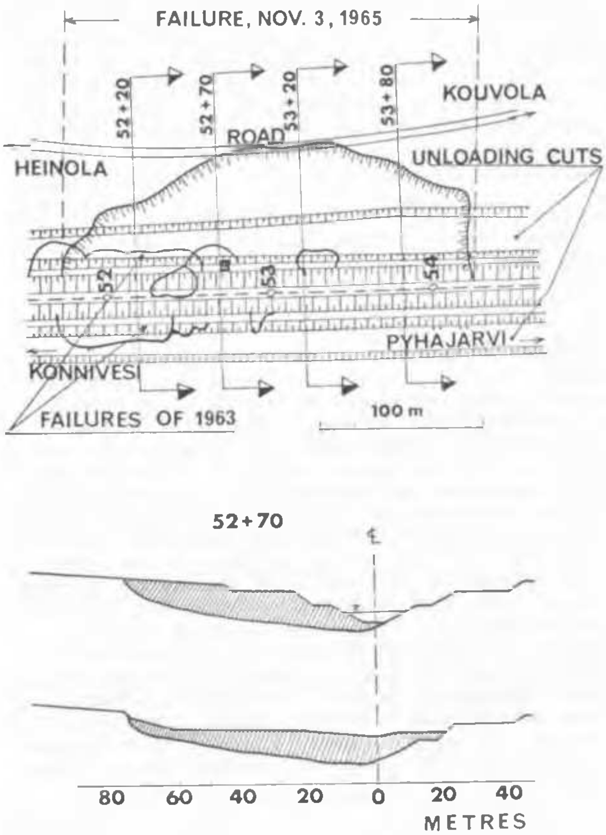

Fig.4 Map and Schematic Cross Section of the Failure of November 1965

4. GEOTECHNICAL PROPERTIES OF THE SOILS IN THE CANAL AREA

The soils in the canal and in the failure area are a part of a large deposit of glacial clay sedimented in a long but rather narrow valley. The geotechnical properties of the clays in the upper part of the lower canal (failure area) and in the upper canal are shown in table 1. The figures are median values of all test results.

Table 1 Geotechnical Properties of the Clays in the Canal Area

Site	Clay %	w %	w_L	w_P	S_t	τ_f^v kg/cm^2
Failure area	58	53	53	26	16	0.30
Upper Canal	51	44	54	23	8	0.50

The clays in the canal area are slightly over consolidated. In the failure area the overconsolidation ratio p_c/p_o=1.5 ... 2.0 and in the upper canal 2.5 ... 3.0. (pc = consolidation pressure, po = effective overburden pressure).

The undrained shear strength (vane strength τ_f^v) is furthermore illustrated by figure 7, a and b, which shows cumulative frequency curves of vane strength and after-peak values of it. The arrows in the figure indicate shear stresses in the critical slip circles calculated by ϕ = o method.

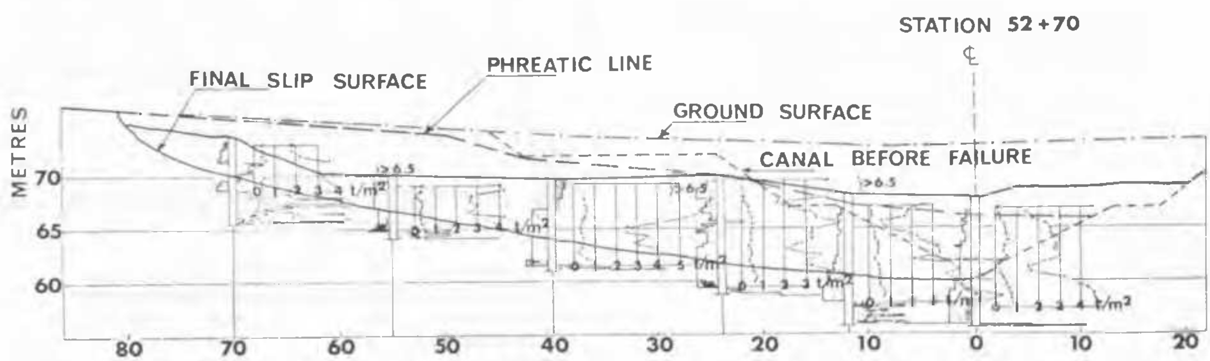

Fig.5 Cross Section from the Failure Area after the Slide, Station 52 + 70

Fig. 6 Photograph Showing the Upper Part of the Failure

5. SPECIAL INVESTIGATIONS

5.1. EFFECT OF VANE SHAPE

The conventional vane borings in the failure area (section 52 + 70, figure 3) are made by a vane with dimensions 5.5 × 11.0 cm (diameter D × hight, H). In order to obtain information of the vertical and horizontal components of undrained shear strength, additional borings were performed (year 1964) using vanes 6.6 × 6.6 cm and 7.8 × 3.9 cm (D×H) in five points in the section 52 + 70 (before the failure).

In each point the maximum shearing moment was measured by different vanes at every 0.5 m in vertical direction and the average moments, for each borehole and for all three vane types were calculated. From the results the average horizontal and vertical shear strength components τ_f^{vh} and τ_f^{vv} were calculated by the method given by Aås (1965).

The results are rather sensitive for the distribution of shear stresses in the end surfaces of the vane. Table 2 shows the ratio τ_f^{vh}/τ_f^{vv} in different boreholes. In column 1 even distribution is assumed and in column 2 triangular.

Table 2 Results of Investigations Concerning the Effect of Vane Shape. Station 52 + 70.

Site of borehole	$\tau_f^{vh} / \tau_f^{vv}$	
	1 Even	2 Triangular
12 m.left fr. cl.	0.89	1.18
16 m. "	0.78	1.04
20 m. "	0.96	1.28
24 m. "	0.88	1.17
40 m. "	0.60	0.80
Average	0.82	1.09

It appears from table 2 that if an even distribution of shear stress in the ends of the failure envelope is assumed, the average horizontal shear strength $\tau_f^{vh} = 0.82\ \tau_f^{vv}$. A triangular distribution gives an average ratio 1.09, which is a more reasonable value and in good agreement with the fall-cone tests carried out on both vertical and horizontal faces of undisturbed samples from the same station. They did not give any significant difference of strength in vertical and horizontal direction. These values equal to or less than unity are an indication of overconsolidation(Jakobsson 1955, Aas 1965) observed in compression tests, too. Another explanation could be stratification (Soveri & Hyyppä 1959), which, however, has not been observed.

5.2 AFTER-PEAK VALUES OF VANE STRENGTH

The many slips, which have occurred in the slopes of this canal, indicate that the values of shear strength obtained by conventional vane borings, where the points of maximum shearing resistance were used in determination of the shear strength, were misleading. Therefore an attempt was made to check if the strength would show any "residual value", if the rotation of the vane was continued over the peak value of the torque. Two determinations of that kind were done at the site of the failure of November 1965 before its occurrence. They were carried out at station 52 + 70 16 m and 25 m to the left from the centre line at every 0.5 m in vertical direction in both boreholes. Afterwards additional determinations have been made in the upper canal (station 42 + 00 and 43 + 00) in the vicinity of many minor slips.

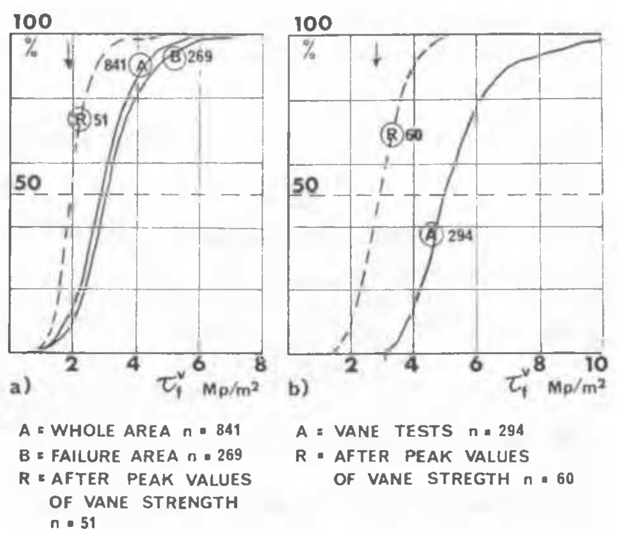

Fig. 7 Cumulative Frequency Carves of Shear Strength
a) Upper Part of Lower Canal
b) Upper Canal

The after-peak values thus obtained are presented in form of cumulative frequency curves in figure 7 and from station 25 m left from the center line in shear stress... vane rotation coordinate system (the torsion angle of the extension rods is eliminated) in figure 8. From the last mentioned it appears clear how the shear stress after the maximum point first decreases rapidly. When the rotation angle, however, has grown to 70 ... 80 degrees it approaches a rather constant value, which has been defined as the after-peak value, τ_f^{vr}, of the shear strength. Its average relative magnitudes compared with the peak values ($r = \tau_f^{vr} / \tau_f^{v} \times 100\%$), angles corresponding to the peak values, number of tests in each borehole, n, and standard deviations of the determinations are shown in table 3. In calculation of α both the torsion of the extension rods and the initial disturbance in the beginning of many curves (fig.8) are eliminated.

Table 3 Results of the Determination of the After-Peak Values of Shear Strength

Station	Borehole left fr.cl.	n	α degrees	r %
42 + 00	15 m	16	15.3 ± 2.3	52 ± 3
- " -	39 m	17	12.9 ± 3.8	57 ± 7
43 + 00	14 m	13	16.2 ± 3.3	59 ± 15
- " -	34 m	14	18.1 ± 3.7	56 ± 12
52 + 70	16 m	13	11.5 ± 2.1	46 ± 7
- " -	24 m	13	11.3 ± 2.0	49 ± 8
x/ 53 + 20	20 m	14	23.6 ± 6.2	39 ± 5
x/ - " -	40 m	11	15.8 ± 3.2	44 ± 8

x/ Failure Area, Tests Made After the Failure

It should be noted that the median of the after-peak values in the failure area (fig.7) is 1.9t/m², which is quite the same as the calculated shear stress in the critical slip circle in the same area calculated by $\phi = 0$ method.

5.3 PORE PRESSURE MEASUREMENTS

In order to obtain information of the pore pressure ten pietzometers were installed in the slope at the station 52 + 70. Their location is shown in figures 9 and 10.

The filter tips, which had an area of 226 cm² and a diameter of 3.3 cm, were pushed into the slope using steel tubes with a diameter of 3.4 cm. The measuring unit was mercury manometer similar to that generally used in the triaxial apparatus. The manometers were installed in a cabin erected on the terrace at level 67.

The measurements began on July 13th 1964 and they were continued uninterrupted to November 3rd 1965 at which time the cabin was completely destroyed by the failure.

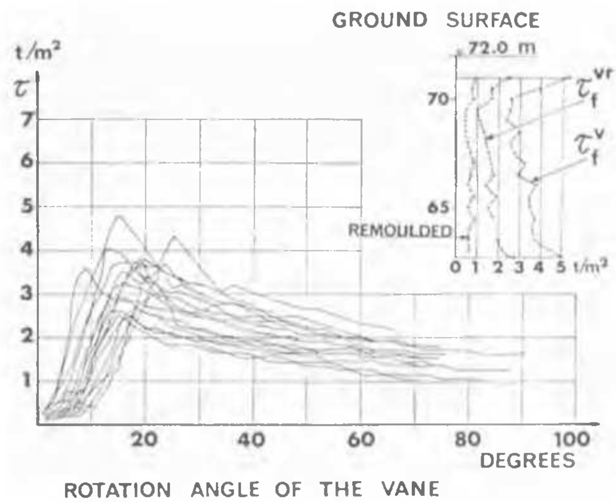

Fig. 8 Results of the Measurements of the After-Peak Values of Shear Strength

The results of the measurements are presented in figure 9. They show clear seasonal pressure variations. The largest values were measured in springtime during the melting of snow and in autumns, when rains, eventually combined with thawing, caused sharp pressure peaks. The lowest values were observed in midwinter and in summer. The largest variations, about 0.3 kg/cm², were measured in the upper part of the slope. The piezometers located in the toe of the slope follow closely the variations of the water level in the canal.

For stability calculations the measurements were plotted as equipotential lines of the pore pressure. One of them is shown in figure 10, which presents the last measurements before the slide. During the spring and autumn they indicate an appreciable pressure gradient towards the canal and often simultaneously also towards the glacial till under it.

In the end of January 1965 the piezometers showed a sudden drop of the pressure. At that time the canal was excavated from the level of 67 m(terrace) to its final depth, the level of 61.9 m. This was done by a dragline and keeping the water away from the canal. The lines of equal changes of pore pressure, Δu, are shown in figure 1. It shows, of course, that the effect of the unloading was biggest near the toe of the slope and hardly visible on the top of it.

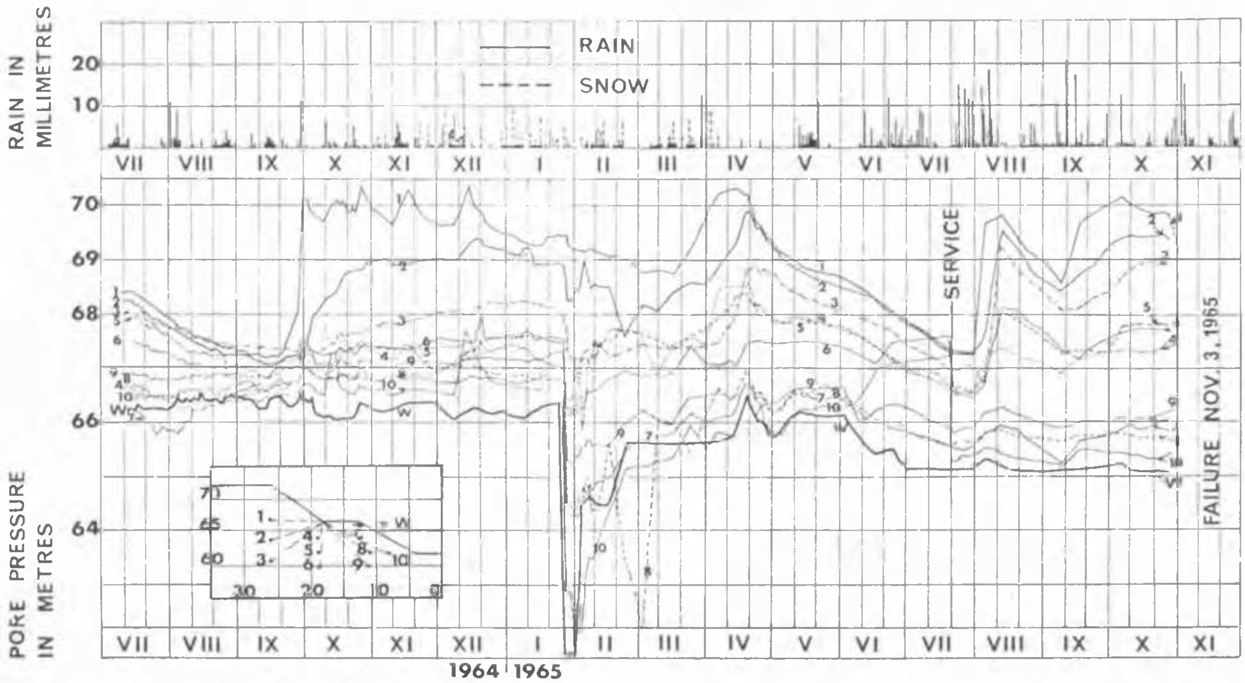

Fig. 9 Results of Pore Pressure Measurements at Station 52 + 70

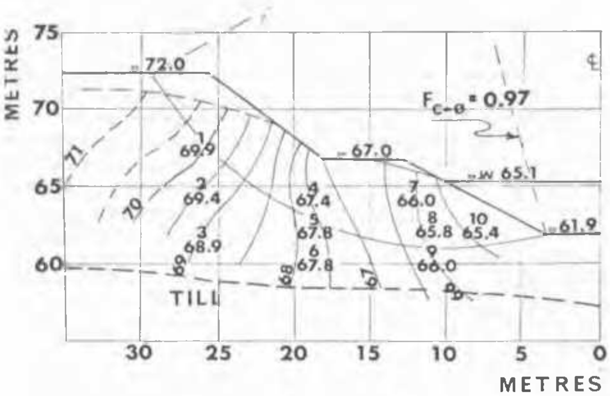

Fig. 10 Equipotential Lines of Pore Pressure at Station 52 + 70, October 27th, 1965

The last readings before the piezometers were destroyed by the slide on November 3rd 1965 (figure 10) were done on October 27th. The meters were read once more on November 2nd , but the results disappeared with the measuring cabin during the slide next day. The man, who had for a long time done the readings, could, however, remember that the piezometer number 1. which was pushed closest to the top of the slope, in spite of rains showed an appreciable lower pressure than before. It indicates that deformations occurred in the upper part of the slope before the failure.

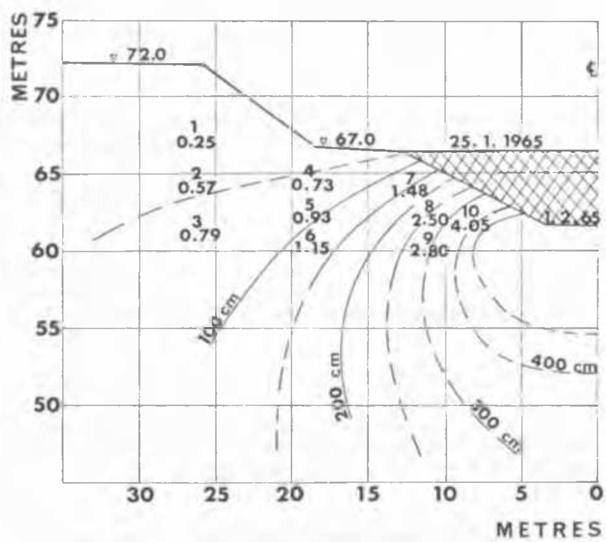

Fig. 11 Lines of Equal Change of Pore Pressure due to the Rapid Excavation

5.4. TRIAXIAL TESTS

In order to determine the effective shear strength parameters, c' and ϕ', many series of triaxial tests were performed. Initially consolidated undrained tests (CIU-tests) were carried out. Afterwards in order to check the results, two series of drained tests (CD-tests) were done, one with samples taken from the upper part of the lower canal (failure area) and the other one with samples taken from the upper canal. In all CIU-tests both $(\sigma_1-\sigma_3)$ max. and (σ_1'/σ_3')max. failure criteria were used. In CD-tests they coincide. The results of all tests in one

Table 4 Results of Triaxial Tests

Station	Borehole metres from the centre line	CIU-tests								CD-tests			
		$(\sigma_1-\sigma_3)$ max.				(σ_1'/σ_3') max.							
		n	r	c'	ϕ'	n	r	c'	ϕ	n	r	c'	ϕ'
43 + 00	33m left	28	0.96	0.174	22.1	24	0.99	0.117	27.3	15	1.00	0.122	27.6
50 + 20	38m right	11	0.97	0.031	22.1	8	0.98	0.022	26.7				
52 + 70	14m left	36	0.95	0.147	20.5	30	0.99	0.098	28.2				
52 + 70	24m left	15	0.98	0.070	25.7	13	1.00	0.107	27.8				
52 + 70	24m right									16	0.99	0.049	27.7
55 + 20	24m right	8	0.94	0.063	22.2	7	1.00	0.062	26.3				
55 + 70	16m left	26	0.99	0.098	25.7	25	1.00	0.083	28.7				
55 + 79	22m left	8	0.97	0.061	21.7								
62 + 80	16m left	23	0.93	0.105	19.0	20	0.98	0.084	27.0				
Total		155				127				31			

borehole were plotted as points in $1/2\,(\sigma_1'-\sigma_3')\ldots 1/2\,(\sigma_1'+\sigma_3')$ coordinate systems and connected with a right line calculated with the method of least squares. The values, c' and ϕ' thus obtained are shown in table 4. In the table c' is given in kg/cm^2, ϕ' in degrees, n is the number of tests and r the coefficient of linear correlation of the tests.

In CIU-tests the consolidation pressure varied between 0.25 ... 4.00 kg/cm^2 and in CD-tests between 0.10 ... 0.80 kg/cm^2, which did not exceed the effective consolidation pressure in the nature.

From table 4 one can easily see that $(\sigma_1-\sigma_3)$ max. failure criterion gives rather low values of ϕ' and that there are great differencies in the values of c' and ϕ' in different boreholes. If (σ_1'/σ_3')max. failure criterion is used, the values of ϕ' are practically the same as the corresponding values given by the CD-tests and appreciably greater than the values obtained with $(\sigma_1-\sigma_3)$ max. failure criterion. In the CIU-tests, the former failure criterion gives failure deformation between about 10...15% and the latter one about 5...10%.

A trend similar to that described above is observed for instance by Bjerrum & Simons (1960). In the final stability calculations the values of c' and ϕ' obtained by CD-tests were used.

All the investigations, except CD-triaxial tests, reported in this chapter (5) were available before the great slide of November 1965.

6. STABILITY CALCULATIONS

For stability calculations two sections, stations 52 + 70 and 43 + 00, with triaxial tests and pore pressure measurements were available. The former section was located in the centre of the area of the great failure and the latter section, still intact, in the vicinity of many minor slips. In c-ϕ analysis the Bishops (1955) simplified method was used. All calculations were made by an electronic computer (IBM-360). The results are presented in form of critical stress envelopes (Kenney 1966) in figure 12. They show that at station 52 + 70 the safety factor some days before the slide was 0.97 (figure 10) and in the previous spring 1.00. Compared with the final length of the failure the critical slip circle is, however, rather small (figures 3 and 5), which indicates that instead of one single movement there probably has been a rapid retrogressive failure.

In the upper canal the lowest calculated factor of safety is 1.16 (figure 12), which corresponds the pore pressure measurements shown in figure 13. The critical slip circle is rather small and of the same magnitude as the real failures occurred in the vicinity. In this section the clay is brittle and clearly overconsolidated and the real stability therefore may be lower than the calculated one (Skempton 1964).

Safety factors obtained by conventional ϕ = o method are 1.55 at station 52 + 70 and 1.40 at station 43 + 00. If instead of the shear strength the after-peak values of it are used the corresponding factors decrease to 0.83 and to about 0.90. The last value is not the lowest one which belongs to a very deep slip circle, but corresponds to the

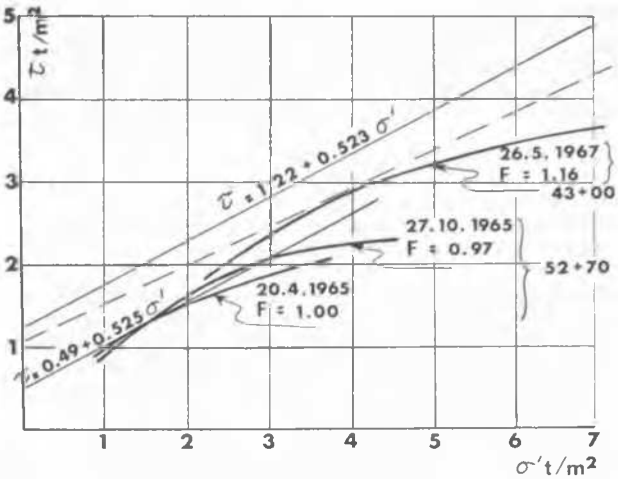

Fig. 12 Results of Stability Calculations at Station 52 + 70 (Failure of November 3rd, 1965) and 43 + 00 (Upper Canal, Stable Section)

most critical c-ø -circle, which is of the same magnitude and shape as the real slip circles in the vicinity.

The shear stress in the long slip surface in the centre of the failure of November 3rd 1965 (station 53 + 20) is practically the same as the after-peak values of shear strength measured at the same depth (at station 52 + 70 16 m and 25 m left from centre line). If a c-ø analysis is made using the long slip surface shown in figure 5 and using the drained values of c´and ø´, a safety factor $F_{c-ø}$=1.7 is obtained, which indicates that there has hardly been a single movement along this surface but a rapid retrogressive failure.

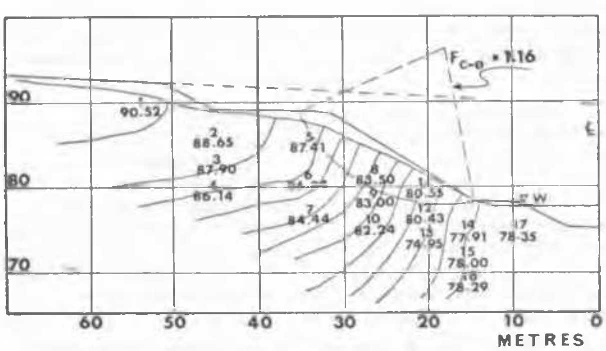

Fig. 13 Results of Pore Pressure Measurements at Station 43 + 00, May 26th 1967

7. CONCLUSIÓNS

The case presented in this paper demonstrates once again the unreliability of the conventional ø = o method for calculations of the long-term stability of slopes in fat slightly overconsolidated clays. If it is used, it seems to be necessary to check that $F_{ø=o} \geq 1$ also if the after-peak values of undrained (vane) shear strength are used.

The simplified c-ø analysis based on measured pore pressures has proved to be quite reliable.

The shear strength parameters c´and ø´, should be determined by CD-tests using consolidation pressures not exceeding the same pressure in the nature. If, however, CIU-tests are used, it seems to be necessary to check which one of the two common failure criteria $(\sigma_1' - \sigma_3')$ max. or (σ_1'/σ_3') max. yields same values as drained tests.

ACKNOWLEDGEMENTS

The investigations were made under the direction of the author during the construction of the canal by the Finnish Board of Roads and Waterways. The system for pore pressure measurements is designed by Mr. L. Pitkäkoski.

In addition valuable suggestions concerning both investigations and interpretation of the results are given by prof. K.V. Helenelund, prof. T.C.Kenney and Dr. L.Bjerrum, for which the author wishes to express his gratitude.

REFERENCES

Aas, G., 1965, A Study of the Effect of the Vane Shape and Rate of Strain on In-Situ Shear Strength of Clays, Proc. VI th Int. Conf. Soil Mech. and Found. Eng., Montriol, Vol. 1, p. 141...145.

Bishop, A., 1955, The Use of Slip Circle in the Stability Analysis, Geotechnique, Vol.5, No. 1, p.7...17.

Bjerrum, L., Simons, N., 1960, Comparison of Shear Strength Characteristics of Normally Consolidated Clays, ASCE Research Conf. Shear Strength of Cohesive Soils, Boulder, p. 711.. 726.

Jakobson, B., 1955, Isotropy of Clays. Geotechnique, Vol.5, No 1, p.23...28.

Kenney, T.C., 1967, Slide Behaviour and Shear Resistance of a Quick Clay Determined from a Study of the Landslide at Selnes, Norway, Proc. Geotechn. Conf. Oslo, Vol.1, p. 57...64.

Skempton, A.W., 1964, Long-Term Stability of Clay Slopes. Geotechnique, Vol. 14, No. 2, p. 77...102.

Soveri, U., Hyyppä, J.M.I., 1959. Influence on Shear Strength of the Structure of Varved Glacial Sediments. (Orig. Finnish with English summary), Rakennusinsinööri 3, p.3...7.