A NEW APPROACH TO EVALUATION OF CLAYEY SLOPES DEFORMATIONS

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ABSTRACT

New approach to deformations and stability of open-pit slopes composed by clayey rocks is considered, based on consideration the lateral earth pressure as a main moving force and the swelling process followed by shearing deformations as a main factor of reducing the rock strength along weak contacts at the early stages of clayey slope failure. Slope landslides in consolidated clays occur along the weak shearing surface at the bottom of the slope. Critical height of the slope may be estimated by $h = 2c/(\xi\gamma \tan\alpha - \gamma' \tan\varphi)$, where ξ is the coefficient of lateral pressure; $\xi = \nu/(1-\nu)$; ν is the Poisson's ratio; γ is the unit weight; c is the cohesion; φ is the angle of internal friction of clay; γ' is submerged unit weight; $\gamma' = \gamma - \gamma_o$; γ_o is the unit weight of water; α is the angle of the slope. In the case where the critical parameters of the slope are exceeded, the process of horizontal sliding surface development will extend deep into rock the mass to the distance L from the slope toe, which is equal to $L = h(\xi\gamma \tan\alpha - \gamma' \tan\varphi)/(2\gamma' \tan\varphi \tan\alpha)$. As a result of swelling of the clay at the weak horizontal contact, an actual value of coefficient of friction $\tan\varphi_c$ becomes less than initial value ($\tan\varphi_c < \tan\varphi$). The failure of the open-pit slope in West Kazakhstan region with parameters $h=100$m, $\alpha=23°$, $\tan\varphi_c=0.05$, $L>2000$m showed that clay behaviour in the moment of rupture may be the same as that of brittle stiff rock.

INTRODUCTION

There are several large landslides on the open-pit slopes composed from clayey rocks in the eighties/nineties in the USSR. In order to understand reasons of occurrence of such catastrophic phenomena, that are beyond the current theory of slope stability, E. Galustian [1, 2, 3], assumed an effect of so called "gravitational wedge" that pushes the resistance prism into the mined-out space of the pit. Really, visible picture, at the first sight, resembles such a process: severe subsidence of the surface of the area near the slope with simultaneous advance the slope mass into the mined-out space as if a sinking wedge were moving into the intact mass and cutting off the slope body from the rest of rock mass. But more detailed analysis of kinematics of considered landslides shows that "wedging" is not a reason rather consequence and final stage of complex processes which followed taking off the weight load during excavation. The zone of gravitational wedge is, in fact, the zone of compression only at the first stage of the failure, but, becomes the zone of tension just after that moment, and the "wedge" itself does not sink into the dipper domains of the rock mass but gets to failure and moves, as debris, horizontally towards the pit, pushing the resistance prism into the worked out space even if it were a retaining wall. Observed subsidence of the surface of the active pressure prism is due to decrease in cohesion and thus limitation of ability to move as a rigid body. Supposed by the author [4, 5, 6] mechanical scheme of development of deformations of the open-pit slopes as a horizontal stress-relief in the rock mass around pit is applied below at the analysis of slope failure in the open pit No. 3 PGMK in Shevtshenko, West Kazakhstan.

Process of deformation of the rocks within the slope zone involves through several stages in particular strength mobilization and at the same time decrease of resistance along some weak elements of the structure, which contributes to the process of taking off the weight load. Firstly, in the area of the pit, vertical deformations occur and, immediately afterwards, horizontal displacements take place as a result of a lateral thrust. The horizontal deformations in the area close to the slope may lead to decrease shear resistance on an undersurface of clayey rock mass, and the rock block separated from below, will move towards the open-pit space, just as a specimen of clay, after removal of the load, rebounds through decompression. The rock mass within the slope (so called prism of resistance) is bending as a thick plate of variable rigidity, constrained at ends and from below, loaded by the lateral thrust initiated by excavation. When a limit equilibrium is attained, the softened rock in the slope may fail. The rock mass failure may be a smooth process if the strength of rocks along the sliding surface is determined by friction (or viscosity); this may be a dynamic process if cohesion is a main strength parameter; this may be a stream-like moving, if, because of high water pressure, there is no cohesion or friction in the loose soft rock constituting the slope.

THEORY OF THE SLOPE DISPLACEMENTS

Stresses

Horizontal displacements of the open pit slopes take place during excavation long before the slope failure, the lateral thrust forces being released. Rock mass being under the effect of the own weight, is a medium with the Poisson's ratio $\nu>0$, something average between an absolutely rigid body with the Poisson's ratio $\nu=0$, and a fluid with the Poisson's ratio $\nu=0.5$, and therefore, lateral stresses $\sigma_{x(y)}=\xi\sigma_z$ are accompanying the gravitational stress $\sigma_z=\gamma z$ (γ is the unit weight of the rock, z is the depth of the point under consideration, ξ is the factor of the lateral thrust). The values of the lateral thrust depend on boundary strain conditions: -in an "infinite" rock mass, far from the open pit, where lateral strains vanish ($\varepsilon_x=\varepsilon_y=0$), the lateral stresses are $\sigma_x=\sigma_y=\xi_0\gamma z$ with $\xi_0=\nu/(1-\nu)$; -in a semi-infinite rock mass uncovered along the y-axis, it is the strains ε_y that become zero($\varepsilon_x\#0$, $\varepsilon_y=0$), and the lateral stresses are $\sigma_x=\xi_x\gamma z$, $\sigma_y=\xi_y\gamma z$, where ξ_x and ξ_y are constants, depending on ν, $\xi_0>\xi_y>\xi_x$; -in a "quarter-infinite" rock mass, uncovered along both x- and y-axes, both ε_x and ε_y horizontal strains do not go to zero, and the lateral stresses are equal to zero in the wall whereas the stresses within the rock mass depend on the distance from the wall and ν. Here ν is the Poisson's ratio; $\xi_{x,(y)}=\sigma_{x,(y)}/\sigma_z$; ξ is so called coefficient of lateral pressure, axes x and y lie in a horizontal plane, z is a vertical coordinate axis.

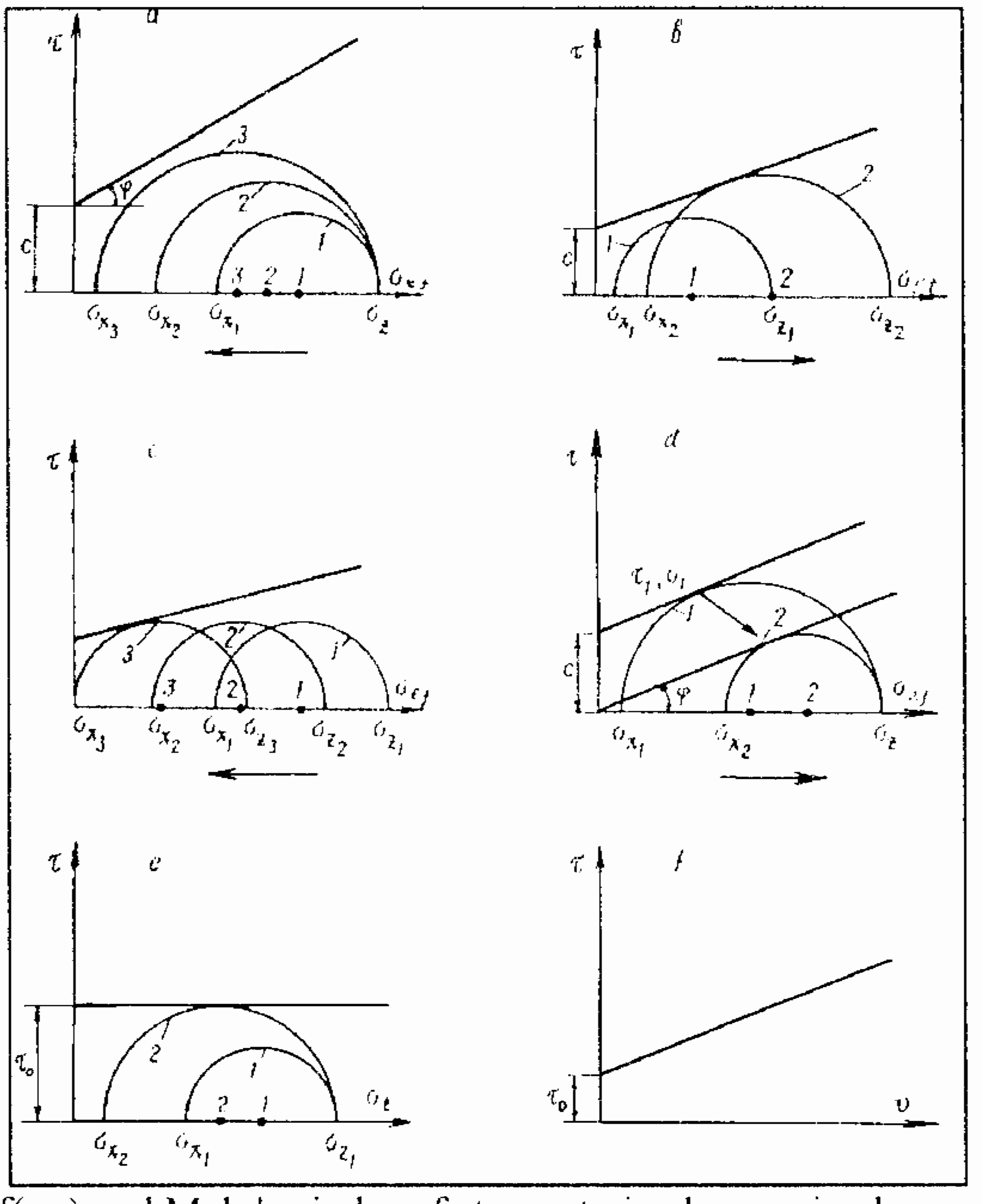

Fig. 1. Graph σ=f(τ) and Mohr's circles of stress-strain changes in clays composed the slope during: a)horizontal rebound; b)raise of the effective stress with lowering the water pressure; c)dropping the effective stress with raising water head; d)rupture at σ_{eff} >0; e)failure at σ_{eff}=0 (liquefaction); f)flow of viscous liquid.

Rock mass considered infinite in initial ("natural") conditions, as shown in Fig. 1a, Mohr's circle 1, becomes semi-infinite after driving of a trench along the y-axis. On the outline of the slope, removing the lateral thrust out of the mined-out space is equivalent to applying a tensile load with intensity $\sigma=\xi_0\gamma z$, which results in horizontal deformations and subsequent reduction of lateral thrust within the destressed strip ($0<x<L_x$) near the open-pit. Mohr's circle (Fig. 1a) changes from position 1 (σ_z ,σ_{x1}) to the position 2 (σ_z ,σ_{x2}) in the area near the trench, with $\sigma_{x1}>\sigma_{x2}$. If, additionally, a transverse trench is driven along the x-axis, the analogous process of rock mass deformations gives rise to stress-relieving along the strip near the open-pit, parallel to the y-axis ($0<y<L_y$). Thus, in the vicinity of the origin of the coordinates, the rock mass stress conditions approximate the conditions of one-dimensional compression, that is $\sigma_x=\sigma_y=0$ and $\sigma_z=\gamma z$ (Fig. 1a, circle 3).

Process of rock mass destressing is limiting by the compression strength of clayey rocks [σ]:

$$[\sigma]=2cb^{0,5}/(1-b\xi) \qquad (1)$$

where c is the cohesion, $b=(1+\sin\varphi)/(1-\sin\varphi)$, φ is the angle of friction. The failure of clayey rock takes place at $\sigma'_z=[\sigma]$, $\sigma_z'=\gamma' z$, (σ'_z is the effective stress, $\gamma'=(\gamma z-p)/z$, p is the water pressure at the depth z).

Removal of the weight load from the slope area and subsequent horizontal stress-relief in the clayey rock mass around the pit are accompanied by reduction of the water pressure. In conditions of arid dry climate, the clayey rock is prevented from swelling because of absence of precipitation, so that effective stresses within the slope area remain initial ones. As $\sigma'_{x1} < \sigma'_{xo}$, the Mohr's circle center displaces to the left (Fig. 1b), which may lead to the failure of the clay. If in some area of clayey rock mass the compression strength limit $[\sigma]$ is achieved and clay fails, the lateral thrust will increase drastically as it is shown in Fig. 1d. The intensity of deformations of rock mass, that is their magnitudes and the velocity of displacements, depends on the difference between the horizontal stresses before and after failure. In the regions with humid climate, clayey rock swells after relieving and restores its initial pore pressure conditions. In the case, that the failure of rock mass, composed by "sensitive" soft clays, is connected with the growth of pore pressure (Fig. 1c), the rock, after cohesion is lost, behaves like viscous liquid (Fig. 1e). On liquefaction, the velocity of the flow of the rock mass as a Newtonian liquid is $v=m_o/\mu(\tau-\tau_o)$, where m_o is the thickness of the sliding block of the rock mass, μ is the viscosity of the fluidized rock (Fig. 1f).

Deformations

Let us consider a pattern of slope deformations, where a vertical wall of height h_o is driven in clayey rock from the earth surface to a horizontal thin and soft parting of thickness m_o, whose strength parameters φ_c and c_c are less than those of a clay bed (φ and c). Then one may assume that a probable type of deformation of clayey rock mass near the wall will be sliding on a horizontal contact plane between the consolidated clay and the weak clayey parting toward the trench with expansion of moving rock mass even if it were a spring subjected by the tensile load. The distance L_x to that the deformations will advance into the rock mass may be determined by equating moving forces, that is, the lateral thrust forces distributed along a vertical cross-section, to resisting forces (strength), that is, friction and cohesion along the weak contact plane (Fig 2), using the following equations:

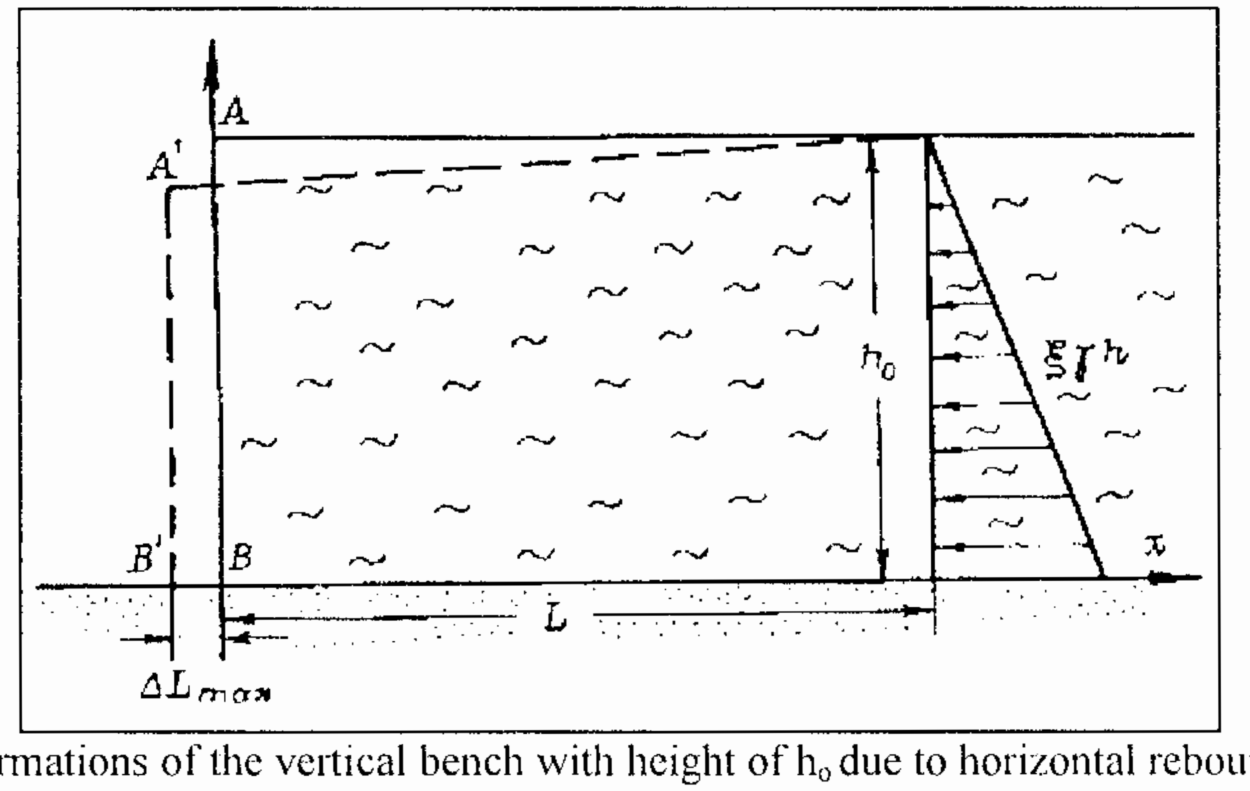

Fig. 2. Deformations of the vertical bench with height of h_o due to horizontal rebound; L_o is the length of influence zone; dL_{max} is the deflection of the crest of the wall.

$$\xi_x \gamma (h)^2 = c_c L_x + \gamma ' h_o L_x \tan \varphi_c \qquad (2)$$

$$L_x = \xi_x \gamma (h_o)^2 / [2(c_c + \gamma ' h_o \tan \varphi_c)] \qquad (3)$$

Horizontal displacement $dL_x(x)$ of the points within the $0<x<L_x$ zone near the trench may be obtained by:

$$dL_x(x) = \xi_x \gamma (L_x - x)^2 / (4EL_x) \qquad (4)$$

where E is the modules of elasticity of the clay. Maximum movement dL_{xmax} at the crest of the wall at x=0 is equal to:

$$dL_x = \xi_x \gamma h_o L_x / 4E) \qquad (5)$$

In the vicinity of the pit wall, where the shear stress on the weak contact exceeds the strength of the soft parting, its cohesion vanishes to zero and the deformation zone extends over the intact rock mass to the distance L_x':

$$L_x' = \xi_x \gamma h_o / (2 \gamma ' \tan \varphi_c) \qquad (6)$$

and the maximum value of displacement at a crest point, dL_{xmax}', will be

$$dL_{xmax}' = (\xi_x \gamma h_o)^2 / (8 \gamma ' E \tan \varphi_c) \qquad (7)$$

Formation of weak contact planes

Equations (2)-(7), obtained above, may be used to assess conditions of formation of a weak contact plane in homogeneous clayey rock mass uncovered by the trench. If the condition of sufficient strength of clay is fulfilled, that is, $[\sigma] > \sigma_z'$ (Eq. 1), and the limit equilibrium at the bottom points is reached according to the Coulomb equation

$$L_x c + \gamma ' h_o \tan \varphi = \xi_x \gamma h_o / 2 \qquad (8)$$

the zone the horizontal displacements may advance deeper into the rock mass to the distance L_x according to the following simple expression:

$$L_x = \xi_x \gamma h_o / (2 \gamma ' \tan \varphi) \qquad (9)$$

of which differs from Eq. (6) only in that φ_c is changed to φ. If the slope angle α is not equal to 90^o, one must take into consideration the weight of the prism of resistance on the slope:

$$L_x = (\xi_x \gamma - \tan \varphi / \tan \alpha) h_o / (2 \gamma ' \tan \varphi) \qquad (10)$$

Increase of porosity of clayey rock mass, accompanying the process of horizontal expansion, contributes to the inflow of the porous water to the area of contact parting of softening clay from neighboring layers, which leads to the growth of the humidity and to the further decrease of the strength of clay within the contact area. Thus, due to swelling of clay, the boundary of the influence of the open-pit extends into the rock mass, being homogeneous in native state, to the distance that corresponds to the case discussed above for the weak clay parting at the bottom of the pit, and hence may be evaluated using Eq. (6).

Brittle failure of clayey mass

After completion of formation of weakened zones at working floor levels of the open pit, the stability of the rock mass composed of stiff "overconsolidated" clays depends mainly on the strength of the "constraints" of the deforming slope body, that is, on the strength of the intact rocks situated along the boundary of the zone of stress relieving and at the flank sides of the slope. This strength is assumed to be sufficient to prevent further horizontal displacements whereas the high clayey wall may fail in the same manner as a specimen under compression loading. At the moment of rupture, the rock mass, losing cohesion, transforms from the coherent body with the lateral thrust near zero into the loose medium being affected by the vertical weight load and makes the slope body move in horizontal direction due to increase of the value of the coefficient of lateral thrust ξ that, in case of cohesionless rock, is equal to Terzaghi's coefficient of earth pressure ξ_r:

$$\xi_r = \tan^2(\pi/4 - \varphi/2) \qquad (11)$$

Horizontal component of the stress tensor $\sigma_x = \xi_r \gamma z$ within the failed part of the prism of active pressure is not in equilibrium with the resisting strength of the rock mass, and, hence it forms an active moving force to be applied to the prism of resistance. Stress relaxation occurs in the failed rocks essentially in the same way as in intact rocks with the exception of substantially increased magnitude of stress-strain modules because the compressibility of clayey debris is higher than that of clay itself.

ANALYSIS OF THE SLOPE FAILURE IN THE OPEN-PIT No. 3 IN KAZAKHSTAN

Objectives of study

An uranium ore deposit has been mined by the open-pit No. 3 using transportless mining system for the uppermost pit bench with a height of 10 to 17 metres and for the basic lowermost bench with a height of 30 metres. Intermediate benches, each 12.5 metres in height, are excavated using motor transport. A 1 metre ore bed is overlapped by a very thick (up to 80m) formation of Paleogene clays, of Neogene clays (up to 50m) and Quarternary sands (20m), and underlaid by Paleogene clays (200 to 300m) and Cretaceous sandstones. Paleogene clays are dense (γ=19kN/m^3; w=0.33; n=0.47); their cohesion is c=150-260 kPa, angle of friction is 14°. There are two aquifers in the rock mass under consideration: the upper in lenses of the Quarternary sands and the lower in Cretaceous sandstones with initial head of groundwater up to 100m above the ground. The ore field is situated in the West Kazakhstan in the region of arid climate with annual norm of precipitation of about 100mm and evaporation beyond 1m per year. The No. 3 open-pit develops a working front 2-3km in length in North-West direction, annual advance reaching 200m. A frontal working slope has the height 110m and an average angle 10^{o} , an unworking sides of the open pit have an angle of inclination about 16^{o}.

In December 1987, after shortening the length of the workings slope, its northern part, where the excavations had been stopped 6 months ago, slided into the mined-out space of the open pit. The section of the slope with a length of 950m, a height of 110m and an average angle of 22^{o} collapsed during some minutes, with the lower benches sliding over the distance of 150m and with the upper benches subsiding up to 60m [2]. Investigation of the reasons of the landslide begins with the surveying of the relief of the post-sliding surface, and after filling in the gap, is being continued through boring wells. Analyzing the obtained data (Table 1, Fig. 3), one can see that four lower benches moved as a unit block, translatorily, with an initial shape of the ground kept intact whereas the upper benches as well as the area adjacent to the slope which had been displaced immediately after the lower benches were distroyed, subsided and expanded.

Table 1. Deformations within the cross section V11-V11 (Fig. 3a) (maximum displacements)

Sect .No.	Parameters of the sections, m				Disp. , m	Strains		
	before sliding		after sliding			ε_x	ε_z	ε_y
	heigh t	width	height	width				
1	55	160	55	160	150	0	0	0
2	100	120	65	180	120	0.35	0.5	0.02
3	105	90	85	125	60	0.27	0.39	0.01
4	105	50	100	65	25	0.05	0.3	0.2
5	105	40	102	50	10	0.03	0.2	0.16
In all	-	460	-	580	-	-	0.17	-

Since there are no marker layers in homogenous clays of which the slope is composed, the survey data are referenced to the positions of intervals with extreme values of moisture content. In view of the fact that core data as well as visual observations show no distinct relationship between the variations in moisture content and lithology, it may be assumed that the position of anomalous moisture content zones is to be related to the position of working floors of the open pit before failure. According to the data obtained while drilling the boreholes on different sites of the landslide area (Fig. 4), extreme values (i. e. neighboring maxima and minima) of water content within the area adjacent to landslide as well as within the subsidence sections No. 5 and No. 4 (Table 1) may be traced to the following depths:

maxima: +8 m, +14 m, +25 m, +38 m(floor +40 m), +58 m(floor +55m), +63 m (floor +65 m);

minima: +4 m, +18 m, +60 m, +70 m.

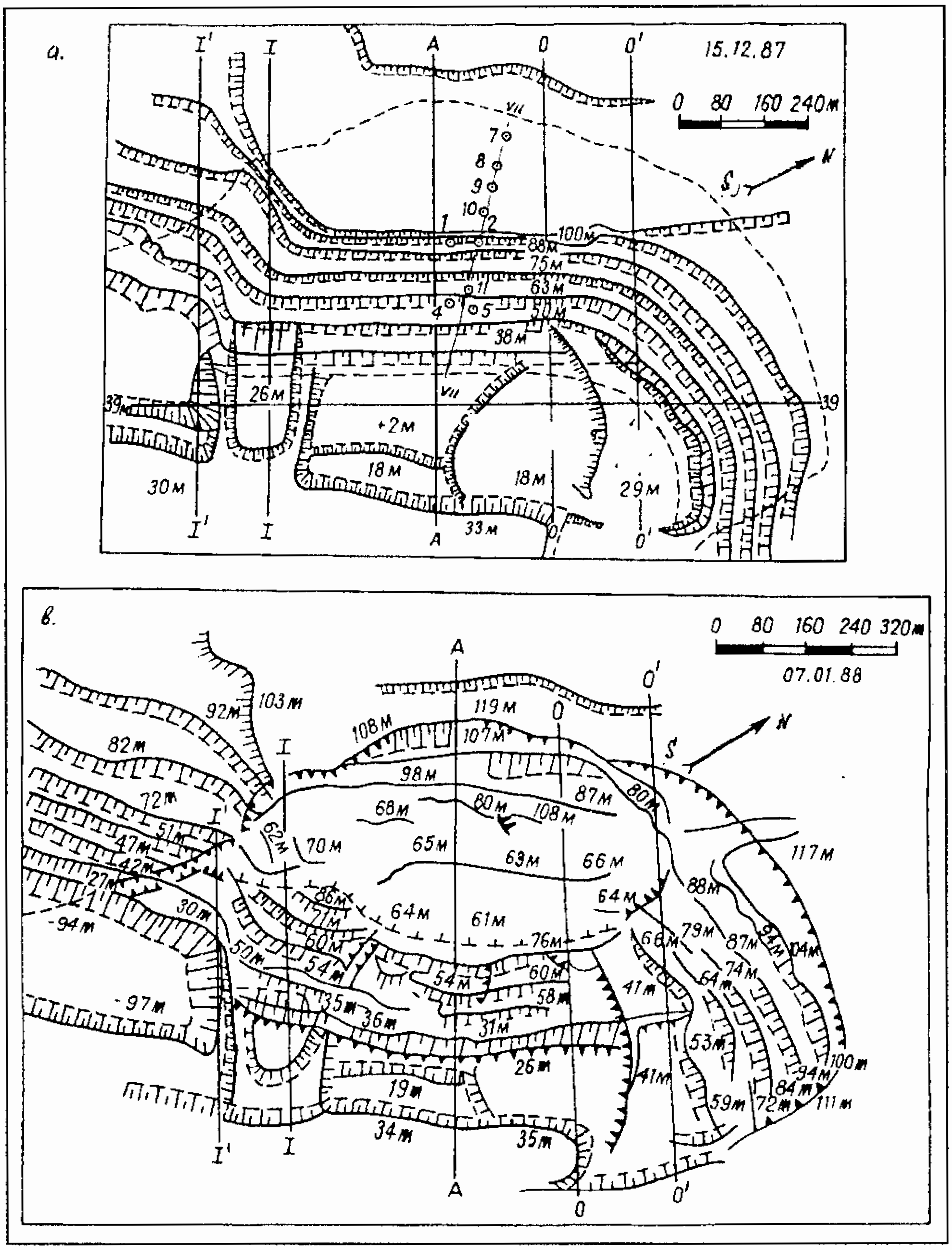

Fig. 3. Mining maps of the northern part of the western slope: a) 15. 12. 87 (before sliding); b) 07. 01. 88 (after sliding); 1-bench elevation; 2- boring hole.

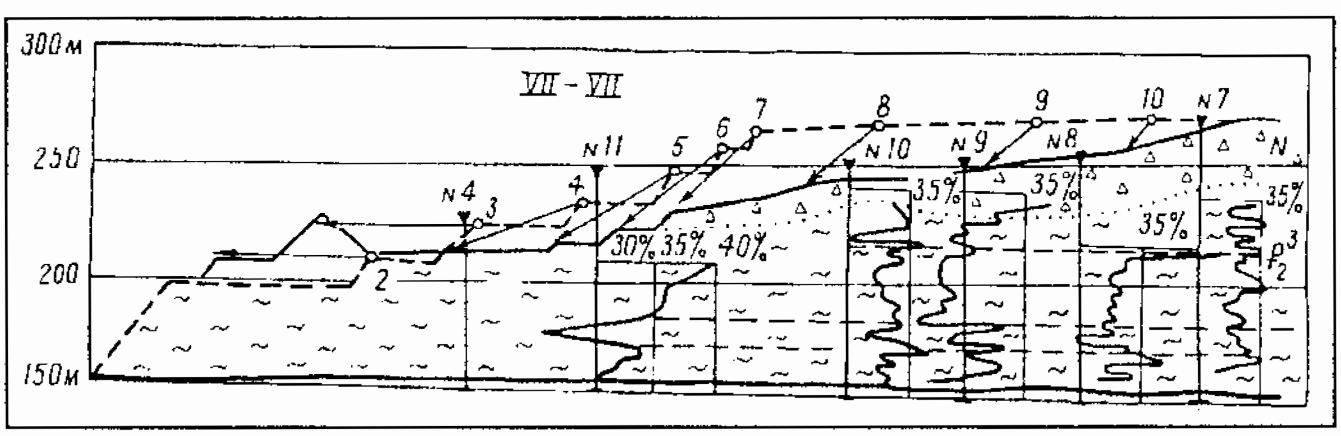

Fig. 4. Cross-section AA through the center of the slided slope. Value of 30% of water content graph coincides borehole axis; --- slope contour before sliding; ____ slope contour after sliding; ← - vector of displacement.

Within the prism of resistance (section No. 1), the positions of anomalous humidity zones are somewhat lower. Within the section No. 2 there is only one minimum of water content (+18m). Thus, we can ascertain that only the section No. 2 adjacent to the prism of resistance failed completely whereas the sections Nos. 3, 4, and 5 were deformed more smoothly. Just as the displacement values are varying across the strike of the slope, so they are varying with position of the open pit section in question in reference to the north end and the south end of the sliding slope, as indicated in Table 2.

Table 2. Deformations in the other cross-sections of the open pit

No.	l_s	h	α^o	w_r	w_p	w_l	dL_r
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
I'-I'	0	90	17	150	300	450	20
I-I	120	110	19	220	320	540	50
A-A	320	105	20	180	270	450	90
0-0	620	110	25	200	210	410	120
0'-0'	780	120	18	200	380	580	20

(1) a number of an open pit cross-section, (2) a distance from the south end of the pit wall, m. Parameters of a slope before sliding: (3) a height, m; (4)a dip angle, (°); (5) a width of resistance prism, m; (6)a width of pressure prism, m; (7)a width of a sliding area, m; (8)a displacement of resistance prism, m.

Additional geological investigations

To gain greater insight into the mechanism of a landslide a series of holes have been drilled since 1988 in the No. 3 open pit, involving determination of moisture content and strength parameters of clays, measurement of the earth surface displacement and rock movement along weak contacts, measurements of pore water pressure. The most important fact established from results of analysis of moisture content profiles is that the rock mass movement under the action of the lateral earth pressure plays a crucial role in forming the weak contact planes. As it is shown in Fig. 5, the zones of the anomalous, as compared to the background values, water content are encountered just strictly at the levels of all the open pit berms and may be followed deep into the rock mass. As a rule, the anomaly is recorded on a small vertical interval of depth as a maximum and its associated minima of water content. The plane of sliding of overlying rocks on underlying rocks is a plane of separation beyond which the horizontal stresses are abruptly increasing by $\xi\gamma h_r/2$ (h_r is the height of the open pit bench), and, at the same time, is a zone of maximum tensile strains, and hence area with a negative pore pressure that causes swelling of the clayey parting within it and dewatering of the adjacent rock mass. As it can be seen from the moisture graphs, there are discrepancies in monotonically decreasing values of the water content with increasing depth at the levels of berms, which is assumed to be attributed to sudden jumps of lateral strains. The jump in deformations, $d\varepsilon=\xi\gamma h_i/E$ (h_i the depth of an i-th berm), is related to origin of the sliding surface and exists further due to rebound of the rock mass moving into the pit. A potential (pressure) of water suction, p_c, which is equivalent to the strain, is given by

$$p_c = \gamma_o h_c = \beta\, d\varepsilon E = \beta\, \xi\, \gamma h_i \qquad (12)$$

where h_c is the height of the capillary rise, β is the coefficient of the load transmission to the pore water pressure, γ_o is the unit weight of water.

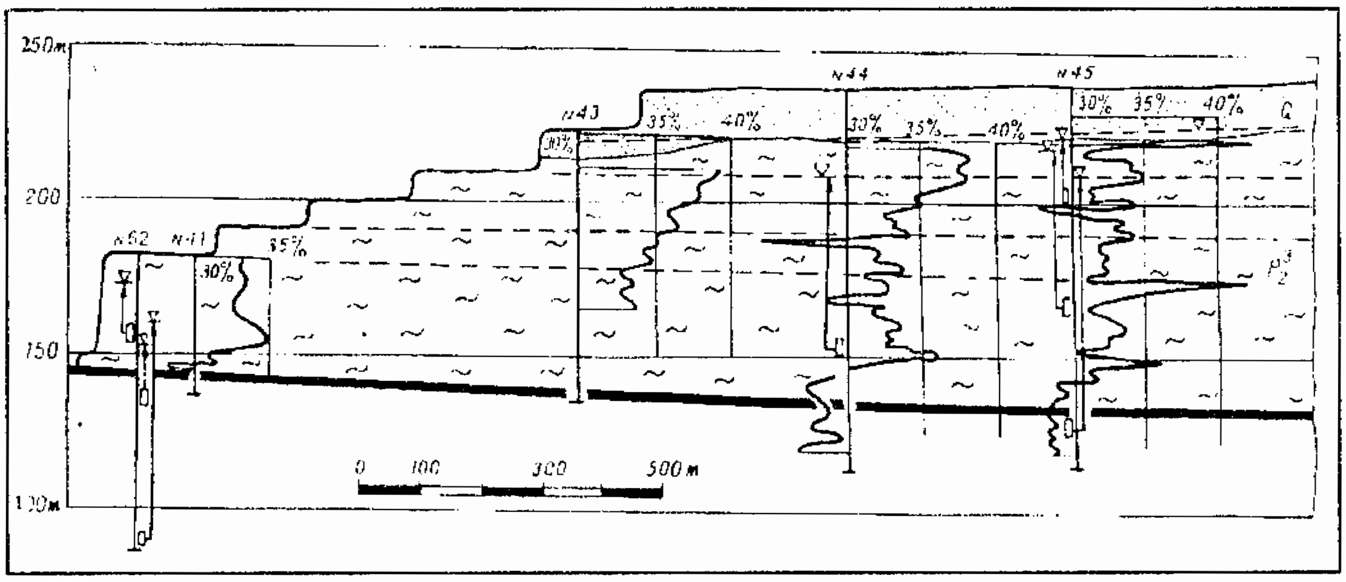

Fig. 5. Cross-section through the central part of the working slope of the open-pit No. 3 of PGMK. w(x) is the water content; ^ is the water level.

In the strata underlying the sliding surface, and in the lower part of overlying strata the water flow is arisen towards the area of the sliding surface which is a zone with a negative water potential. It looks like a pumping pore water into deforming small interval, that is to be recorded as a minima of water content in "consolidated" clays near its maximum which is characteristic of a shear stress zone. Process of pumping of he water into the swelling zone occurs at about constant boundary condition $S_o=h_{ci}$. Since the water intake from the constant source (Quarternary sands) is superimposed on the process of water content redistribution, the moisture profiles lose the sharpness of their peaks, and the average moisture is increasing in direction from the outer boundary of the zone of rock loosening to the open-pit as may be seen from the water content profiles. The increase of the average rock humidity is confirmed by analysis of the core specimens, obtained from boreholes situated at the distance 700m and 350m from the slope (No. 45 and No. 44, respectively), at the first pit floor (No. 41) and at the fifth pit floor (No. 62). Graphs w(x, t) have been plotted by using the method of type curves according to the formula:

$$dw = dw_o \operatorname{erfc}(x^2/4at) \qquad (13)$$

where x is the distance from the well No. 45, dw_o is the maximum reduction of the average rock humidity related to that obtained from the well No. 45; $a=k/\mu$; k is the coefficient of permeability; μ is the coefficient of storage for clays. According to calculations, there are $a=1m^2/d$ (No. 44) and $a=0.\ 44m^2/d$ (No. 43), and $dw_o=0.\ 055$ (No. 44) and $dw_o=0.\ 09$ (No. 43) provided that dw=0 at the well No. 45, where there is no effect of leakage from Quarternary sands. Assuming, further, the storage $\mu=5\times10^{-4}m^{-1}$ (No. 44) and $\mu=1\times10^{-3}\ m^{-1}$(No. 43), we obtain constant value of $k=5\times10^{-4}$ m/d and suction potentials $S_o=110m$ (No. 44) and $S_o=90m$ (No. 43), respectively. As

$S=p_c=dw_o/\mu$ is the drawdown of the pore water pressure due to the horizontal relieving of the rock mass, we may calculate the value of active porosity n_o from the equation [5]:

$$n_o=k_o h_c^2/22 \qquad (14)$$

here $h_c=p_c/\gamma_o$, h_c is being estimated in metres of water column and γ_o being as usual the unit weight of water. The obtained value of $n_o=0.25$ is half as much the average value of total porosity of clays. By the assuming the coefficient of friction of clayey rocks does not depend on the depth and that the well No. 45 is situated at the boundary of the upper sliding surface , one may evaluate the magnitudes L_i of other sliding surfaces (Table 3).

Table 3. Calculated values of L_i.

No.	Depth of sliding surface, m	Distance to the boundary of the swelling zone, m, from		
		a bench floor	the open-pit crest	the well No. 45
1	25	500	500	0
2	37	1100	750	250
3	47	1300	900	400
4	60	1700	1200	700
5	85	2000	1700	1200

The calculation was fulfilled according to Eq. (6) with $\xi=1$ and $\tan\varphi_c=0,05$.

Observations of the water heads have been conducted with using special water pressure transducers installed in the wells and separated by the intervals of swelling clays. The results of the observations (Table 4) have revealed an effect of stabilization of the water levels in wells No. 45 and No. 63 situated far from the open-pit slope, a slow lowering of the water level in the well No. 44 (5 metres per year) under influence of the approaching of the upper working floor to the well No. 44, and quick drop of the water pressure (14-24m during 2.5 months) in the well No. 62 that is bored lowermost open-pit bench with the height of 40 metres.

Table 4. Pore water pressure in clays

Well No.	Date	Depth, m, of the pressure gauge	Water pressure, m	Water level, m
62	26. 8. 91	110	68. 8	41. 2
	5. 11. 91		54. 6	55. 2
	26. 8. 91	40	22	18
	5. 11. 91		-0. 8	40. 8
	26. 8. 91	20	22	+2
	5. 11. 91		-2	22
44	30. 9. 90	86	72	14
	12. 12. 90		65. 8	20. 2
	28. 12. 90		60. 7	25. 3

45	26. 8. 91	105	78. 2	26. 8
	28. 12. 91		78. 2	26. 8
	26. 8. 91	70	53. 5	17. 5
	28. 12. 91		53. 6	17. 4
	28. 8. 91	35	18	17
	28. 12. 91		19. 1	15. 9
63	26. 8. 91	120	95	25
	28. 12. 91		94	26
	26. 8. 91	80	54. 9	25. 1
	28. 12. 91		54. 5	25. 5

Pressure of 1m water column is equal to 10kPa.

It should be noted, that because of the excavation, the vertical relieving and softening of the slope area itself takes place, but this process is not followed by the quick swelling of clays because of the lack of aquifers and because of hot climate that is responsible for the conditions of water evaporation from the surface of the slope.

Analysis of the landslide 1987

A stable North part of the working slope with an average height of 105m, a length along a open-pit crest of 1300m and a slope angle of 20°-22°, had been taken out of operation since July 1987. The mining operation were being continued only on the south half of the working slope. By December 1987, the south-east end of this inactive part of the slope was cross-strike cut off some distance deep into the rock mass (i. e. 350 metres at the upper floor, 150 metres at the +50m middle floor, and 120 metres at the +4-m ore floor). At the bottom of pit, a waste-rock dump was formed along the stopped part of the nonoperating slope, the dump height being 14 metres generally and up to 31 metres at the south boundary of the area under consideration(Fig. 3a). In the night 16. 12. 87 the slope movement occurred involving the lower benches from +4-m floor up to +80-m floor; the moving masses fell into the worked-out space within 5 minutes. Then, at the same speed, the failure both of the rocks on the +100-m floor and overlying rocks occurred. The boundary of the sliding area corresponds to the boundary of cutting off the inactive slope (Fig. 3b, Fig. 6, schematically). Before landslide, the north part of the slope was in near-limit stable state until it was cut off at the south-west end. By that time, due to discussed above effect of the horizontal unloading, a set of weak horizontal contact planes in swelling clays was formed as a result of lateral rock relieving with cohesion equal to zero and small (0.05) coefficient of friction. The propagation of the softening zones extended over two kilometers from the slope. The lateral thrust coefficient within the slope was about zero (at least, less than 0.2γ), because the one-dimensional compression strength of clays

$$[\sigma]=2c\tan(\pi/4+\varphi/2)=550\text{kPa} \qquad (15)$$

somewhat exceeded the average effective stress σ_{av}'

$$\sigma_{av}'=\left(\int_0^h \sigma' dz\right)/h=(\gamma-\gamma_o)h/2+\delta\gamma_o=520\text{kPa} \qquad (16)$$

within the resistance prism (slope section 1, see Table 1, h=70m, δ=20m, where δ is the water level).

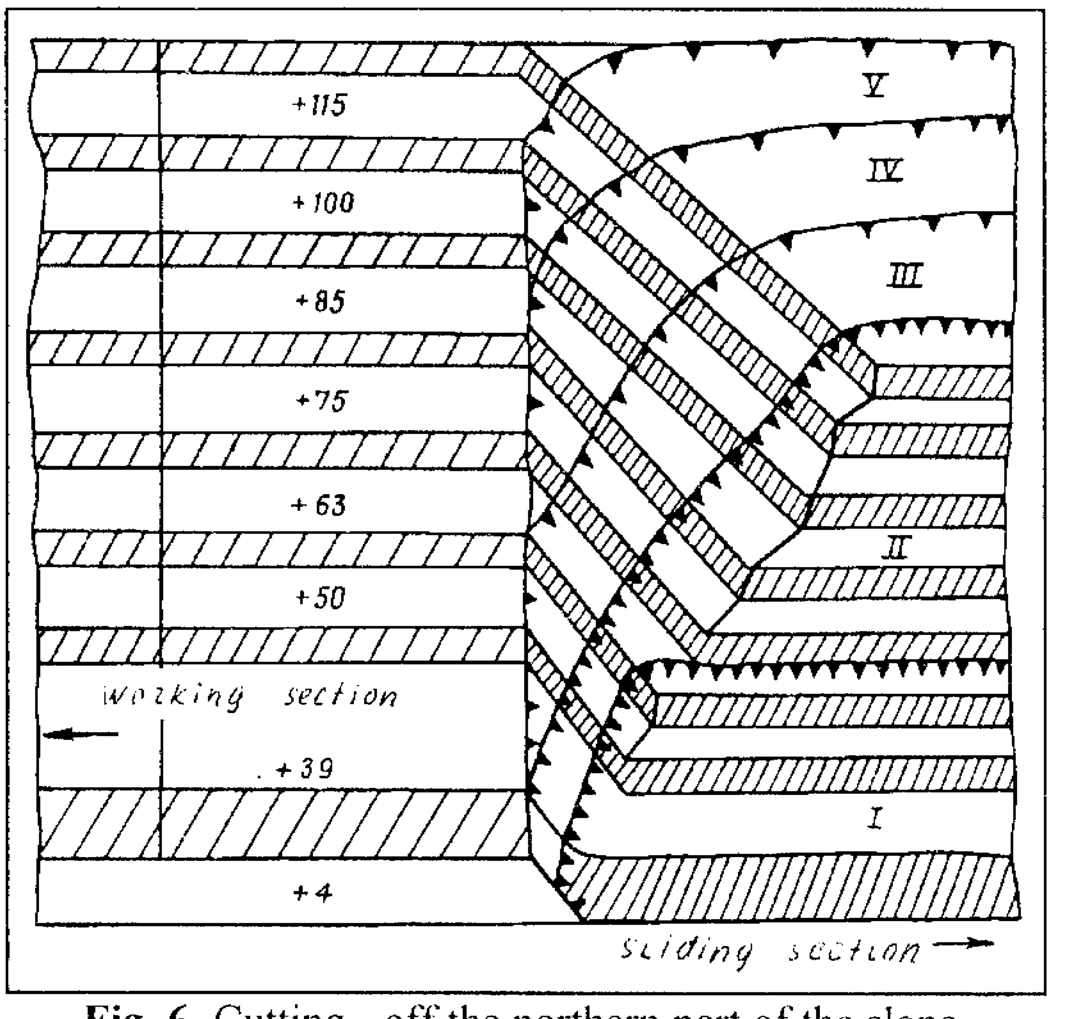

Fig. 6. Cutting - off the northern part of the slope.

The situation began to change with cutting off the working part of the slope from its inoperative part. Firstly, the rigidity of constraint of the south end of inactive section of the slope became progressively lower up to vanishing to zero at the floors near the ore zone. Secondly, forces of lateral thrust $\sigma_y=\xi_y\sigma_z$ were released, which contributed to the appearance of shearing forces acting normally to the cut-off end of the inactive section of the slope. Along the vertical surfaces directed in accordance with the slope extend, the slope body was cut off from the adjacent mass as soon as the shearing force exceeded the cohesion of clays. An argument in favor of such a development of rock movement process is the fact established by Javorsky [8] that at the floors of the cut-off end of the north inactive section a set of vertical fissures appeared in accordance with the slope extent just before the landslide. Preliminary stage of the failure of the slope ended with the bending of the rock mass within the resistance prism (section 1, see Table 1) which began translatory moving toward the rock waste dump. Adjacent section 2, i. e. the prism of active pressure, having lost the residual lateral thrust due to the moving of the lower section 1, went over into the state of uniaxial compression under its own weight, σ_x=0. With the coefficient of lateral thrust equal to zero, the rock mass within the active pressure prism began to fail since at the height beyond 80-100m the average effective stress σ_{zav}' exceeded the compression strength $[\sigma]$ of the rock mass. At first, the fully (from the bottom to the upper floors) cut-off section 2 (Table 1) and, then, the partially cut-off sections 3, 4, and 5, in succession, went to failure, becoming loose debris mass without cohesion, and with a high values of compressibility, and with a high coefficient of lateral pressure ξ_r=0.6 at φ=15° according to (11).

As the lateral thrust did not exceed 0.1-0.2 of the weight load of rock before failure and the lateral thrust subjected to the resistance prism increased up to 0.4-0.5 of the weight of rock after failure, the further movement both of the resistance prism and of failed prism of active pressure into the mined-out space tended to dynamic phenomena like rock burst. As shown in Table 1, discontinuity of rock mass deformations corresponds to discrete character of cutting off the end of the failed slope (Figures 3 and 6). Section No. 2 (Table 1) was cut off from its bottom at the +75-m horizon while section No. 3 was cut off only above +63-m horizon, and section No. 4 above +88-m horizon. Thus, if the height the section No. 2 is 85m above the +15-m sliding surface the upper part with the thickness of 35m is to be weakened through undercut, while within the section No. 3 that has the height of 90 metres the 30-metres thick mass is weakened; and within the sections No. 4 and No. 5, both of these 100 metres, the mass of thickness 25 metres is weakened. It is interesting to evaluate Young's modules of the failed rock mass. Using the Hook's equation in the form

$$E=d\sigma_{y.i} L_i /2dL_i \qquad (17)$$

one may obtain the following values: E_2 =140kPa, E_3 =180kPa, E_4 =240kPa, E_5 =60kPa. Here $d\sigma_{y.i}$ is the residual lateral thrust, $d\sigma_{y.i}=0.3\gamma(h_i - h_{i-1})$, "i" is the number of the section under consideration, L is the length, dL is the length increment of the section after failure.

Dynamics of the slope rupture was connected with brittle (in the sense discussed above) failure of the stiff clays, followed by occurrence of the surplus horizontal force F_s which magnitude per unit length of the slope is equal to:

$$F=\xi_s \gamma h^2 /2 \qquad (18)$$

where ξ_s is the coefficient of surplus lateral thrust, and h is the height of the section. According to this approximate equation, the force $F_{s.1}$ =20MN/m arising in the moment of failure of the section No. 2 caused the resistance prism (its mass is 650t/m) to move a distance of 150m with acceleration 0.3g=3m/s^2 within 10 seconds (calculation is performed using the second law of Newton). The section No. 2 was moving for 11 seconds in the same manner traveling the distance of 120m with an acceleration a=2.25m/s^2 (unit force $F_{s.2}$ =65MN/m, unit mass m=3000t/m). The section No. 3 was moving under the action of the unit force $F_{s.3}$ =65MN/m (displacement d=60m, unit mass m=5000t/m, and the time of motion t=9.5s). The section No. 4 was moving during 7 seconds (d=25m, m=6000t/m, $F_{s.4}$ =65MN/m), and the section No. 5 was moving during 7 seconds as well(d=10m, m=7000t/m, $F_{s.5}$ =65MN/m). These calculations pertain to the central cross-section of the sliding slope, where the braking effect of the "constraints" shows itself only slightly. The deformations of the cut-off south end of the slope might be likened to bending strain of a cantilever beam at the first stage of the movement, and later on, after the moving mass had met the rock-waste dump placed here, the displacements might be represented as bending of a beam with constrained ends. The validity of this assumption is supported by calculation of the bending parameters of the sliding sections (Table 5) with using the equation:

$$dy_{max} =ql^4/384EI \qquad (19)$$

where dy_{max} is the maximum displacement of the section, q is the excessive unit force q=F_s , EI is the bending rigidity of the beam, l is the length of the section.

Table 5. Parameters of sliding sections (E=1MPa)

Section , No.	Height ,m	Width ,m	I, m^4 $\times 10^{-8}$	dy_{max}, m	Calculated q, MN/m	values l, m
1	55	160	0.2	30	20	600
2	100	120	2	60	65	500
3	105	90	3	35	65	500
4	105	50	9	15	32	650
5	105	40	20	10	16	1000

Unlike the calculation of displacements caused by dynamic factors while crushing clays, one should not take into account the whole distance of movement to calculate the bending deformations, but only the part of the movement pathway that is due to slowing down the mass movement on account of the resistance of the constrained beam ends. This part of the movement pathway is a difference between the displacement of the rock section in question and the displacement of the next rock section, because the bending strain is responsible for difference in displacement rather than for total displacement (Table 5, dy_{max}). A comparison between actual and calculated values "dy" along the sliding slope is tabulated in Table 6.

Table 6. Comparison between actual and calculated bending deformations

Distance, x, m, from the southern end	$x=x_n/l$	Relative displacement, dy/dy_{max}	
		calculated	actual
120	0.15	0.24	0.23
320	0.4	0.92	0.54
620	0.78	0.49	0.77
780	0.95	0	0

As is seen from Table 6, the actual function of displacements is asymmetric and quite different from the calculated function, and this fact is not unexpected one, because, in reality, the southern end of the sliding slope was not rigidly constrained, but moved some distance as a free end of a cantilever beam (about first 20 metres of its displacement), and then, setting against the rock-waste dump, this end had a possibility to turn like a hinge, while the northern end of the slope, being a smooth transition region between the working slope and the intact rock mass, was more rigidly constrained.

CONCLUSIONS

The fact that landslides along the weak planes are far from standard, was pointed out by many experienced specialists in geotechnics, among which are our Professor Maslov N. N. , famous A. Skempton, Norweyan, Swedish, and Canadian researchers. N. N. Maslov wrote as early as in the forties, that landslide motion along a cylindrical surface rarely occurs in nature, and supposed an original method of horizontal forces to improve existing calculation methods. A. Skempton explained appearance of landslides of stiff overconsolidated London clays by their partly softening followed by quick rebound. Landslides descriptions in English and Canadian Journals, studies in Norway and Sweden in the 50s, and in Canada in the 80-90s, lead close to the thought about a decisive role of the lateral thrust in forming weak contacts at the base of slopes composed by clayey rocks. The landslide in open-pit No. 3 considered above, large in amount (about 3-5 Millions m^3), and unique in the failure pattern, was a matter for scientific inquiry and enabled one to obtain a great body of information about behaviour of clays in conditions of the intensive technogenic loading. The author does not pretend to comprehensive analysis of this very interesting problem, but only tried to touch on some main aspects (displacement, swelling, rupture), in order to construct a logically correct picture, and made an emphasis on the leading factors of controlling the processes of the clayey rock mass deformations. Analysis of the data collected in the period of the research work by VNIMI at the open-pit No. 3, showed that the author's hypothesis about formation of technogenic weak contacts explains the causes of the slope rupture in 1987 and allows to predict the behaviour of the rock mass over the period of further mining operations.

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