Research of underground water heads influence on the landslide of the Angren open-pit slope.

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ABSTRACT

In 1987 the mass of the working slope of brown coal Angren open-pit began to move towards the wrought out space with the speed firstly of some centimetres per month till in the past periods of acceleration to 20-50 mm/day. The process of the slope sliding continued more than 10 years. The western slope with the height of 80-110m above the sliding surface (the whole height of the slope more than 200m) at the distance about 1 km from south to north edge dismissed at the distance more than 10 m in horizontal direction. The volume of the displaced mass (more than 50 Mio. m^3) was represented by loams and gravels of Quarternary age, clays, sands and limestones of Paleogenic and Cretacerous age. There were two aquifers within the exposure mass, in gravels (rate of inflow 1000 m^3/h) in the upper part of the mass, and in sands and weak sandstones (50-60 m^3/h) at the bottom of the displaced mass. The sliding surface was situated along the contact between Paleogenic (Suzaki) sands (sandstones) and clays.

Three main factors which prevented the slope from failure are under investigation-specifics bending deformations of the slope considered as construction based on weak foundation and constrained in its edges;-unstationary process of relaxation of the active moving forces due to the clay viscosity resistance during mass displacement;-strengthening of clays under influence of their rebound in period of excavation in hot and dry climate. Analysis of field data of pore pressure measured by special gauges installed into the well bored on the slope and data of survey observations for the slope displacement as well as visible and simple mechanical approach allowed to work out the landslide practically without such special measures as drainage and lowering of the slope angle.

THE ROLE OF GROUND - WATER HEADS IN DEVELOPMENT OF A LANDSLIDE ON THE ANGREN OPEN - PIT COAL MINE

INTRODUCTION

The opencast mining practice poses the problem of control of slope parameters under limit equilibrium and slope deformation at early stages of landslide formation. During the mining activities, the speeds of slope movement and the sizes of deformation concentration areas must not disturb the normal operation of the open pit, that is to say, the slope displacement and deformation rates, admissible from technical considerations, must not lead to a sudden slope slide and failure. In modern practice, geomechanic forecast includes computation of the field of deformations by using analysis of displacement surveying, geologic and hydrogeologic features of mine fields, and mechanical properties of rocks making up the slope mass. Based on results of the forecast, one may efficiently schedule the mining operations, taking into account a possible unfavourable development of events and using reliable methods for the landslide control such as unloading at the upper portion of slope and piling of waste rock against its lower portion, reduction or enlargement of excavation front, and drainage.

Here we consider problems of a deformation computation theory for an open - pit slope body composed of horizontally - bedded rocks with weak contact planes, hydrogeologic conditions of the Angren brown coal basin[1], and Angren open - pit slope deformations.

1. DEFORMATIONS OF THE SLOPE COMPOSED OF HORIZONTAL BEDS WITH A WEAK CONTACT PLANE UNDERCUT BY THE EXCAVATION.

Under these conditions, after the limit equilibrium is established,

$$P_d = P_h \qquad (1)$$

Here P_d is the force which tends to displace the slope mass, and P_h is the force which tends to hold it in place.Referring to Fig. 1 ,

$$P_d = \xi \gamma h^2 / 2 \qquad (2)$$

and

$$P_h = cL + \gamma L h \tan \varphi / 2 \qquad (3)$$

where

$$\xi = \nu / (1 - \nu) \qquad (4)$$

c is cohesion, φ is friction angle for the weak contact plane ; ν is Poisson's ratio ; and γ is volume weight.

Referring to Fig. 1,

$$L = h / \tan \alpha \qquad (5)$$

where h is the height and α is the angle of the slope.

A partial loss of strength on the weak contact plane entails the slope movement towards the mined - out space, the block ABC involving an adjacent block DBCE into movement. The length of the block DBCE is

[1]Angren is a small town within 100 km east of Tashkent, Uzbekistan

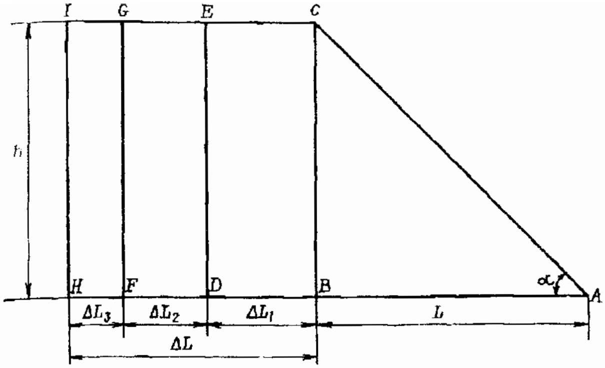

Fig. 1. Sequence of development of slope deformations.

$$\Delta L_1 = c\,L \,/\, (\gamma\, h \tan\varphi + c), \qquad (6)$$

and after cohesion on the length ΔL_1 is lost, a next block will be involved into sliding along the length ΔL_2.

$$\Delta L_2 = c\,\Delta L_1 \,/ (\gamma\, h \tan\varphi + c), \qquad (7)$$

and so forth until a total length ΔL of the softening blocks is such that the friction compensates the loss of cohesion on the whole length of the surface of sliding:

$$\Delta L = c\,L \,/\, \gamma\, h \tan\varphi = c \,/\, (\gamma \tan\varphi \tan\alpha) \qquad (8)$$

This is, in general way, the development of a landslide, assuming that there is no limitation on its extension along the strike. In real situations the landslide development depends on the finite dimensions of the slope along the strike, on rheology of clayey material on the weak contact plane, and on the failure pattern of blocks close to the slope face. The main obstacle to further stepwise propagation is a restriction of a slope portion affected by sliding, at the edges along the strike. In this case the displacements are limited to the values of the order of

$$y = P_a\, l^4 f(x/l) \,/\, E\, I \qquad (9)$$

where y is horizontal displacement; P_a is intensity of force which causes the slope displacement ; l is the length of the slope portion affected by the sliding, which is considered as a beam of triangular cross-section with a bending stiffness E I, where E is modulus elasticity, and I is cross-section moment of inertia with respect to a neutral axis ; $I = L^3\, h \,/\, 36 = h^4 \,/\, 36\, (\tan\alpha)^3$; x is distance from the left-handed end of the beam ; and f(x/l) is beam- bending function dependent on the pinching conditions at the beam ends.

The resistance to sliding due to rock strengthening on the restrained portion of the slope does not take effect immediately but only as bending displacements have occurred (Fig. 2). The displacement rate may be determined according to the law of viscous friction :

$$\tau = \mu \, \partial v / \partial z \tag{10}$$

where τ is shearing stress, μ is viscosity factor ; v is speed of displacement ; and z - axis is normal to sliding surface. At first, when the forces of lateral restraint are equal to zero,

$$\tau_{max} = P_{a\,max} / L \tag{11}$$

$$v_{max} = \tau_{max} m_0 / \mu = P_a m_0 / \mu L = P_a / \mu_p \tag{12}$$

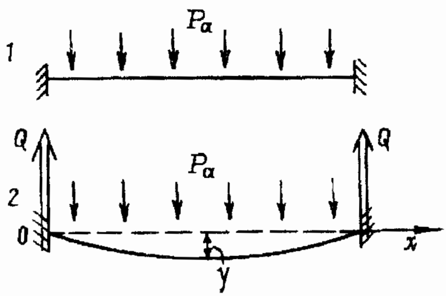

Fig. 2. Relaxation of forces which induce the slope movement.

where $P_{a\,max}$ is maximum unbalanced force responsible for sliding ; m_0 is thickness of the clayey gouge on the weak contact plane; L is the length of the contact plane; and $\mu_p = \mu L / m_0$.
Once within the restrained ends of the bending beam reactive forces Q_r appear

$$Q_r = P_a l / 2 \tag{13}$$

that depend on the maximum value $y_{max} = y (x = l / 2)$ of the beam, so that

$$Q_r = y_{max} E I / 2 l^3 f(0.5) \tag{14}$$

$$\text{where } y_{max} = \int v \, dt = \int (P_a / \mu_p) \, dt \tag{15}$$

the force inducing the displacement in the middle section of the beam decreases to the value of P_a,

$$P_a = P_{a\,max} - 2 Q_r / l \tag{16}$$

and consequently the displacement process will slow down.

$$Q_r = P_{a\,max}\, l\, \exp(-\rho t) / 2 \quad (17)$$

$$\rho = y_{max} E I / \mu_p l^4 f(0.5) \quad (18)$$

$$v = P_{a\,max} \exp(-\rho t) / \mu_p \quad (19)$$

The displacement value may be determined by

$$y = P_{a\,max} [1 - \exp(-\rho t)] / \rho \mu_p \quad (20)$$

or by

$$y = \lambda [1 - \exp(-\rho t)] \quad (21)$$

whereupon it is not difficult to determine both $\lambda = P_{a\,max} / \mu_p \rho$ and ρ by the method of type curves.

The failure of rocks of which the landslide body is composed will occur if the tension stresses resulting from the bending deformation exceed the tension strength $[\sigma_t]$ of rocks within the area adjacent to the restrained ends of the beam :

$$\sigma = M_{max} / w \leq [\sigma_t] \quad (22)$$

where w is a resistance moment. The failure will occur as well if, with the maximum values of displacement having been reached, the shearing strength of rocks within the restrained section is less than the value of shearing force

$$Q_r = \int (c + \sigma \tan\varphi)\, dF \quad (23)$$

where in the right -hand side of the formula there is a sum of cohesion and friction for the sectional area of ABC.

Because of the loss of rock strength due to ruptures across the strike, the displacement of the slide body is further progressing as the deep cracks are formed deep into the slope mass, according to the pattern described above for the "infinite" slope.

2. INFLUENCE OF UNDERGROUND WATER.

To take into account influence of underground water on rock deformations it is necessary to represent the stress state of rock mass and the properties of rocks as a function of the pore pressure. According to the Terzaghi and Bishop principle of the pore pressure 'neutrality', the relations between the forces which tend to displace the slope mass and the forces which tend to held it in place may be corrected quite adequately by merely introducing the effective normal stresses into the Coulomb formula as follows

$$[\tau] = c + \sigma_{eff} \tan\varphi \quad (24)$$

with

$$\sigma_{eff} = \gamma h - p \tag{25}$$

where p is pore water pressure at the depth h. Eq. (25) can be written in more general form, which allows it to be used in the earlier deduced relations, i. e.

$$\sigma_{eff} = \gamma_p h \tag{26}$$

$$\text{where } \gamma_p = \gamma - \gamma_0 (p / \gamma h) \tag{27}$$

γ and γ_0 being volume weights of rock and water, respectively.

To take into account the variations of strength properties of clayey material depending on water content W, based on the laboratory test results, e. g.

$$[\tau] = [\tau(\sigma, W)] \tag{28}$$

it should be required that we be able at least to forecast the changes of rock humidity with varying load and pressure of ground water.

The relationships W (σ_{eff}) for clayey soils can be obtained when studying the process of clay softening due to unloading during removal of overlying strata. Incidentally the argillaceous rocks either are softening (as within an area close to an aquifer, where the pore pressure is plus-signed and humidity increases as loading decreases) or consolidating (farther away from the aquifers, where the pore pressure is minus-signed.)

Assuming that the permeability and water yield of clays are not humidity dependent, one could describe decreasing water pressure (drawdown) S in clay during its softening (Fig. 3) as

$$S = b\, t\, F(Fox) \tag{29}$$

$$F(Fox) = 1 + [1 + 1/2(Fox)^{-0.5}]\, \mathrm{erfc}\, [1/2(Fox)^{-0.5}] - \exp[-(4\, Fox)^{-1}] / (\pi\, Fox)^{-0.5} \tag{30}$$

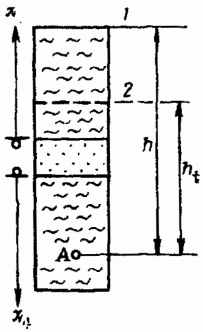

Fig. 3. Calculation of water pressure in clays : A is point under consideration ; h_0 depth before excavation ; h_t depth at the moment t ; 1 earth surface ; 2 surface of an open - pit bench.

Here $b = \beta\, v_{op} (\gamma / \gamma_0)$; β is a coefficient of pore water pressure transfer ; v_{op} is the speed of open-pit deepening ; γ is the volume weight of rocks ; $Fox = at / x^2$, where $a = k_o / \mu^*$, k_o being the coefficient of filtration and μ^* the water storage of clay ; x is the distance to an area in contact with an aquifer; t is the elapsed time from the start of opencast operation.
Of interest are both extreme cases. With $x \to \infty$,

$$S(\infty) = \beta\, v_{op}\, \gamma\, t / \gamma_0 = \beta\, \gamma\, (h_o - h_t) / \gamma_0 \qquad (31)$$

where h_o and h_t are the depths of a reference point under question before the mining activities and at the moment t, respectively,

$$\sigma_{eff}(\infty) = \gamma\, (1 - \beta)\, h_t + \beta\gamma\, h_o - p_o \qquad (32)$$

With $x \to 0$, $S(0) \to S_{aquifer}$, and hence,

$$\sigma_{eff}(0) = \gamma\, h_t - p_o \qquad (33)$$

where $p_o = \gamma_o\, h_{aquifer}$, $h_{aquifer}$ is the height of the water column within the nearest aquifer above its overlying (underlying) strata.
The rate q of water absorption by the softening clays from the aquifer can be evaluated by the equation

$$q = \beta\, v_{op}\, \gamma\, k_o\, t^{0.5} / \gamma_o\, (\pi\, a)^{0.5} \qquad (34)$$

3. ANALYSIS OF THE SLOPE DEFORMATIONS OF THE ANGREN OPEN PIT.

In spring 1987 a landslide developed on the working side of the Angren open pit, which made itself evident in formation and growth of a deep crack on the surface at the distance of 100 metres from the uppermost open-pit bench, in motion of the bench No. 6 composed of Suzak sandstones forward on the light green clays, and in formation of similar but less pronounced movements at the lower benches composed of kaolinic clays. According to survey measurements starting in May 1987, the landslide could be classified as a slow continuous movement of the upper part of the open-pit slope towards the mined-out space, which reached within two years 4. 3 m in the middle portion of the landslide body within the boundaries of the benches above the sliding surface and 0. 4 m at the bench No. 6 below it (Fig. 4).

Geologic features of the open-pit side on which the mining operations are carried out are shown in Fig. 5. At the upper benches made up of shingles, with the levels of seepage depressing in NS direction, an aquifer of Quarternary period has been tapped, water being partly discharged by means of trenches into the Angren river ($Q = 700\ m^3 / h$) and partly transferred over the benches to the pit bottom ($Q = 600\ m^3 / h$). Within the contact area of chalky sandstones and kaolinic clays, stripped by the benches, water of the Cretaceous - Paleogene aquifer crops out, whereas water seepage from the sands and sandstones is limited to isolated areas and is no more than $10\ m^3 / h$ because of failure of these rocks as the result of blasting up to 100 metres deep into the slope mass. Transmissivity T of the shingles aquifer is about 100 m^2/d, and of sandstones is about 10 m^2/d.

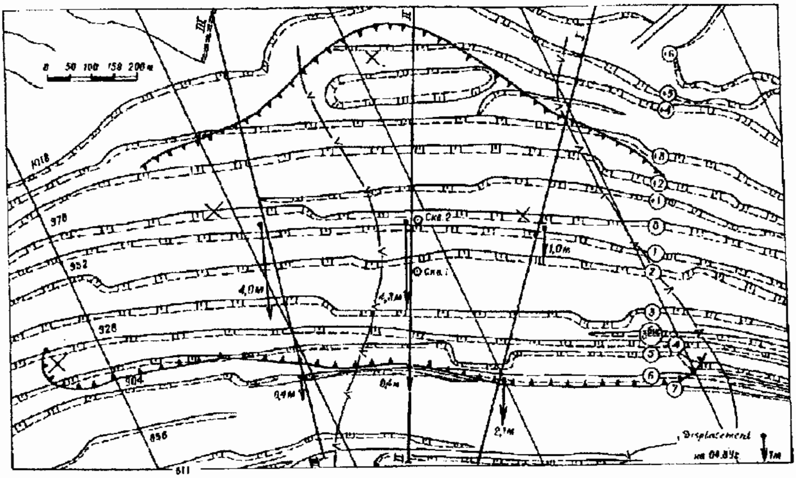

Fig. 4. Mining plan of the Anqren open pit (April, 1989): outline of the landslide body; I - II - III survey lines.

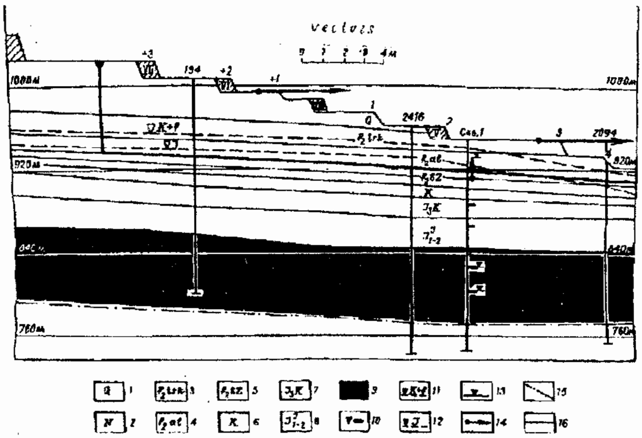

Fig. 5. Geologic profile (II - II) : 1 shingle, 2 conglomerate, 3 clay, 4 limestone, 5 clay and sand, 6 sand and clay, 7 speckled kaolin, 8 grey kaolin, 9 coal, 10 transducer, 11 water table in Cretaceous-Paleogene aquifer, 12 water table in coal bearing strata, 13 deep cracks, 14 displacement vector, 15 tectonic dislocations, 16 sliding surface.

In order to estimate the real water tables in the sliding body, provision was made for drilling of two boreholes with the pore pressure transducers to be installed in them. One of the boreholes was completed and equipped at the bench No. 1, 946 m, in the middle portion of the landslide body. The second borehole designed two benches above the first, in the shingles, was not instrumented because of an accident. The first borehole with diameter of 200 mm was drilled to the depth of 200 metres down to the roof of the Palaeozoic complex (Fig. 5). After logging, seven pore pressure transducers DC -13 suspended on a wire line were lowered down the borehole : three lower transducers (Nos. 1, 2, 3) in coal seam and coal-bearing strata ; two transducers (Nos. 4 and 5) in kaolinic grey and red clays ; the transducer No. 6 close to the slip surface ; and the transducer No. 7 in Suzak sandstones. The results of the pore pressure measurements are given in Table 1.

Table 1. Pore pressure (metres, water column).

No.	Depth, m	28. 5. 89	29. 5	30. 5	3. 6. 89	6. 6	16. 6	28. 6	5. 7. 89	2. 8. 89	11. 9. 89
1	145	180	155	136	9	10	7	10	10	3. 5	3
2	120	150	125	120	10	3	- 2	12	-2	-2	-3
3	100	118	97	98	68	10	0. 5	-5	-2	-2	-3
4*	80	83	-	-	-	-	-	-	-	-	-
5*	60	65	-	-	-	-	-	-	-	-	-
6**	40	43	27	-	-	-	-	-	-	-	-
7	35	35	18	20	20	21	5	8	5	7. 5	18

*Transducer is inoperative since May 29, 1989.

** Transducer displaced as a result of filler clay consolidation.

After the pore pressure stabilisation early in June 1989 it could be concluded that in response to the intense softening processes in the underlying confining bed of the Cretaceous - Paleogene aquifer the water pressure in it drops to the point where water becomes capillary or film water (Table 2). This effect is most pronounced at small depths where the pressure transducers Nos. 4 and 5 have failed the next day after installation, probably, for the reason that the water pressure is found to have been below an " absolute vacuum " (10 metres water column) and it may not be in principle measured by mechanic systems.

Table 2. Calculation of coefficient β by means of Eq. 31.

Depth h_t, m	Initial depth h_o, m	Initial water pressure p_o, m	Measured water pressure, m	β
145	225	165	+5	1
120	200	150	-3	0. 95
100	180	140	-3	0. 9
80	160	130	-10*	0. 9
60	140	120	-20*	0. 9
35	115	105	+10	-

*obtained at β = 0. 9.

Under these conditions there is no need to carry out the drainage within the slope mass. Influenced by the unloading process, the rocks of which the slope mass made up are not practically softening (Table 2, $\beta \approx 1$). At the rock unloading rate of 10 metres per year the underground water stream in Jurassic strata will not reach the point where the borehole is located so that the water pressure p here is to be calculated as

$$p = p_o - \beta\gamma (h_o - h_t) \qquad (35)$$

The effect of water loss from the aquifer either due to evaporation through overlying bed or because of softening process in the confining beds of aquifer on the flow regime in the aquifer itself may be estimated by using the equation for the one-dimensional flow at the leakage to another aquifer. The boundary conditions are (Fig. 6) : $S (L) = S_1$, $S (0) = S_o$, $q (L) = 0$.

$$S = S_1 \, ch [(L - x) / B] \qquad (36)$$

$$q = T S_1 \, sh [(L - x) / B]/B \qquad (37)$$

$$L = arch (S_o / S_1) \qquad (38)$$

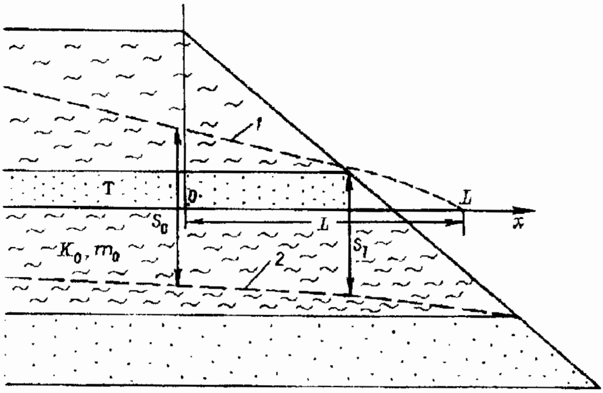

Fig. 6. Water loss from an aquifer within the slope area.

where S is a difference of water pressure heads in the basic aquifer and the aquifer interacting with the basic one ; T is the transmissity of basic aquifer ; $B = (m_o T/k_o)^{0.5}$; m_o and k_o being the thickness and coefficient of filtration of a bed between the aquifers. With using these relationships one can calculate the parameters of interaction of the aquifers and an average water loss from the aquifer within the slope area (Table 3), expressed as $q_o = Q / L$.

Table 3. Calculation of the rate of the stream loss in the slope zone.

Aquifer	Rate of water inflow to the slope zone, m^2/d	Rate of water seepage on the bench, m^2/d	Length of the seepage zone, m	B, m	q_o, m / d
Cretaceous - Paleogene	5	0. 8	700	600	0. 007
Jurassic	0. 8	0	2000	1000	0. 0004

4. NON-STATIONARY SLOPE DEFORMATIONS.

The derived relationships to determine the slope deformations include the strength - strain parameters whose determination is feasible only from results of back analysis. These parameters are coefficient of lateral earth pressure ξ, cohesion and friction angle (c and φ) of weak contact plane, coefficient of viscosity μ and thickness m_o of the contact clay gouge being deformed, and elasticity modulus E. How much true each determined parameter is depends on the earlier accumulated errors of calculation of other parameters related to the parameter under consideration. It should be pointed out that our calculations have been concerned with the average data both over the open - pit mine as a whole and along the length of the slope in strike. This fact has head an immediate impact on the calculation results.

The sliding movements in the slope body of the Angren open- pit mine began at following parameters : Slope height between the sliding surface and the edge of the uppermost bench 80 - 100 metres (on average, 100 m) ; slope angle 10° ; and horizontal extent of the sliding surface, on average, 550 metres. After a rear deep crack had formed on the surface of the uppermost bench in the middle portion of the landslide body (an average distance ΔL to the initial deep crack is about 150 m), the major rupture in the rock mass being deformed extended from the uppermost bench edge to the next two of lower benches as view in plan. The rock mass outlined by these discontinuities began moving translatorily towards the mined - out space. With an aim of preventing the slope body from sudden failure, the upper benches in the middle portion of landslide were in 1987 -1988 rearranged with a wider spacing between them. This rearrangement ensured that the involvement of a new series of blocks could be brought under control : even though the average depth of the pit being measured in the Suzak clays increased to 110 metres, the slope angle in the middle portion of the slope decreased to 9 degrees (the length of the sliding surface increasing to 700 metres).

Since, as is evident from the present knowledge of tectonics of the Angren coal deposit, the rock mass is tectonically disturbed just where the deep cracks have formed, one can evaluate the force which tends to displace the slope body from Eq. (2) as follows

$$P_d = \xi \times 10^2 \, \mathrm{MPa} \qquad (39)$$

at h = 100 m and $\gamma = 2$ t/m^3. The relation between cohesion and friction on the sliding surface can be obtained from Eq. (6) by

$$c = 76 \tan \varphi \tag{40}$$

with $\Delta L = 150$ m and $L = 550$ m, as well as the relation between lateral earth pressure coefficient and friction on the sliding surface by

$$\xi = 9.7 \tan\varphi \tag{41}$$

Assuming that $\xi = 0.3$, one can calculate c and $\tan\varphi$ as

$$\tan \varphi = 0.031,\ c = 23.6 \text{ kPa} \tag{42}$$

$$\text{and } P_a = c\,L = 13 \text{ MPa m} \tag{43}$$

Since the block in that the deep crack had formed lost the cohesion on the length ΔL, causing the landslide's movement, but another deep crack had not developed, the unbalanced i. e. active components of the forces initiating slope movement were being transferred in the course of deformation to the end sections of the beam under study. With measured displacement y (t) for the pit bench No. 1 in the middle portion of the slope area, one can obtain from Eq. (21) two sets of parameters : at an initial stage of deformation process, $\rho = 2300^{-1}$ day^{-1} and $y_{max} = 10$ m ; and later, beginning since winter 1987, $\rho = 860^{-1}$ day^{-1} and $y_{max} = 8$ m. (Fig. 7).

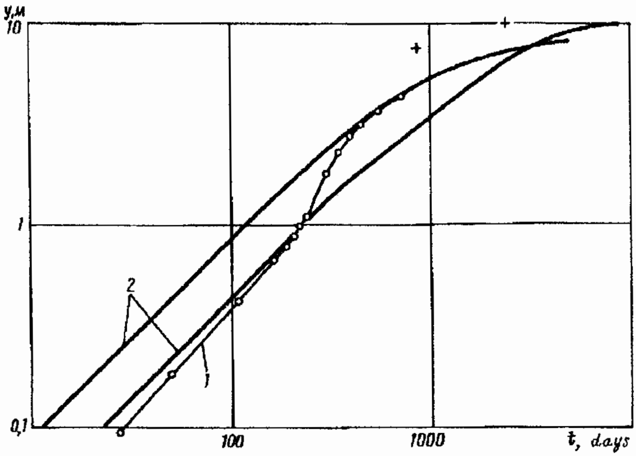

Fig 7. Diagram of slope displacement (bilogarithmic paper) : 1 and 2 actual and calculated displacement, respectively.

However, while taking into account the displacements y(x) in various areas of the slope body and using the known relationships, which describe the bending of the beam rigidly restrained and hinged at its ends, one can obtain the values as shown in Table 4.

Table 4. Bending displacements.

Distance x to the left - hand end of the beam, m	Displacement y(x), m	y/y_{max} actual	x / L	y/y_{max} rigid restrained beam	y / y_{max} simple beam
200	0	0	0. 11	0. 13	0. 31
750	4. 0	0. 89	0. 42	0. 96	0. 92
1100	4. 3	0. 96	0. 62	0. 90	0. 94
1260	2. 1	0. 47	0. 74	0. 58	0. 65
1400	1. 0	0. 22	0. 81	0. 4	0. 58
1750	0	0	1	0	0

As the rigid restraint most closely corresponds to the actual conditions, the required parameter E has been calculated based on the formula (see Eq. 9)

$$y_{max} = P_a L^4 / 384 EI \qquad (44)$$

With $L = 1750$ m and $I = 4.6 \cdot 10^8 m^4$, there will be E = 8 MPa and E = 9 MPa for the initial stage and the further stage of observation, respectively. This increase of modulus of elasticity may be attributed to the fact that these calculations ignored the increase of the moment of inertia of the beam in bending after an additional detached mass was involved into movement along with the main landslide body.

5. CONCLUSIONS.

On the basis of the field investigations and the calculations carried out, an analysis of deformations of the open- pit slopes with flat- lying beds has been provided. A forecast of the landslide development is given based on mechanical behaviour of rock mass, taking into account the impact of ground - water pressure and of overburden removing on the state of stress of rock mass. This analysis, as applied to the conditions of the Angren coal fields, will allow for choosing the best methods of burden removing within the area of landslide.