**Calculation of the Deformations of Surface Mine Slopes Composed of Steeply Dipping Strata**

**By A. N. Ryumin**

Senior Research Worker, Russian Institute of Mining Geomechanics (VNIMI) St. Petersburg

## ABSTRACT

In 1966 a landslide occurred at the Bogoslovka open pit in the Northern Urals. It was not a catastrophic rock mass movement but for a long period from its appearance up to the present, this landslide has been responsible for damage to the upper berms of the eastern open pit slope. This is because of translation and deflection and subsidence of weak steep strata which form the floor of the "Moschny" coal seam and which has a thickness of 50 to 80 m.

In 1972 a landslide of the same kind occurred for the first time at the Kuznetsk coal basin on the slope face of the Vachrushev open pit, and later on, in the nineteen-eighties at four other open pits in the Bai-Salair region with steeply dipping strata.

The investigation of these cases show that the landslides above may be attributed to the same process of exfoliation and disintegration due to bending formations of stratified rocks. A high water head within the footwall and a lower permeability of strata across the strike allows for a lateral shearing force component and, at the same time, decreased friction on the bedding planes.

An analytical calculation scheme was developed using a limit equilibrium equation with the bedding planes in the rock mass represented as a composite two-dimensional plate. This calculation approach may be used for estimation of slope stability for natural and artificial slopes within a rock mass composed of steeply dipping strata.

## INTRODUCTION

The conventional engineering methods for prediction of rock mass behaviour in open pit coal mining are mostly based on the assumption that the rock mass behaves as a loose medium being stiff as the elastic strains are developing up to the failure, which results in total disturbance of the rock mass stability in the slope area or in movement of the entire slope. The mined-out space along an inclined sliding surface cutting off the slope area from the adjacent rock mass, on which the condition of limit equilibrium is satisfied (strictly speaking, this limit is a little exceeded [1]).

Nevertheless, it should be noted that, under certain conditions, the stratified rock masses are subjected to long-term quasi-elastic strains and are, without complete failure, gradually losing their stiffness because of disturbing the planes of weakness between the layers. The failure processes of this type are especially characteristic for open pit mining of coal deposits where there are steep layers in the footwall.

For the first time, the deformations of the footwall had been observed in the Urals open pit coal mining in Mesozoic weak rocks in the seventies and, later on, in more hard Permian rocks in Kuzbass and in Carbonian rocks in Ekibastus. During this deformation process the sedimentary rock mass within the opencut slope area is separated into individual sheets or packets of 5 m to 20 m thickness and deflected towards the mined-out space, with the opencut benches laterally moving without failure (at the slope height of about 100 m the displacements reach 20 - 30 m).

All observations carried out in situ at numerous deformed opencut slopes showed, that there is a new type of slope deformation during which the slope body is not moving translatory as a rigid body cut off from the rest of rock mass but is deflecting under bending loads.

The detailed analysis of the situation indicated the existence of a relationship between the rock masss deformations and the active lateral earth pressure as a result of stress relief in horizontal direction due to extracting the coal. It revealed also a very considerable role of sub-surface water pressure that contributed to increasing shear forces and decreasing frictional strength between the layers. All the observed deformations in the stratified rock mass were clearly bending deformations and therefore, in order to formulate a mathematical approach to evaluate the strata displacements and deformations, the theory of composite plates should be used in its simplest version.

## STATEMENT OF THE PROBLEM AND SOLUTION ALGORITHM

The present investigation deals with the estimation of equilibrium conditions and the calculation of displacements of opencut slopes in steep-dipping strata under ground water pressure.

In order to evaluate the bending deformations in the slope area, it should be reasonable to consider a so-called prism of resistance or a slope body which is cut off from the rock mass both by one horizontal plane passing through the pit bottom and three vertical planes, one of which separates the slope body from the adjacent rock mass (or footwall) and is directed along the strike whereas two other planes are directed across the strike and pass at the left and the right ends of the sliding slope area. The rock prism in the slope area is considered further as a composite plate that is pinched along its edges and from below (Figure 1a). The deformation behaviour of the plate can be treated as a simultaneous bending of two beams one of which is a frontal beam regarded as infinite on the strike and rigidly fixed at the open pit bottom level (Figure 1b) while the other is a longitudinal beam that is restricted from below and pinched on the strike (Figure 1c). Each of two imaginary beams is subjected to a bending load ($Q_x$, $Q_z$) which is a component of the total plate loads ($Q_s$). In order to obtain a satisfactory solution, it is important to determine the role of either beam in forming the total bending strain. Having assumed at first, that the beams are bending independently under the forces $Q_x$ and $Q_z$ , we obtain the formulas to find the bending strain components $dy_1(z)$ and $dy_2(x)$ which are caused by bending moments applied to the frontal beam and longitudinal beam respectively. Then, using a condition of the statics and a principle of solutions multiplication, we may find two relationships between $Q_z$ and $Q_x$ to be found on the one hand and the known resultant plate load $Q_s$ on the other hand.

Of great importance is knowing a limiting size of either beam, ie critical height $h_{cr}$ and critical length $l_{cr}$ when the rock mass presented as a composite beam loses its initial high bending rigidity as a solid unbroken body and begins to behave as a group of thin beams with little bending rigidity. At that moment, the rock mass within the slope area deflects towards the mined-out space as a group of layers sliding on each other. The critical equilibrium state of the slope is a special state when the limit equilibrium is reached only on the planes of contacts between layers, since the deformations have appeared but the rock mass strength on the planes of fixing is still high enough to prevent the slope body from its total failure.

## REDUCTION OF THE TWO-DIMENSIONAL PROBLEM

In order to obtain a physically clear pattern of calculation of the plate deformation, we can combine two one-dimensional problems to provide the required solution of two-dimensional problem. For this purpose, we can use the so-called principle of solutions multiplication that is valid for the functions which satisfy the Poisson equation. In accordance to this principle [2], the two-dimensional problem solution for dy(z,x) might be presented as product of the one-dimensional problem solutions that have been derived under properly transformated boundary conditions. For the plate considered (Figure 1) we obtain

$$dy(z,x_o) = dy_1(z) \;.\; dy_2(x_o) / dy_{2max}(x) \qquad (1)$$

or

$$dy(x,z_o) = dy_2(x) \;.\; dy_1(z_o) / dy_{1max}(z) \qquad (2)$$

where $dy_1(z)$ and $dy_2(x)$ are horizontal displacements of the frontal and the longitudinal beams respectively, at the points z and x under the bending loads in the one-dimensional problem.

Since $dy(z_o,x_o) = dy\,(x_o,z_o)$, it follows from this that

$$dy_{1max} = dy_{2max} \qquad (3)$$

and since the bending strains of the beams and the acting loads are linearly related, we may write using (3):

$$dy_{1max} = Q_z \,.\, f_1\,(z)_{max} \qquad (4)$$

and

$$dy_{2max} = Q_x \,.\, f_2\,(x)_{max} \qquad (5)$$

where $f_1\,(z)_{max}$ and $f_2\,(x)_{max}$ are the bending functions of the deflection line of the beams at the points of the maximum bending strains.

Taking into account the conditions of the statics

$$Q_s = Q_z + Q_x \quad (6)$$

where $Q_s$ is the load acting on the plate, we obtain

$$Q_z = Q_s / (1 + \lambda), \qquad Q_x = \lambda \,.\, Q_s / (1 + \lambda) \quad (7)$$

where

$$\lambda = f_1(z)_{max} / f_2(x)_{max} \quad (8)$$

## BENDING STRAINS OF THE SLOPE AS A CANTILEVER FRONTAL BEAM

The pit slope is considered as an infinite beam (that is, the case $\lambda = 0$) bending in the frontal direction. The bending deformation of the frontal beam can be described by comparing the moment of shearing forces and resistance moment on the planes of weakness in the rock mass [3]. The frontal beam is loaded by external lateral forces evenly distributed along the beam's length (Figure 1b), such as by lateral earth pressure (Figure 2a).

$$q_1 = \xi \gamma (h - z) \quad (9)$$

or hydrostatic underground water pressure (Figure 2b)

$$q_2 = \gamma_o (h_o - z) \quad (10)$$

acting on the plane at the boundary between the slope area and the footwall.

Here are $\xi$ lateral earth-pressure coefficient expressed as $\xi = \nu / (1 - \nu)$; $\nu$ Poisson's ratio, $\gamma$ unit weight of rock, h height of opencut slope, $h_o$ height of water head, here $h_o = h$; z vertical distance measured from the plane of pinching the beam, that is, from the open pit bottom, and $\gamma_o$ unit weight of water.

Out of two types of lateral forces which are horizontal components of the active resultant force, the force of the greatest magnitude, ie mainly $q_2 = \max(q_1, q_2)$ has to be taken into account when calculating the moment of shear forces.

The shear strength $\tau$ resisting sliding one layer of the composite beam upon another are cohesion and friction on the side surfaces of strata within the beam discussed and is determined by means of the Coulomb-Terzaghi type equation

$$\tau = \sigma_{eff} \,.\, \tan \varphi_1 + c_1 \quad (11)$$

where $\sigma_{eff} = q(z) - p(i,z)$, q(z) being external lateral load, applied to the plate ABCD at the height z (see (9), (10)), and p(i,z) ground water pressure on the contact planes between the i-th and i+1-th layers at the height z above the pit bottom, $c_1$ cohesion and $\varphi_1$ angle of friction on the contact surfaces of layers (Figure 3).

Furthermore, a reactive force and its moment are induced at the pinched section of the beam and hence prevent the layer rotation and displacement within the pinched portion.

It is important to know whether the stratified rock mass has sufficient strength along the planes of weakness to resist elastic bending deformations, caused by horizontal lateral forces.

The solution of this problem may be a general one, but in order to simplify the calculation procedure, consider a special case of the strongly vertical layers forming the slope body. In this case, gravitational forces do not act on the vertical planes and the beam is subjected to the external horizontal force, q, and prevents the layers from sliding relative to each other only owing to the strength along the contact planes of layers.

The deformation process (in the sense mentioned above) in the frontal beam is initiated as the moment of shearing forces $M_s$ and resistance moment $M_r$ are equal. It is simple to show, that

$$M_s = q \, . \, (h - z)^2 / 6 \qquad (12)$$

and

$$M_r = \sum_{i=1}^{N} [\{ \int_0^{h_i} (\sigma_{eff,i} . \tan\varphi_{1,i} + c_{1,i}) dz \} m_i] \qquad (13)$$

where $m_i$ and $h_i$ are the thickness and the height respectively, of the i-th layer out of N layers within the beam in question.

As can be seen from (13), the expression for the resistance moment is obtained by addition of moments of friction and cohesion on the contacts between the layers. This expression, in general, cannot be simplified to the algebraic formula in the case of varying values of $\varphi_{1,i}$, $c_{1,i}$ and $m_i$ for different layers.

Since, as a rule, individual properties of layers and their contact planes are not known, it is important to derive a practically applicable expression for the idealised homogeneous medium with $\varphi_{1,i}$ and $c_{1,i}$ and $m_i$ being constant.

Assuming that $m_i = m_o$ = const and taking into account that $i = (h-z)/m_o \tan \alpha$ , one may obtain the critical height of the open pit slope as a function of the strength parameters on the contact surfaces as follows:

$$h_{cr} = 3 \, . \, c_1 / \gamma' \qquad (14)$$

where $\gamma'$ is a conventionally taken value of shear forces on contact surfaces, as defined depending on the lateral force and ground water conditions (Figure 2) within the slope area (Table 1).

As the limit equilibrium of shearing and resistance moments is reached, the separation of contact planes between the layers will occur as a result of lack of coherence and the bending deformation will take place.

Table 1.

| Ratio | Ground water conditions within the slope area | |
|---|---|---|
| $q_1 / q_2$ | totally drained slope | totally watered slope |
| more than 1 | $\xi \gamma (\tan\alpha - 2\tan\varphi_1)$ | $\xi \gamma (\tan\alpha - 2\tan\varphi_1) + \gamma_o \tan\varphi_1$ |
| less than 1 | $\gamma_o (\tan\alpha - 2\tan\varphi_1)$ | $\gamma_o (\tan\alpha - \tan\varphi_1)$ |

The horizontal displacement $dy_1$ within the deflection line of strata can be defined by means of the differential equation

$$d^2 (dy_1) / dz^2 - dM / EI_1 \tag{15}$$

at

$$dM = M_s - M'_r \tag{16}$$

$EI_1$ is the bending rigidity of the beam and is defined by

$$EI_1 = N(z) . E . m_o / 12 \tag{17}$$

N(z) is a number of layers within the beam section, $M'_r$ is the resistance moment due to the friction between the layers which are moving relative to each other on their contact surfaces while bending within the previously intact beam/plate; it is assumed that in consequence of disturbing the contact planes the cohesion along them will not exist and hence $M'_r$ has to be evaluated by (13) at $c_{I,i} = 0$.

By carrying out the summation of $dM_i / EI_{1,i}$ over all the layers in the beam we obtain

$$d^2 (dy_1) / dz^2 = 2\gamma' . (h-z)^2 / E(m_o)^2 \tag{18}$$

from where, at $dy_1 = 0$ (at $z = 0$) and $d(dy_1) / dz = 0$ (at $z = 0$), the horizontal displacement $dy_1$ may be obtained by

$$dy_1 = \gamma' h^4 . (z_n)^2 . (z_n^2 - 4 . z_n + 6) / 6 Em_o^2 \tag{19}$$

where a normalised value $z_n = z/h$.

The maximum of the horizontal displacement at $z_n = 1$ is expressed by

$$dy_{1max} = \gamma' h^4 / 2 Em_o^2 \tag{20}$$

and used for calucaltion of $\lambda$.

## BENDING STRAINS OF THE SLOPE AS A LONGITUDINAL BEAM

In order to carry out the deformation behaviour analysis for the plate under study, one should dwell on the question as to how important is the effect of finite dimensions of this plate [4], [5], [6]. Consider for this purpose the plate as a longitudinal beam being cut off from the rock massif on the pinching plane of the frontal beam (see above) and restrained along the strike (ie the case $\lambda$ is equal to infinity, see (7)) Figure 4.

The analysis of the conditions that contribute to initiation of deformations of the longitudinal beam is similar to that of the frontal beam above [4].

The critical length of the open pit slope at which the bending rigidity is lost as a result of rock mass loosening is expressed by

$$l_{cr} = 4 \cdot c_1 / \gamma' \qquad (21)$$

if $h = h_{cr}$ and if cohesion $c_{1h} = c_1$ and friction angle $\varphi_{1h} = \varphi_1$ are not dependent on the movement direction of the layers. The horizontal displacement of the beam subjected to the load $Q_x$ uniformly distributed along the beam's length l, may be described by the equation [7]:

$$dy_2 = Q_x \cdot 1^4 \cdot f(x/1) / EI_2 \qquad (22)$$

where x distance from the end of the beam, f (x/1) bending function depending on the conditions of pinching the ends of the beam, $EI_2$ bending rigidity.

If the central axis moment of inertia $I_2$ is defined for non-layered media (that is, before reaching the critical state) by means of equation

$$I_2 = h^4 / 36 (\tan \alpha)^3 \qquad (23)$$

it will be defined for the layered (that is, after reaching the critical state) rock mass composed of N layers, at the layer thicknesses $m_o$, by

$$I_2 = h^2 \cdot (m_o)^2 / 24 \tan \alpha \qquad (24)$$

that is, the bending rigidity will decrease at least by $2/3.N^2$ times, N being the number of layers which form the opencut slope. The active force $Q_x$ has to be determined as

$$Q_x = \xi \gamma \cdot h^2 / 2, \quad \text{if } \xi \gamma > \gamma_o \quad \text{or} \quad Q_x = \gamma_o \cdot h^2 / 2, \quad \text{if } \xi \gamma <= \gamma_o \qquad (25)$$

In order to find $dy_{2max}$ and $f_2(x)_{max}$ the approximated relation

$$dy_{2max} = \gamma'' \cdot 1^4 / 6 Em_o^2 \qquad (26)$$

may be used, where $\gamma''$ is to be adopted from Table 2.

Table 2.

| $q_1 / q_2$ | Ground water conditions within the slope area | |
|---|---|---|
| | totally drained slope | totally watered slope |
| > 1 | $\xi \gamma (\tan \alpha - 2 (h/1) . \tan \varphi_{1h})$ | $\xi \gamma (\tan \alpha - 2(h/1) . \tan \varphi_{1h}) + \varphi_o (h/1) . \tan \varphi_{1h}$ |
| <= 1 | $\varphi_o (\tan \alpha - 2(h/1) . \tan \varphi_{1h}$ | $\gamma_o (\tan \alpha - (h/1) . \tan \varphi_{1h})$ |

## EVALUATION OF λ

With substitution of $dy_{1max}$ (20) and $dy_{2max}$ (26) into the equations (4), (5) and then, taking into consideration, that here $Q_x = Q_z$, we obtain from (8)

$$\lambda = 3\gamma' . h^4 / \gamma'' 1^4 \quad (27)$$

By the time the greater deformations have occurred at $h = h_{cr}$ and $1 = 1_{cr}$, the value of λ is about 1 and hence the active load is distributed equally between the frontal and longitudinal beams. With the opencut slope length increasing, λ is rapidly decreasing and, at $1/h_{cr} = 3$, $\lambda = 0.03$ only.

## COMPARISON OF ACTUAL DATA

In April 1972, the upper bench at the Vakhrushev open pit of the Kuzbass coal basin slid down for the first time under similar conditions, accompanied by significant surface ground movement in the vicinity of the open pit [8]. The results of open pit surface survey and of in situ measurement of bench slope deformations indicated (Figure 5) that the slide movement was not restricted to the overlying loams, which was supposed from the visual observation, but involved also opencut benches composed of the basement rock. At the open pit district No 2 (West Part) where the slope slide occurred, the average slope height was 100 m, the slope angle was 30°, the deformed slope length was 300 m, coal bearing strata consisted of sandstones, claystones and siltstones and the Permian coal seams were inclined at 70 to 75 degrees into the rock massif. The upper bench containing loams of about 15 m thickness was entirely destroyed. Cracks, grooves and steps as a consequence of sliding were encountered 300 m along the strike of the slope face and 150 m into the slope body.

The water tables in the blasting holes on the pit benches before the slope failure stood about 3 to 5 m below the berm level and therefore one may use the calculation pattern applied for the totally watered opencut slope. The condition of limit equilibium is satisifed at following values of cohesion and friction angle on the contact planes of layers for $h_{cr} = 100$ m (see Table 3, according to formula 14):

Table 3.

| | | | | | | |
|---|---|---|---|---|---|---|
| $c_l$ (kPa) | 197 | 150 | 100 | 50 | 20 | 0 |
| $\varphi_1$° | 0 | 8 | 16 | 23 | 27 | 30 |

From these parameters, the values of $c_1$ = 100 kPa and $\varphi_1 = 16°$ are supposed to be a most probable pair of parameters for the Kuzbass coal field and correspond to the least values of contact plane strength characteristics obtained in the tests on the one-plane shear test device.

Knowing the maximum value of horizontal displacement, $dy_{max}$ = 10 m, the average friction angle, $\varphi_1 = 16°$ and the layer thickness, mo = 3.7 m, one could find from (20) the effective Young's modulus, E = $10^6$ kPa.

Table 4 contains the results of the measurements of horizontal displacements carried out at different times.

Table 4.

| Elevation of pit benches (m) | May 1972 | | October 1972 | | April 1973 | |
|---|---|---|---|---|---|---|
| | Height above pit bottom (m) | Horizontal displ. (m) | Height above pit bottom (m) | Horizontal displ. (m) | Height above pit bottom (m) | Horizontal displ. (m) |
| 385 | 100 | 10 | 110 | - | 110 | 16 |
| 365 | 80 | 9 | 90 | - | 90 | - |
| 345 | 60 | 4 | 70 | 3.3 | 70 | 7.3 |
| 325 | 40 | 2 | 50 | 2.5 | 50 | 4.5 |
| 305 | 20 | cracks | 30 | 2.5 | 30 | 2.5 |
| 285 | 0 | - | 10 | 0.8 | 10 | 0.8 |

The results obtained during the mine survey on 15 May 1972 were compared with the values calculated from (19) (Table 5).

Table 5.

| Elevation of pit benches (m) | Height above pit floor (m) | Horizontal displacements (m) | |
|---|---|---|---|
| | | measured | calculated |
| 385 | 100 | more than 10 | 10.0 |
| 365 | 80 | 10 | 7.7 |
| 345 | 60 | 4 | 4.8 |
| 325 | 40 | 2 | 2.5 |
| 305 | 20 | cracks | 0.7 |

Taking into consideration, that there are layers of thickness ranging from 0.5 m to 10 m and even up to 15 m within the open pit slope area and that the different lithological features of thin-laminated claystones on the one hand and of the massive coals and sandstones on the other hand pre-determine a rather significant variability of Young's modulus of different layers, one may consider that the measured and calculated horizontal displacements evaluated without distinguishing individual properties or layers agree reasonably well.

Figure 5 shows the results of ground movement measurements that have been carried out from April to October 1972 by using the marked points established on the berms of pit benches.

The measured values are compared to the calculated ones. By means of the fomulas (1, 2, 19, 26) the horizontal displacements at the points x and z have been calculated. Using equation (26) the non-dimensional relationships $\beta_1(z) = dy_1 / dy_{1max}$ and $\beta_2(x) = dy_2 / dy_{2max}$ are found and then $dy = dy_{1max} \cdot \beta_1(z) \cdot \beta_2(x)$ is obtained at $\lambda = 0$.

As seen from Figure 6, the general trend to decreasing horizontal displacements downwards and away from the centre of the deformed area is followed quite strictly. The actual displacements are not symmetric with respect to the centre line and are greater than the calculated ones. That the actual deformations fail to follow the symmetrical pattern can be explained by the fact that the rock in the south end of the pit is pinched more rigidly than that in the north end, because the brachysincline in the South is a closed fold and forms the boundary of mining activity (and so the profile pinching), whereas the northern boundary of the slope slide area is controlled by an extensive zone of gradually decreasing height and the slope angle of the pit wall.

Thus, the difference between actual and calculated deformations principally results from the adopted calculation pattern based on the assumptions which are only partially realised at the working opencut slope.

An additional check-up on the reliability of the calculation method took place after the open pit had been deepened in 1972-1973 up to 112 m deep at the same value of pit slope angle. The greatest displacements of the upper bench observed in Spring 1973 reached 16 m and had a reasonable agreement with the predicted greatest displacements calculated by

$$(dy_{max})'' = (dy_{max})' \cdot (h'' / h')^4 \qquad (28)$$

where $(dy_{max})''$ and h'' are the displacement and the height of the open pit after deepening, whereas $(dy_{max})'$ and h' are the same parameters before deepening.

## CONCLUSIONS

The above approach to the calculation of opencut slope deformations does not pretend to be mathematically strict and accurate because of rather rough approximations as far as the bending deformations analysis of the plate is concerned. It nevertheless enables a reliable qualitative assessment of the effect of bending deformations on the rock mass rigidity factors.

From a practical point of view, the algorithm of using the suggested relationships is intended for obtaining necessary quantitative strength and deformation characteristics of the rocks as far back as at initial stages of mining operations. The strength and deformation behaviour of the rock mass within the slope area is analysed using the results of rock movement observations on the opencut slope as a whole under the state of stress approaching the limiting state of stress within the slope portion subject to strains. On the base of these strength and deformation data one may design the stable non-operative open pit banks under boundary conditions also taking into account the permissible surface displacements and, if it is necessary, one may plan a drainage system or introduce changes in the excavation order with variations of the permissible working front length along the strike.

From the general point of view, the suggested interpretation of stress state of stratified rock mass can provide a physically clear basis for a better understanding of such complicated phenomena as earthquakes, tectonics, landslides and other processes in the earth crust, ancient and recent, natural and antropogeneous, as far as they are connected with changing the stress-strain state of sedimentary rocks.

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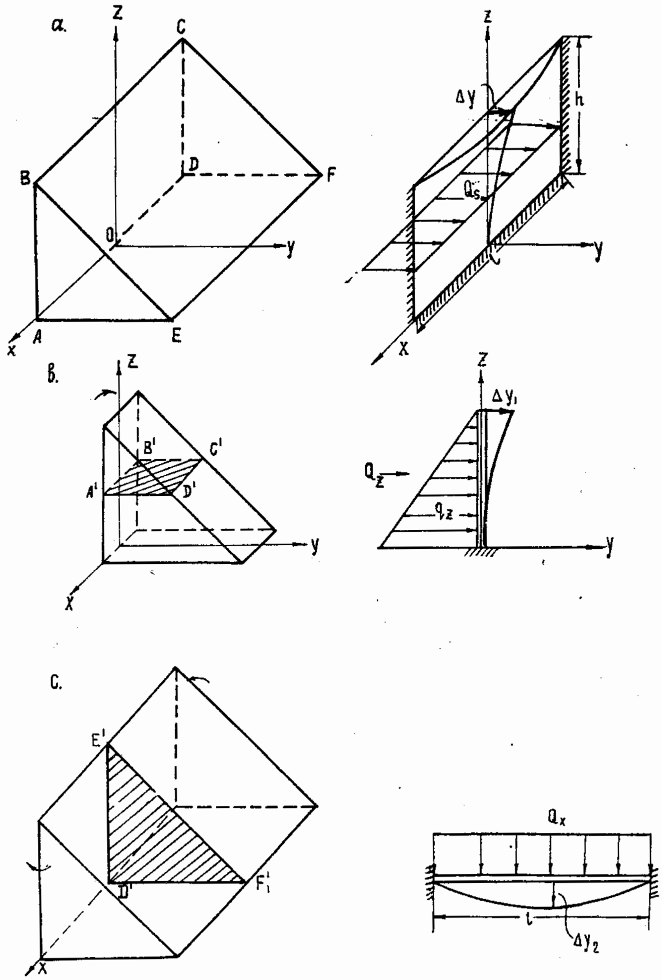


**Figure 1. General scheme of opencut slope bending calculation.**
(a) Slope-body prism cut off from adjacent rock mass is considered as a plate restrained at its longitudinal edges and from below and subjected to a lateral load with intensity of q(z) per unit area (or $Q_s$ per unit of length) on its rear side.
(b) Vertical strip of unit length, cut out the slope body prism, as a frontal cantilever beam fixed from below and subjected to a load with intensity of $q_z(z)$ per unit area (resultant force $Q_z$).
(c) Slope body prism undercut from below at the level of the open pit bottom as a longitudinal beam fixed at both ends and subjected to uniformly distributed load with intensity of $Q_x$ per unit of length (as resultant of load with intensity of $q_x(z)$ per unit area).
Arrow is direction of bending; shaded shown cross sections of beams, used in calculation of moments of inertia I1 and I2.

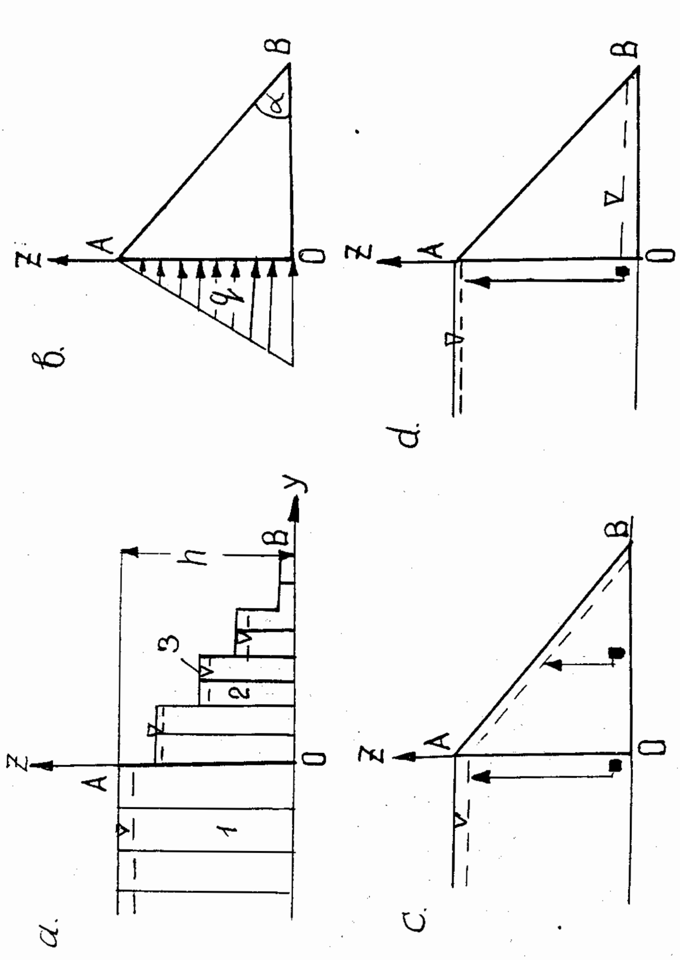


**Figure 2. Cross sections of the slope body prism.**

(a) General view of layered slope, 1 - footwall area, 2 - slope area, 3 - surface of a bench, ∇- ground water level.

(b) Distribution of external load with intensity of q(z) per unit area subjected to vertical edge OA of slope body prism OAB.

(c) Totally watered slope area. High ground water level are attributed to a poor drainage affect due to low permeability of rocks across the strike of the stratified structure.

(d) Totally drained slope area (usually due to horizontal drainage wells bored from the bench at the level of the pit bottom).

Shaded point with arrow means position of water pressure gauge.

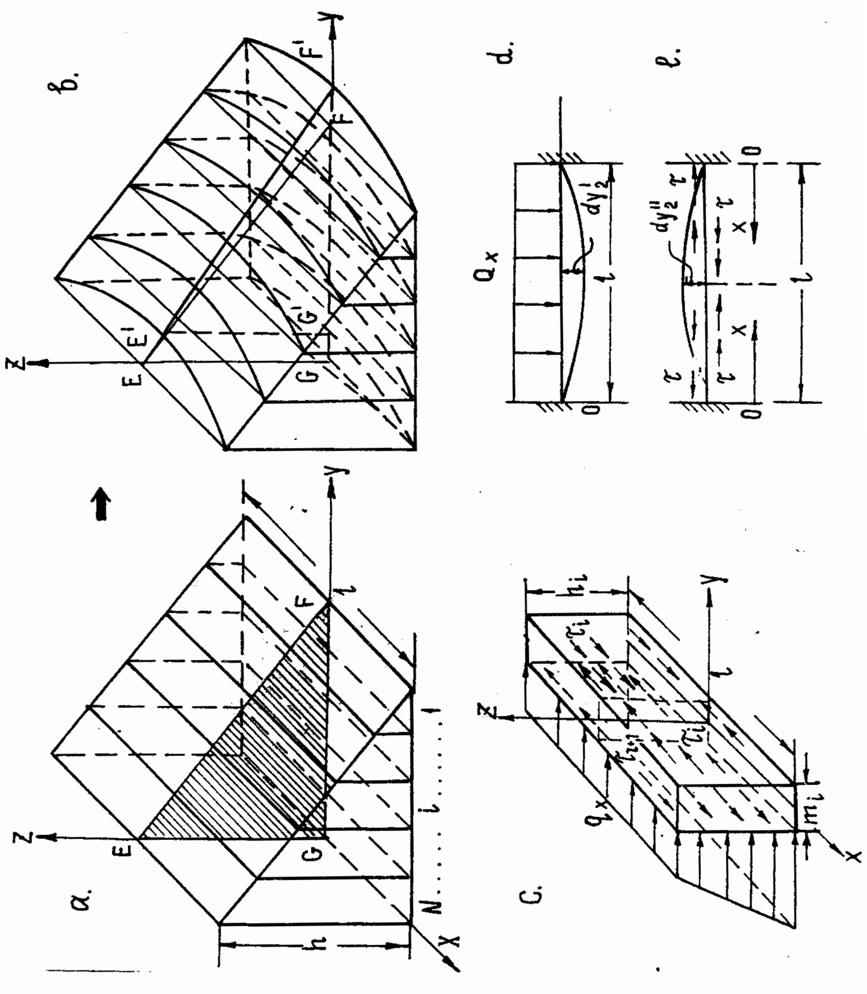

**Figure 3. Bending of the frontal beam.**

General view of the slope strip before (a) and after (b) deformation; i - number of layer; ABCD is a section at which the bending rigidity $I_1(z)$ is determined.

(c) Diagram of external load q subjected to the i-th layer; shearing resistance stresses $\tau$ create a moment preventing the i-th layer from sliding relative to i+1-th and i-1-th layers in vertical direction.

(d) Deflection curve of the beam before and after bending.

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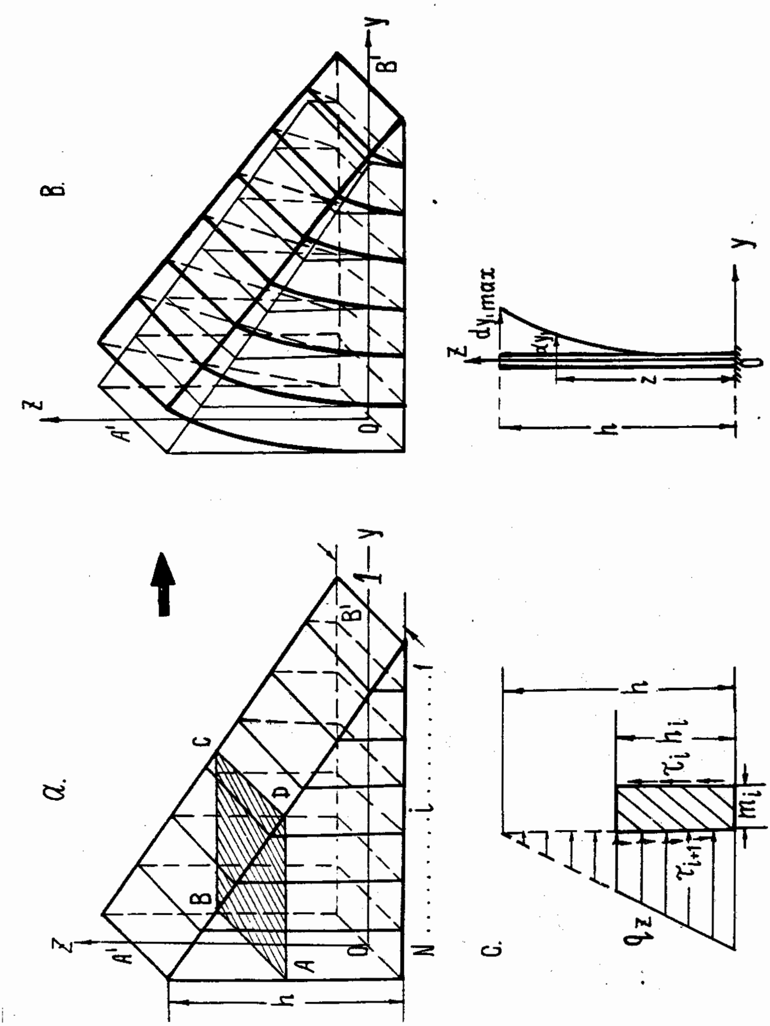

**Figure 4. Bending of the longitudinal beam.**

General view of the slope undercut from below before (a) and after (b) bending deformation; EFG is a section at which the bending rigidity $I_2$ is determined.

(c) Longitudinal bending of i-th layer subjected to external uniformly distributed load with intensity $Q_x(z)$ per unit of length (as resultant of load with intensity q(z) per unit area subjected to rear side of the layer under consideration); cohesion and friction strength $\tau$ create a moment preventing the i-th layer from sliding along adjacent layers in longitudinal direction.

(d) Deflection line of the beam due to only external load with intensity of $Q_x$ per unit length.

(e) Imaginary deflection line of the beam subjected to only resistance frictional forces.

(f) Resultant deflection line of longitudinal combine beam.

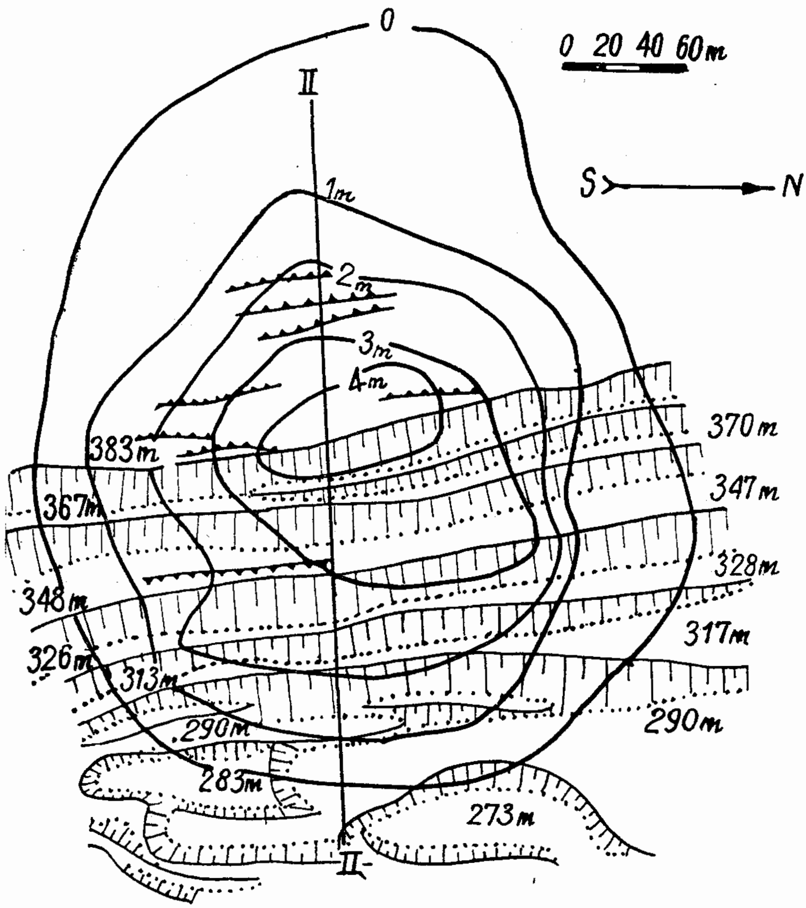


**Figure 5. Mining plan and profile of the slide area within the extraction district No 2, Vakhushev open pit**

(a) Mining plan with isolines of displacements in metres (Summer 1972); geared lines correspond to cracks.

(b) Profile 11-11 with displacement vectors (from 29.4 to 17.10.72). In the coal bearing series consistent of intermittent fine grained sandstones, siltstones and coal beds, only the coal seams are shown in the Figure.

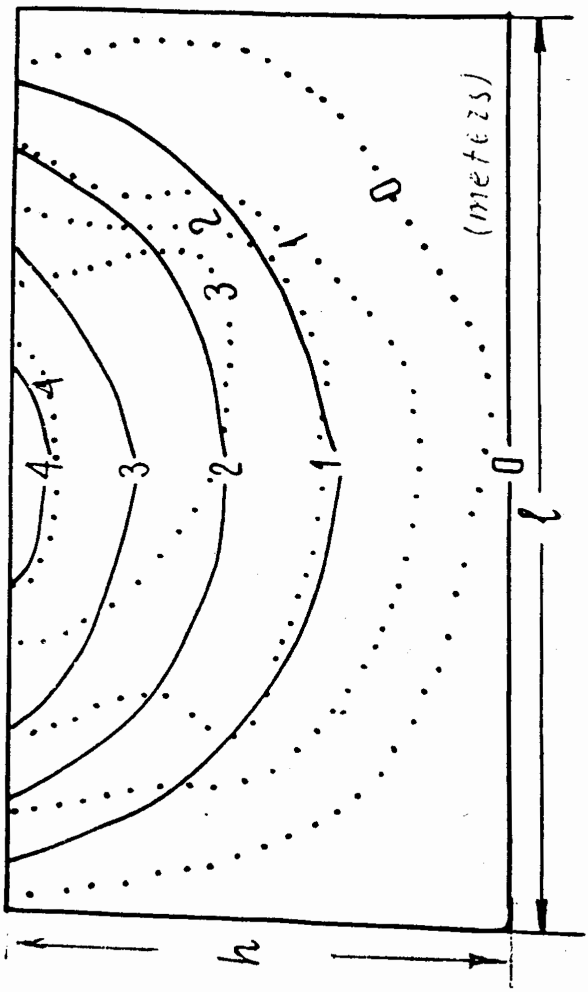

**Figure 6. Comparison of calculated (full lines) and measured (dotted) values of displacements at the district No 2 (in metres)**