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Research on Slope Stability of a Certain Open-Pit Mine

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During these thirty years and more, open-pit mining has advanced a lot due to the rapid development of gigantic excavation machine and its frequent application to mines as well as the advantage of the open-pit mining method. Nowadays the designed mining depth has already reached over 600 to 700m. With the rapid development of modern socialist construction in China, large and high slopes of coal, metal, non-metal as well as hydro-pivotal project have come into construction and use one by one. Foreign design of a certain open-pit mine disagreed with practice. Unstability of staired slope was discovered only two years after the mine came into operation and several stairslidings occurred so to damage the excavators and stop the railway transportation. Hence twice redesigns were made in 1961 and 1963 for the final slope boundary. Owing to inadequate knowledge of slope failure essentials and the main affecting factors, and lack of detailed investigation and scientific analysis, satisfactory results were not obtained. Wuhan Institute of Rock and Soil Mechanics of Academia Sinica, under the guidance of its director, Pro. Tan Tjon-Kie, took an comprehensive research on this open-pit mine slope stability. After two years of in situ investigation and test, and one year of calculation and analysis, a safe, economical and reasonable angle of slope and the corresponding measures to ensure slope stability were put forward at last. Since then mining practice has proved that slopesliding or other similar hazards did not occur for nearly 20 years, the slope was kept stable and the mining operation went on smoothly. The main principles of consideration and investigation and research work for slope stability are described below.

1. It is quite essential to make a clear investigation of in situ geological and hydrogeological environment and their relation to engineering in order to deal with the slope stability because they are decisive to maintain slope stability. This is the foundation for the whole research program. So ground survey and mapping of the slope area, 1000-2000m tunnel investigation, about 2000m drilling and acoustic method by borehole and seismic method etc. were used to determine state of rock mass, and geomechanical method was applied for analysis. Geomechanical investigation and research has two purposes:

1) Study the deformation process of ore body and the rock mass around the ore body, assess the orientation and extent of geological residual stress field by the deformation process of rock mass;

2) Make a distinct investigation of the fractures within the scope of 2-3 H (height of slope) around the ore body, which are formed inside the rock mass due to the complicated stress field around the slope and much related to slope stability.

The surrounding rock of the ore body has the following deformation processes:

- 1) Initially under the action of NNE-SSW principal compressive stress, some NWW folds and large-scale thrust faults and accompanying torsional and tensile fractures etc. were generated.
- 2) After the rock deformation under the action of principal compressive stress, NNE folds and thrust faults parallel to axis occurred on the background of NWW formation, which indicates the action of NWW-SEE principal compressive stress.
- 3) Torsional fracture planes spreading in both NE and NE directions develop widely in the whole area and are superposed on the formation traces

of the above two systems. They demonstrate mostly in the form of joints and some as fractures, which indicate the stress movement of S-N compression.

4) Besides the above, in the mining area S-N spreading compressive planes exist, but the trace can be seen only in some locations and on small scale, which shows the rock mass also experienced EW compressive stress movement.

Hydrogeological investigation results show that the hydrogeological conditions are relatively simple in this area. The main ground water supply is precipitation and has no hydraulic link with the river and lake around. According to pumping data, the seepage coefficient of crack under condition of water is small.

In the light of geomechanical, geological, hydrogeological investigations, slope layout, mining technique etc. the sloping area can be divided into 6 sections:

Section 1. It is composed of ore body, marble and diorite of different-sized blocks (from a few centimeters to over 2 meters) which have apparent angles and irregularity in order and no orientation in arrangement. Iron and calcium are bonded inside the rock so as to make the segment solid and compacted. No faults and other weak planes of some extent and orientation are further discovered passing here. Therefore it can be regarded, on a large scale, as well-bonded breccia, which has uniformity and integrity to a certain extent and is favourable for the slope stability.

Section 2. Strike of main weak planes of this section is NNE almost normal to slope strike. Also, a set of faults (F_3) of NWW or nearly EW strike cross the rock mass. In view of slope stability, the former is favourable and the later is unfavourable. Except a few faults being well-mixed planes, most planes are compressive or/and torsional, and well extensive horizontally and

vertically. Most are crushed breccia, part are mylonite and no water is deeply in the fractured zone.

Section 3. Two faults are the separate-line of this section. Main strike of rock layer in this section is parallel to stope. Groud surface inclines along the slope, turns backward in deep part and inlines southward. Main planes affecting rock stability are stratification planes (along the sliding plane), NWW compressive planes, torsional fault F_{11} and tensile fractured zone F_{23} . Except F_{23} being watered zone, other faults are not water-bearing. Stratification planes in the middle and upper parts incline along alope and affect the slope stability greatly, but affect small after turning backward and inclining southward. Faults F_{23} , F_9 , F_{11} , F_{13} in slope shoulder may be within the extension after the slope was formed, and fractures may occur along the faults and stability be damaged. Rock mass near slope toe is relatively intact and no larger faults appear. The designed excavation depth is -96m and slope height is 230m. Section 4. Dependents of rock stability in this section are mainly the bedding weak planes parallel to the stope, nearly SN tensile fractures, and F_{25} torsional fractures. Tensile fractures and bedding weak planes cut the rock mass into several "triangle blocks" and lead to the failure of upper part in slope easily.

Section 5 and Section 6.

2. Slope stability mainly depends on the slide factors and the resistance conditions against slide factors. These factors are:

- 1) Strength and deformation properties
- 2) Gravity
- 3) Residual stress
- 4) Dynamics by blast and excavation
- 5) Forces by underground water movement
- 6) Stress by earthquake
- 7) Temperature stress caused by temperature change

In order to investigate comprehensively and correctly the sliding factors and slide-resistance factors, the following test researches were carried out:

- 1) In situ tests of rock strength and deformation characteristics and lab tests of strength and deformation behavior for rock samples;
- 2) Residual stress measurements near the mining area and project site;
- 3) Chemical analysis of underground water, mineral and physico-chemical analysis of filling materials in weak intercalations;
- 4) measurement and research of dynamic effect by blast;
- 5) photoelastic test for the materials.

3. Test research on rock strength and deformation properties

Controlling factor of rock slope stability is mainly the tectonic strength of rock mass and the deformation properties, not the strength and deformation of rock itself.

Based on the said above, the stope can be divided into a few geological-mechanical sections with different boundary conditions and features, in which the mechanism of slope sliding may differ. In typical case, it can be regarded that the upper 1/3 part of slope are the weathered and tensile zones; the middle 1/3 part is basically sloping shear zone where slopeslide mainly occurs along bedding weak planes; the lower 1/3 part is stress concentration zone where gliding planes may cross rock mass or bring about larger deformation so as to cause slope unstability due to high stress concentration by earth stress and

gravity. (Fig.1)

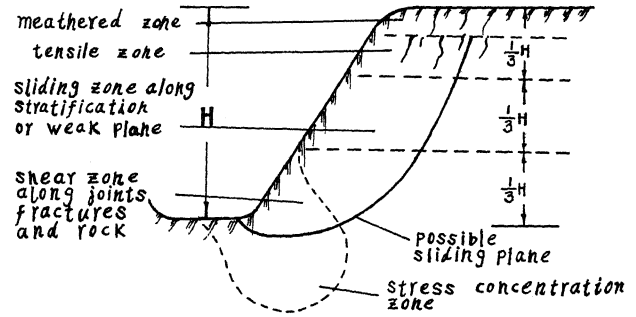


Fig.1 Slopeslide

Purposes of test research on strength and deformation properties of rock mass are:

- 1) Find out the reliable and objective strength of fractured plane;
- 2) Find out objective strength of rock mass without fracture plane.

Hence different tests of rock mechanics should be carried out at different locations of slope, i.e., strength and deformation tests of fractured plane, stratification plane and weak intercalation plane are mainly done from lower to middle part of slope. The occurrence and strike of these planes are along the slope strike, and the tilt is generally parallel to sloping angle.

Taking into account ground and underground water, shear test under saturation should be done besides quick shear test. Since fractured plane is rheologic and rheologic phenomena are divided into two kinds: 1) deformation increases with time; 2) strength decreases with time, strength rheologic test of rock mass and weak planes must be done to determine deformation-time relationship curve at stepped loadings in order to ensure the long stability of slope for several decades. (Fig.2)

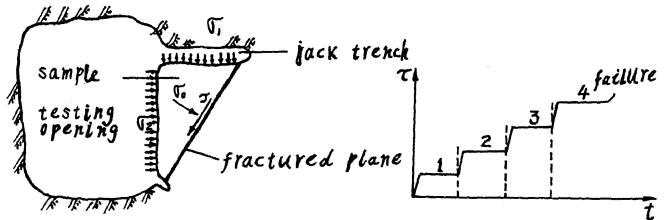


Fig.2 Weak plane shear rheological test and stepped loading

Rock mechanics tests in deep part of slope depend on the local geological conditions, fracture orientation should be paid special attention to and slope toe problems are mainly solved.

Results of rock mass strength and deformation property tests are shown in Tab.1.

Ser. No.	Dep. of Test	Geology	No. of Samples	σ kg/cm ²	ϕ degree	Remarks
1	45	5-10cm thick weak intercalation, fillings of white calcareous spar powder, coarse grain, crystal, fluid, smooth, poor bonding	2	2.5	28	
2	150	weak plane occurrence N35° W/L SW65° 0.5-0.8cm thick weak intercalation with filling of calcareous spar crystal and breccia	4	1.5	23.3	
3	150	weak plane occurrence N80° W/SW65° within the fracture zone of Fault F15, well-developed bedding fissures, filling of carbonite and peaty materials	2	1.3	30	
4	120	well-developed bedding fissures, occurrence N15° W/SW65° green argillaceous filling, 0.1-0.2cm wide smooth and closed fractured plane, 0.5cm thick intercalation	4	1.7	37.3	

Tab.2 Results of weak plane shear test and shear test of rock mass

No. of Test Opening	Dep. of Test Opening (M)	Geology	No. of Sample	Shear Resistance		Friction		Remarks			
				τ	σ	α	β				
1	45	1-6cm thick intercalation, filling of white calcareous spar coarse grain, crystal, smooth, humid, poor bonding	10	6.25	37.8	35.1	2.08	26.6	weak plane shear		
3	150	white small-stripped marble, fissures are mainly closed and bedding fractured plane flatter, fillings of carbonate peaty, brownish gray siliceous materials etc. 100-200cm	4	1.4	39	2.23	0.132		weak plane shear		
2	150	grey white stripped marble, well-developed fissures not large fissure width, mostly closed, filling of crystal calcareous spar	5	4.95	44	40	440	24	23	27	rock mass shear
5	210		7	34.6	60.5	18.0	54	15.1	48	rock mass shear	

τ*-failure L**limit S***scale.

From deformation property tests it is obtained that elastic modulus for rock mass is about $70 \times 10^4 \text{ kg/cm}^2$, for fault fractured zone is about $7-20 \times 10^4 \text{ kg/cm}^2$; deformation modulus for rock mass is about $20-40 \times 10^4 \text{ kg/cm}^2$ and for fault fractured zone is about $1-6 \times 10^4 \text{ kg/cm}^2$. From the test results it can be seen that shear rheological strength of weak plane is generally less than shear strength of weak plane. For the rock mass of the same layer, e.g. at 45m depth of Test Opening I, rheological strength is less about 30% than shear strength while shear strength of rock mass is much higher, even several times higher than shear strength of weak plane (both c, ϕ).

4. In situ stress measurement and determination For long in analysis of rock slope stability, stress state has hardly been paid attention to and only gravity action been considered. Along with geomechanical development and a lot of in situ tests, it is understood that besides the gravity of rock mass itself the residual stress from geological tectonic movement exists. Sometimes the later is several times larger than the gravity and often changes in some areas (earthquake zone or large hydraulic project zone etc.) So it is one of the important influence factors for slope stability.

Generally stress of rock mass consists of 1) gravity 2) residual stress by geologic tectonic movement 3) pore pressure 4) dynamic force by blast or earthquake 5) stress redistribution by excavation or reservoir construction. Our tests mainly aim at determining residual stress but we take it into account that stress state in rock mass is a comprehensive phenomenon.

Rock stress, especially the residual stress, will exert adverse effect on slope, which demonstrates in the following aspects:

1) High stress concentration will take place at slope toe after excavation which may cause part of rock mass to be in plastic state so as to reduce the strength of rock mass and produce large deformation. (Fig.3)

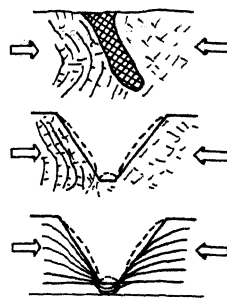


Fig.3 Effect of geological residual stress on slope

2) Due to nonuniform relief of stress after excavation, original rock mass loses its equilibrium, or even fails, so as to reduce the strength.

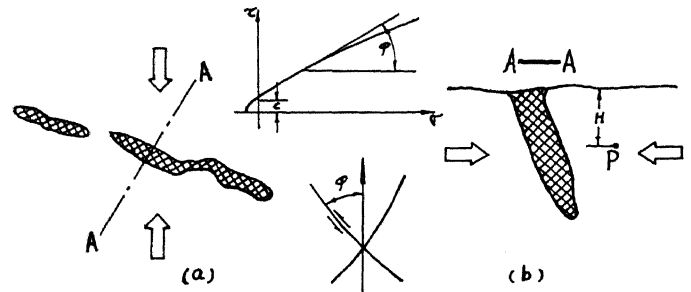


Fig.4 Interrelation among stress field, displacement, strength and fissures

3) Comprehensive action of various stresses increases the gliding force.

The concrete tasks of measuring stress include: 1) measure stress field in large area; 2) measure the stress field in rock mass around the ore body.

Since the rock mass experienced a lot of tectonic movements and have been affected by various joints, fissures, faults, the stress state is very complicated. Besides the tectonic stress field in the area, there is stress field originated in the process of ore generation (including temperature stress field). It is required to make detailed geomechanical investigation at first to know the history of rock deformation and stress action of the whole area as well as the process of ore generation. Then, on the basis of the above, stress measurement is done.

We selected two locations for deep-hole stress relief tests some distance away from the ore body to determined stress field of the large area. And in 5 testing tunnels drainage galleries and inclined shaft in upper, middle, lower layers and different parts of the slope, over 80 points of stress measurements were carried out to determine the stress field of rock mass around the ore body.

What we applied are three kinds of stress relief method, i.e., deep-hole (100-120m down from ground surface), shallow-hole (a few meters within the rock wall of the tunnel) and wall surface (determine the local surface strain of rock) methods. All of them are to cut rock off the rock mass, to determine the elastic deformation of rock due to stress relief by separation, and then to calculate back the stress on the basis of known rock elastic modulus. (Fig.5)

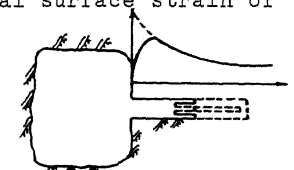


Fig.5 Schematic diagram of stress measurement (shallow borehole)

The stress relief method assumes 1) rock is elastic body 2) rock is homogeneous and isotropic. Measurement points should be selected in as much homogeneous place as possible and away from cracks. By tests the mean value of stress field in the large area was determined as $\sigma_1=100-145\text{kg/cm}^2$, $\sigma_2=76\text{kg/cm}^2$, horizontal azimuth angle of principal stress changes gradually from N30°E to N82°E at 19m down to 110m underground.

Tab.3. Shallow-hole stress releasing results After eliminating stress concentration effect

Point No.	Max-change of ratio of principal stress	mean value of principal stress	action orientation of principal stress
1	198-114	160	N82°-NW10'
2	270-120	150	N82°-NW5'
3	174-383	378	NW6°
4	270-100	350	NW7°
5	203-560	400	N85°-SN
6	325-396	347	NW6°-SN

Results indicate that the horizontal residual stress normal to mining slope strike is about 100-200kg/cm² and weights as 2-4 times as the overloading. It is smaller in shallow part than in deep part. There is a local but complicated stress field near the ore body. Local jointed fractures and adjacent faults affect stress severely, and the stress normal to fault orientation is often small.

5. Dynamic measurement by blasting
It is quite essential for blast dynamics to affect the slope stability. Slope problem for the open-pit mining slope can be divided into two for research solution. One is mass sliding of the large and high slope, called as large slope problem. Another is terraced slope, i.e. small slope problem. Excavation and blasting method influences small slope greater. Vertical-hole blasting is apt to cause rock layer failure at 10-20m below the ground surface. W is much great during pumping water, even water pressure test can't be carried out. It is rather dangerous for a slope of dozens or hundreds of meters height to have such a layer. Raining water may also infiltrate, so after several years friction angle and strength of the fractured plane with weathered minerals will decrease further and this fractured zone will bring about instability. (Fig.6 a,b,c)

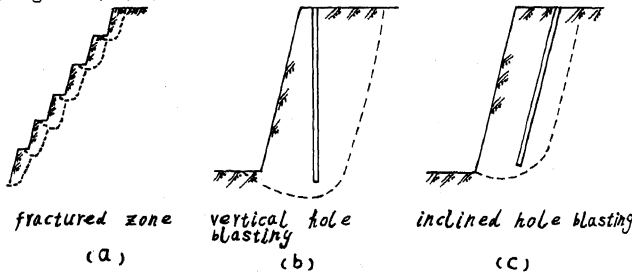


Fig.6 Fractured zone and comparison between vertical hole and inclined hole blastings
Theoretically, in isotropic homogeneous medium the effect of inclined-hole blast is only about 60% of vertical-hole blast. Millisecond blast results in even smaller synthetic wave, quicker attenuation and less effect. Contents and purposes of blast dynamics measurements are:

- 1) Research on the effect of stress wave on de rock mass, determination of acceleration, velocity and displacement of the tunnel surface under blast action;
- 2) Comparison of blast effectiveness between vertical and inclined hole blast and the effect on deep rock mass (due to lack of test condition this comparison test was not completed). Test method is shown as in Fig.7. a, b. The material is marble in test site. All blasts are group blastings of shallow-buried column charging. Blast site is on the surface and test tunnels are underground. The tunnels are circular (with 2m in dia. and 15m in length) and square (with 2x2m section and 30m in length) respectively. 2 and 4 profiles of acceleration measurements are respectively for these two tunnels.

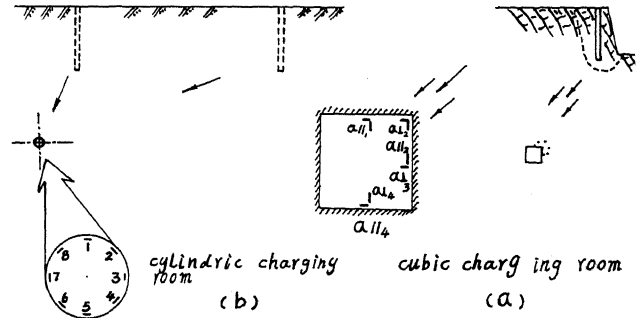


Fig.7 Schematic diagram of blast test

Tab.4. Measurement results in circular tunnel

Ser. No. of blast	Rock	Charging Method	Blasting Method	Max. No. of Blast Hole & Arrangement	Area, Wave Distance (m)	Max. Effect Velocity	Measured Acceleration	Remarks
1	marble	nitrocellulose 2836	25 millisecond electric cap	12 in two rows, 6 for each	6600	60.6	49.8	
2	marble	nitrocellulose 3600	25 millisecond electric cap	14 in two rows, 7 for each	6600	70.7	32.6	
3	marble	nitrocellulose explosive	fuse blast	14 in one row	6600	52.4	28.8	

Tab.5. Measurement results in square tunnel

Ser. No.	Rock	Charging Method	Blasting Method	Max. No. of Blast Hole & Arrangement	Area, Wave Distance (m)	Max. Effect Velocity	Measured Acceleration	Remarks
1	marble (marble in the vertical face)	nitrocellulose 9700	electric cap	28 in two rows	31	759.03	73.0	
2	nitrocellulose explosive	2700	electric cap	19 in one row	44	5500	59.03	73.5
3	nitrocellulose explosive	2700	electric cap	10 in one row	119	6263	2.30	2.43
4	nitrocellulose explosive	2700	electric cap	11 in one row	96		5.38	2.85
5	nitrocellulose explosive	10000	fuse	35 in one row & 17 in two rows	157	6826	0	1.79
6	nitrocellulose explosive	8000	fuse	35 in one row	55	6750	4.62	3.57

It is proved by the experiment that the current vertical-hole blast method severely affects the stability of terraced slope. Within 50m blast pulse acceleration can reach dozens of g, e.g. the max. accelerations is 59.03g in the distance of 44m by charging of 6.7 tons so that square tunnel is damaged greatly.

Blast pulse acceleration attenuates quickly with the increase of distance. By calculation of data from circular tunnel, the equation of axial particle acceleration excited by shallow-buried column charge in marble is obtained as follows

$$\frac{a_2}{a_1} = \left(\frac{\phi_2}{\phi_1}\right) \cdot \left(\frac{r_1}{r_2}\right)^4 \quad \text{or } a = 0.208 \times 10^6 \cdot \phi \cdot r^{-4}$$

which indicates that acceleration decreases with r to the fourth power.

It can be seen from the square tunnel data that 1) the max. pulse acceleration is at the angles of the square tunnel;

2) Blast compressive wave transmits to tunnel and reflects on the surface of tunnel.

When tensile stress generated by the reflective

wave is greater than the tensile strength of rock, tunnel is damaged and rock blocks falls down. Failure often occurs at the upper corners which are nearest to blast source.

6. Photoelastic simulation test and research on filling components in rock cracks and their certain physico-chemical properties.

The purpose of the former is to determine stress distribution, principal stress trace, stress concentration coefficient and tensile force area in rock slope under action of geologic tectonic movement and gravity (in terms of plane problem). The later is to carry out the surface area, ion exchange and differential thermal analysis, and to analyse the amount of various ions contained in ground water within the rock cracks so as to understand the effect of physico-chemical properties of crack fillings on rock strength. The conclusions drawn in this ore area are 1) ion exchange amount and surface area of crack fillings in all sections are smaller, stability of weathered minerals is better, surface activity is not so large. The fillings have a good absorption of water so as not to produce dilatation.

2) There are more HCO_3^- , SO_4^{2-} in ground water within rock cracks, which exerts erosion on carbonatite and reduces rock strength after long years. On the other hand, high content of SO_4^{2-} in ground water may react chemically with unstable minerals such as isolated Ca^{2+} in fillings so as to produce some mineral like gypsum $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$, thus fissure plane will become smooth and indexes like weak plane rheology, cohesive force, friction coefficient etc. will be reduced. But the adverse effect is not too great for as long as decades of years operation in a mining slope.

7. To meet the needs of production, quantitation should be provided besides qualitative analysis. The core work in quantitative evaluation is analytical computation which depends on correct qualitative analysis and reflection of objective environment.

1) Computational analysis method

Limit equilibrium analysis or limit equilibrium diagram of vertical strip method is employed for plane problem. It is simple and in correspondance with distribution of in situ jointed crack groups. According to assumption of action force on strip boundary planes there are four cases: (Fig.8, a, b, c)

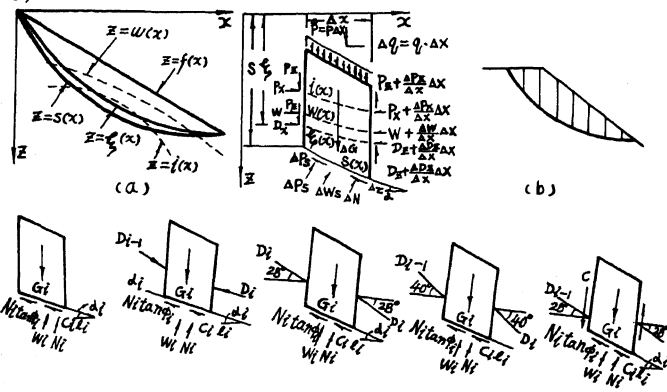


Fig.8 Force action analysis for strip-block boundary planes in calculation
 (1) There is contact force between strip boundary planes. Assume that the trace $\xi(x)$ is known, so several $\xi(x)$ are computed respectively and the minimum of safety factors is what we need. This is abbreviated as D_c method.

(2) Assume that the orientation of contact force is parallel to sliding plane, i.e., $D_y/S(x)$. This is abbreviated as D_s method.

(3) Assume that there is no any contact between strip boundary plane, i.e., $D=0$. This is abbreviated as D_0 method.

(4) Assume that contact force of strip boundary planes and the normal line of boundary plane have a certain fixed angle or there is still cohesive force. This is called as D_θ method. In calculation of each profile, several slope-slice arcs are assumed respectively based on geological condition and 3-dimensional location of weak plane, and each arc is calculated and the corresponding safety factor is obtained. (Fig.9)

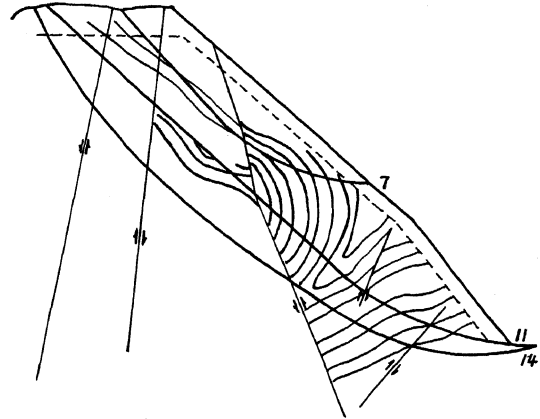


Fig.9 Probable sliding plane in a certain profile (calculate respectively)

- 2) Reflection of factors in computation
- (1) Stress concentration due to earth stress has already reached or surpassed strength index of rock mass or weak plane in some part of the slope, which does harm to rock mass and weak plane. Thus the shear strength index (friction) after shear failure from in situ test is used in computation. It is also considered that ground water may infiltrate into crack planes.
- (2) With reference to underground water table data obtained by geological drilling and in consideration of the water table change along with mining tunnel shaping, "sweating curve" in profile for calculation is obtained. Based on (1), supporting float force acts on the sliding plane and the force depends on the sweating curve. Generally water pressure is not considered on the strip boundary planes because this boundary plane is assumed and affects little as a whole. Area revised coefficient is not considered in computation of water pressure. In fact, it is not possible that water pressure acts on the whole plane. Therefore doing so is more safe.
- (3) Static pressure due to crack water (after storm) in the crack walls above the sweating curve should be regarded as additional water head pressure in checking the computation.
- (4) Shear strength of fractured plane is reduced due to saturation of ground water. Test data under saturation indicate this effect.
- (5) After analysis of ground water and physico-chemical properties of filling materials in weak planes, it is considered that these two factors are not necessarily taken into account in reduction of strength.
- (6) Based on drilling water pressure test and ground surface investigation, the surface fractured thickness is about 20m; and based on blast

observation about 10m thick rock mass normal to the slope was damaged. Strength indexes in these two parts are reduced correspondingly in computation.

(7) In the case of terraced slope, blast seismic effect is considered; in the case of whole slope, the effect is appropriately taken into account to determine the safety factor.

(8) In the area of 25m width of inverted axial part in geological investigation, tensile joints develop better, the strength index should be decreased correspondingly in computation.

3) Shear strength index

Field test results are applied with lab test results as comparative reference. Shear strengths of weak plane and strata fractured plane are shown in Fig.10.

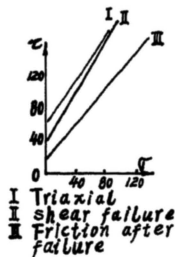
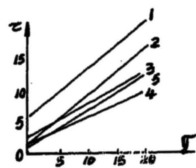


Fig.10a Result of field tests Fig.10b Result of lab tests

Rheologic shear curve is employed as strength index curve of weak plane in the direction of slope. (With consideration of slope operation as long as decades of years).

When sliding plane is not bedding fractured one, a simple treating of strength as anisotropic one is applied, i.e., the strength oval is the basis of conversion of strength index.

In addition, for strength the following are specified:

- (1) In the fractured section of slope top, $c=0$
 $\phi=280$ (for weak plane)
- (2) In the fractured zone of inverted axial part,

$$c = \frac{c}{1.2}, \quad \text{tg}\phi' = \frac{\text{tg}\phi}{1.2}$$

- (3) In the fractured section of slope, $c^n = -\frac{c}{5}$,

$$\text{tg}\phi'' = \frac{\text{tg}\phi}{1.2}$$

c and ϕ can be obtained in terms of oval.

4) Achievement from computation

- (1) Results from D_c and D_s methods are quite approximate, D_s method is simpler and reasonable.
- (2) Computed result in terms of saw-tooth sliding plane is the same as that of bedding slip.
- (3) Water pressure effect on sliding plane is severe (even up to over 200%).
- (4) There is little difference between computed results with consideration of superposed earth pressure and without.

8. Conclusions

- 1) Research on mining slope stability needs combination of geology and mechanics, of theory and practice. It is essential to do geomechanical analysis in geological investigation, which is the basis of other works. Not only current state of rock, but also the history of rock mass development should be known so as to predict the possible development of future.
- 2) Tectonic residual stress, its orientation and intensity play an important role in keeping slope stability. Residual stress is related to development history of local geological formation, i.e.,

to history of rock mass deformation and stress action of the whole area.

3) Rock mechanical strength and deformation characteristics are main factors of resisting slope slide. This strength is the tectonic strength that is, the strength of fissures and weak intercalations between rock strata or rock blocks, and the strength of rock itself is secondary. Practice proves this in that field shear and rheologic strengths of weak plane are much lower than the lab shear strength.

4) The physical boundary conditions of the strength and deformation property test must be taken into account with combination of engineering and mechanical conditions of slope. Strength indexes are obtained by different tests of different boundaries respectively at top, waist and bottom part of slope. For example, at top the tensile test is the main, at the waist part the shear and rheologic tests of fractured weak plane in the direction of slope are the main, while at the bottom shear test of rock itself along the fissures is the main. Also saturation and rheology etc. must be considered since slope stability should last several decades of years. Generally rheological strength is less, sometimes even over 20-30%, than shear strength.

5) After the investigation in such a comprehensive way as above, the conclusions from analysis and computation are in good agreement with the actual situation. Practice of these twenty years have proved that all slopes are stable and safe that were treated in the light of suggested reasonable slope angle and required measures.

9. References

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