Selection of basic data for numerical modeling of rock mass stress state at Mirny Mining and Processing Works, Alrosa Group of Companies

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Abstract. The influence of structural features on the strength and elasticity modulus is studied in rock mass in the area of Mirny Mining and Processing Works. The authors make recommendations on the values of physical properties of rocks.

Analysis and prediction of geomechanical behavior of rock mass in the course of mineral mining uses the method of mathematical modeling, as rule.

In the numerical modeling and solution of geotechnical problems, the input data are governed by many parameters: mineralogy and crystal structure of rock mass, presence of weakening planes and weakening intensity, orientation of fractures, weakening nature, hydrogeology, etc. these parameters mostly condition strength and deformation characteristics of rocks, load-bearing capacity and stability of exposed elements of mine system, and dimensions of the latter [1]. So, it is important to correctly select data for the numerical modeling.

The most complicated and least theoretically validated problem in geomechanics is studying strength and deformation characteristics of rocks. The current estimation procedures for the mechanical properties of rocks involve:

— in situ measurement of strength and deformation characteristics by means of full-scale testing of large specimens commensurable with the dimensions of structural elements of mining systems;
— determination of physical parameters from the inverse problem solution given the known nature of deformation, generated stresses and measured strains, based on the available analytical solutions or numerical modeling;
— inclusion of the rock mass structure influence by means of so-called structural weakening coefficients that can be determined from direct tests or by calculation.

The simplest way of transition from the mechanical properties in core tests to the properties of rock mass is allowed by the introduction of the structural weakening coefficient (Constructions Norms and Regulations SNiP I-94-80) which is determined as a number of joints per running meter. This coefficient ranges from 1 (solid intact rock mass) to 0.1 (more than 10 fractures per running meter).

The most popular relations to find the structural weakening coefficient are described below [2].

As recommended by Slesarev [3], compression strength, tension strength and cohesion of rock mass can be given by:

\[
\sigma_{\text{com}}^m = K_1\sigma_{\text{com}}^0; \sigma_{\text{tm}}^m = K_2\sigma_{\text{tm}}^0; C^m = K_3C^0,
\]

where \( K_1, K_2, K_3 \) are structural weakening coefficients.
where $K_1 = 0.3–0.35$; $K_2 = 0$ at dense jointing, $K_2 = 0.01–0.1$ at closed joint, $K_2 = 0.01–0.05$ at intensive micro-jointing; $K_3 = 0.6–0.9$ in rock mass with a network of micro-joints, $K_3 = 0.05–0.25$ in rock mass with closed contact fractures, $K_3 = 0.01–0.02$ in rock mass with weakening bedding planes with slickensides.

According to Fisenko [4] the structural weakening coefficient $K_{sw}$ for the estimation of rock cohesion and ultimate compression strength is

$$K_{sw} = 1/(1+a\times\ln(H/d)),$$

where $a$—coefficient governed by rock specimen strength and jointing, grows from 1 to 7 as strength of specimens increases from 1 to 100 MPa; $H$—average size of exposure; $d$—average size of elementary block in rock mass. The size of elementary blocks is determined by the spacing of joints.

The deformation modulus can be estimated using procedures by Ruppeneyt [5]:

$$E_M = E_0/((1+5N\eta)/8); E_M = 1.6(1+1/k)E_0; \eta = \delta(\beta d),$$

where $E_M$ and $E_0$—deformation moduli of rock mass and rock specimen, respectively; $N$—number of joints in a system; $\delta$—opening of joints, mm; $d$—average distance between the edges of neighbor joints; $\beta$—relative area of hard rock contacts (Ruppeneyt recommended $\beta = 3\times10^{-3}$); $k$—fracture porosity which is a ratio of volume of pores (fractures) to total volume of material. The first equality is valid in rock mass containing no more than 4 systems of joints; the second equality is for the case of chaotic jointing when rock mass behaves as granular medium.

Aside from the structural weakening coefficient, it is common to use the coefficient of long-term strength. Assessment of long-term strength in rocks has currently insufficient theoretical backup and is a complicated problem.

A feature of stress state studies is the impossibility to measure actual stresses directly in a solid medium. The stress state is examined indirectly, through measurement of other values connected with stresses by different relations. For this reason, an apparatus to convert measured values to stresses and the model of a medium included in this apparatus are the key points in the theory of measurements.

In mines of Mirny Mining and Processing Works, stresses are measured using the method of parallel drilling (by the Institute of Mining, Siberian Branch, RAS), borehole slotter and the approach based on “memory” of rocks possessing property of creep: when such rocks are subjected to long-term loading, structural rupture results in development of both elastic and plastic strains [6].

Nonuniformity of deformation characteristics influences initial stress state of rock mass. The data of in situ measurement of initial stresses are widely scattered within the same deposit both in point observations and in averaged values obtained in different areas of the deposit. Such scatter cannot be explained only by experimental imprecision or by data processing errors. The most significant factor of such pattern is the nonuniformity of the stress field conditioned by the rock mass structure, faulting and different elastic characteristics of rocks composing the deposit.

There are many procedures available for the processing of experimental results. The traditional method is the averaging of stresses determined in different areas but at the same depth in rocks; then, the averaged values are used to plot stress–depth curves. These curves are used later on as the boundary conditions in calculation of stress–strain state of enclosing rock mass around underground excavations and to determine factor of safety of underground structures.

In solving problems in geomechanics, the differentiation approach should be used in setting boundary conditions. When an underground excavation has such dimensions that its influence zone occurs within a uniform rock mass (layer), it is possible to take in situ stresses measured in this layer. Otherwise, to make the stress state calculations more accurate, it is necessary to refine boundary conditions by solving some problems on nonuniform rock mass.

For the reliable determination of the initial stress field parameters, point measurements should be carried out on a grid based on variation in properties of rocks and characteristics of jointing.

Deposits developed by Mirny Mining and Processing Works have complex geology. Ore bodies, enclosing rock mass and backfill have different strength and deformation characteristics. Enclosing
Rock mass is presented by alternating rocks of medium strength (limestone, dolomite) and rocks of high plasticity (rock salt) with inclusions of dolerite sill.

In situ tests on stress determination at actual elevation of –365 in Mir Mine revealed anisotropy of deformation characteristics of dolomite.

The lateral earth pressure coefficient varies from Dinnik’s value to 1.2.

An emphasis should be laid on influence of brines on the strength of natural rocks and backfill.

The effect of highly mineralized water of Metegerso-Ichersky aquifer of strength of kimberlite, enclosing rock mass and backfill depending on length of exposure is in detail described in [6].

**Table 1.** Strength characteristics of rock suites at Mir kimberlite deposit (determined in tests of samples).

| Suite          | Occurrence depth, m | Thickness, m | Bulk weight $\gamma^\star 10^2$, MN/m$^3$ | Uniaxial compression strength $\sigma_{com}$, MPa | Cohesion $C$, MPa | Internal friction angle $\varphi$, deg |
|----------------|---------------------|--------------|-----------------------------------------|-------------------------------------------------|-----------------|--------------------------------------|
| Ilginskaya     | 0–170               | 170          | 2.34–2.68                               | 4.3–41.5                                        | 2.1–11.7        | 23–40                                |
| Verkhokolenskaya | 170–315             | 145          | 2.40–2.53                               | 8.3–22.4                                        | 2.4–8.9         | 23–41                                |
| Metegerskaya   | 315–430             | 115          | 2.40–2.65                               | 17.6–24.0                                       | 4.4–7.0         | 27–35                                |
| Icherskaya     | 430–520             | 90           | 2.42–2.75                               | 29.0                                             | 7.8–10.5        | 28–36                                |
| Dolerite       | 520–555             | 35           | 2.74–3.00                               | –                                                | 30.0            | 26                                   |
| Charskaya      | 555–855             | 300          | 2.55–2.63                               | 24.0–37.5                                       | 9.3–10.6        | 35–36                                |
| (–187.4/–210.3// –495.6/–513.6) |                 |     |                                          |                                                  |                 |                                      |
| Olekminskaya   | 855–1045            | 190          | 2.65–2.79                               | 39.0–69.0                                       | 11.2–21.4       | 36–38                                |
| (–495.6/–513.6// –681.7/–701.6) |                 |     |                                          |                                                  |                 |                                      |
| Tolbachanskaya | 1045                | –            | 2.72–2.75                               | 54.0–60.5                                       | 14.8–17.1       | 36–37                                |

**Table 2.** Strength and deformation characteristics of enclosing rock mass around Mir kimberlite pipe (determined in tests of samples).

| Rock                  | Young’s modulus $E$, GPa | Poisson’s ratio $\mu$ | Density $\gamma$, t/m$^3$ | Compressional strength, MPa | Tension strength, MPa | Internal friction angle $\varphi$, deg | Cohesion $C$, MPa |
|-----------------------|--------------------------|-----------------------|---------------------------|-----------------------------|-----------------------|----------------------------------------|------------------|
| Dolomite, anhydrite   | 30.0                     | 0.24                  | 2.5                       | 35                          | 3                     | 35                                     | 10               |
| Limestone             | 40.0                     | 0.28                  | 2.6                       | 51                          | 4.8                   | 37                                     | 11               |
| Sill of dolerite      | 55.0                     | 0.22                  | 2.8                       | 52                          | 6                     | 33                                     | 13               |
| Halogen-carbonate strata | 12.0                 | 0.30                  | 2.1                       | 22                          | 2                     | 38                                     | 5                |
| Kimberlite            | 21.0                     | 0.26                  | 2.45                      | 31                          | 3.3                   | 35                                     | 9                |
| Main backfill         | 1.0                      | 0.18                  | 1.7                       | 4                           | 0.4                   | 30                                     | 0.9              |
| First cut layer backfill | 2.0                    | 0.20                  | 1.9                       | 6                           | 0.5                   | 30                                     | 1.2              |
Table 3. Strength and deformation characteristics of enclosing rock mass around Mir kimberlite pipe in stress–strain state calculations.

| No. | Suite | Rocks | Deformation modulus E, GPa | Poisson’s ratio, ν | Bulk weight ρ, t/m³ | Cohesion, C, MPa | Internal friction angle, φ, deg |
|-----|-------|-------|---------------------------|------------------|-----------------|----------------|------------------|
| 1   | Metegerskaya (act. elev. 57///–54 m) | Dolomite, limestone, chalky clay, gypsum anhydrite | 7.2 | 0.25 | 2.59 | 4.35 | 31 |
|     | Icherskaya (act. elev. –54///–130 m) | Gypsum anhydrite and dolomite | 12 | 0.35 | 2.64 | 4.95 | 35 |
| 2   | Kharyalakhskaya (act. elev. –130///–161 m) | Rock salt interlaid with dolomite and anhydrite | 8.5 | 0.33 | 2.15 | 2.5 | 37 |
| 3   | Charshaya (act. elev. –161///–507 m) | Dolomitic limestone, halite, dolomite, anhydritic dolomite | 13.3 | 0.22 | 2.69 | 5.1 | 36 |
|     | Olekminskaya (act. elev. –507///–703 m) | Kimberlite | 7.7 | 0.25 | 2.45 | 2.58 | 35 |
|     | Tolbachanskaya (act. elev. –703///–883 m) | Backfill | 0.5 | 0.18 | 1.7 | 0.6 | 30 |

It is also evident that monolithic rock masses have higher deformation modulus and are natural stress raisers; the value of stress concentration depends on the size of discontinuities caused by deformation, ratio of elastic characteristics and other factors. Jointed areas, contact zones and backfill feature lower actual stresses and relaxation zone.

Thus, solution of the specific practical problems in geomechanics requires particular care in setting boundary conditions. The geomechanical modeling of rock mass calls for specific attention to the selection of data on strength and deformation characteristics of rocks.

The recommended values of physical properties for the first-stage calculation in geomechanical modeling are compiled in Tables 1–3.

A geomechanical model of a specific geotechnical situation is refined by the data of geomechanical monitoring. At this stage, the initial information and generalized and adjusted for the numerical modeling.

References
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