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Geotechnical approach to reinforcement system of underground openings in a gold mine

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ABSTRACT: This paper describes rock mass classification techniques and monitoring of the behaviors of artificial roof supports which were performed to establish the stability of underground openings and the reasonable excavation method at a gold mine of vein type. The Q-system which is one of the classification techniques can provide optimum support systems, depending on the conditions of rock masses and openings at this mine. In addition it is confirmed from the monitoring results that the excavation design using the artificial roof supports, made of reinforced concrete and of H-beam with wire mesh, is valid for stability of the openings in the cut and fill stoping.

1 INTRODUCTION

One of the objects of rock mechanics practice is to assess the stability of underground structures, such as mines and tunnels. There are analytical methods, empirical methods and observational methods for the design of the underground structures (Bieniawski,1984).

The analytical methods include numerical procedures (finite element, finite difference, boundary element), analog simulations (electrical and photoelastic) and physical modeling, and utilize the analyses of stresses and deformations around openings. Since, unlike conventional structures such as buildings or bridges, the underground structures have an excessively large number of relevant parameters, they can not be easily treated by the analytical methods.

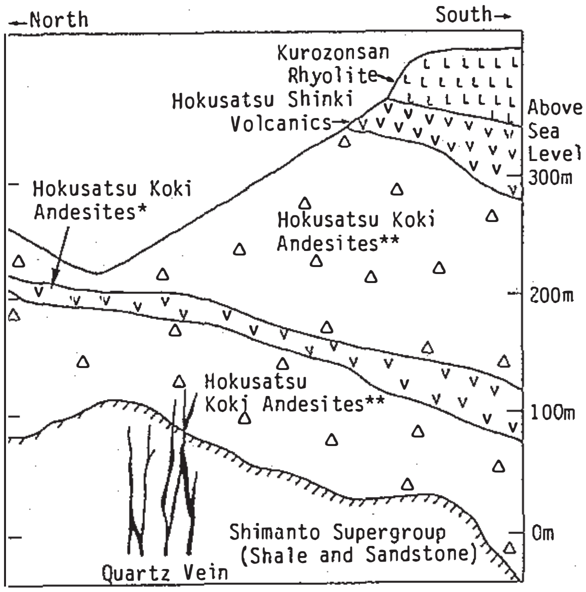
The empirical methods utilize statistical analyses of underground observations. The qualitative description of geotechnical materials by means of classification techniques has a long development history. While only one soil classification system, introduced by Casagrande, is widely used today for soils, many qualitative or quantitative rock mass classification systems have been proposed to describe rock mass conditions. From a practical point of view, the application of different classification systems to the same ground conditions may be desirable to benefit from individual advantages or to compensate for factors neglected by some (Kaiser et al.,1986).

The observational methods rely on actual monitoring of ground movement during excavation to detect measurable instability. These include the New Austrian Tunneling Method.

In this study, quantitative assessments of rock mass conditions were performed at the Hishikari gold mine to establish a more optimum support system, using two classification systems. In addition, the behaviors of two types of the artificial roof supports, one made of reinforced concrete and the other of H-beams with wire meshes, were monitored in order to reasonably design the cut and fill stoping method.

2 GEOLOGY, ORE DEPOSIT AND UNDERGROUND STRUCTURE

The Hishikari gold mine, which is now regarded as one of the highest grade gold mines in the world, is located in the Hokusatsu district, Kagoshima Prefecture, Japan. The deposit comprises an epithermal gold-silver bearing quartz-adularia vein (Abe et al.,1986). Several major veins and numerous veinlets have so far been encountered in an area some 800 m by 100 m. These veins strike 45° to 70°E and dip 70° to 90°N, and show extremely high gold contents in the region between 0 m and 100 m above sea level (250 m and 150 m below the portal, respectively; see Fig.1). Their widths vary from a few centimeters to 8 meters. The veins occur in both Pleistocene



*Andesite Lava:**Tuff Breccia

Fig.1 Schematic cross section of geology.

andesitic volcanics and sedimentary rocks of late Cretaceous to early Palaeogene age.

This deposit was discovered in 1981 during a scout-drilling program carried out by the Metal Mining Agency of Japan (MMAJ). In December 1982, twin parallel inclined shafts were developed to approach the orebody, and the first cross cut intersected the orebody in July 1985. The shafts have a grade of 17 % (10 degrees) and a size of 4.8 m width by 3.8 m height. Thereafter, main levels were in turn driven at 100 m, 70 m, 40 m and 10 m above sea level.

3 ROCK MASS CLASSIFICATION

The excavation of the orebody is now being advanced smoothly to a certain extent, while it is necessary to establish a more optimum support system. This is realized by a rock mass classification. The present study uses two classification systems: the rock mass quality (Q-system) developed by Barton et al.(1974) and the rock mass rating (RMR) system by Bieniawski(1974).

The rock mass quality number Q is calculated by

$$Q=(RQD/J_n)\cdot(J_r/J_a)\cdot(J_w/SRF), \qquad (1)$$

where

RQD : the rock quality designation,
J_n : the joint set number,
J_r : the joint roughness number,
J_a : the joint alteration number,
J_w : the joint water reduction factor, and
SRF : the stress reduction factor.

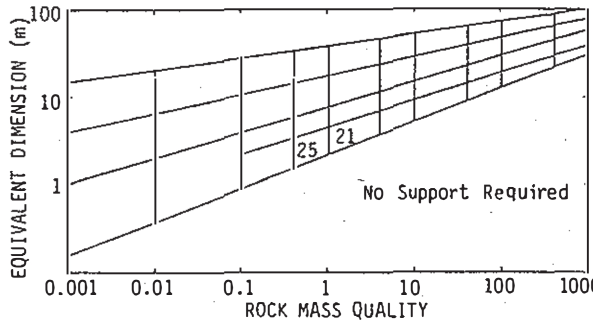


Fig.2 Support chart of Q-system.

The three terms on the right hand side of Eq.(1) express rock block size, joint shear strength and confining stress, respectively. The range in values of Q is from 0.001 for extremely poor rock to 1,000 for excellent rock. In addition Barton et al. defined the equivalent dimension (D_e) as

$$D_e=(\text{span or height of opening})/(ESR), \qquad (2)$$

where the term ESR is called the excavation support ratio and resembles the reciprocal of a safety factor. The value of 1.6 is adopted in permanent mine openings and 3 to 5 in temporary mine openings.

The RMR is obtained by summing five parameter values and adjusting this total by taking into account the joint orientations. The parameters included in the system are rock material strength, RQD, joint spacing, joint roughness and separation, and ground water. The RMR value can range between zero for extremely poor rock and 100 for excellent rock, and provides a relationship between the stand-up time and the maximum unsupported span for a given rock mass.

The RMR system may give guidelines for the selection of support, which depend on such factors as the depth below the ground surface (in-situ stress), opening size and shape, and the method of excavation. However, Bieniawski(1976) showed only the support guideline for a typical opening and excavation in his paper. On the contrary, the Q-system can choose a support system among 38 categories, based on the relationship between the rock mass quality and the equivalent dimension. Figure 2 shows the support chart, where the boundary of the maximum unsupported span is drawn using the following equation:

$$D_e=2\cdot Q^{0.4}. \qquad (3)$$

The Q-system considers not only the tunnel sections in two dimensional plane strain

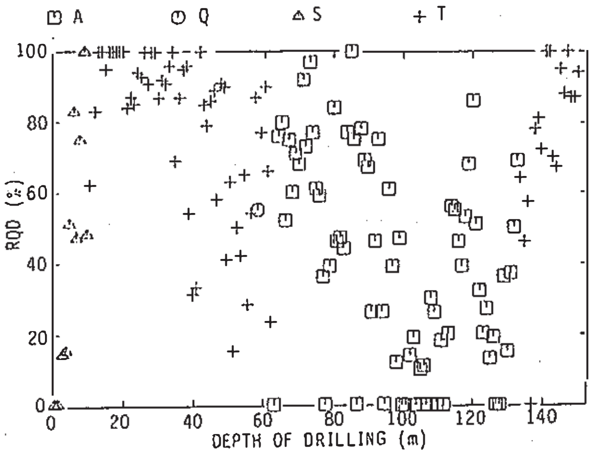


Fig.3 Distribution of RQD values along a drilling line. A;andesite, Q;quartz, S; shale and sandstone, and T;tuff.

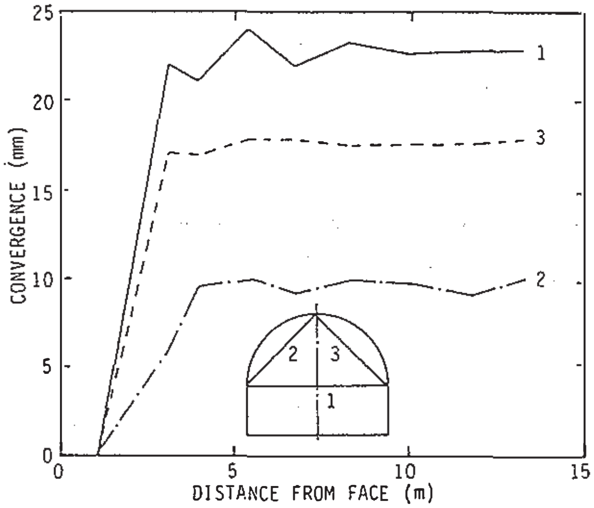


Fig.4 Convergences of the opening supported based on the Q-system.

condition but also crossings and bifurcations. Since, unlike tunnels in civil engineering, the Hishikari gold mine has a complicated underground structure such as inclined shafts, main levels, cross cuts, drifts etc. and their opening widths vary from point to point, the Q-system is preferable to the RMR system.

4 SUPPORT SYSTEM RECOMMENDED BY ROCK MASS CLASSIFICATION

For the reason described above, the support system recommended by the Q-system is mainly presented here. The parameters of RQD, J_n, J_r and J_a were evaluated for the typical and worst conditions of rock masses in this mine; the RQD values were not determined from the observation of rock walls but from that of drill cores which had been abundantly obtained during exploration and development. Figure 3 shows an example of the RQD distribution along a drilling line. Since in this line the RQD values considerably decrease at 100 to 110 meters in drilling depth, it is presumed that joint distribution is dense in this region and the rock mass is poor. The RQD values varying widely as shown in Fig.3 were handled with a statistical procedure to obtain their representative values for the typical and worst conditions.

The value of the joint water reduction factor was regarded as unity at the sections under dry condition, but as 0.5 at the inclined shafts for development of lower levels because of ground water inflows. The parameter of the stress reduction factor was also regarded as unity, considering that rock stress is medium in the Hishikari mine.

Table 1 shows the ratings for host rocks of andesite, tuff and Shimanto super-group (shale and sandstone) without inflows. Although the ratings for the quartz veins were additionally evaluated, the result does not appear in this table because the vein conditions varied considerably from point to point and it was difficult to arrange the ratings. For the drifts, therefore, the conventional support system was used: steel sets of H-beams with wood blocks. The recommended support systems is also shown in Table 1, using the equivalent dimension of 3 meters (the span of opening, 4.8 m and the excavation support ratio, 1.6). It can be seen from Table 1 that the typical rock masses have no need of support. Considering a loose part of rocks in the roofs, however, the support system for the worst rock masses was actually

Table 1 Evaluated Q values and recommended supports for two conditions of rocks without inflows.

Rock	Condition*	Q value	Support
Andesite	T	11	No support
	W	1.3	#21
Shale and sandstone	T	18	No support
	W	1.3	#21
Tuff	T	40	No support
	W	2.5	#21

* T:typical and W:worst.
#21:shotcrete of 5 cm thickness.

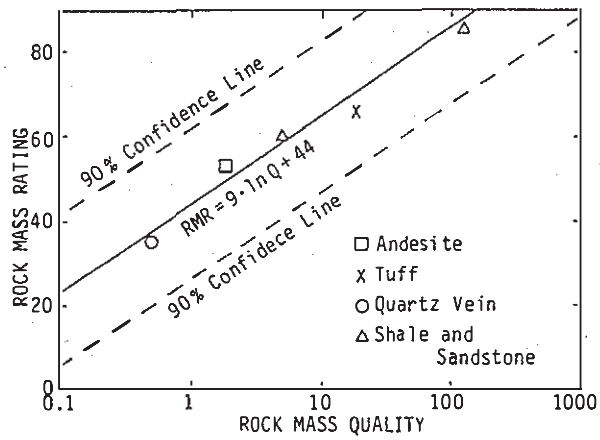


Fig.5 The Q values and the RMR evaluated for rock masses in the Hishikari mine.

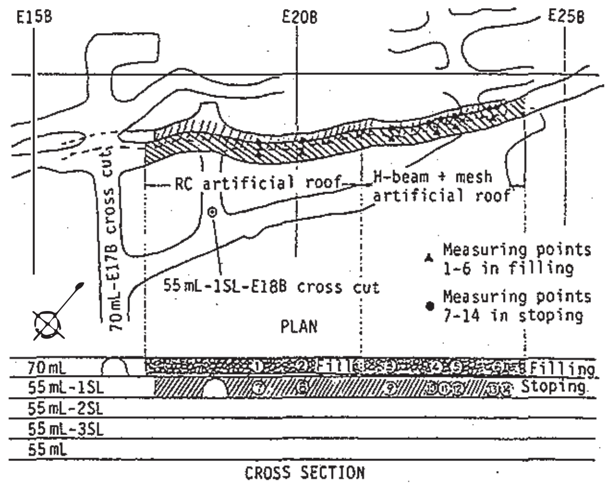


Fig.6 Plan and cross section of test site of the downward cut and fill stoping.

chosen.

The recommended support system of the category 21 (shotcrete of 2.5 to 5 cm thickness) was tried in a site of tuff, and the convergences of the opening (4.0 m width by 3.457 m height) were measured there. The convergences, as shown in Fig.4, increased until the face stood about 3 m apart from the measuring point, but remained constant thereafter. It was judged from this result that the support of shotcrete is sufficient for stability of the opening, but it was also predicted that the shotcrete support would decrease the working efficiency. Therefore, instead of the shotcrete, the support system of wire mesh (ϕ5 x 100 x 100 mm) and rock bolt (ϕ25 x 2000 mm; 1 m spacing), corresponding to the ability of the shotcrete, was determined to use actually for all conditions.

For the support at the crossings without ground water inflows, the joint set number in Eq.(1) was multiplied by the value of 3.0 according to the suggestion by Barton et al.(1974); i.e. the Q value was decreased to one third and the support category 21 was changed into the category 25 consisting of wire mesh, rock bolt and shotcrete. The support category 25 was applied to the inclined shafts of the Shimanto super-group in the lower levels with inflows as well.

The RMR values were also evaluated to confirmed the following relationship which was developed from 117 case histories by Bieniawski(1976):

$$RMR=9\cdot\ln Q+44. \quad (4)$$

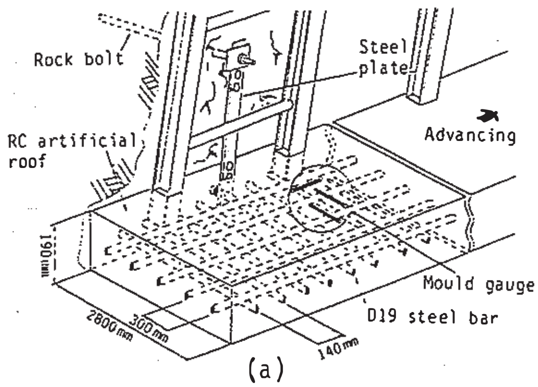
Figure 5 shows the Q values obtained for the worst condition and the RMR for the typical condition in the same rock masses at some places. Eq.(4) is valid when rock masses are evaluated under such different standards in the Q-system and the RMR.

5 DOWNWARD CUT AND FILL STOPING METHOD

It is premised that all ores are excavated and no pillars remain, and that workers do not work below unsupported stoping areas for safety. Thus, the downward cut and fill stoping method with artificial roof supports was determined to be adopted.

The test stoping over a length of 71 m were performed directly below the artificially supported drift of 70 m level E17B which was filled over a length of 73 m. Figure 6 shows a plan and cross section view of the testing site of the downward cut and fill stoping. The two kinds of artificial roof, the reinforced concrete (RC) roof and the H-beam roof shown in Fig.7, were tested. The strains and deformations of the roofs were measured by mould gauges in two directions in RC and by strain gauges put on H-beam respectively during the advance of the face. The strain of the support of the lower stoping area was also measured by strain gauges on the middle of the supporting member simultaneously. In addition roof sags were measured by the deformeter connecting with anchored invar wire.

In the design of artificial roofs, the load of fill material only was considered and the load from side walls was neglected. The RC slab was 19 cm thick, and D19 steel bars were arranged with spacing of 14 cm and 30 cm at a right angle. Allowable stresses of reinforcement and concrete are 1,800 kgf/cm^2 and 112 kgf/cm^2 respectively. H-beams (H100 x 100 x 6 x 8 mm) were arranged with spacing of 50 cm and connected



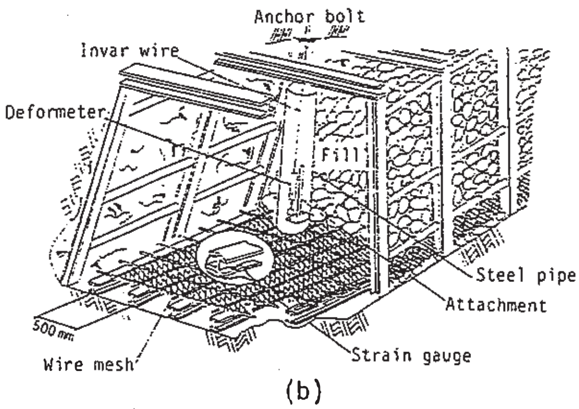


Fig.7 Artificial roofs of RC and H-beam with wire mesh, and their measuring systems. (a);RC and (b);H-beam with wire mesh.

at each end by splicing plates. The artificial roof of H-beams with two kinds of wire meshes were supported by steel sets with wood blocks.

The sags of the RC roof were -9.1 mm and -14.4 mm at 2 measuring points, respectively, just after the face passing. Since 96 % and 78 % of the deformations were recovered when the face had proceeded forward, the RC roof can be considered to be elastic. The H-beam roof caused a sag of 56 mm. The sag did not recover but maintained a

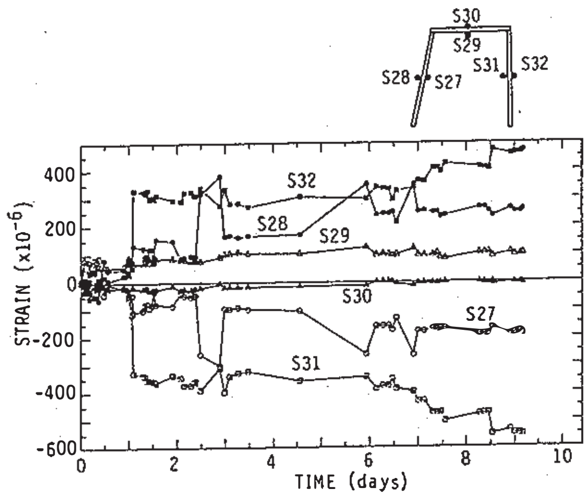


Fig.8 Strains of the steel sets at measuring point 8.

Table 2 Measured and calculated strains in steel sets at lower stoping area.

Artificial roof	Measuring point	Condition*	Strain in steel sets (x10^{-6}) Left vertical member	Horizontal member	Right vertical member
Reinforced concrete	7	A	350	80	-220
		B	230	40	-60
		C	360	430	-960
Reinforced concrete	8	A	130	50	-330
		B	140	60	-330
		C	380	330	-960
H-beam and mesh	9	A	150	40	-30
		B	80	0	-50
		C	370	490	-960
H-beam and mesh	10	A	300	100	-200
		B	700	900	0
		C	480	680	-1240

* A:value just after the face passing, B:final value and C:calculated value.

constant value. When the face passed through, the strains in the artificial roofs varied remarkably due to the change of moment. However, they became constant values with time and did not show excess values.

Figure 8 shows the strains of the steel set at the measuring point 8 in the lower stoping area, with advancing of the face. Table 2 shows the measured and calculated strains of the steel sets under different conditions at some measuring points. The estimated values were obtained from an assumption that all load of the fill material would apply to the steel sets through the wood blocks.

The RC roof is regarded as "self support roof" because the measured strains at the measuring points 7 and 8 are small. In the H-beam roof, the measured strains are relatively small compared with the calculated ones. Generally the load applied to the steel set is smaller than the total load from filling material. This reason is that the strength of fill material retained somewhat and the total weight did not apply to the support.

It is confirmed from these monitoring results that the artificial roofs can carry the applied load and the design is valid for the stability of the openings in the cut and fill stoping.

6 CONCLUSIONS

To obtain more optimum reinforcement systems of underground openings at a gold mine, two rock mass classification techniques were applied and investigated, and the behaviors of two types of artificial roof support were monitored. The results are as follows:

1. Considering that the gold mine has a complicated underground structure, The Q-system is preferable to the RMR.
2. A relationship is established between the Q value evaluated for the worst condition and the RMR for the typical condition in the same rock masses.
3. The Q-system provides a reasonable support system, made of rock bolt and wire mesh, for openings of host rocks under the conditions of two dimensional plane strain and no ground water inflow; tunnel crossings or inflows add the reinforcement of shotcrete to the support system.
4. The design of the artificial roof supports, made of reinforced concrete and of H-beam with wire mesh, are valid for the stability of the openings in cut and fill stoping.

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