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Evaluation of the rock property around TBM tunnels using seismic reflective survey data and TBM driving data

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Abstract: The relationship between the reflection number obtained from seismic reflective survey and the rock strength value obtained from TBM excavation is examined, and the procedure of the conversion from the reflection number to the rock strength value is proposed. Subsequently, geostatistical method is employed to evaluate the rock properties ahead of the tunnel face and around the tunnel with good precision, using both the seismic reflective survey data and the TBM driving data for the purpose of the tunnel driving and enlargement. The applicability of this evaluation method is examined at the actual tunnel site.

1. Introduction

Economical efficiency and safety of tunnel excavation depends critically on a detailed understanding of rock conditions to be encountered at the working face. Unanticipated joints, faults, or shear zones could lead to potentially hazardous conditions which may result in work stoppages and resultant claims and disputes. In the past, pilot core boring from the tunnel face was often performed out of necessity. However, this exploration method is relatively costly, because the excavation work must be suspended while thepilot hole is drilled. As a result, demands have risen for the development of new prediction system of the geological conditions ahead of tunnel face.Thus, a new system of seismic reflective survey has been applied at several Japanese tunnel site since 1999 by the authors (Yamamoto 2003). By observing seismic wave from multiple sources including the TBM cutting noise, this system creates a 3D isometric map of the geological structure with the "Reflection number", which is proportional to the Reflection Coefficient, some 100 meters out and up to 30 meters along the tunnel alignment. Authors have applied it to predict rock property ahead of the tunnel face in the several projects from February 1999. The images produced by this system have been used satisfactorily to manage the APC risk by encountering unforeseen able geological conditions.

On the other hand, the authors have developed the TBM excavation control system in hard rocks in 1999 for monitoring the geological conditions at the face simultaneously with theTBM advance, in order to realize rational excavation in many types of rocks and geological conditions (Shirasagi 2001). In this system, the rock strength is calculated using the TBM driving data such as thrust, torque and penetration rate. This rock strength value shows good correspondence with the rock mass quality, which is one of the most important values for designing the support.

This paper describes the relationship between the reflection number obtained from seismic reflective survey and the rock strength value obtained from TBM excavation, and the procedure of the conversion from the reflection number to the rock strength value. Subsequently, geostatistical method is employed to evaluate the rock properties ahead of the tunnel face and around the tunnel with good precision, using both the seismic reflective survey data and the TBM driving data for the purpose of tunnel driving and enlargement. The applicability of this evaluation method is examined at the actual tunnel site.

2. TBM driving data and 3D seismic reflective survey data

In this study the TBM driving data automatically acquired during excavation and the 3D seismic reflective survey data are used for the prediction of the rock properties ahead of and around the tunnel face.

TBM driving data

During the TBM excavation, several kinds of data such as thrust load, cutter torque, penetration rate, cutter revolutions, etc., are successively recorded as a function of tunnel distance. In the TBM Excavation Control System, these data are used to perform real-time calculations of the rock strength value proposed by Fukui et al. (1999), which is defined as

$$\sigma_C = \frac{F_N}{CPe} \tag{1}$$

where F_N= resultant thrust force; Pe = penetration rate; and C= constant. Fukui et al. (1999) showed that the rock strength value has good correspondence with rock mass classification and Schmidt rebound hammer test results, and provides a useful method for properly selecting a supporting pattern for rapidly changing rock mass characteristics.

3D seismic reflective survey data

The concept of seismic reflection for imaging ground conditions in three-dimensions is shown in Fig. 1. For each source and receiver of known location, the locus of all possible reflector positions defines an ellipsoid. For a sufficient number of sources and receivers forming a three-dimensional array, each boundary/reflecting horizon can be identified as an area where a majority of ellipsoids intersect. This system usually uses ten accelerometers as receivers and twelve sources arrayed three dimensionally, so that innumerable ellipsoids, which are enough to define reflecting horizons, can be drawn through the combinations of source/receiver points in the explored space.

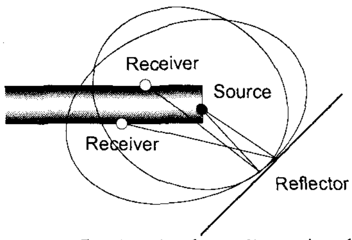

Fig. 1. The concept of using seismic reflection for three-dimensional imaging of ground conditions.

The reflecting horizon generated by faults, fractured zones, rock boundaries or cavities, is identified by the 2 dimensionally local change in the "reflection number" that shows the contrast of the seismic velocity between media k and media $k+1$. Reflection number γ_k is given by

$$\gamma_k = C \times \frac{V'_{k+1} - V'_k}{V'_k + V'_{k+1}} \tag{2}$$

with the provisional seismic velocities

$$V'_k = C_0 V_k \tag{3}$$

$$V'_{k+1} = C_0 V_{k+1}, \tag{4}$$

which are proportional to the seismic velocities of media k and media $k+1$ (V_k and V_{k+1}), respectively where C and C_0 are constants.. Furthermore, equation (2) can be transformed to

$$V'_{k+1} = \frac{1 + \frac{\gamma_k}{C}}{1 - \frac{\gamma_k}{C}} V'_k \tag{5}$$

with the constraint

$$C \geq 0. \tag{6}$$

3. Relationship between rock strength and reflection number

Conversion of reflection number

Since the reflection number obtained from 3D seismic reflective survey is a relative value that implies the degree of seismic velocity change, this value may not be directly correlated with the rock strength value computed from TBM driving data. Thus, the spatial distribution of reflection number was converted to the spatial distribution of rock strength through the following procedure.

From the 3D seismic reflective survey, we can have the reflection numbers every 2.5m along the tunnel alignment, as shown in Fig. 2. At these points, the reflection number γ is converted to V' using equation (5) (Fig. 3) with an appropriate value of C and a possible V'_1 (e.g. 2.5km/s)

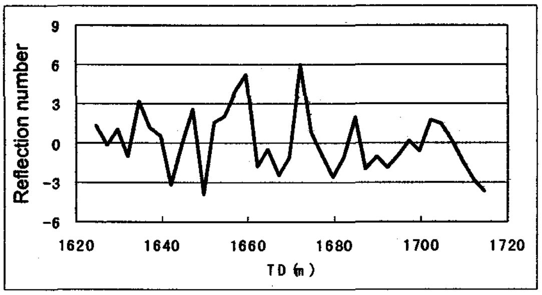

Fig. 2. Reflection number along the tunnel alignment.

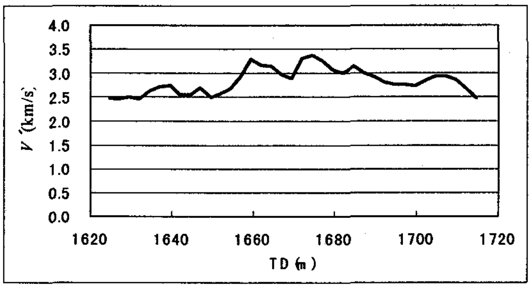

Fig. 3. Provisional seismic velocity along the tunnel alignment.

In general, a relationship between seismic velocity and rock strength value shows that the greater the rock strength value is, the faster the seismic wave travels. Taking this relationship into account, the rock strength σ can be induced from V'. The rock strength calculated from the TBM driving data is plotted in Fig. 4.

The mean and variance of V' as $m_{V'}$, $s_{V'}$, respectively and the mean and variance of σ as m_σ, s_σ, respectively, , can be given by

$$m_{V'} = \frac{\sum V'}{N} \tag{8}$$

$$s_{V'} = \frac{\sum (V' - m_{V'})^2}{N} \tag{9}$$

$$m_\sigma = \frac{\sum \sigma}{N} \tag{10}$$

$$s_\sigma = \frac{\sum (\sigma - m_\sigma)^2}{N} \tag{11}$$

where N denotes number of the data.

With these values, the rock strength can be induced by

$$\sigma = m_\sigma + \sqrt{\frac{s_\sigma}{s_{V'}}}(V' - m_{V'}) \tag{12}$$

Using this expression, the rock strength value can be calculated as shown in Fig.5. Pursuing this conversion enables us to make comparative discussion of the correlativity between the rock strength values converted from reflection number and the rock strength value from TBM driving data (Fig. 6). This figure implies that the rock strength ahead of the tunnel face can be predicted approximately, if such comparison is carried out as TBM advances after seismic reflective survey, and the rock strength around the tunnel can be estimated with better precision when TBM excavation is completed in the section of seismic reflective survey. The former gives useful information for the tunnel driving, while the latter gives useful information for the tunnel enlargement.

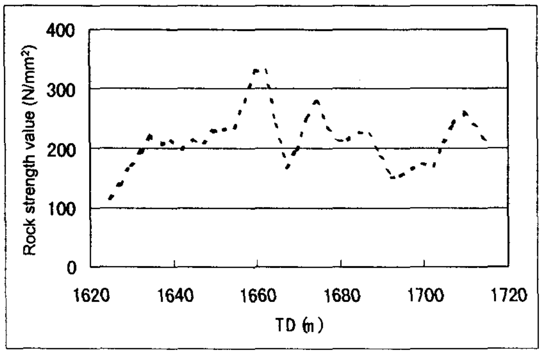

Fig. 4. Rock strength derived from TBM driving data.

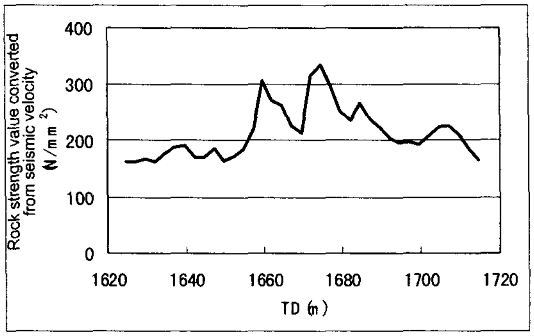

Fig. 5. Rock strength value induced from V'.

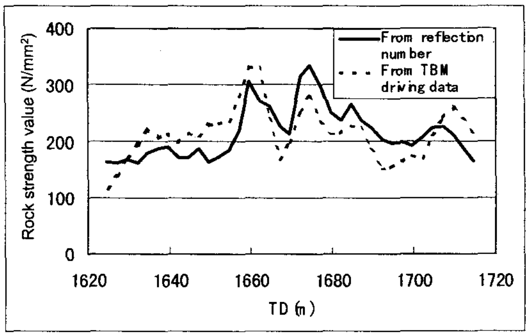

Fig. 6. Comparison rock strength value from reflection number with one from TBM driving data.

Correlation between rock strength value from TBM driving data and one from reflection number

In the section from TD 1400m to TD 1720m of Kanaya tunnel (excavation method: TBM pilot and enlargement method, tunnel diameter: 16m, total length: 4,454m), which is the scope of this study, 3D seismic reflective survey was conducted three times at each of three points of TD 1400m, TD 1520m, and TD 1620m. The results of 3D seismic reflective survey were passed through a conversion process, and the comparison was made between the rock strength value induced from reflection number and the rock strength value calculated from TBM driving data at each point, and the correlation between two types of data was examined. Fig. 7 shows the summarized correlation, containing all the data from three points, TD 1400m, 1520m and 1620m.

Fig. 7 evidently shows a positive correlation between the rock strength value induced from reflection number and the rock strength value calculated from TBM driving data at the relevant point for each of three 3D seismic reflective surveys. The coefficient of correlation is as a high as 0.866, in which all the data at the above three points are plotted. This assures that the proposed method of conversion is reasonable. Thus, the method allows us to have significant rock strength from the reflection number obtained by 3D seismic reflective survey.

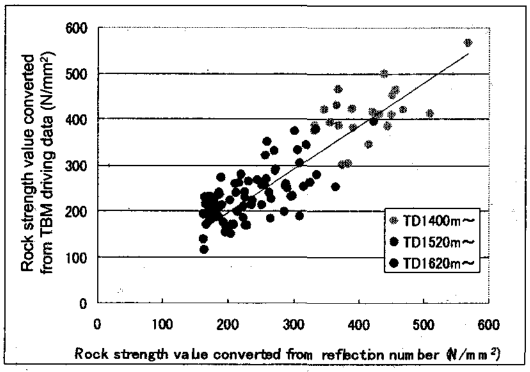

Fig. 7. Correlation between rock strength value from TBM driving data and one from reflection number

4. 3D geotechnical modelling using geostatistical technique

Outline of modelling

This study mainly intends to establish the geotechnical model with high reliability to ensure selection of the most reasonable tunnel support pattern when the enlargement is to be implemented in a large-section tunnel. In this chapter, a 3D geotechnical model of large-section tunnel based on geostatistical technique is proposed. Further, the practicality of this modelling technique is validated through comparison of that the generated model and the result of geologic observation.

The 3D geotechnical model for a large-section tunnel is created by the following procedure:

1) To take the rock strength value calculated from TBM driving data as the variable of interest that represents the geotechnical feature of rock mass.
2) To implement 3D seismic reflective survey during TBM excavation in order to obtain 3D seismic reflective survey data to cover the area around the large-section tunnel in the form of 3D data with the intervals of 2.5m.
3) To pick up those data at the spots at which rock strength values have been calculated from TBM driving data out of the 3D seismic reflective survey data. To examine the correlation between the picked up data with the rock strength, and to convert the 3D seismic reflective survey data into rock strength values based on the correlation.
4) To conduct 3D interpolation of the available rock strength values converted in Step 3, which is in the form of 3D data with the intervals of 2.5m, using the geostatistical technique in order to take into consideration the anisotropy or fabric of geological structure, which has spatial correlation , and then, to create a 3D geology model based on the data obtained through the interpolation.

The 3D geotechnical model of the large-section tunnel created through the above procedure enables us to estimate not only the rock strength distribution but also the location of major discontinuities including fracture zone, fault, geological boundary, etc..

Interpolation by geostatistical technique

Performing Steps 1 to 3 in the above-mentioned procedure, the rock strength value is obtained in the form of 3D data with 2.5m intervals. This study uses geostatistical technique for interpolation of rock strength value data to take into consideration the anisotropy or fabric of geological structure, which has spatial correlation in order to create a "geologically reasonable" 3D geology model of large-section tunnel. The following section describes the geostatistical technique.

Outline of geostatistics

Geostatistics is a statistic method to process an event involving spatial variation. One of the largest purposes of geostatistics is to estimate spatial distribution of the data that has spatial correlation with good precision from limited sample data. The method features stochastic treatment of spatial dispersion in the process of estimation.

The method of estimation by geostatistics:

1) Describes spatial characteristics (spatial structure) of the variable of interest, and models random field that represents spatial correlation from the observed value.

2) Estimates the values at arbitrary points at which values have not beenobserved, using the predetermined random field model (the method is called Kriging).

Step 1 estimates mean function and covariance function that describe the average values, variances, and spatial dispersions of the variable of interest based on the observed values. This study uses a Variogram function to describe the spatial structure. Step 2 estimates the values at arbitrary points, , as the linear combination of the observed values based on the model determined in Step 1. This step performs spatial interpolation of the variable of interest.

Modelling of spatial correlation

To estimate the target variable using geostatistical technique, random function of the variable must be modelled using the data obtained from the field.

Pairs of samples are evaluated by computing the squared difference between the values. The resulting dissimilarities are related to the separation of sample pairs in space. Experimental variogram that describes the average dissimilarities of pairs of samples is defined as

$$\gamma(h)=\frac{1}{2N(h)}\sum_{i=1}^{N(h)}\left[Z(x_i+h)-Z(x_i)\right]^2 \tag{13}$$

where x_i= location of sample in space, $Z(x_i)$ = random variable, h = distance between a pair of samples, and $N(h)$ = number of pairs of samples with respect to h.

A Variogram can be considered as an indication how the influence of a certain point changes as the distance from point increases, and the change has features that vary with the nature of the aimed subject. In addition, to find an adequate Variogram to express that nature is the prerequisite for application of the Kriging method when to estimate a spatial correlation.

Besides Variogram estimated value determined by equation (13), to make approximation using an theoretical Variogram model is called application of model (Fig. 8). The reason why such application is necessary is because the value of Variogram must be calculated at an arbitrary value of "h" in the Kriging equation in order to implement spatial interpolation by Kriging, as explained in the first place. For the purpose or simplifying the calculation, a simple mathematical expression model is normally used for application.

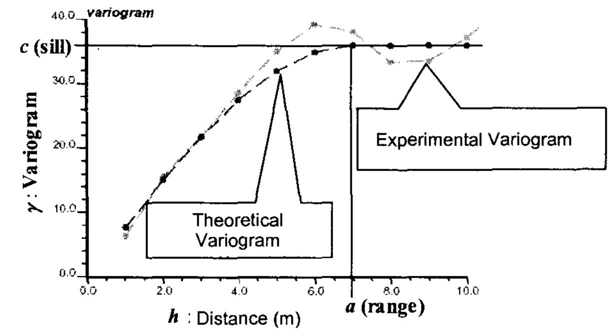

Fig. 8. Variogram.

For the better understanding of the spatial structure, experimental variograms are commonly fitted with spherical model;

$$\gamma(h)=\begin{cases} c\left[\dfrac{3}{2}\cdot\dfrac{h}{a}-\dfrac{1}{2}\cdot\left(\dfrac{h}{a}\right)^3\right] & h\le a \\ c & h>a \end{cases} \tag{14}$$

where a is the range and c is the sill, both of which are parameters calculated by the model application. In most cases, the influence of a certain point decreases as the distance increases, and finally becomes null at a certain distance. This distance, h, at which the point has no influence, is called a "range", and the value of the Variogram for that "no influence" is called a "sill". Further, the following relation (15) is found between the Variogram and the covariance function $C(h)$;

$$C(h) = C(0) - \gamma(h) \tag{15}$$

where $C(0)$ means the sill. This expression indicates that the Variogram behaves in the antithetical manner to the covariance function.

Ordinary Kriging

When a model of the spatial correlation has been created, the Kriging method can estimate physical quantities at points at which observation values were not acquired based on that model. The Kriging method is different from so called classic linear regression in the point that the former takes into account the probabilistic correlativity between data. Every variation of the Kriging method is, basically, estimation by linear regression, and the estimator at an arbitrary position can be expressed by

$$Z^*(x_0) - \mu(x_0) = \sum_{i=1}^{n} \lambda_i [Z(x_i) - \mu(x_i)] \tag{16}$$

where $\mu(x_0)$ is the expected value $E[Z(x_0)]$ of the random variable $Z(x_0)$.

To determine the estimator from Expression (16), kriging weight (λ_1, λ_2, ..., λ_n)] should be found. The equation to achieve the above is the Kriging equation, which should be determined, basically, to meet two requirements: the estimation error variance must be minimized; and the estimator must be universal. In this study, the ordinary Kriging;

$$\begin{cases} \sum_{i=1}^{n} \lambda_i \gamma(|x_j - x_i|) + \eta = \gamma(|x_j - x_0|) & j = 1,2,\cdots n \\ \sum_{i=1}^{n} \lambda_i = 1 \end{cases} \tag{17}$$

is used for discussion.

Estimation of rock strength

Performing interpolation by the geostatistical technique, a 3D geotechnical model (rock strength distribution) can be obtained as shown in Fig.9. In this figure, only the parts at which the rock strength value is low are displayed, and other parts are omitted for the better visual display. The parts with darker blue indicate the spots of lower rock strength value. The section indicated by dotted lines on the diagram is the portion excavated by the TBM.

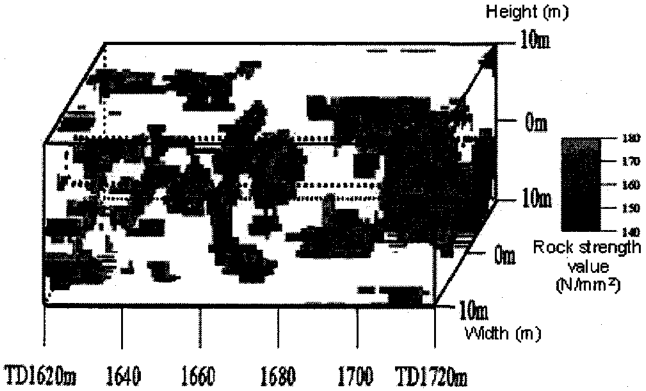

Fig. 9. The 3D rock strength distribution.

Comparison with geological situation

The observation result of TBM pilot tunnel is depicted by developing the record of the observation of internal surface of the TBM pilot tunnel, including the left wall (SPL), the crown, and the right wall (SPR) as shown in Fig. 10. It shows the locations of discontinuities and fractured zones. Major discontinuities are demarcated with black bold line. Fig. 11 shows the rock strength distribution in the vertical cross section of the tunnel. The rock strength value is relatively lower at each spot around the intersection of discontinuities than the other parts. Therefore, 3D geology model is in good consistency with the geologic observation result.

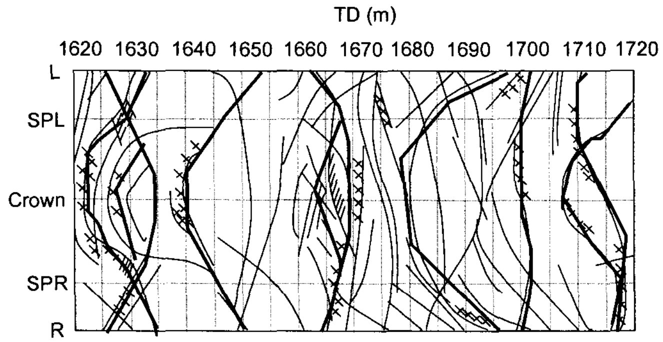

Fig. 10. Locations of discontinuity.

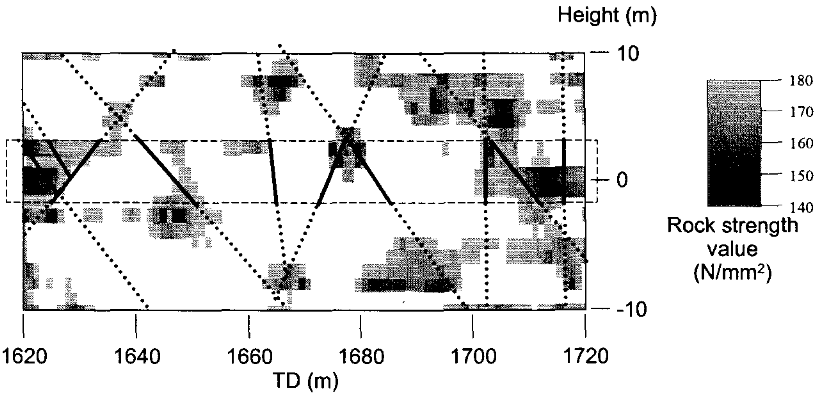

Fig. 11. Positions and direction of discontinuities.

5. Conclusion

In this study, the relation between the reflection number derived from the 3D seismic reflective survey and the rock strength value obtained from the TBM driving data is examined and the technique of conversion from the distribution of reflection number to the distribution of rock strength value is proposed for the purpose of the tunnel driving and enlargement. The geotechnical model described by rock strength is created using geostatistical technique in order to reproduce the anisotropy or fabric of geological structure taking the spatial correlation of the data into account. The applicability of this method is validated through the comparison between the generated geotechnical model and observation result in the actual tunnel.

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