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Performance of a Large Diameter Tunnel in Weak Rocks

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SYNOPSIS : The correlation of tunnel movement versus rock mass quality was investigated using actual monitored data as well as theoretical studies. Results revealed that meaningful empirical correlation between the commonly used rock mass rating system and tunnel deformation can be obtained only if geological structure and in-situ stresses are taken into account. In this respect, the commonly used rock mass rating system is not very suitable for such purpose. A new parameter using rock mass strength normalized by in-situ stress level appears to be more suitable for establishing the relationship between tunnel deformation and rock mass quality.

INTRODUCTION

In the tunnel engineering practice in Taiwan, empirical methods are used at the design stage to estimate the support required for rock masses with various classes of quality. The support design is generally based on limited geological and rock data available, such as topography, rock formations, weak planes, laboratory properties of intact rocks, etc. obtained from surface mapping and a few borings. At the construction stage, rock supports are assigned at the site based on the rating of rock mass encountered. Supports would be revised if required as a result of tunnel performance assessment based on monitoring data, such as tunnel deformation, support stress, etc. To make such an assessment, empirical guidance based on previous experiences under similar conditions is generally essential. However, there is very little published information or experiences on the tunnel deformation at present.

In this study, monitored deformations of a 16m span highway tunnel driven through relatively weak rock formations was used to establish an empirical correlation between tunnel deformation and rock mass quality. Analytical method was also employed to predict the tunnel deformations for various rock mass qualities. The predicted tunnel movements were then compared with the empirical correlation established. Discrepancy in results was then studied for possible causes including geological ones and the nature of the parameter used for correlation, and a new correlation parameter was suggested.

GEOLOGICAL CONDITIONS AND SUPPORT DESIGN

The tunnel investigated is located along the western foothills to the southeast of the Taipei basin. Rock formations include sedimentary rock interbedded with tuffs of Miocene age in Tertiary period and recent alluvial deposits. The rocks encountered along the tunnel consist mainly of sandstone, interbedded sandstone and Shale and tuff. The geological profile along the tunnel is shown in Fig. 1.

The rocks are generally weak with uniaxial compressive strength in the order of 250kg/cm^2 and elastic modulus of 70,000kg/cm^2 for intact core. The rock formations were disturbed by tectonic movements, and several faults, fracture zones and foldings may be seen in the Figure. The rock masses are in the range of 30 - 60 in RMR rating and 0.4 - 6 in Q system, and may be rated as poor to fair rock.

Typical section of the tunnel is shown in Fig. 2. The monitoring systems referred to in this study are also shown in the Figure. Six types of semi-rigid supports consisting of rockbolt, shotcrete, wiremesh and steel rib, as shown in the Table below, were used to stabilize the tunnel. Based on the on-going rating of rock mass quality with NGI-Q and CSIR-RMR systems, appropriate support type was selected at the site. Monitoring was then carried out to assess the performance of the tunnel-support system.

CORRELATION BETWEEN ROOF SETTLEMENT AND ROCK MASS QUALITY

Two rock mass rating systems i.e., CSIR rock mass rating- RMR system and NGI-Q system which are most commonly used, were adopted for correlating the tunnel movement and rock mass quality. The results for roof settlement and convergences are shown in Figs. 3a - 3d. Roof settlement, which would give the best indication of tunnel movement because of the tunnel shape, was selected for this study.

There was a large scattering of results for both rock mass rating systems. Useful correlation can't be obtained as an empirical guidance. This is attributable to error in measurement, existance of special geological conditions and inadequacy of rock mass rating as a correlation parameter.

Concerning the error in measurement, it includes the improper and late installation of instruments, human error in measurement, etc. To avoid these errors, only the measurement sections installed close to the advancing face (usually less than 1m) and measurement values

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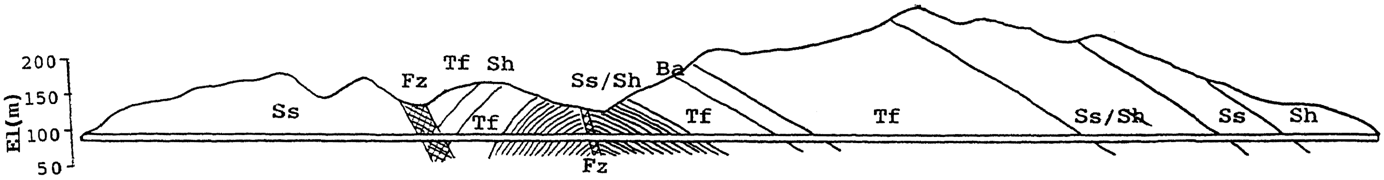
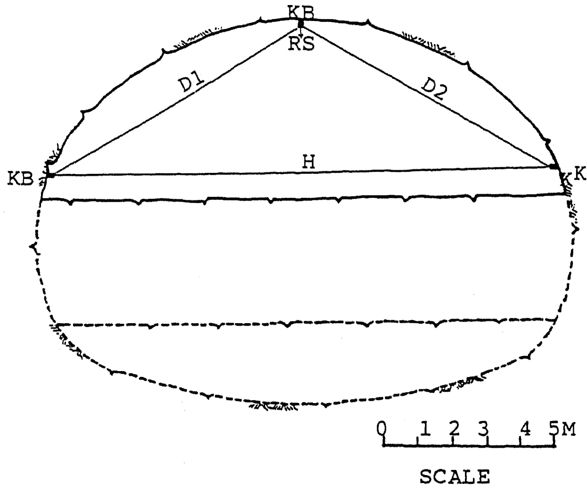


Ss:Sandstone
Sh:Shale
Ss/Sh:Alternation of Sandstone & Shale
Ba:Basalt
Tf:Tuff
Fz:Fault or Fracture Zone

Fig. 1. Geological Profile along the Tunnel



H:Horizontal Convergence
D1 & D2:Diagonal Convergences
KB:Convergence Bolt
RS:Roof Settlement

Fig. 2. Typical Section of the Tunnel

Tunnel Support Design

SUPPORT TYPE	Q	RMR	ROCK BOLT	SHOTCRETE	STEEL RIB
I	>40	>77	29 ϕ L=4m LOCALLY	5 cm	-
II	40 ~ 10	77 ~ 65	29 ϕ L=4m @2mx2m	10 cm	-
III	10 ~ 4	65 ~ 56	29 ϕ L=4/6m @2mx1.5m	15 cm	H100x100 @2.5m~2m
IV	4 ~ 1	56 ~ 44	29 ϕ L=6m @1.8mx1.5m	15 cm	H100x100 @1.5m~2m
V	1 ~ 0.2	44 ~ 30	29 ϕ L=6/9m @1.8mx1m	20 cm	H125x125 @1m~1.5m
VI	0.2 ~ 0.01	30 ~ 2	29 ϕ L=6/9m @1.5mx0.8m	25 cm	H150x150 @0.8m~1.2m

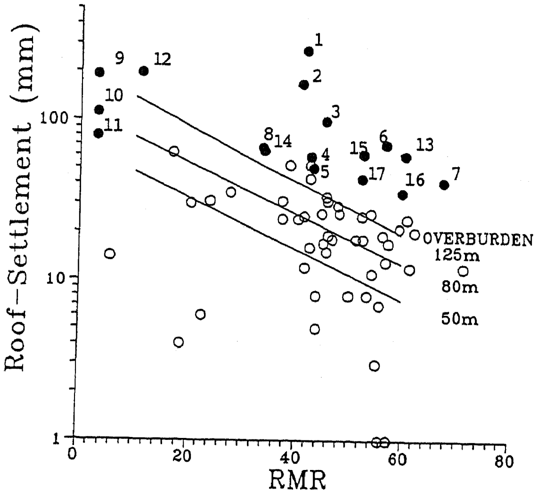


Fig. 3a. Correlation between Roof Settlement and RMR

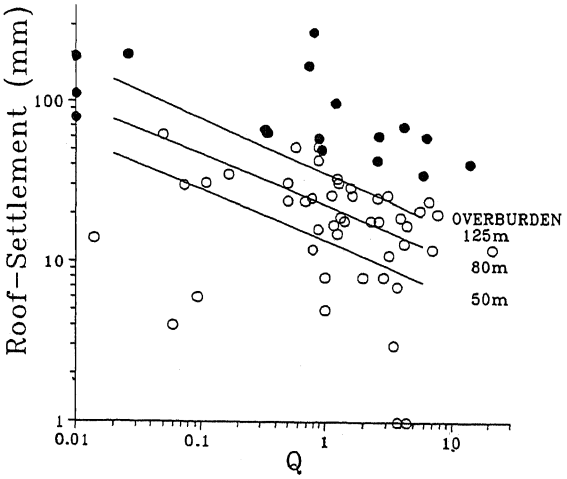


Fig. 3b. Correlation between Roof Settlement and Q

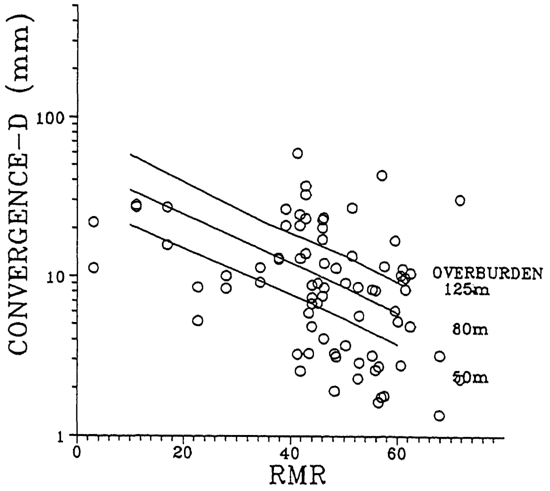


Fig. 3c. **Correlation between Diagonal Convergence and RMR**

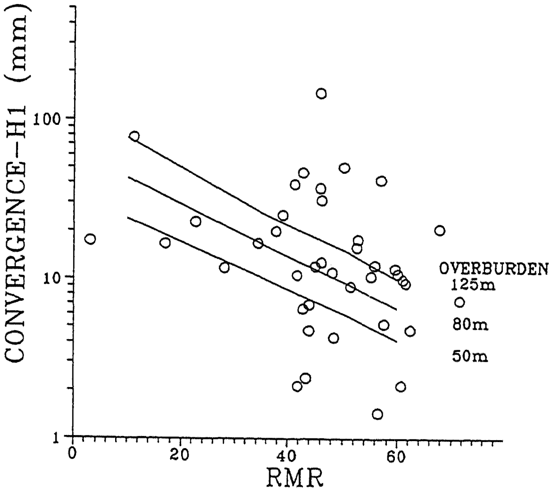


Fig. 3d. **Correlation between Roof Settlement and RMR**

howing a consistent trend were used. herefore, error in measurement is not onsidered to be significant, and the cattering of results appears more likely to be ue to appropriateness of the correlation arameter, i.e., RMR or Q value, and geological onditions which will be discussed in the next ection.

OMPARISON OF MONITORED DATA AND ANALYTICAL ESULTS

lasto-plastic analyses were made by using xplicit finite difference code FLAC (Fast agrangian Analysis of Continua). In the nalysis, Mohr-Coulomb yield conditions were ssumed for the materials. The results for hree overburden thicknesses are shown by the olid lines in Fig. 3a. They show that with the xception of a few data points as shown by the ll dots the measured data fall reasonably well ithin the range of the predicted values.

A review of the geological conditions in the monitoring sections which gave large variations in results revealed that these conditions were present in three different types of areas, i.e., geological structure controlled area, portal area and near fault zone. For data points 1 to 5 (Fig. 3a), the tunnel is very close to a fault zone. A well-developed joint set dipping toward the slope as shown in Fig. 4a existed in the area. Unfavorable stress conditions in relation to the dip direction of the weak planes may be the cause of the exceptionally large roof settlement. The initial stress conditions are given in Fig. 5a. According to Jaeger's study on the effects of angle (β) between weak plane and major principal stress, when β is in the range of 10 to 50 the strength of rock mass decreases significantly. In this case, the angle is within 15 to 50 around the tunnel. Therefore, the strength of rock mass was controlled by the weak plane. The analytical results of tunnel deformation are shown in Fig. 5b. Large movement appeared to be the result of shearing along the predominant joint plane, which is consistent with the field observation in cracks developed. Otherwise, relatively small roof settlement is predicted if homogeneous, isotopic properties of rock mass were assumed (Chern, Chang and Lin, 1992).

For data points 6 to 8, the tunnel is in an area with predominant structure. Either an unstable wedge (Fig. 4b) or a shear zone in the roof (Fig. 4c) caused severe cracking in the tunnel support. Analysis by the continum approach gives poor prediction.

For data points 9 to 12, the tunnel is located in a fault zone. The rock mass is more or less homogeneous. Therefore, prediction analysis gave reasonably good results.

Data points 13 to 17 are for tunnel sections near the portal area. The overburden in these sections is generally small. The tunnel deformation tends to be large in these areas.

If these data points were excluded and the data were separated into low overburden area (50~80m) and high overburden area (80~125m), there would be a much better correlation as shown in Figs. 6a and 6b. Therefore, it may be concluded that from tunnel deformation point of view, rock mass quality, in-situ stress conditions and geological structure are the main controlling factors.

The factors considered in the RMR system, include rock core strength, block size, strength of weak plane, ground water condition and orientation of weak plane. In the Q system, the factors include the block size, strength of weak plane, ground water condition and stress factor. These rock mass classification systems, which were intened for tunnel support design purpose, may not be able to fully reflect the geomechanical characteristics of the rock-tunnel system, especially the stress factor. Therefore, a good correlation can't be obtained by adopting these commonly used rock mass classification systems alone.

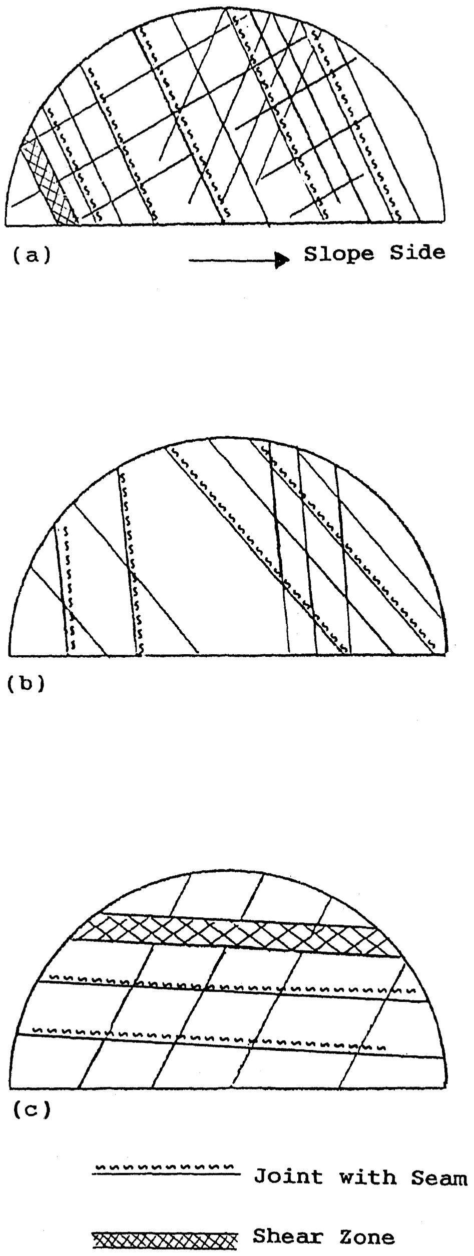


Fig. 4. Special Geological Conditions Encountered

SUGGESTED CORRELATION

The results of the analytical study indicate that the strength and deformation properties of rock mass and the in-situ stress conditions are the most important factors influencing the deformation of tunnel if a site with special geological structure is excluded. In rock engineering, uniaxial compressive strength of rock is often being used as an indication of its competence. If uniaxial compressing strength of rock mass, which can be obtained from failure criteria suggested by Hoek and Brown (1988), normalized by in-situ stress level is used as the correlation parameter, the analytical results for various overburdens collapse into a

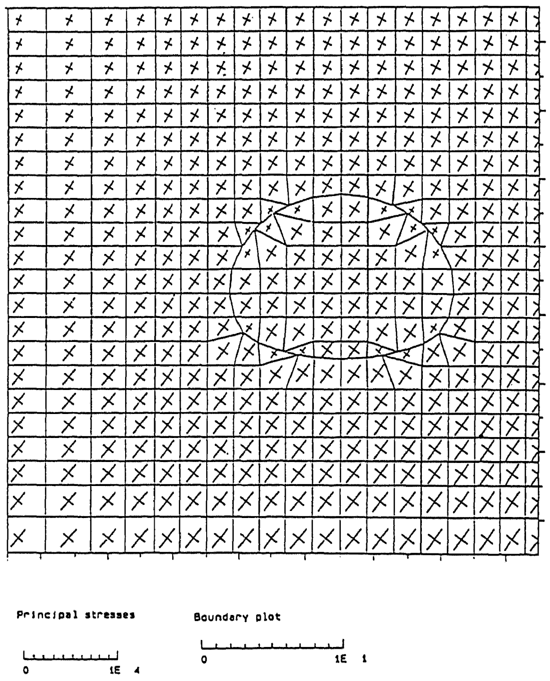


Fig. 5a. Initial Stress Conditions around the tunnel

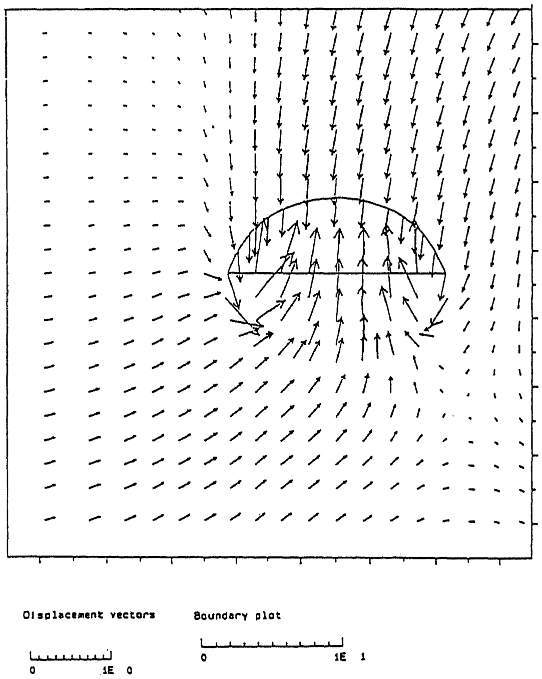


Fig. 5b. Analytical Results of Tunnel Deformation

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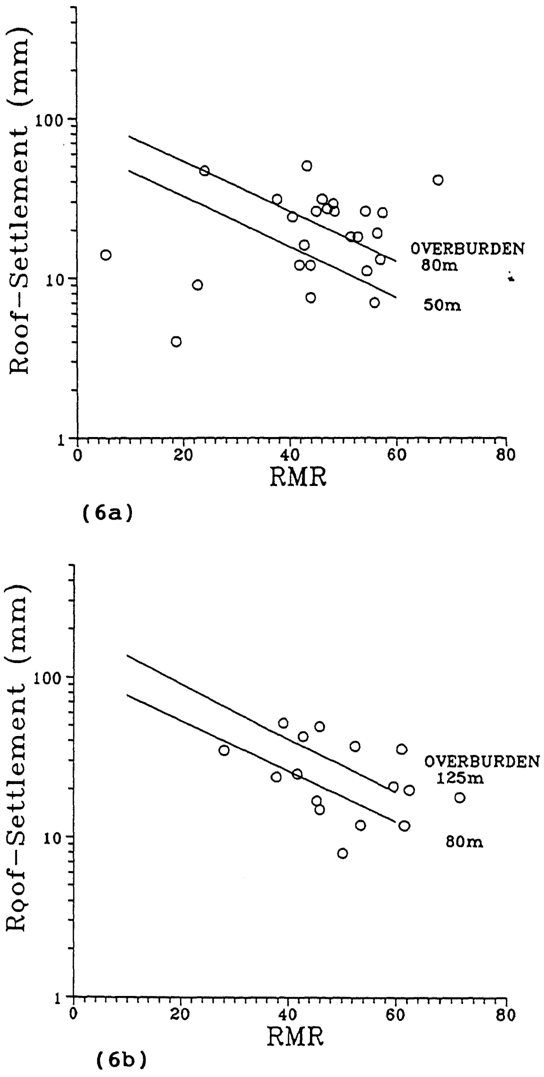

(6a)

(6b)

Fig. 6. Correlation between Roof Settlement and RMR for Different Overburden

fairly narrow zone and a much better correlation can be obtained as shown in Fig. 7.

From these studies, for reasonably uniform rock conditions free from special geological structure and with proper assumptions in rock properties, numerical analysis by the continuum approach can give a fairly good prediction of the tunnel behavior. Empirical relationship between tunnel deformation and uniaxial compressing strength of rock mass normalized by in-situ stress level can be established as a guideline for assessing the tunnel performance. However, it should be noted that the tunnel movement is intimately related to the shape and dimensions of cross-section also. Care should be exercised in using the empirical relationship.

CONCLUSIONS

From the case study of a large diameter tunnel driven through relatively weak rock masses, the following conclusions may be drawn:

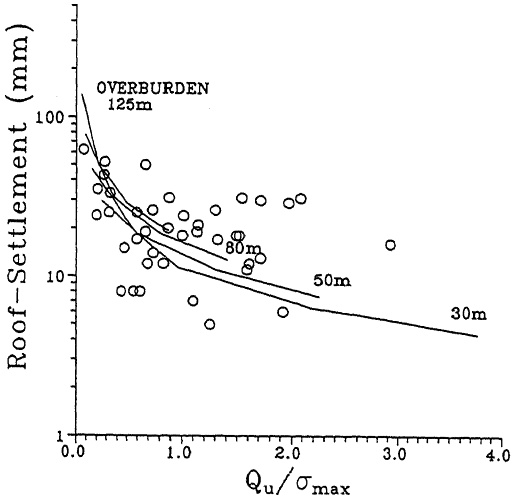

Fig. 7. Correlation between Roof Settlement and Rock Mass Strength Normalized by In-Situ Stress

(1)Direct establishment of empirical relationship between tunnel deformation and the commonly used rock mass classification system is difficult. Meaningful relationship can't not be obtained due to wide scattering of results.

(2)The wide scattering of results is mainly due to geological structure and different in-situ stress conditions existing at the site. Meaningful empirical relationship can be established only for relatively homogeneous rock mass free from special geological structure and similar in-situ stress conditions.

(3)Numerical analysis can be a useful tool in estimating the order of magnitude of tunnel deformation for relatively homogeneous rock mass.

(4)Theoretically, uniaxial compressive strength of rock mass normalized by in-situ stress level appears to be a better parameter than the commonly used rock mass classification systems such as RMR or Q in relating the rock mass quality to the tunnel deformation. However, there is a shortcoming in its practical application due to the difficulty of assessing the parameter in the field. Therefore, a more appropriate approach may be to adjust the current rock mass classification system to reflect the strength and stress encountered in the field.

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