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Engineering geological assessment of the permanent ship lock slope, the Three Gorges Project, China

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Abstract

The permanent ship lock slope of the Three Gorges Project was excavated through the hill of granite massif with the aspect of SE(106°). Some problems of engineering geology (such as statistical features of rock mass structure) were comprehensively studied based on field investigation and numerical analysis. The numerical modelling techniques including damage variable and finite element analysis were used for the detailed study of the effect of excavation and blasting on rock mass quality and slope stability. The analyses indicate that the rock mass is relatively intact and rock mass quality varies mainly from Class I to Class II. Consequently, the overall stability of slope can be ensured except for some minor local unstable blocks.

INTRODUCTION

The Three Gorges Project (TGP), located in the middle reaches of the Yangtze River, China, is the largest hydroelectric project in the world. It is a gigantic multipurpose project, which will generate great many comprehensive benefits, especially in flood prevention, power generation, shipping, and water supply. The TGP consists of a 180 m high, concrete gravity dam with a spillway in its central portion, electric power stations on both sides of the spillway, and navigation structures. The navigation structures are composed of a double-lane, five-step, permanent ship lock; a single-lane, temporary ship lock; and a one-step, vertical ship lift (Fig. 1).

Among these structures, the double-lane, five-step, permanent ship lock is the main part of the navigation constructions and one of the main structures of the TGP. Its geometrical parameters are given in Table 1.

The ship lock was cut through the granite hill with high and steep slopes (Fig. 2). The granite massif showed large-scale anisotropy in the rock mass and remarkable release of geo-stress due to excavation. The main engineering geological studies carried out were statistical analysis of discontinuities, assessment of rock mass quality, slope stability analysis, and selection of slope stabilisation measures. A short description of them is given below.

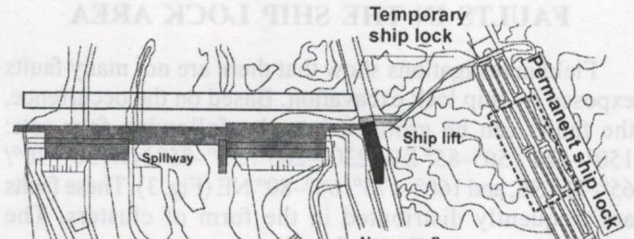

Fig. 1: The layout of the Three Gorges Project

Table 1: Geometrical parameters of the permanent ship lock

Attribute	Dimension
Total length of the ship lock axis	6442 m
Maximum height of the excavated slope	170 m
Vertical height of each slope step	15 m
Orientation of the ship lock axis	106°
Designed cut slope angle	90° for the fresh rock 73° for the slightly weathered to fresh rock 45°for the completely to highly weathered rock

GEOLOGY OF THE AREA

The permanent ship lock area is composed mainly of the Presinian System. It is made up of granite containing xenoliths of gneiss and bands of diabase. Pegmatite veins and dykes of granitic composition crosscut the granite (Fig. 3).

The gneiss xenoliths with the general orientation of N-S (strike)/ 54°–80° (angle of dip) W (dip direction), intersect the third ship lock head and chamber. They have sharp and irregular contacts with the country rock. There are two sets of diabase bands with the orientation of 050°–070°/ 60°–85° NW and 340°–360°/ 65°–85° SW, respectively. The former is

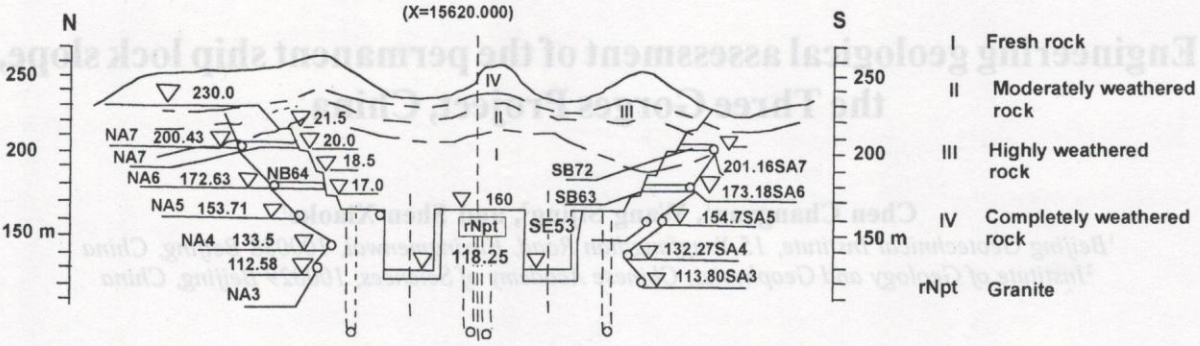

Fig. 2: Cross-section of the permanent ship lock slope

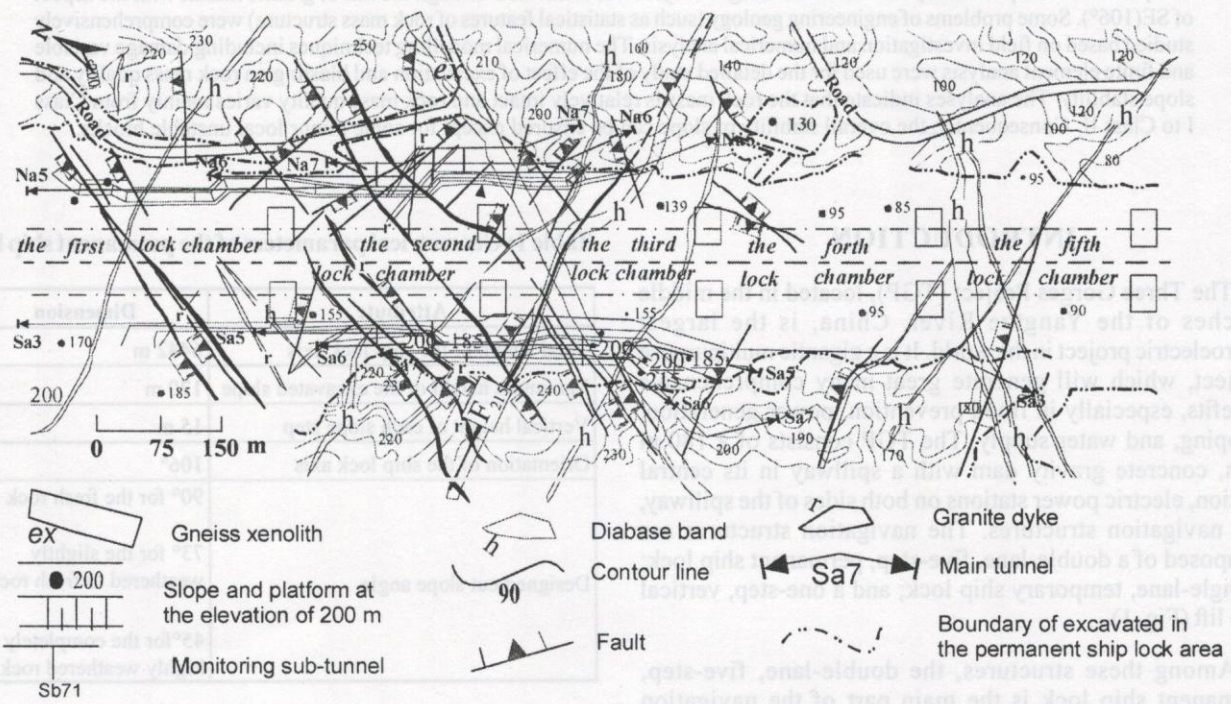

Fig. 3: Engineering geological map of the ship lock area

from several hundreds to several thousands m long and is frequently exposed in the first, third, and fifth chambers as well as in the second and third ship lock heads. It has frequently faulted or fractured contact with the country rock. The latter is of relatively small-scale in comparison with the former and is mainly distributed in the fifth ship lock head.

The dykes and the pegmatite veins show sharp and irregular contacts with the country rock, and can be grouped into the two sets: 100°–120°/ 20°–50° SW and 60°–85°/ 40°–80° SE. Owing to compositional variations and presence of fractures, the rock is differentially weathered in the lateral as well as vertical direction. Based on the degree of weathering, fracture characteristics, mineral composition, and mechanical properties, the rock mass can be divided into the following five zones in the vertical profile: completely weathered, highly weathered, moderately weathered, slightly weathered, and fresh (Fig. 2).

FAULTS IN THE SHIP LOCK AREA

Field investigations show that there are not many faults exposed by ship lock excavation. Based on the occurrence, the faults can be grouped into the following four sets: 150°–170°/ 50°–85° SW, 230°–265°/ 55°–85° NW, 50°–70°/ 65°–85° SE, and 160°–175°/ 60°–80° NE (Fig. 3). These faults are frequently distributed in the form of clusters. The former two sets are the preferred ones.

The first set is constituted by medium- to small-scale faults and is represented mainly by compressive-shear reverse faults that changed later to extensional normals. This set is mostly occupied by some veins and has favourable engineering properties. The spacing varies from 30 to 40 m on the southern slope and from 35 to 50 m on the northern. It accounts for 40–45 % of all faults.

The second set with an average orientation of 240°/ 75° NW makes up about 30% of all faults. It is normally in the state of tensile-shear. These faults contain poorly cemented breccia. The representative fault F_{215}, of an orientation of 230°–250°/ 70°–80° NW is the largest one with little cement and transverses the southern and northern ship lock slopes. Its crush zone with a width of about 4–6 m is composed of alternating zones of weak breccia and fault gouge. Evidences indicate that F_{215} is a tensile-shear type of fault and exhibits unfavourable geo-engineering properties. The other two sets are secondary, and are small and well cemented. They make up about 3–5 % of the total, respectively.

About 15% of the faults are long enough to traverse the whole ship lock slope, and the others are of medium-scale and extend only to the one side of the slope. Most of the faults are well cemented and steep. The fault zone contains mainly cataclastic rocks indicating that the tectonic deformation in the ship lock area was relatively simple and characterised by brittle failure.

The theory of fractal geometry was applied to study quantitatively the complexity of the faults (Table 2). The results show that these faults are of well self-similar, and their correlative coefficients are greater than 0.95. The fractal dimension of the first set is the biggest and the next to it is the second set. The dimensions of these two sets are bigger than those of the entire fault system. Therefore, the first and the second sets are more complex than the others, and the entire fault system is anisotropic and unevenly distributed.

JOINTS IN THE SHIP LOCK AREA

Joints, cleavage, schistosity, and fissures are the predominating penetrative discontinuities in the ship lock area. These structural surfaces and their combination are the dominant factors to control rock mass quality, slope rock stability, and groundwater flow.

We carried out field measurements and statistical analysis of joints mainly in the main drainage tunnel, monitoring sub-tunnel, and on the slope surface from the first chamber to the third one during the first stage of slope excavation. The same work was not carried out in the fourth and fifth chambers due to the presence of deeply weathered rock mass. The statistical analysis of joints from the third chamber is discussed below as an example.

The excavated rock slope in the third and the second chambers is the steepest among the five chambers. The weathering grade of rock exposed on the slopes varies from slightly weathered to fresh. However, large faults and joints are well developed in the area. Owing to the effect of F_{215} and other faults and veins, the joints in the third chamber are well developed, and they are more developed in the southern slope than those in the northern. In the southern slope, the maximum linear density is 5.76 per m and the minimum is 2.53 per m with an average value of 2.95 per m. In the northern slope, the maximum linear density is 3.35 per m and the minimum is 2.2 per m with an average value of 2.70 per m.

The field investigation and statistical analysis revealed that the spatial distribution of joints is characterised by anisotropy, heterogeneity, and non-uniformity, coinciding with that of the faults. There are alternating sparse and closely spaced joints. Close joint spacing is observed near the fault zone where they are becoming wider while moving from it. The joints are closely spaced near the fault zone but become sparsely spaced while moving away from it. This implies that in some cases, the fault controls the distribution of joints, but in other cases, the fault originates from the joints.

Like the faults, the joints are also classified into four sets in the third chamber (Table 3).

The statistics shows that the joints in the third chamber are longer than those in the first chamber, but shorter than those in the second. Most joints exhibit planar surface but some of them are also slightly rough or undulating with some cemented infilling.

The gently dipping joints are sporadically developed in the ship lock rock mass. They are of relatively large-scale with average length of about 15 m, and are of proportional spacing in both the plan and cross-section. Therefore, the gently dipping joints play a significant role in the stability of the rock mass. The preferred shape of these joints is mostly planar and sometimes slightly rough or undulating.

ROCK MASS STRUCTURE IN THE SHIP LOCK AREA

The characteristics of structural surfaces and rock bodies, as well as their combination and arrangement define the rock mass structure. The rock mass structure in the ship lock area can be classified into the following five types: intact, blocky, cataclastic, mosaic, and loose. The main statistical characteristics of these classes of rock mass structure are presented in Table 4.

Table 2: Statistical fractal dimensions of faults in the ship lock area

Fault Set	Fractal dimension	Correlative coefficient
340–355°/ 70–85°SW	1.01–1.05	0.998
60–80°/ 75–85°NW	0.91–0.95	0.997
130–150°/ 70–80°NE	0.75	0.970
10–40°/ 55–75°SE	0.6–0.7	0.965
Entire fault system	0.895	0.982

Table 3: Statistical characteristics of joints in the third chamber

Joint Set No.	Occurrence		Spacing			Length	Volume density	JRC
	Dip direction Range/Mean	Dip Range/Mean	Mean/ Variance (m)	Probability distribution	Fractal dimension	Mean/ Variance (m)		
I	320°–350°/340°	60°–85°/80°	0.80/1.23	Negative exponent	0.89	9/1.15	1.32×10^{-2}	8–12
II	240°–260°/260°	60°–85°/76°	2.21/3.14	Negative exponent	0.62	9.5/2.1	1.26×10^{-3}	8–12
III	50°–80°/70°	60°–80°/70°	4.39/6.50	Negative exponent	0.28	4.5/2.8	0.65×10^{-3}	10–12
IV	10°–40°/17°	60°–85°/70°				3.8/3.1		10–12

Table 4: Classification of rock mass structures in the ship lock area

Type	Joint spacing (cm)	Joint set number	RQD (%)	Intact coefficient (Ip)	P-wave velocity Vp (m/s)	Properties of discontinuity	Structure deformation and weathering
Intact structure	>100	1–2	>95	>0.85	4800–5850	Planar, slightly rough, no or few infillings	Slight deformation, the fresh and slightly weathered rock
Blocky structure	50–100	2–3	95–80	0.75–0.85	4300–5500	Planar, slightly rough or undulating, well cemented	Slight deformation; IV, V order, partly II, III order structure surface relatively developed, the fresh and slightly weathered rock
Mosaic structure	15–40	>3	75–50	0.6–0.7	2600–3500	2-3 sets dominant discontinuities	Near faulted zone, close jointed zone and pegmatitic dykes, the highly and the upper moderate weathered rock
Cataclastic structure	10–20	>4	<30	0.28–0.5	<3000	Weak discontinuities	Strongly deformed fractured zone, the intensively and the upper moderately weathered rock, and confined weathered belt
Loose structure					500–3000		Completely and intensively weathered rock and chambered weathered rock

All phenomena show that the distribution of various rock mass structures is governed essentially by deformation, weathering, and lithology in the ship lock area. The intact and blocky structures are dominant in the areas containing slightly weathered and fresh rock with sparse joints and faults. The blocky structure of rock mass is observed in the areas with moderately to slightly weathered rock with widely to closely spaced joints. Similarly, the mosaic and cataclastic types often occur in some poorly cemented belts, such as near F_{215}, or in the gneiss xenolith and the diabase bands. The loose rock mass structure is seen in the areas with completely weathered rock or confined weathered zone within the fractured rock.

EVALUATION OF ROCK MASS QUALITY

Some current rock mass quality classifying systems, such as RMR (Bieniawski 1974, 1989), Q-system (Barton et. al. 1974), SMR (Romana 1988), the national standard of rock mass rating and quality classification of rock mass in Chinese technical specification for water resources and hydropower engineering geological investigation (National Standard of People's Republic of China 1996), are known methods to evaluate rock mass quality. They can be called as static methods. These methods may not be satisfactory for a complicated rock mass because they are characterised by the fixed indices and linear calculation. In fact, the geological and engineering factors influencing slope rock quality are complex, uncertain, fuzzy, and multi-hierarchical. Thus, the quality of rock mass was assessed by using the static as well as such dynamic methods as multi-hierarchical Fuzzy synthetic evaluation and Fuzzy Neural Network.

As an example of rock mass quality evaluation in the ship lock area, the slopes lying between the first and third chambers are described below. It is an important aspect to determine the weight coefficient of factors affecting slope rock mass quality with Fuzzy synthetic evaluation method. The multi-hierarchical index system is presented in Fig. 4. The modified AHP method (Saaty 1980) is adopted to identify the weight coefficients in this paper.

With the above different methods, the quality of rock mass was evaluated for main draining tunnel, monitoring sub-tunnel, and the slope at an elevation of 200 m (Table 5 and 6).

The above analyses indicate that the classifications of the rock mass quality evaluated using different methods are similar. The rock mass quality varies mainly from Class I (very good) to the Class III (fair) with most of the area lying in Class II (good). The rock mass quality is mainly Class I for the intact and blocky structure, characterised by the presence of fresh and slightly weathered rock with lesser discontinuities. Class III or Class IV rock mass is distributed in the vicinity of fractures and the diabase band, which is characterised by the mosaic and the cataclastic structures. However, Class V exists only in large fault zones (such as the fault F_{215}) and in the completely weathered rock. The rock mass quality is improved with the increasing excavation depth (from Class III to Class II or Class I), which corresponds to the improvement of the rock mass structure (Fig. 5).

EFFECT OF BLASTING AND EXCAVATION

To evaluate quantitatively the weakening effect of excavation and blasting on slope stability and rock mass quality, two numerical modelling techniques including damage variable and finite element were used.

According to damage mechanics theory, rock mass is regarded as a whole damaged medium and the damage variable is used to demonstrate the degree of damage. The damage variable is calculated by means of three-dimensional fracture density (Mauldon 1994). Assuming a line sample of the plane, n (x) is the number of joints crossed by a strip of thickness *dx*, and then, the micro-damage variable can be expressed as:

$$\Omega = \frac{2\overline{N} \times \overline{S}}{W \times \sin 2\alpha}$$

where, $\overline{N}$ is the average number of joints in the sampling area; W is the length of the scan line; α is the angle between the joint set normal and the horizontal direction; $\overline{S}$ is the average spacing of the joint set. $\overline{S}$ and $\overline{N}$ can be directly calculated by surveying joint set in situ.

The analytical results obtained for a monitoring sub-tunnel are listed in Table 7.

The above results imply that the impact of excavation and blasting on the slope surface is the most serious, and that on the sub-tunnel is intermediate, whereas the main tunnel is subjected to the smallest impact.

Table 5: Comprehensive evaluation of the rock mass quality

No.	Site	I	II	III
1	Slope surface (slightly weathered rock)	0	80	20
2	Monitoring sub-tunnel	35	55	10
3	Main tunnel (fresh rock mass)	65	35	0

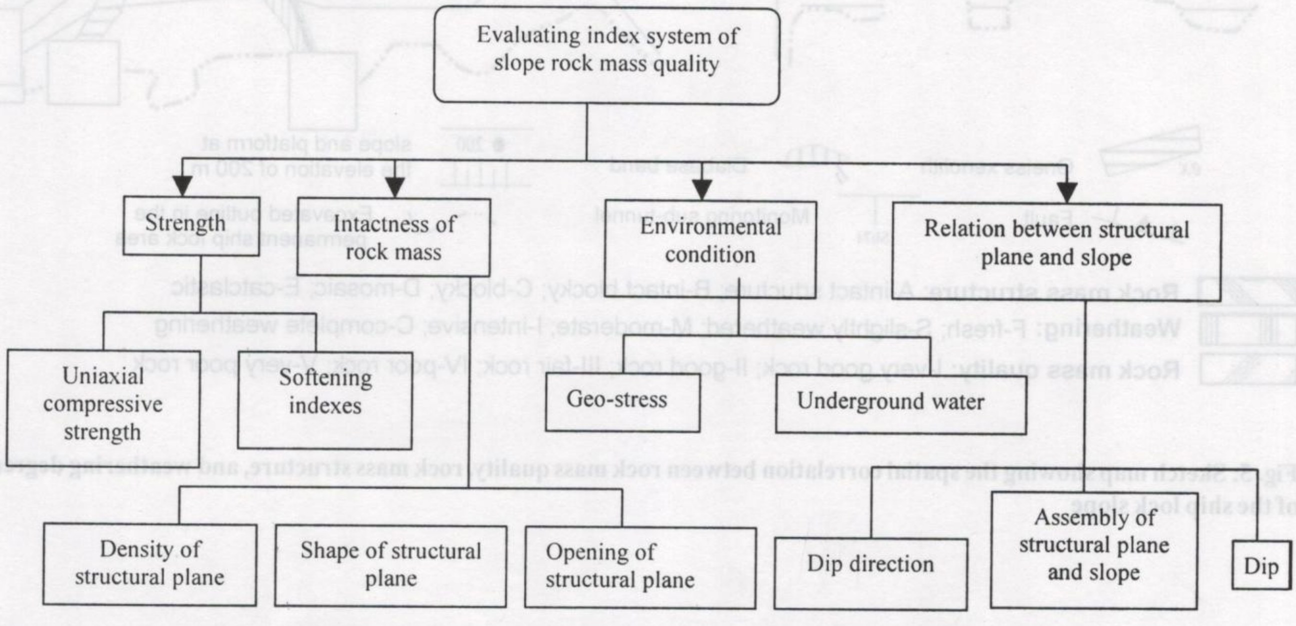

Fig. 4: Multi-hierarchical index system of slope rock mass quality

Table 6: Correlation between rock mass quality, rock mass structure, and weathering grade in the ship lock slope

Rock mass classification	Rock mass structure (granite): Intact	Blocky	Mosaic	Cataclastic	Slightly weathered and fresh granitic dyke	Weathering grade: Fresh	Slightly weathered	Moderately weathered	Highly weathered	Completely weathered
I (very good)	0.95–0.85	0.55–0.7	/	/	>0.70	>0.75	0.30–0.45	0.10–0.30		
II (good)	?0.5	0.1–0.2	/	/	0.10–0.30	>0.20	0.45–0.65	0.20–0.45	/	/
III (fair)	/	?0.15	0.65–0.80	0.50–0.15	/	?0.1	?0.2	0.35–0.50	0.10–0.20	
IV (poor)	/	/	0.20–0.30	0.30–0.45	/	/	/	?0.10	0.40–0.55	/
V (very poor)	/	/	?0.10	0.50–0.65	/	/	/	/	0.15–0.30	>0.85

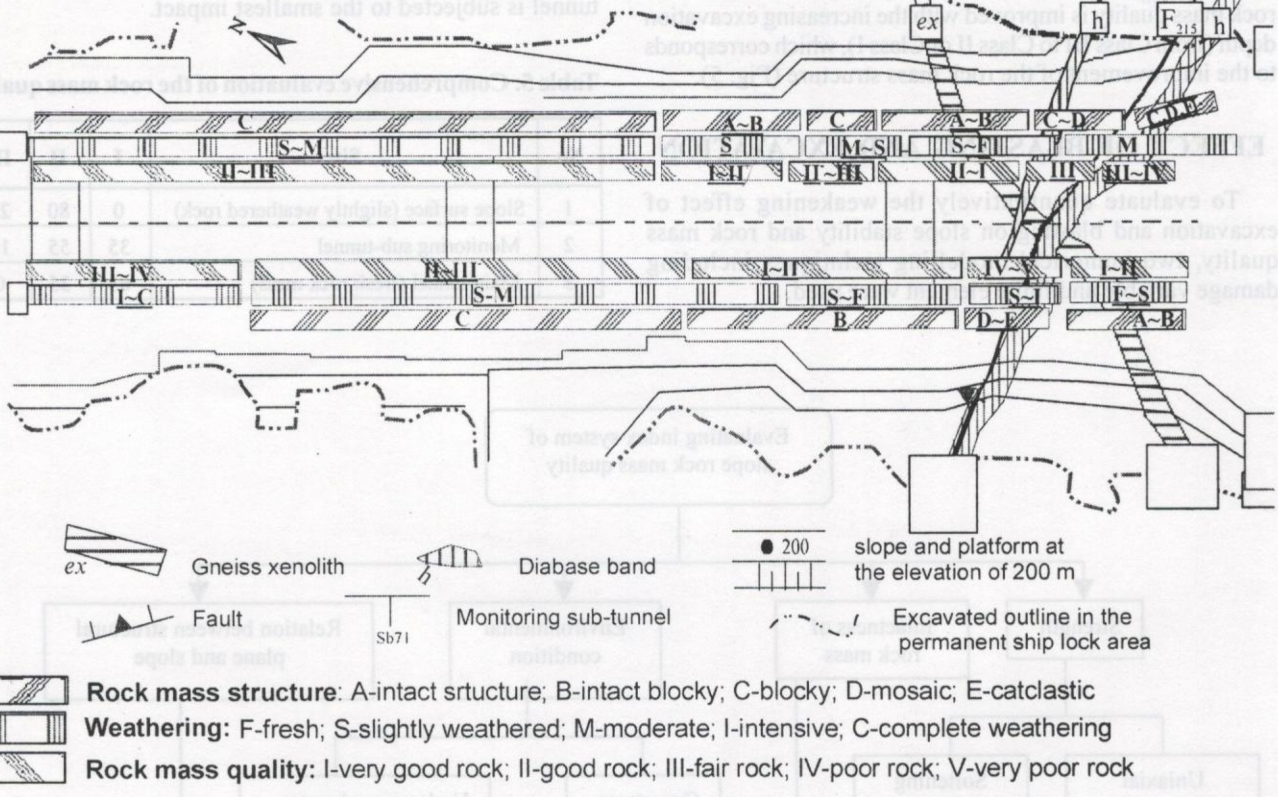

Fig. 5: Sketch map showing the spatial correlation between rock mass quality, rock mass structure, and weathering degree of the ship lock slope

Table 7: Micro-damage variable Ω calculated around a monitoring sub-tunnel

Location	α	$\bar{N}$	W(m)	$\bar{S}$(m)	Ω
Slope surface	60	16	60	0.97	0.6
Sub-tunnel	40	14	18	0.86	0.5
Main-tunnel	60	5	28	1.00	0.4

FINITE ELEMENT METHOD OF MODELLING

To simulate quantitatively the dynamic effect of blasting and excavation on rock mass, the two-dimensional finite element program UDEC was used in the analysis. UDEC can demonstrate the direct dynamic relationship between stress, deformation, and time. UDEC specifies the interactions between a block and its surrounding neighbours with force-displacement laws.

However, it is very difficult to define the dynamic load in the actual event because the load variation with time, magnitude, and extent depends on a number of factors, such as the volume of leaking gas/liquid, energy extent, and extreme conditions such as ventilation and time before initiation of blasting. The dynamic load used herein was defined by Rosengren (1993). The peak pressure of 1 MPa was assumed to be reached after 100 ms.

Geometrical parameters

According to the present construction status, the model is assumed symmetrical. Thus, half of the whole model is selected in the analysis. The geometrical parameters of the model are shown in Table 8.

The rock block is assumed to behave as an elasto-plastic medium conforming to the Mohr-Coulomb yield criterion, but the joint is assumed to follow the Coulomb slip criterion. The properties of the block and joint used in analysis are shown in Table 9.

Modelling results

The modelling results show that the rock mass is slightly affected by excavation, and the tensile stress region develops only in the slope surface with an influencing distance of about 5–6 m. The maximum displacement of slope is 2.8 cm. These results confirm well with the field observations that after excavation, the originally tight joints had the apertures of 4–5 mm. The agreement between the simulated and measured displacements indicates that the model was calibrated correctly. At the same time, the results agree well with that of the modelling by damage variable and rock mass quality evaluated. The impact on the slope surface is the most serious and the sub-tunnel is subjected to intermediate influence, whereas the impact on the main tunnel is the lightest.

Consequently, the surveying and modelling results concurrently show that the excavation-loosened zone is about 6 m deep on the slope surface, and beyond that depth, the effect of excavation is insignificant.

ANALYSIS OF SLOPE STABILITY

According to above analyses and calculations, the rock mass structure is mainly intact and the rock mass quality is good. Various types of structural surfaces are mainly well cemented and steep, and the intersection angle of the preferred structural surfaces and the slope axis is greater than 45°. Though the NWW-striking and NEE-striking joints with a length of 5–7 m intersect the slope at an angle of 20–30°, they are sporadically developed and only about 17 % of them are exposed. Therefore, they will not affect the overall stability of the slope except for small plane sliding and wedge failure. Additionally, the numerical analysis indicates that blasting and excavation affect slightly the overall stability of the slope and the rock mass quality. Consequently, due to favourable rock mass structure, good quality of the rock mass, and rational engineering design, the overall stability of slopes can be ensured and it is unlikely for large-scale block failures to happen. However, minor local unstable blocks controlled by slope surface, platform, and

Table 8: Geometrical parameters used in finite element analysis

Object	Parameters	Object	Parameters
Length	144 m	Slope height	+245 m
Height	90 m	Step height	15 m
Platform width	12 m	Slope angle	73°

Table 9: Block and joint properties used in modelling

Block		Joint	
Parameter	Value	Parameter	Value
Density, ρ (kg/m^3)	2700	Normal Stiffness, K_n (GPa/m)	67.2
Volume modules, g (GPa)	26.6	Shear Stiffness, K_s (GPa/m)	27
Tangible Modules, k (GPa)	16	Cohesion, c (MPa)	0.7
Cohesion, c (MPa)	2	Friction angle, ϕ (°)	41
Friction angle, ϕ (°)	41	Tensile strength, σ_T (MPa)	0
Tensile strength, σ_T (MPa)	2		

structural plane can be formed within the height of one slope bench. The local unstable blocks are formed mainly in the following three ways:

(1) Determinate blocks formed by the intersections of Grade II and Grade III structural planes;

(2) Half-determinate blocks of minor to moderate scales formed only in the upper slope and platform by the assemblage of faults and joints, especially with NWW or NEE strikes; and

(3) Stochastic blocks, made up of joints as major structures. They are generally formed by slope surface and platform with intersection of NWW or NEE-striking joints, especially the long and large joints with moderate or gentle dips.

Therefore, it is feasible and efficient to design a system of rock bolting and anchoring the unstable or potentially unstable blocks.

Failure modes of the ship lock slope

Based upon the investigations and comprehensive analyses, there are mainly four failure modes for the minor unstable blocks, namely, plane sliding, wedge failure, circular failure, and fall.

Plane sliding often occurs along the structural plane with the dip smaller than that of the slope and the azimuth within 20° of that of slope surface. It can be subdivided into single plane sliding and multi-plane sliding based on the structural plane sets. The plane sliding often occurs in the surface of the slope and affects easily the shape of the slope.

Wedge failure is composed of two or more sets of structural planes with opposite dip direction and slope surface. It can be subdivided into two types: wedge cone block and wedge prism blocks. The former is one of the major modes and it is formed with two sets of opposite dipping structural plane, slope, and platform in the upper slope. The latter is secondary and composed of more than two sets of differently dipping planes, slope and/or platform, or another structural plane.

Circular failure occurs mainly in the completely weathered rock and the scale is small.

Fall often occurs in the cataclastic structural rock mass. The unstable block is composed of several sets of crosscutting discontinuity planes, especially near fault zones, joint clusters, and the diabase band.

CONCLUSIONS

The rock mass structure in the ship lock slope of the TGP is simple and relatively intact, and the rock mass quality is good. The tectonic deformation in the ship lock area is relatively simple and characterised by brittle failure. The weakening effect of blasting and excavation is apparent only within a reach of 5–6 m. These are advantageous conciliations to ensure the overall stability of the slope. The whole ship lock slope is stable, but local instabilities also exist in the form of determinate, half-determinate, or stochastic blocks, and rock bolting and anchoring measures can be applied to control them.

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