Stability Analysis by FEM on New Large Shiplock Slopes in Yangtze River

유한요소법에 의한 양쯔강 신설 대수로사면 안정검토

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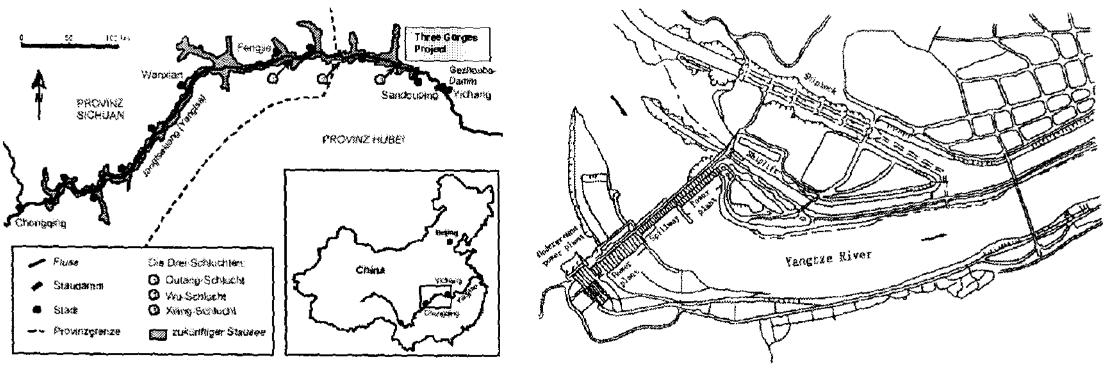
개 요 : 중국 Three Gorges Project의 대수로사면 안정성은 설계와 시공측면에 있어 주요 관심사가 되었다. 사면 굴착으로 인한 제하과정에서 암반은 역학적으로 불안정한 상태에 놓인다. 본 논문은 FEM(2D-3D)를 이용하여 단층 암반 굴착으로 인한 암반사면의 안정성을 평가하였다. 해석결과, 굴착 후 수로사면의 양측 수직벽과 분리울타리의 중간·상부에서 인장응력과 전단손상영역이 주로 발생하였다. 해석결과를 토대로 대규모 사면활동에 대한 안정성을 확인하였으나, 시공단계에서 국부적 사면활동을 방지하기 위한 록볼트와 록앵커 등의 보강이 필요한 것으로 검토되었다.

Key words : Stability, Permanent shiplock slopes, Three gorges project, Finite element method, Excavation

1. INTRODUCTION

Three Gorges Project (TGP) is the largest water conservancy project ever built in China, and in the world. With normal pool level (NPL) at 175m, the total storage capacity of the reservoir is 39.3 billion cubic meters. The TGP is a multi-purpose hydro-development project producing comprehensive benefits mainly in flood control, power generation, and navigation improvement.

The Three Gorges dam is a conventional concrete gravity dam, 181m high above lowest foundation, and 2,309m long. The upstream face is vertical and the downstream face slopes at 0.7 (horizontal) to 1.0 (vertical), similar to most gravity dams. The dam site is situated at Sandouping of Yichang City, Hubei Province. The bedrock of the dam site is sound and intact granite that has intruded through the limestone. The faults and fissures in the bedrock are less developed. The permeability of the rock mass is slight in nature. The river valley is relatively open and broad at the location of Sandouping dam site, providing opportunities for diversion of the river during construction, and the granite provides an excellent foundation for the dam. The general layout plan of TGP is shown in Fig. 1.



(a) Scheme of Three Gorges Project (b) Layout of the New Large Shiplock Slope

Figure 1. Plan of Three Gorges Project

The permanent shiplock, a double-line continuous five-stage flight structure with a total chamber length of 1607m, is provided to lift or lower vessels up to 10,000 tonnes through 113m, from 62m downstream to 175m upstream. The shiplock is a massive structure, which is constructed by deeply and widely excavating into the mountain on the north bank of the Yangtze River. To tally 40 million m3 of rock excavation has been carried out. As shown in Fig. 2 and plate 1, the permanent cut slopes in both weathered and fresh granitic rocks have been formed as a result of such excavation. The slopes comprise two steep upper cut slopes and four vertical ones. The maximum height of the cut slope is about 170m. A 60m wide central rock barrier is kept between the north and the south chambers. The dimensions of each chamber (between successive locks) are 280 × 34 × 5m (length × width × minimum water depth).

The entire excavation work started in April 1994 and ended in May 1999. After excavation, the high slope of the lock is lined with thin concrete, and a series of strengthening measures are applied to ensure the safety, including rock body drainage, prestressed tendons and high strength rock bolting, shotcreting, etc. A wide range of safety monitoring systems has been provided. There are 1500 instruments embedded in the lock. 3600 prestressed tendons and 100,000 high strength structural rock bolts have been installed. The shiplock has been put into operation since 2002.

The unloading process due to the large scale excavation would definitely have some negative effects on the mechanical properties of the rock mass in the permanent slopes. So during design and construction stages, the stability of the permanent shiplock slopes after excavation is among the key issues of the TGP, having drawn considerable interest from the geotechnical communities in China and around the world. Many relevant researches based on different approaches, such as field investigation, in-situ test, physical and numerical simulation, displacement back analysis and so on, has been carried out, and a number of achievement have been obtained (e.g. Chen 1999; Feng et al. 2000; Deng & Lee 2001; Sheng et al. 2002).

This paper presents some work which the first author had carried out during the feasibility study and design analysis of slope of permanent shiplock. An elasto-plastic finite element method (FEM) (2D and 3D) was applied to study the characteristics of stress and displacement distribution of the rock mass. The results obtained are used to evaluate the stability of the permanent shiplock slopes under the effects of excavation.

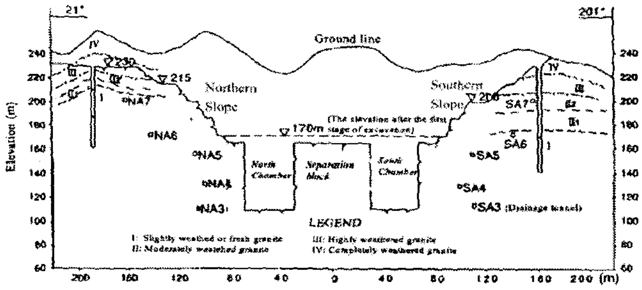


Figure 2. Typical cross-section of the permanent shiplock chambers

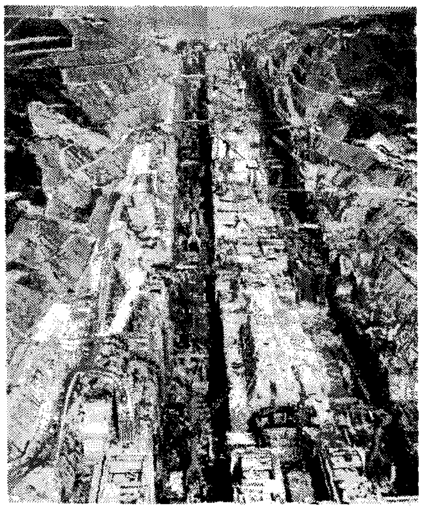


Photo. 1 Site photograph illustrating the on-going construction of the permanent shiplock cut slopes

2. ROCK MASS PROPERTIES

The permanent shiplock slopes comprise amphibole-plagioclase granite, which is generally sound and intact with a very limited distribution of schist xenoliths and veins. The granite is completely decomposed at the ground surface. The degree of weathering decreases with an increase in depth, until reaching the fresh rock. Based on the type and macrostructure of the rock block, the integrity and mechanical properties of the rock mass, and the hydrologic properties, four different grades of weathering in the granite are identified as follows (shown in Fig. 2): (a) slightly weathered and fresh (I); (b) moderately weathered (II); (c) heavily weathered (III); and (d) completely weathered rock (IV). The moderately weathered granite can be further subdivided into an upper part (II2) and a lower part (II1), based on the differences in weathering characteristics and engineering geologic properties.

Faults are uncovered on the vertical sidewalls of the shiplock chambers and the upper surface of the central rock barrier. In the central barrier and southern slopes, the fault set with the strike direction of north to northwestern (NNW) is the most highly developed, followed by the set with direction of northeastern to east (NEE). While in the northern slopes, the NEE fault set is the most developed, followed by the NNW and NNE (north to northeastern) fault sets.

A relatively large number of laboratory tests were conducted during the design stage. The test results showed that the slightly weathered or fresh granite blocks have a uniaxial compressive strength and a deformation modulus greater than 120MPa and 60GPa, respectively, which indicate that the granite blocks were sound and competent. For the purpose of numerical analysis, the values of the mechanical properties of the granites and faults have been summarized in Table 1, based on the geological investigation and in-situ test. These parameters include unit weight, Young's modulus, Poisson's ratio, friction angle and cohesion.

Table 1. Parameters of granites and faults adopted in numerical analysis

Rock type	I	II	III	IV	Fault
Unit weight (kN/m^3)	27.0	26.8	26.5	26.5	26.5
Young's modulus ($10^3 kN/m^2$)	40.0	15.0	1.0	1.0	5.0
Poisson's ratio	0.22	0.24	0.30	0.30	0.35
Cohesion (kN/m^2)	1.8	1.0	0.35	0.35	0.5
Friction angle (^{o})	60.9	52.4	45.0	45.0	45.0

3. IN-SITU STRESS FIELD

A total of 60 sets of in-situ stress measurements were made in thirteen deep drillholes using the over coring and "hydraulic fracturing techniques" in the permanent shiplock area. These measurements were used to regress and define the in-situ stress field. For example, in the area at the head of third lock chamber, the in-situ stress field has following form

$$\sigma_x = 4.4982 + 0.01168H \quad (MPa) \tag{1}$$

$$\sigma_y = 4.7152 + 0.01027H \quad (MPa) \tag{2}$$

$$\sigma_z = 1.6628 + 0.03039H \quad (MPa) \tag{3}$$

$$\tau_{xy} = 0.4048 + 0.00005H \quad (MPa) \tag{4}$$

$$\tau_{yz} = 0.7470 - 0.00046H \quad (MPa) \tag{5}$$

$$\tau_{zx} = -0.04720 + 0.00008H \quad (MPa) \tag{6}$$

where, H is depth below ground surface(m), and compressive stress is define to be positive

4. NUMERICAL ANALYSIS (FEM)

A 2D FEM simulation was first performed on two typical cross-sections. Then 3D numerical modelling was carried out. In order to simulate the rock mass deformation under more unfavourable conditions, the bolts and cable anchors adopted in actual supporting are not considered in the course of simulation by FEM. The rock mass is considered to be an elasto-plastic medium in the FEM analysis. The no tension yield criterion has been used for the completely weathered and highly weathered granite and fault. The Drucker-Prager yield criterion was used to define the yield surface of the slightly weathered and fresh granite.

The simulation of excavation process by FEM can be described as follows: (1) First, the released stresses are taken as equivalent loads and imposed counter on the excavated openings; (2) Second, the displacements and stresses of current excavation step are solved by FEM; (3) Then, the obtained displacements and stresses are taken as the initial values of the next step, and the excavation problem is thus solved step by step until the final excavation step.

4.1 2-Dimension Analysis

4.1.1 Deformation

The mesh of cross-section No. 16-16 for 2D FEM analysis is presented in Fig.3.The main deformation occurs in both chosen sections due to the excavation unloading. Fig. 4 shows the displacement vectors of cross-section No. 16-16. It can be seen that the left slope moves toward right and upper, theright one develops toward left and upper, and the central rock barrier moves mainly toward upper and spreads a little on both sides. The main displacements take place in the two steep upper cut slopes and upper part of the central barrier. The maximal values of vertical displacement vector for two sections are 24.77 and 48.37mm respectively. Deformation monitoring after excavation revealed maximum deformations of 54mm, resulting from relaxation of the upper rock.

4.1.2 Stress Redistribution

Stress redistribution due to the excavation unloading has been evaluated by considering the original in-situ stress field. Fig. 5 illustrates a typical example of the redistributed stress field of the cut slopes after excavation in the section No. 16-16. The major principal stress in the granites below the two shiplock chambers and the central rock barrier remain in a horizontal direction. Near the surface, the direction of the major principal stress becomes almost parallel to the excavated slope surface, while the direction of the minor principal stress is perpendicular to the cut surface. The stresses can drop off to become tensile near the exposed surface. In the middle and upper portions of the central barrier, the stresses become almost zero. The phenomena of stress concentration take place in the toes of the vertical cut slopes, where the computed maximum compressive stresses are between 25.0 and 36.0MPa. Such high levels of compressive stresses have obviously resulted from the up to 170m excavations.

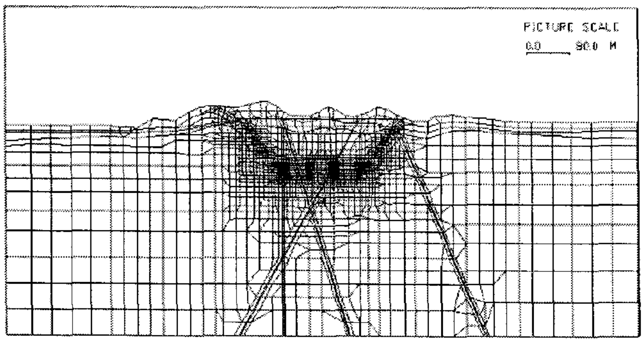


Figure 3. Mesh of section for 2D FEM analysis 16-16

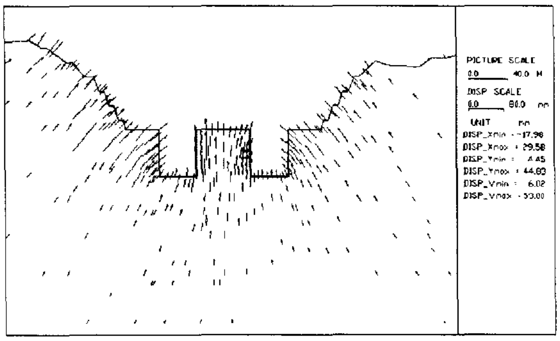


Figure 4. Displacement vectors of cross-section 16-16

4.1.3 Plastic Yield Zones

Relatively large plastic zones occur in the upper half of the central rock barrier and in the triangular zones of upper part of the northern and southern vertical cut slopes. At the slope toes, plastic yield zones are also predicted for both the two sections. Fig. 6 shows the plastic yield zones in cross-section No. 16-16. It is noted that in most of these plastic yield zones, the calculated stresses are low in compressive state or high in tensile state. This is because the excavation induced an unloading process in the unexcavated granites.

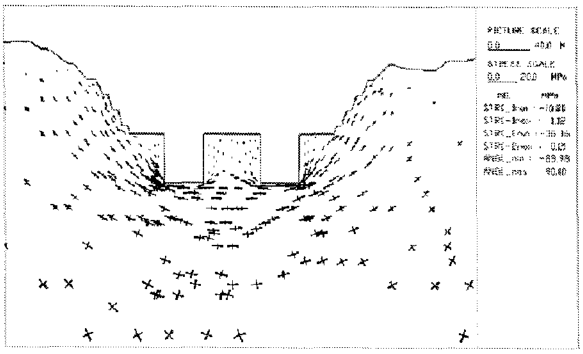


Figure 5. Stress vectors of section 16-16

Figure 6. Plastic yield zone of section 16-16

4.2 3-Dimension Analysis

The computation domain for 3D FEM analysis is 700m in length (X-direction), 320m in width (Y-direction) and 365m in depth (Z-direction), as shown in Fig. 7. The domain contains 8165 nodes and 7185 elements (eight-nodal isoparametric continuum elements).

The simulation results basically conform to the deformation pattern of the rock mass obtained from the 2D FEM analysis as discussed above, that is, the rebounding deformation occurs due to the unloading excavation. Fig.7 illustrates the computed vertical displacement contour. The horizontal and vertical displacements calculated from 3D analysis are in general lower than the results from 2D simulation. For example, the maximal values of vertical displacement, for the above-mentioned two sections, are 17.1 and 20.12mm in 3D case respectively. The main reason is that the mutual constraint effects between the adjacent cross-sections have not been taken into account in the 2D analysis.

The stress distribution and plastic yield zones obtained from 3D FEM analysis also display minor differences to those of 2D FEM calculation. The region of tensile stress takes place near the surface of the slope and in the upper part of the central rock barrier.

The results of numerical modelling and prediction provided a basis for slope stability and deformation analysis as well as design. It is understood, however, that the elasto-plastic finite element analyses have been carried out based on the assumption that the rock mass is continuous, with a simplified excavation process. This assumption will, of course, differ from the actual field situation.

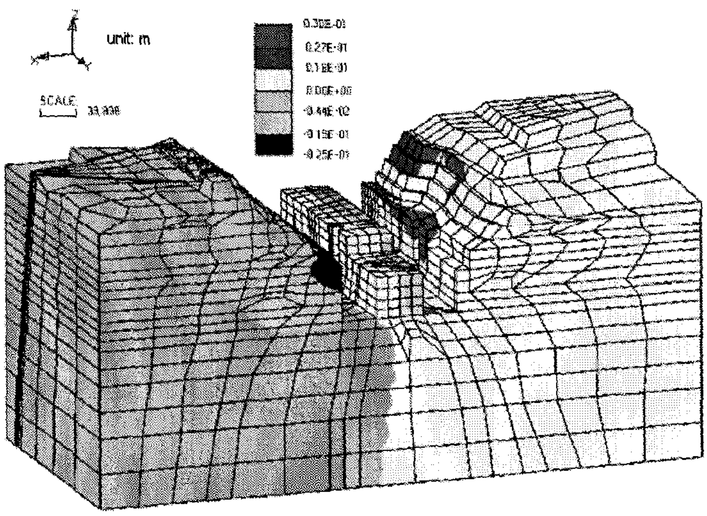


Figure 7. Vertical displacement contour from 3D FEM analysis

5. CONCLUSIONS

This paper employs the finite element method (2D and 3D) to evaluate the stability of permanent slopes under the effects of excavation. Our major findings can be summarized as follows:

1) After excavation, the deformation displays a rebounding pattern and the values of the displacements are within the accepted range.
2) The shear-damaged zones mainly occur at the middle and upper parts of central rock barrier and vertical walls. The dominant sliding path would not be formed by the extension and linkage of the local potential sliding surfaces. The global stability of the TGP permanent slopes can be ensured in this respect. The reinforcement may be required to prevent the local collapse during construction and operation stage.
3) Special attention should be paid to the central rock barrier, which has been unloaded in three directions after excavation and changed from compressive stress state to tensile state. Efficient monitoring measurements and support treatment must be adopted to control the excess deformation of the central barrier in order to guarantee the long-term safety of the TGP.

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