Geotechnical Aspects of Underground Construction in Soft Ground, Kusakabe, Fujita & Miyazaki (eds)
© 2000 Balkema, Rotterdam, ISBN 90 5809 1 066

Some deformation observations in underground openings

N.Gurung, Y.Iwao & K.Ishibashi
Department of Civil Engineering, Saga University, Japan

S.Hongo
Geology Group, Department of Civil and Architecture, Electric Power Development Company, Tokyo, Japan

ABSTRACT: This paper outlines geotechnical experiences on deformation observations of Lam Ta Khong project, Thailand. A case history on soft ground hydropower tunnels in quaternary deposits of clay/mud stone is sited. Adopted support patterns for tunnels and shafts based on the engineering geological rock mass classification were verified on stability by convergence responses. Deformation monitoring was part of the geotechnical evaluation for the safety of the support systems. Typical tunnel movements with respect to the face advance over elapsed time at various stations and depths are presented. In soft clay/mud stone underground construction, rates of lateral deformations are found to depend upon the pore water and effective earth pressures.

1 INTRODUCTION

The project is located nearby lake of Lam Ta Khong in Nakhorn Ratchasima province, which is about 180 km north-east of Bangkok. The area consists of Jurassic sedimentary rocks of Khorat (Phu-Kradung) formation mainly as sandstone, sand/siltstone, siltstone and quaternary deposits of clay/mudstone. Major underground structures are Access tunnel to Powerhouse, Central Powerhouse cavern, Drainage tunnel in Upper Pond, Penstock tunnels (horizontal, vertical and inclined), Headrace, Power Cable and Tailrace tunnels. Most of the hydropower tunnels in sedimentary rocks were excavated by conventional drill and blast method. Typical tunnel advance rates were 1.5 to 3.5 m per round with 1.1-1.8 kg/m^3 of specific explosive charge. Some soft ground part of tunnels in clay/mudstone (ST-3) of Upper Access tunnel to Penstock, Upper Horizontal tunnel to Penstock, Drainage tunnel in Upper Pond, etc. were excavated by mechanical excavators and road head cutters without use of any explosive charges. Standard support pattern based on the rock mass classification was used and the deformation performances of tunnel were observed by regular deformation monitoring programs. Deformation monitoring frequency for the project was specified based on the daily rate of movement and the advanced face distance. Typical tunnel movements with respect to the face advance over elapsed time at various depths are presented. In deeper stations, the deformations due to excavation are observed to be relatively smaller. Some aspects of geotechnical observations and deformation monitoring about the project may be obtained in Gurung & Iwao (1998). Brief discussions on the deformation of tunnel support due to excavation and the deformation analysis related to soft ground cases are presented.

2 GROUND CLASSIFICATION

Soil or Rock mass classifications are mainly used to solve problem of support design, construction of underground structures and intend to reveal geotechnical information to carry out safe, effective and reasonable excavation. Out of a number of overseas and Japanese classifications, the project rock mass classification was based on the Japanese classification system of the Electric Power Development Corporation (EPDC, 1992), Japan. In this classification system, geological factors and geotechnical properties are recorded in digital expression and mainly the type and strength of rock mass are considered mainly. Fundamental geological factors for rock mass classification are state of weathering, hardness and joints spacings. Other input details used in the project were representative rock and soil type, stability of face, degree of weathering, hardness, rock quality designation (RQD), rock mass rating (RMR), joint details like strike/dip, aperture, infilling, roughness, state and persistence, inflow and support type. Underground geotechnical observations consisted regular mapping, describing, photographing and rock mass classification (EPDC, 1996) on full surfaces of excavation.

Five states of weathering, hardness and joint spacing are designated to define the rock mass class. Weathering states are defined as: very fresh, fresh, fairly fresh, weathered and strongly weathered based on degrees of the minerals altered and weathered. Hardness states are defined as: very hard (A), hard (B), moderately hard (C), moderately soft (D) and soft (E). The identification index for approximate uniaxial compressive strength from the hardness is presented in Table 1. Joint spacing ranges are measured as: over 200 cm, 200-100 cm, 100-40 cm, 40-20 cm and 20-5 cm. Based upon these three basic geotechnical parameters, sandstone and siltstone of the project area were classified as SS-1, SS-2, SS-3, ST-1, ST-2 and ST-3. All clay/mudstones and weathered siltstones (weathering state higher or equal to the degree 3), were categorised in ST-3 class. Soft quaternary deposit of clay and mudstone as well as sedimentary rocks of sandstone, siltstone, silt/sandstone were also found during underground excavations in the project site. Figure 1 illustrates sketch on geological profile along the upper tunnels. Representative types of rock or soil encountered in the excavation were sandstone, siltstone, clay/mudstone, alternation of sandstone and siltstone.

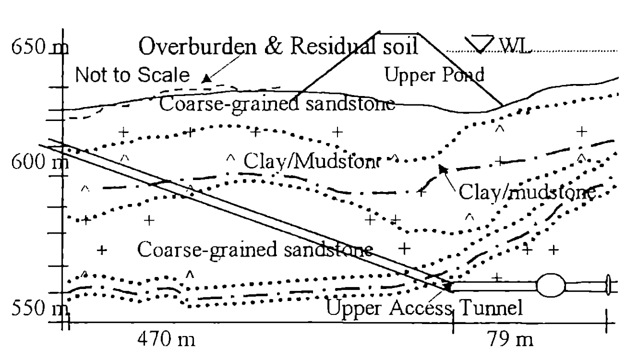

Figure 1. Sketch of upper access tunnel to penstock.

Sandstone (SS-1 to SS-3), thinly banded alternate layers of sandstone/siltstone (SS/ST-1 to SS/ST-3) and siltstone (ST-1 to ST-3) rocks were observed in Access tunnel to Powerhouse, Arch Heading, Bench Excavation, Tailrace and Penstock tunnels, etc. These tunnels exist in parallel alignment in plan of Upper Pond area.

Most of the longitudinal section for both tunnels falls under clay/mudstone (ST-3) zone. The worst zones of soft swelling clays of ST-3 category were encountered in excavation of Drainage tunnels, Upper Horizontal Tunnels and Upper Access Tunnel to Penstock. The project correlations for rock mass types, hardness and RMR values are shown in Table 1. Probable cohesion and friction of rock mass may be tentatively estimated (Hoek and Brown, 1980) using the Geomechanics classification. Based on standard of the rock mass class, the excavation patterns (blasting pattern or cutting tools) and the primary support pattern are designed. Regular rock mass classifications, geological mappings (sidewalls, front face and roof), photographing, geotechnical descriptions (joint surveys, core drillings, stereographic analysis, sonic tests, Lugeon tests, etc.) and deformation observations were conducted during progress of excavation. Generally, the rock mass classification system by qualitative method is not fully consistent (Yufu 1995) with that by quantitative method. Rock classes by other systems of classification namely rock mass rating (RMR of Bieniawski, 1989), rock structure rating (RSR of Wickham et. al 1974) and comparative evaluation by the Q-system (Barton et. al 1974) were often applied to assure the adequacy of the project rock mass classification (EPDC, 1996).

3 STANDARD SUPPORTS FOR OPENINGS

Underground excavation especially in soft ground needs proper support to prevent the loosening or loss

Table 1. Correlation of the project rock mass classes.

Rock mass class			H	RMR	Cohesion	Friction
SS	ST	C/M	A		(kPa)	(deg)
			B			
SS-1			C	> 60	>300-400	25-45
SS-2	ST-1	-	D	50-60	200-300	25-35
SS-3	ST-2	-	E	40-50	100-200	15-25
	ST-3	ST-3	E	30-40		

Note: SS-Sandstone, ST-Siltstone, C/M-Clay/mudstone
H-Hardness, RMR-Rock Mass Rating

of ground, excessive wall movements, surface settlement, property damage and human injury. The project specification (EGAT, 1995) explains both temporary and permanent supports for underground structures. The standard support patterns (permanent) for diameter <3 m openings is illustrated in Table 2. The pattern of support requirements was primarily based on the rock mass classification. It was later modified after deformation observations for the New Austrian Tunnelling Method (NATM). The basic intention was to integrate underground opening into an overall tunnel support system with the surrounding ground (soil or rock formations) achieving economy, flexibility and stability. To enhance the NATM principle, measures like (a) geomechanical behaviours of ground were considered, (b) adverse states of stress and deformation (as far as applicable) were avoided by appropriate support in time, (c) admissible deformations were permitted and (d) regular deformation checks were monitored. Steel ribs were used in the early stations driven through colluvium. Patterned rock bolts were installed usually after every two rounds of tunnel advance. Shotcrete with or without wire mesh was placed after every three rounds of excavation. Majority of adopted specified supports in sandstone and siltstone worked satisfactorily, except in some cases of the problematic slaking siltstone and swelling clay/mudstone (ST-3). Slaking deterioration in moisture was major problem in siltstone portion and the immediate spray of shotcrete was adopted as a countermeasure to avoid further softening, slaking and weathering degradation. Soft ground swelling from percolated rainwater in the Drainage tunnel and the Upper Access tunnel to Penstock resulted in increased lateral earth pressures and the horizontal diameter had moved by significant amounts in extreme cases.

The typical support pressures (p_i) and average effective time of delay for support reaction are presented in Table 3. In the case of soft clayey ground, the rigid steel rib supports were seen to be less effective; such as in the wet clay/mudstone (ST-3) zone, where the sidewall earth pressure had increased drastically after rains and a collapse had occurred. The overall performances of the standard supports of the project were satisfactory. Using other comparable engineering rock class classification systems, i.e. the EPDC, RMR and Q-system, field and laboratory tests and numerical finite element models, the consultant and client independently checked and verified the stability and deformation of the adopted standard support patterns.

Table 2. Standard support pattern of tunnel D < 6 m.

Rock Class	SS1	SS2	SS3	ST1	ST2	ST3
Shotcrete *1	7	10	10	10	10	15
+ Reinforcement	-	-	Wire mesh	-	Wire mesh	Wire mesh
Rock Bolt *2	-	2	2	2	2	3
Pieces/Section	-	4	4	4	4	6
Spacing (m)	-	1.8	1.5	1.8	1.5	1.2
Steel Support	-	-	-	-	-	H 100
Spacing (m)	-	-	-	-	-	1.0
Percentage (%)	-	-	-	-	-	20

Note: *1 - Thickness, cm. *2 - Length, m.

4 DEFORMATION MONITORINGS

The powerhouse cavern was smoothly excavated (Dragados, 1996) in several stages of arch, bench and side in order to minimise ground loosening and rock over break. Top Gallery tunnel is located above the Powerhouse. Drainage Gallery and Draft Gate

Table 3. Typical support pressures (p_i).

Support types	p_i ranges	Effective time
Rock bolts	0 -552 kPa	Several hours
Shotcrete (5-20cm)	0.34 -1.4 MPa	Several hours
Steel sets	0-2.76 MPa	One day to weeks
Concrete lining	0.7-3.45 MPa	Weeks to months
Steel lining	3.4-20.7 MPa	Months

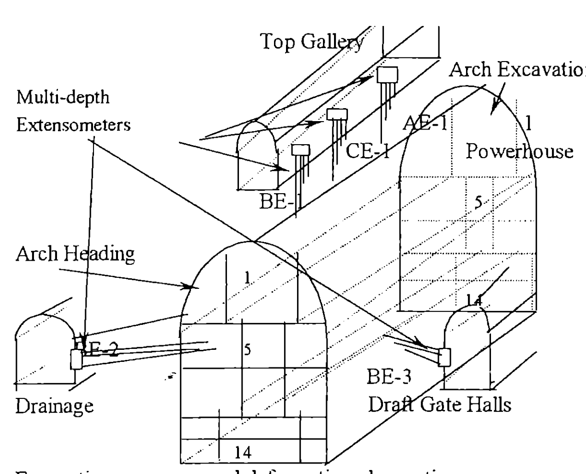

Figure 2. Excavation stages for powerhouse caverns.

Hall are aligned on sides of the central Powerhouse cavern (Fig. 2). Deformations in the surrounding rock stratums due to excavation were monitored regularly by multi rod extensometers. The specification for convergence mention that the face advance must be farther than five times diameter and the final movement rate must be less than 1 mm in a day for long period. But convergence rates (Bieniawski, 1984) of the order of 0.001 mm per day for assumed stable condition, 0.05 mm per day for alarming to large chambers and 1mm per day for recommended follow-up measurements.

4.1 *Observations by multi-position extensometers*

Responses of various geological strata due to underground excavation can be effectively monitored by multi depth extensometers. Multi rod extensometer placed in borehole is designed to measure relative displacements in soil or rock mass in underground constructions. Deformations in the surrounding rock stratums due to arch excavation in the powerhouse cavern were monitored regularly by multiple bore hole extensometers. Extensometers were installed in planes to check stress-deformation states. Typical deformation plot (BE-1) is shown in Figure 3.

Movements of geological strata at various depths of 14.8 m, 12.3 m, 9.3 m and 5.3 m are illustrated. The deepest ground stratum, which is also the nearest to the Arch excavation, was mostly affected by disturbances. Parallel trends of deformation with decreasing intensity for shallow depths, are visible. Extent of disturbed zones and the states of stresses for ideal cases may be analytically evaluated.

4.2 *Observations by tape extensometers*

All underground openings were routinely monitored by simple tape extensometers at an interval of thirty meters or less. The extensometers with ± 0.01 mm accuracy were being used. Three steel bar pins inserted as usually, one on crown and two others on sidewalls, made monitoring stations. Diametrical (A & B) and diagonal (C) lengths were measured. Relative movements of diagonals and diameters (ΔA, ΔB and ΔC), are monitored in each cross section along the longitudinal intervals of 30 m or less. Convergence rate was a criterion to study not only the support performance and but also the adjacent blast effects. Deformation levels of siltstone are comparable to soft ground tunnels. Rock mass deformations were found to depend on the face advance, the time of support application and the rock mass type. Typical measurement sections (A, B, C) are shown in Figure 4. Deformation rates and face advances of the tunnels were regularly measured. Often, parts of readings (ΔA or ΔB) were lost due to obstructions arising from ventilation pipes.

Measurements were followed till a long-term convergence of less than ± 1.00 mm per day was assured. The specified convergence criteria for stable tunnels based upon the movement rate per day is listed in Table 4. A collapse of opening in clay (ST-3) zone near entry portal in Drainage tunnel had occurred after heavy rainfall. Increase in the pore water pressure obviously reduces the shear strength. Pre-failure lateral deformation increased dramatically and sudden collapse occurred within a few days. Drainage and additional support in the forms of wire-mesh, shotcrete, rock bolts and steel ribs were

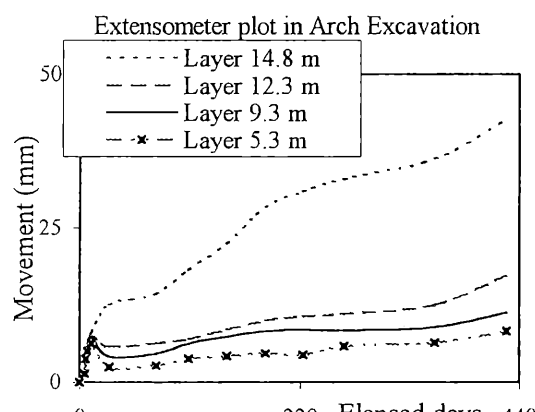

Figure 3. Deformations at sta. 0+51.5 from (BE-1).

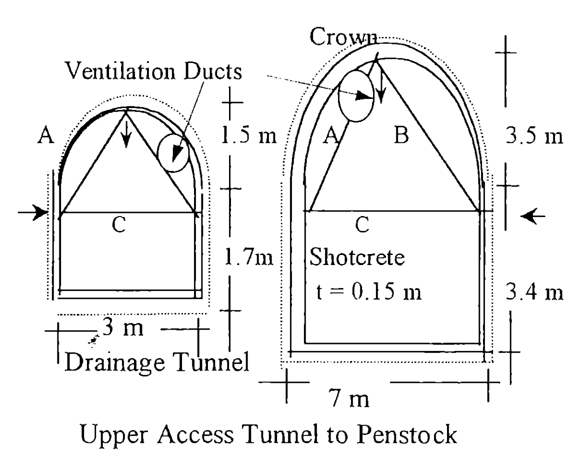

Figure 4. Typical sections for observations.

Table 4. Specification criteria for convergence.

Deformation Rate (mm/day)	Face Advance Distance (D)	Monitoring Frequency
Movement > 10	0 - 1	Once or twice a day
10- 5	1 - 2	Once a day
5- 1	2 - 5	Once every two days
Less than 1	more than 5	Once a week

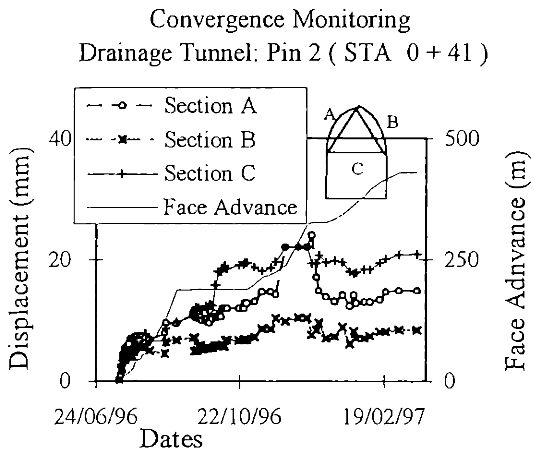

Figure 5. Deformation plot in drainage tunnel at sta. 0+41 m.

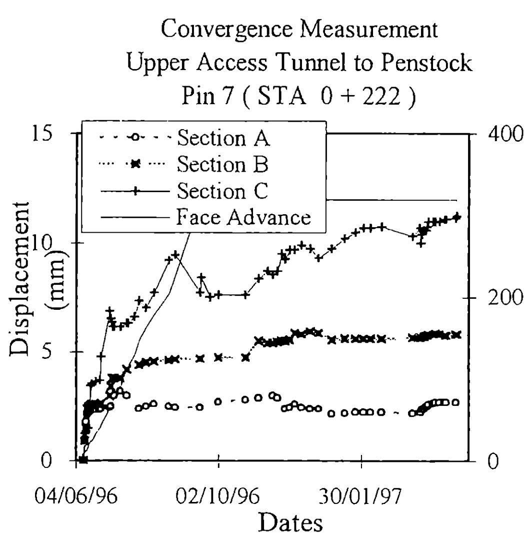

Figure 6. Deformation plot in upper access tunnel to penstock at sta. 0+222 m.

applied to counteract the problems of excess pore water, high earth pressure, continual swelling and large lateral deformation of underground openings.

Typical deformation observations at various stations for the Drainage tunnel are presented in Figure 5. Higher lateral deformations and smaller diagonal movements relative to crown are observed. Increasing rate of movements were attributed to the lateral pressures due to variation in inflows of groundwater from seasonal rains.

Some portions of Drainage tunnel showed creep, plastic and squeezing behaviours. Slow accumulation of load from time dependent movement and large strains exhibited. A part of section of Drainage tunnel just after entry portal had collapsed after heavy rainfall despite heavy steel rib supports. Drainage and additional supports and were provided to restore the opening. French drains, weep holes and perforated pipes were installed at regular intervals for drainage. Reduced pore water pressures and added supports had significantly helped to maintain balance with excavation stresses in the soft ground openings. In most cases, vertical settlements of crown were observed to be relatively smaller. It may have been reduced by arch action and stress redistribution. Horizontal deformations (ΔC) appear larger than the diagonal movements (ΔA, ΔB) relative to crown. It was attributed to high lateral earth pressures. Typical deformation plots at selected stations are presented in Figure 6 for the Upper Access tunnel to Penstock. The lateral deformation (ΔC) values are comparatively higher in clayey portion of the openings which may be probably due to some expansive minerals in the clayey soil. Deformation monitoring also helps to estimate the states of stresses. Lateral stresses can be easily estimated from convergence records. Stability of openings can be back analysed (Sakurai, 1998) from behaviour of rock mass strains.

The process of back calculation to estimate states of stresses from deformation records can be found in Chandler and martin (1994). Dramatic increase in lateral deformation within few days prior to the collapse in Pin 1 was noted but a part of deformation readings was lost for Pin 1 due to excessive bulging after rain and relocation of ventilation pipes.

5 CONCLUSIONS

The first hand support based on soil/rock classification need improvement after actual observations of the ground response and the support performance. Deformation observations help to monitor not only the effectiveness of support systems but also the safety and stability. Deformation values in the opening of sedimentary rock mass are relatively lesser than in the opening under soft quaternary deposit. Standard support patterns, that worked satisfactorily in sedimentary sandstone and siltstone needed further strengthening in case of the swelling quaternary deposit of clay/mud stone.

Lateral deformations are relatively higher than diagonal deformations for shallow depth openings of Drainage and upper Access tunnel. General deformation patterns for deep openings are found to be rather low. Subsequent blast disturbances were thought to be responsible for cumulative deformations in the opening of sedimentary rock mass. Seasonal ground water, lack of drainage, strength reduction due to softening, fissure expansions by pore water pressure and excessive lateral deformation are observed to be more critical for tunnels in cohesive clayey soil (ST-3). Unlike, deformation in the sedimentary rock, which had shown direct dependence on the rock mass type, deformation in the quaternary deposit of clay/mud stone showed dependence on seepage water, swelling of clayey component and effective lateral earth pressure behind the sidewalls of the tunnels. Lateral stresses, (tension) crack from pore water, ground softening, squeezing and other time dependent viscous behaviour were found to control the long-term movements in soft ground cases. In soft ground construction, high horizontal deformations might be encountered due to excessive lateral earth pressure. The swelling and shrinkage of clayey minerals, slaking and deterioration by geological weathering, softening and pore water pressure increase, etc. from various sources affect the weak regions. Adequacy of the adopted supports in soft ground tunnel must be verified by following the long-term convergence monitoring combined with frequent visual inspections.

REFERENCES

Barton, N., R. Lien & J. Lunde 1974. Engineering classification of rock masses for the design of tunnel support. *Rock Mechanics,* 6(4): 189-236.

Bieniawski, Z.T. 1984. *Rock mechanics design in mining and tunnelling.* Balkema Rotterdam.

Bieniawski, Z.T. 1989. *Engineering rock mass classifications.* Wiley, New York: 251.

Chandler N.A., & C.D. Martin 1994. The influence of near surface faults on in situ stresses. *Proc. 1st North American Rock Mechanics Symposium,* Austin. Nelson P.P. & S.E. Laubach. (eds.) Balkema, Rotterdam: 369-376.

Dragados 1996. Central de Lam Ta Khong en Tailandia. *Boletín De Noticias, Número* 84-Octubre, 1996: 4-5.

Electricity Generating Authority of Thailand 1995. *Specification of tender document.* Lam Ta Khong Pumped Storage Project, Klong Pai 5, Nakhorn Ratchasima, Thailand.

Electric Power Development Co., Ltd. 1992. Rock mass classification and evaluation. *Engineering Geology,* Special Issue, Japan Society of Engineering Geology, 1992: 20-22.

Electric Power Development Co., Ltd.1996. Guide for geotechnical observation in underground excavation. *Addendum Document of Lam Ta Khong Pumped Storage Project,* (2): 1-9.

Gurung, N. & Y. Iwao 1998. Geotechnical observations and deformation monitoring in Lam Ta Khong Hydro power tunnel, Thailand. Kumamoto Conference, JSCE. III-144: 652-653.

Hoek, E. & E.T. Brown 1980. *Underground Excavations in Rock.* Inst. Min. Metall., London: 527.

Sakurai, S. 1998. Back analysis based on field measurements with particular reference to underground openings. *Proceedings of Int. Conf. on Geomechanics / Ground Control in Mining and Underground Construction,* Wollongong NSW Australia, (1): 3-12.

Yufu, Z. 1995. Principal conversion methods for rock mass classification systems used at home and abroad. *Bulletin of the Int'l Ass. of Engg. Geology,* Paris (51): 81-88.