1Ö^h International Mining Congress and Exhibition of Turkey-IMCET 2003. & 2003, ISBN 975-395-605-3

An Example and Comparison of Old and New Tunnel Support Estimation Methods

C. Ağan, A. Turabik & M. M. Güven
DSI General Directorate, Geoteclmical Services and Groundwater Department, Rock and Soil Mechanics Section, Ankara, Turkey

ABSTRACT: The stability of the rock structures in mining and geotechnic fields depends on the in-situ stress state and mechanical properties of the rock mass and the geometry of the structures. The stabilities and the chosen of equipments (support types, etc.) depend on the in-situ stresses. But, because of measuring difficulties and high costs, the in-situ stress measurements are very rare. They can be measure by technological methods like flat-jack, photoelasticity, hydraulic fracturing, thermal fracturing, acoustic emission, etc. But, because of high costs and difficulties of field works, they are not widely preferred by engineers.

So, lor recent years, engineers began to use computer aided technologies and designs. They can be 2 or 3 dimensioned and simulate in-situ stresses, deformations, displacements, etc. But, to increase the confidence ot this technology as possible as higher, the input data (E, c, 0, x, etc.) have to be taken with in-situ experiments and studies.

But, because of high programs costs and high experience requirements, directs the engineers to simple method which is consisted of laboratory tests, old methods (Terzaghi, RMR, Q, etc) and charts in foundation design, tunnel support design, etc.

In this report, the problems and mutabilities of an injection tunnel caused by old methods will be examined. And then, the tunnel direction and support load will be designed again with computer aided simulation program to emphasize that old systems (without in-situ data) lead into errors and they must not to be prefened by engineers.

1 INTRODUCTION

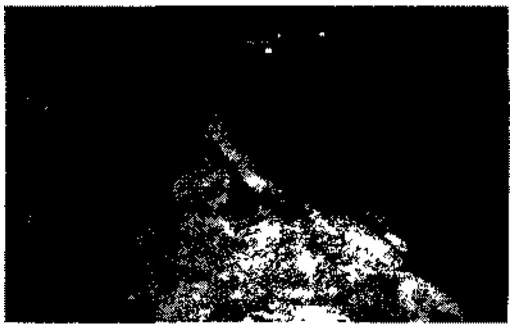
This report is dealing about the causes and repairs of the swellings and deformations at the left side injection and transportation tunnels of Çokal Dam at Çanakkale (Figure I). For this reason, some in-situ experiments, measurements and laboratory experiments have been done.

2 GEOLOGY OF STUDY FIELD

Çokal Dam field has a flysch formation with eocene aged. Formation has a gray-brown color, lateral sequenced and composed with sandstone, siltstone and claystone.

The thickness ot strata are changeable between 3-12 m and their declivities are IO°-20° SE.

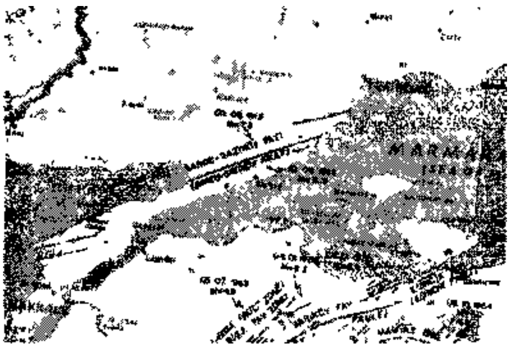
Sandstones are generally stable but the places where siltstones and claystones have majority, the alterations are too much. By the existence of fault



Fıguıe l. Defoimatioiv- al tunnel

zones, the alterations are increased. Sandstones are generally hard and cemented fresh surfaces that have brown-red color. Siltstones and claystones have generally thin strata and they are fragile.

The research field is in the I. grade earthquake region according to the Ministry Decision with number 96/8109 and date 18.04.1996. It is just 2 km away to an active tault (Figure 2). This tault is a component ot North Anatolian Fault zone and this zone is 60 km in length and 10 km in width Some destructives and liquefaction effects of this fault can be observed at a distance ot 200 km (Erdem and Guven 2000).



Fıguıc 2 Teuonic map of Mainnu ,i Region

Also some secondary fractures intersect with the left side tunnels (Tozak 1999). These secondary fractures altered the surrounding rocks. The leakage of surface and underground water also increases the alterations.

3 GENERAL SITUATION OF TUNNELS

The tunnels excavated in flysch formation. They are nearly 4,2 m in height and 4 m in width. They have horseshoe shape. Shotcrete (15 cm) +wire mesh+steel support (1-16) are used for suppoit system. Surrounding rock was classified as 4" class according to Terzaghi.

There is no deformation that can be observed at the right side tunnels. Only low amount of sweat and drips can be observed.

At the left side tunnels, by the effects of fault, the flysch formation locally became clay. Locally, big deformations and large amount of water flow can be observed.

4 THE STUDIES AT THE LEFT SIDE INJECTION AND TRANSPORTATION TUNNELS

4.1 Deformation studies

At fust, the deformed steel supports were cut and swelled parts were sciapped. Then the aperture between each steel support elements decreased from I m to 0,5 m and locally to 0,25 m especially at tault zone. The steel support elements tied to each other from the bottom. Then gravel material spread to the floor.

4.2 Deformation measurements

After the earthquake, which occurred at Marmara Sea on the dale of 28.02 2002, total station used for deformation measiuements at different sides and different places. After investigations ot these measurements, it was observed that the biggest deformations (50 cm) took place at the bottom side. The bottom of steel supports became closer to each other. It was also observed that the tunnels were buried within the time.

43 Drainage studies

By the leakage of water to the inner and surrounding parts ot tunnel, it caused to swelling. Because of the fault zone, the flysch formation became clay by the time. So that the number of drainage holes increased. Perforated pipes were set to the places where the leakages can be observed. It is aimed to take the water into the tunnel and then to the outer side of tunnel. By the measurements I lt/sec leakage was determined.

4.4 Studies for determination of the Yield zone

For determining the tunnel load, it is aimed to find the size of yield zone. Time was not enough for setting ol the extensometer or other systems. So piessuremetcr was chosen for this purpose. 4 locations were determined for cxpeiiments. 7 drilling holes were excavated which have 12-15 m in depth. Systematically pressuremeter experiments were done at these drilling holes.

Locations of pressuremeter experiments:

I- U-PRSK-1: It is diilled at transpoıtatıon tunnel at 135 km. It is drilled at tunnel roof and the surrounding rock is strong. By the experiments, 34-40 kg/cm" limit pressure and 550-1500 kg/cm" elastic module values were obtained.

II- U-PRSK-2: 4 holes were drilled with 45° angle at transportation tunnel. It was at 208 km and at fault zone. It was seen that approximately 3 m ot surrounding rock was yielded completely By the experiments, 3-9 kg/cm^2 limit pressure and 50-60 kg/cm" elastic module values obtained.

III- E-PRSK-3. It was drilled at the root ot injection tunnel at 270 km. 5-13 kg/cm" limit pressure and 65-130 kg/cm^2 elastic module values were obtained.

IV- E-PRSK-4: It was drilled at the bottom of injection tunnel at 280 km. The experiment results were similar to the E-PRSK-3.

It can be seen that big detoi mations occuued at the bottom of the tunnel (Figure 3) These icsults confirm the bumng ot tunnel and the lesults ot detoi mation measurements

hguie 1 Detu initiation ot yield zone hy PtessuiemclLi Test

4 5 Laboiaton it suit?

Foi deteımınatıon ol the kind ot clay undıaınaged samples weie taken toi expeiiments Because at lust seen the causes ot these detoi mations weie swelling ot clay The ıesults and evaluation ot expenment lesults *die* piesented in Table 1 and Table 2

Dunng the spillway excavation watei soi tie was seen Watei sample was sent to laboiatoty toi analyzes Within analyzes 4660 |.ıhos/cm electııcal conductivity and 2990 mg/L dissolved solid matenals aie found These ıesults mean that this watei soıted tıom depth levels

By the ıesults it is undei stood that the clay has a big swelling potential and the detoi mations caused by the watei come horn tault ciacks The eaithquake also ıncıeased the detoi mations

Table 1 Lıboıatoıy test ıesults

	Plastic Clay	Clay Gavel Sand
0 075(111111)	80-90	16-40
LL *C/r)*	42-44	32-33
PL(%)	21-22	15-17
PI(%)	21 23	14-18
Class	CLC1	GC
Gs(gi/cm)	27 28	27
yn (gi/cm')	2 1-2 3	2 2-2 3
Wn (%)	10 20	4-5
SI«)	80-100	26 56
Swelling Pi essuie (kg/cm)	1 0-2 26	-
Swelling %	4 5 12	-
UCS (kg/cm')	3 97	-

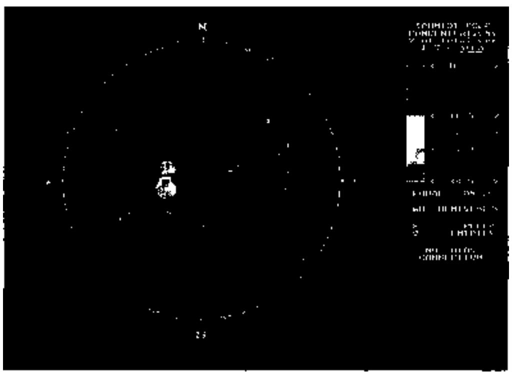
Tible 2 Evaluation ot I iboi not y ıesults

	Stable Pat ts	Detoi mated Paits
Liquid limit (LL)	28-42	42-44
Plastic index (PI)	10-23	20-23
Gain petcent undei #200 mesh (%)	79 CL	90-100 CI
Swelling (9f)	1 8-3 0	S 12
Swelling (kg/cm")	0 6 0 7	1 00-2 26
Wn(%)	8-16	12-21
SC/,)	67 75	7-S-100
Gs (gi/cm)	2 78-2 80	2 77-2 80
Kind ot clay	111 it	11 lit

S TUNNEL ROUTE

5 / ln\ i stıı >alıon\ about ninnc I toute

Both geological investigations and field observations it ıs obseıved that these tunnels cross a lot ot tault cracks For analyzing the effects ot these faults to the tunnel the dnections ot faults and tunnels are mai keel to the stereonet system in computer (Figuie 4) All ol the dip angles weie assumed as 30°



Fmute 4 Conioulis ot fault zones

As shown in Figure 4 injection tunnel (1) is affected by only tault F5 (2) The othei faults do not eltect the injection tunnel But the transportation tunnel (8) is eitected by faults F2 (4) F4 (5) estimated tault (6) and Ece fault (7) This result is similar to the detoimation studies results Because, the biggest deloi mations had occurred at transposition tunnel (Table 3)

Table 1. Faulk ni Figure 4

Contour No	
1	Injection Tunnel
2	F5 Fault
3	F3 Fault
4	F2 Fault
5	F4 Fault
6	Estimated Fault
7	Ece Fault
8	Transportation Tunnel

5.2 Investigations about tunnel load, stability and deformations

Before the tunnel was excavated, the rock is determined as 4[U] class rock according to the Terzaghi. Referencing this system, rock load was determined as 0,7-1,0 kg/cm" and supports are selected according to this load. After these deformations, the investigations began.

Two different paths are fallowed. At first path, the pressuremeter experiment results are used. According to these results, 3 m of surrounding rock yielded. By calculations rock load was calculated as 6-7 kg/cm". And also, the swelling pressure found in laboratory was approximately 1,5-2 kg/cm^2.

At the second path, computer program was used to find the rock load. The inputs are given in Table 4.

Table 4 Inputs, of program

Input Data	Data Values
Excavation limits	4 m width, 4 m height, horse nail section
Over burden height	Max 60 m
Material	Alterated flysch
Young module	1000 kg/cm^2
Poisson ratio	0,33
Specific weight	2,7 ton/m*
Cohesion	2 kg/cm"
ф	30°
Characteristic-	Plastic

As shown in Figure 5, the maximum stress value is approximately 5-6 kg/cm" and it is applied to roof and the corners of bottom. During the observations at tunnel, it is seen that the maximum deformations occurred at the roof and bottom, especially at the corners. This is the evidence that, the computer analyzes are also true.

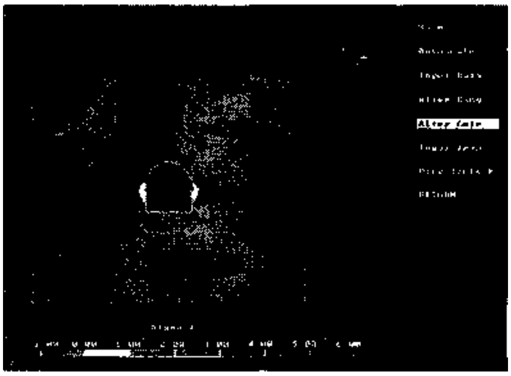


Figure S. Vertical stress on tunnel section

6 CONCLUSIONS

The multilateral investigations, which are held in Çokal Dam, demonstrate that;

- Drainage studies had been done correctly
- The primitive methods (Terzaghi, Q, etc.) must not be used alone for supporting and route selection. They can be changeable from one person to another and deceptive. In today technology, these methods must be only used for controlling and confirming the studies. Because, the primitive methods are based on experiences. But, it is true that the underground is always changeable and these methods can be insufficient. For this reason, during these calculations rock and soil mechanics principles must be taken into consideration and must not be avoided from in-situ experiments for obtaining the requiring parameters. Primitive methods do not have validity at today technology.
- All of the discontinuities and faults directions must be determined before selecting the route of tunnels.
- If the working field is in the fault zone, NATM methods and circular section must be chosen. The invert concrete must not be neglected.

REFERENCES

Ağan. C , Turabik. A.. 2002. Çokal Barap Sol Sahil Ulaşım ve Enjeksiyon Galenleri Kaya Mekaniği Etüt Raporu. *Enerji ve Tabii Kaynaklar Bakanlığı, Devlet Su işlen Genel Mıdıırlıığu. Jeolekıık Hi:metler ve Yeraltısuları Daireli Başkanlığı. Kava ve Zemin Mekaniği Şube Mnıhırlitğıı, Rapor* No:257

Bagualın. F.. Jezequel. F.J.. Shields. H.. 1978. Pressuremeter and Foundation Engineering. *Trims. Tech. Publication.* Clausthal. Germany

Bieniawski. Z.T., 1984. *Rack Mechanic': Delikli in Mining anil Tiiiinelliiif;.* A.A. Balkema Publisher

Canadian Manual on Foundation Engineiing I97S Canada Pi intmg and Publishing Ottowa Canada

Erdem IU Güven M M 2000 Çanakkale Çokal Baran Temel Zemini Sıvılaşma Analizi Rapoıu *Etuıjı* u *Tabu Kımıaklaı Bakanlığı De\lcl Su İşlen Genci Mıdıuı lıığıı Jeotekıık Hızmetleı \t Yeıcıllısılaıı Dairesi Başkanlığı Ka\a* t *e Zemin Mekaniği Şube Mıdıu lıığıı* Rapoı No 222

Erdoğan II Ba\kal I Frdem IU 1988 Şanlıurla-Tugav komutanlığı Yerleşim Sahası Zemin Etüt Rapoıu *Ba\ ınılılık u. likan Bakanlığı De\let Su İşlen Genel Mıdıılıığıı Jeotekıık Hı:ııııtleı u Yeıullısılaıı Daıııcsı Başkanlığı Kastı \ı Zemin Mekaniği Şube Mıdıu lıığıı* Rapoı No I 10

Hoek E Bıay JW IW Kaya Şev Stabılıtesı *TMMOB Jeoloji Mühendisten Odan Umulan i* Baskı

Obeıt L Duvall W I 1967 Rock Mechanics and The Design ot Stıuctuıes in Rock John Wiley and .Sons Inc

The Menard Pressuremetei Inteipietation and Application ot Pıessuıemeıer Test Results to Foundation Design *l9Ts Sah Sails* No 26 Pans

Tozak A Z 2000 Çanakkale-Gelıholıı-Gokbuet Pıojesı Çokal ßarap Dolusavak Şeşleri leoteknık Rapoıu *Ener/ı ie Tabu KıiMiakhıı Bakanlığı Detlef Su İşlen Genel Mıdıu lıığıı D.SI* HI *Bilice Mıdıu lıığıı Jeotekıık Hizmetler \e)eıallnulaıı Daıılsı Başkanlığı* Balıkesir

Ulusay R 2001 Uygulamalı Jeotekıık Bılgıleı *TMMOB Jtoloıı MıılıenıliKİeıı Odası Yavınlaıı* 38 4 Baskı