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Non-deformable support system application at tunnel-34 of Ankara-Istanbul high speed railway project

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Abstract. Non-Deformable Support System (NDSS) is one of the support system analysis methods. It is likely seen as numerical analysis. Obviously, numerical modeling is the key tool for this system but not unique. Although the name of the system makes you feel that there is no deformation on the support system, it is not true. The system contains some deformation but in certain tolerance determined by the numerical analyses. The important question is what is the deformation tolerance? Zero deformation in the excavation environment is not the case, actually. However, deformation occurred after supporting is important. This deformation amount will determine the performance of the applied support. NDSS is a stronghold analysis method applied in full to make this work. While doing this, NDSS uses the properties of rock mass and material, various rock mass failure criteria, various material models, different excavation geometries, like other methods. The thing that differ NDSS method from the others is that NDSS makes analysis using the time dependent deformation properties of rock mass and engineering judgement. During the evaluation process, NDSS gives the permission of questioning the field observations, measurements and time-dependent support performance. These transactions are carried out with 3-dimensional numeric modeling analysis. The goal of NDSS is to design a support system which does not allow greater deformation of the support system than that calculated by numerical modeling. In this paper, NDSS applied to the problems of Tunnel 34 of the same Project (excavated with NATM method, has a length of 2218 meters), which is driven in graphite schist, was illustrated. Results of the system analysis and insitu measurements successfully coincide with each other.

Keywords: non-deformable support system; weak rock mass; tunneling; tunnel support; support analysis

1. Introduction

Support design is one of the important works in tunneling. Support design should include a solution consisting of three dimensional complex components. Optimum solution can only be obtained by evaluation of data with those 3D complex components (Feng and Hudson 2008). Tunneling is 3D operation. Shear force, axial force and bending moment, exposed to support, give

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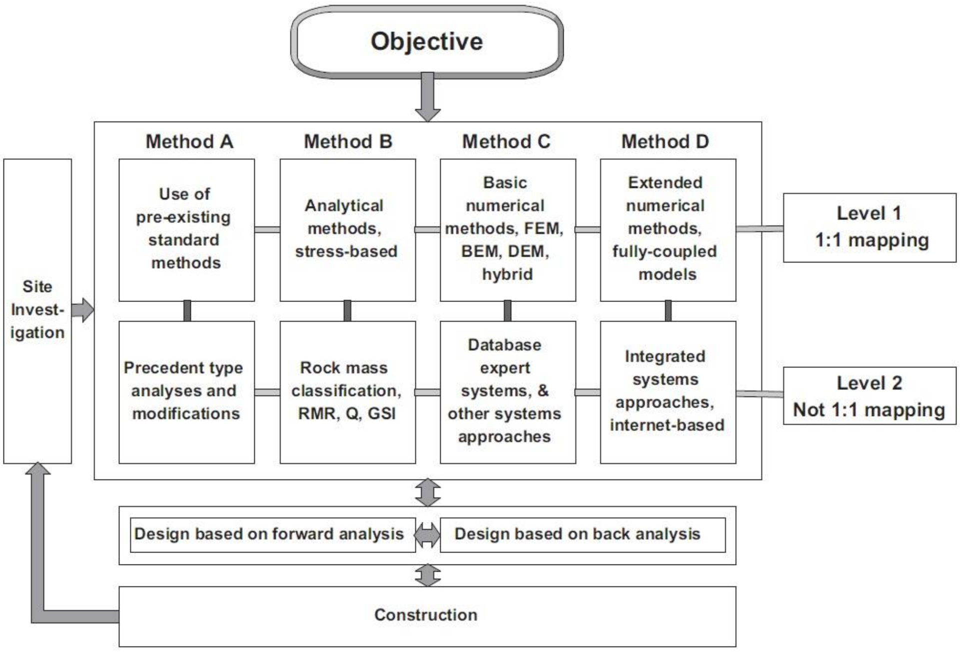


Fig. 1 Flowchart of rock mechanics modeling types used to support rock engineering design (Feng and Hudson 2008)

important information on support performance. Tunnel face motion, stresses in front of the tunnel face and measures taken to those effects can only be determined by 3D analysis (Aksoy and Onargan 2010, Aksoy *et al*. 2012a). 2D design of that complex structure brings lots of problem. 3D empirical or semi-empirical solutions, however, take long time. The easiest way of 3D solutions is numerical modeling, nowadays. There are many methods in numerical modeling. Most popular ones are Finite Element, Finite Difference and Boundary Elements. Feng and Hudson (2008) illustrate the principals of numerical modeling in Fig. 1, which is very easy method and it incorporates long observations.

There are different methods in support design. The most popular ones are Convergence-Confinement Method (Einstein and Branco 1991, Carranza-Torres and Fairhurst 2000, Panet and Guenot 1982, Curran *et al.* 2003, Oreste and Oggeri 2012), Analysis of the behavior of a yield-control support system (Barla *et al.* 2011, Cantieni and Anagnostou 2009), probabilistic analysis (Chen *et al.* 2007), analysis of interaction between tunnel support and ground convergence (Shin *et al.* 2011), semi-empirical support analysis (Singh *et al.* 1995) and they prove themselves in many important projects.

The NDSS is a 3D method serving the same purpose with the other methods but, contrary to the others, it also takes account many complex parameters in support design. NDSS is primarily composed of nine stages as it is stated in Fig. 2.

The first stage, same in the other methods, is defining the rock material and the rock mass (Aksoy *et al.* 2010, Aksoy *et al.* 2012c). The second stage is the phase which separates the method from the others. At this stage, the time dependent behavior of rock deformation and time-dependent deformation behavior under different loads are defined. In the third step, rock mass

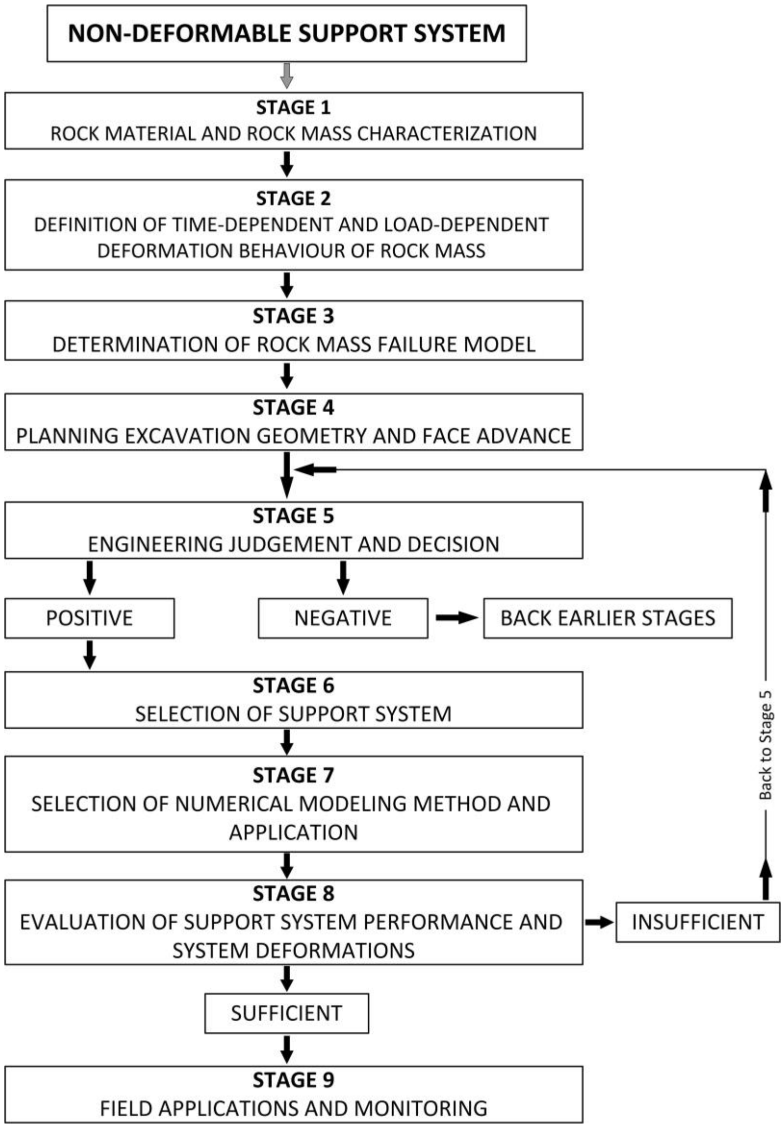


Fig. 2 Stages of NDSS

failure criteria is determined based on the data obtained. At this stage, the designers either use the developed models like Hoek-Brown, Mohr-Coulomb, Jointed Rock Mass etc., or use a newly derived model by using the first and second stages. This newly derived model is introduced to the numerical model in the 7th stage. In the fourth stage, face advance and excavation geometry are defined according to previously set rock mass behavior. In the fifth stage, prior to the numerical modeling, the system is evaluated with engineering judgement. If the designer is not satisfied the results in this stage, the concerned stage should be revised. If a problem does not appear, a support system based on experience and calculations are determined.

In numerical modeling, all kinds of support elements can be inserted into the system and can be integrated into the model. Sometimes the cement injection or chemical injection can also be integrated into the model by making the appropriate field experiments (Aksoy 2008, Kucuk *et al*. 2009). If this stage, 6th stage, can be passed without any problem, one can go through the 7th stage which is the selection of numerical modeling technique. In this stage, Finite Element Method, Finite Different Method, Boundary Element etc. are considered and the one that will give the most accurate results is selected.

In the 8th stage, the results of analysis obtained from the definition of previously obtained rock mass deformation behavior, support system, excavation geometry are evaluated and the performance of the support system is discussed. If the results are not satisfactory, it will return to the 5th stage. If the results are satisfactory, field application and monitoring stage will start as the final stage. If necessary, some revisions can be made in the support system in this stage. With this system, there is also the possibility of questioning the environmental impact of the tunnel beside the support performance. The environmental impact is one of the factors in the success of the tunnel projects (Hasanpour *et al*. 2012, 2015, Miranda *et al*. 2015, Aksoy *et al.* 2006).

The most important difference of NDSS from the other methods is that NDSS may contain time dependent behavior of rocks. The first application of this support design system was performed by Aksoy and Onargan (2010) in İzmir 2nd stage metro tunnel construction. Afterwards, Aksoy *et al*. (2009) applied the same system to İzmir 3th stage metro tunnel construction. This method was applied to Ankara-İstanbul High Speed Railway Project Tunnel-13 and Tunnel-35 construction (Aksoy *et al*. 2012a, Aksoy *et al*. 2014). Aksoy *et al*. (2012b) were also used same method at the main drift of Omerler underground coal mine. The system gives satisfactory results in all tunnel applications. The performances of support system are evaluated with in situ measurements.

This paper is based on the application of this system to construction of Tunnel-34 of Ankara-İstanbul High Speed Railway Project. It was determined that the NDSS design was successful when looking at in situ measurements.

2. Project characteristics

The purpose of the Ankara-Istanbul High-Speed Railway Project is to reduce the travelling-time between the biggest two cities, Ankara and Istanbul, and increasing the percentage of railway in transportation by creating a fast, comfort and safe transportation facility. The old railway route between Ankara and Istanbul is 576 km and all of them are electrified and signalized. When the Ankara-Istanbul High-Speed Railway Project finishes, a railroad, which is electrified, signalized, two-line, will be constructed. Ankara-Istanbul will become around 533 km and total travel time will be 3 hours which is now 7 hours. Köseköy-Inönü is the second stage of the project and has 150 km length which is designed as section-1; 95 km (Köseköy-Vezirhan), section-2; 55 km (Vezirhan-Inönü). The engineering structures of the project are given in Table 1.

Table 1 Engineering structure of Ankara-İstanbul High Speed Train Railway construction second section

	Section 1	Section 2	Total
Length	95 km	55 km	150 km
Viaduct	18 (6.120 m)	13 (6.582 m)	31 (12.702 m)
Tunnel	13 (25.700 m)	19 (27.210 m)	32 (52.910 m)
Open-Cut Tunnel	-	1 (1.090 m)	1 (1.090 m)

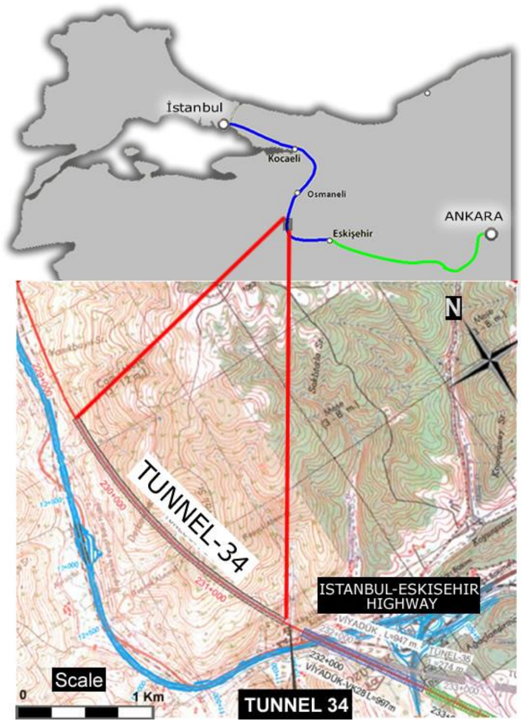


Fig. 3 Location of the Tunnel 34

The Tunnel 34 has been excavated in the second stage of the Ankara-İstanbul High Speed Railway Project. The main geological unit is graphite schist. For the excavation and support of the Tunnel 34, the primary support system has been proposed. There has been encountered some deformation problems on the primary support system such as more than predicted deformations. Earlier support systems have been revised again for the solution of the encountered problem. This paper is about the solution of this deformation problems and prediction of new support system performance.

3. Geological characteristics of the tunnel route

The Tunnel 34 which has been investigated in detail is located between Bozüyük-Bilecik and one kilometer northwest of Bozüyük (Fig. 3). The tunnel passes from Derbentbaşı ridge and Ayvalı ridge positions. Along the route of the tunnel, the covering thickness is variable and it changes between 5 and 100 meters.

Along the route of the tunnel, Pazarcık melange is observed which is named by Koçyiğit *et al.* (1991). The unit is between Bilecik and Bozüyük and it is represented by many different types of rock offering imbricate structure. In the tunnel location, the base of the unit is not seen and this unit offers a relationship of erosion and in some places faulted with Trias old Karakaya Group at the top and contact with erosion relationship with Bayırköy Formation. The unit consists of rocks with different thickness passing metamorphism in the green schist facies conditions and come one on the top of the other structurally. As common, there are psammites, marbles, migmatite-gnays,

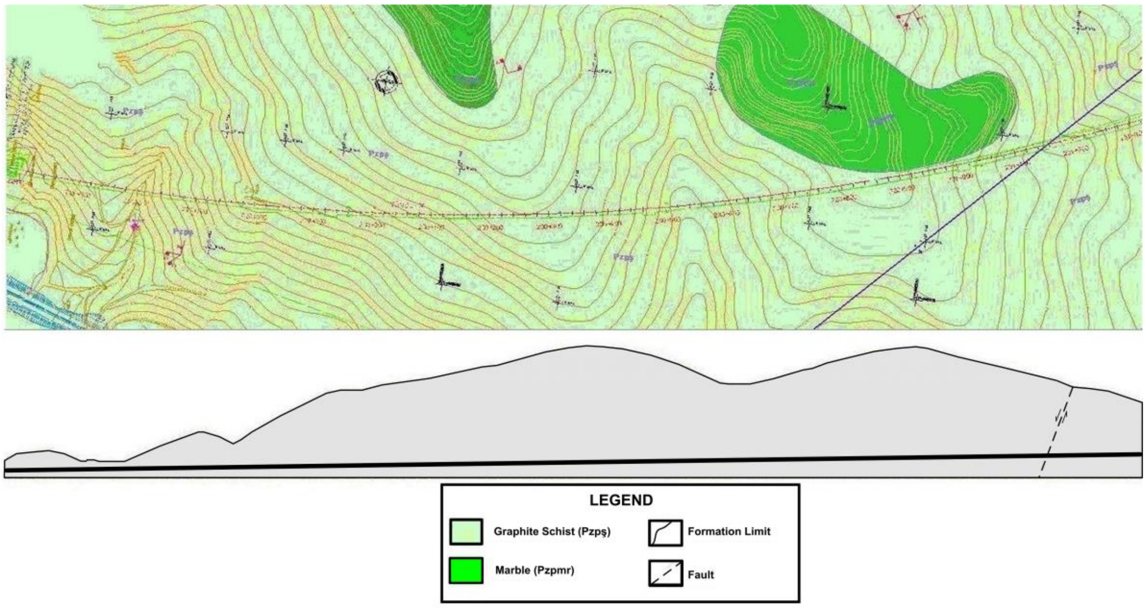


Fig. 4 The geological map and geological cross-section of Tunnel 34 (re-design from Şimşek *et al*. 2009)

Table 2 Geotechnical parameters of the Tunnel 34 (Şimşek *et al.* 2009)

Uniaxial Compressive Strength *UCS* (MPa)	Geological Strength Index *GSI*	Hoek-Brown Material Constant *mi*	Disturbance Factor *D*	Elasticity Modulus of Intact Rock E_i (MPa)	Natural Unit Weight (kN/m^3)	Cohesion *c* (kPa)	Internal Friction Angle (°)	Deformation Modulus of Rock Mass E_m (MPa)
19	20	10	0.5	4250	25	200	32	194

granodiorite, mega-blocks into the outcropping schists. The unit is cut by quartz and aplitic dykes of Bozüyük Granitoid.

Along the route of the Tunnel 34, encountered unit is graphite schists (Pzpş) (Jemas 2009, Şimşek *et al*. 2009). Graphite schists are black-dark gray- greenish dark gray, clear schistosity, fragmented, medium-very secluded, weak-medium strength. Along the schistosity planes, there is encountered with a couple of marble blocks of 10 m and quartz veins which are up to the thickness of 2 m into the graphite schists which can be separated easily (Fig. 4). The properties of intact rock material and rock mass were given in Table 2.

4. Design of new support system by NDSS

Excessive deformation formed at Tunnel 34 which excavated in graphitic schists. High convergence is normal due to the schistosity and squeezing properties of formation. It is needed to take in the account Barla's (1995) comments of time dependent deformation behavior of tunnel and the analysis of support should be 3D and time dependent.

4.1 Earlier support system and problem encountered

New Austrian Tunneling Method has been used in the Tunnel 34. Excavation phase is composed of 3 main parts. The total height of the tunnel is 11.85 meters, width is 13.75 meters. The total excavation area is 136 m^2 (Fig. 5).

Earlier support system elements applied for supporting Tunnel 34 in graphite schist are given in Table 3. Extreme deformation problems were observed while excavating Tunnel 34 with these support elements.

During the excavating of both inlet and outlet side of the tunnel, measurements have been taken by opto-trigonometric method and the tape extensometer with different periods. As a result of the measurements, displacements have been measured above 80 mm tolerance, which was determined by earlier designer, limit in the location of the inlet side KM:229+365.50 and in the direction of the outlet side KM: 230+800.50 (Figs. 6 and 7). Depending on these movements, there have been occurred deep cracks on shotcrete surfaces (Fig. 8). The measurement periods have been increased at these points and the tunnel closure has been determined. Movements in the direction of the outlet at all points, the deformation have exceeded the excavation tolerances according to earlier support system. By acting together of the top half of the tunnel settled towards the bottom. Since

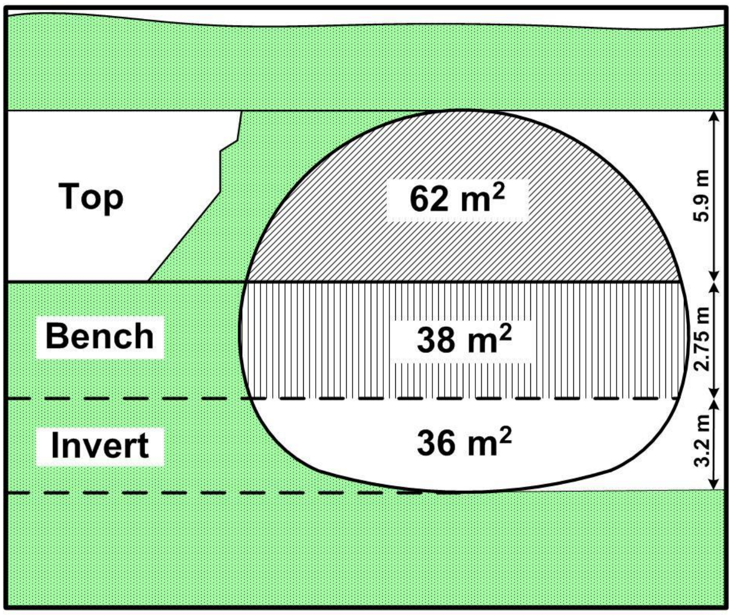
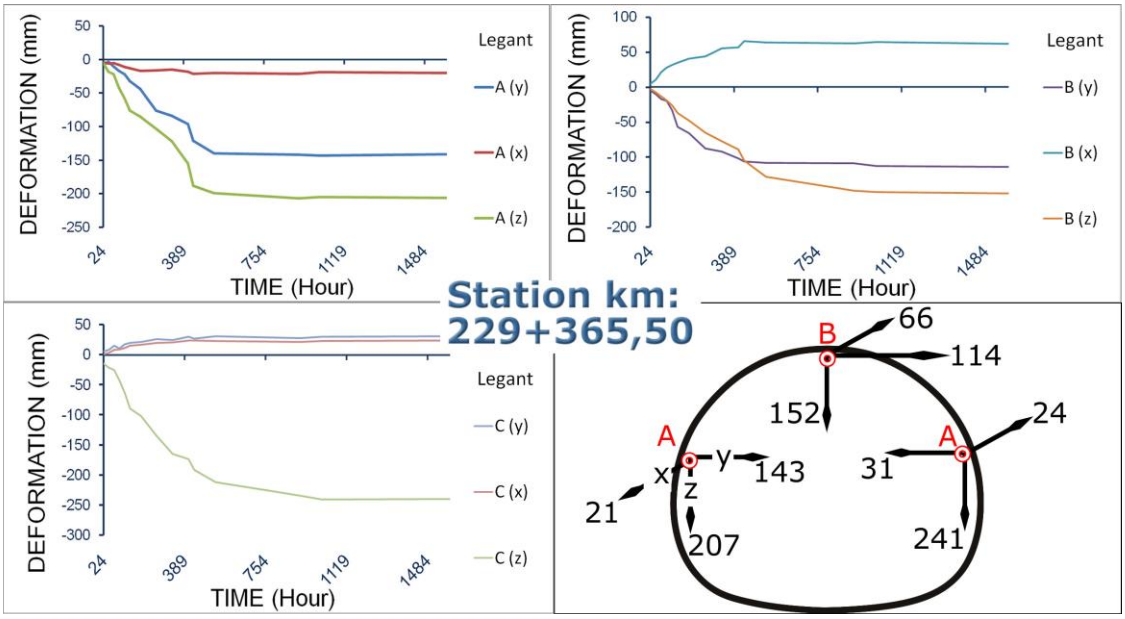


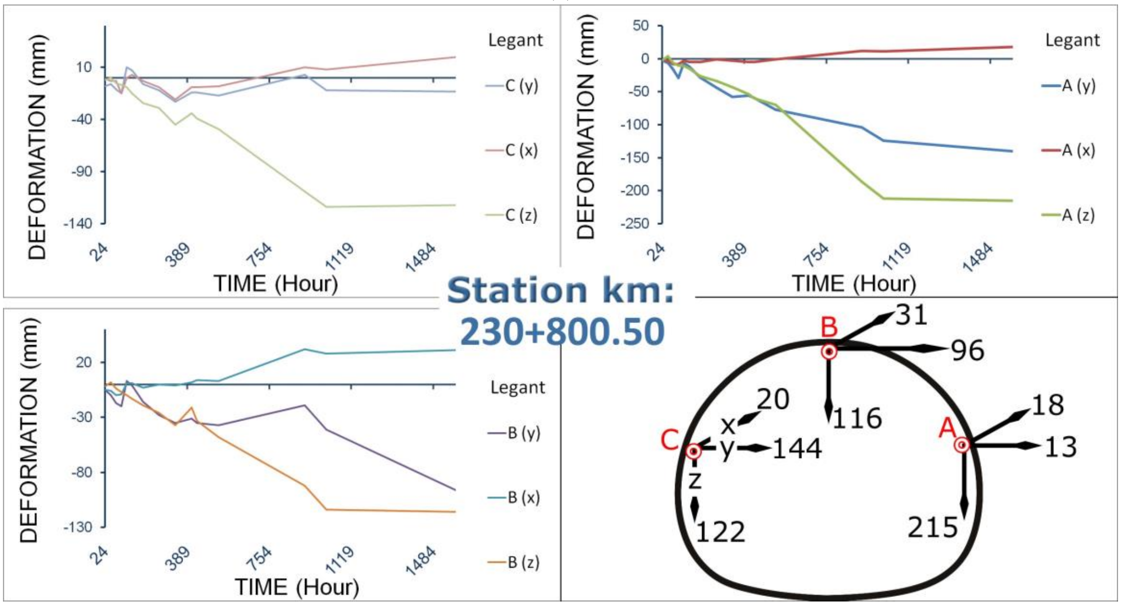
Fig. 5 Excavation geometry of Tunnel 34 (Şimşek *et al.* 2009)

Table 3 Earlier support system elements (Şimşek *et al.* 2009)

Supporting Components	Characteristics
Steel Support	I 160 Profile
Shotcrete	25 cm (C20)
Bolt	SN Type Φ28 (l=4 m and 1×1 m)
Fore poling	1,5” ongoing pipe without injection (axis 30-40 cm distance)
Face Advance	1-1,50 m



(a)



(b)

Fig. 6 Opto-trigonometrik deformation measurements ((a) KM:229+365.50, (b) KM:230+800.50)

amount of deformation is greater than expected, re-shaping should be applied in tunnel. Re-shaping, as known, is very expensive and time consuming operation.

5, 10 and 15 meters arm length with 2 pieces rod extensometer has been applied right and left shoulder regions of the tunnel and measurements have been taken periodically in order to determine the tunnel plastic zone (Fig. 9).

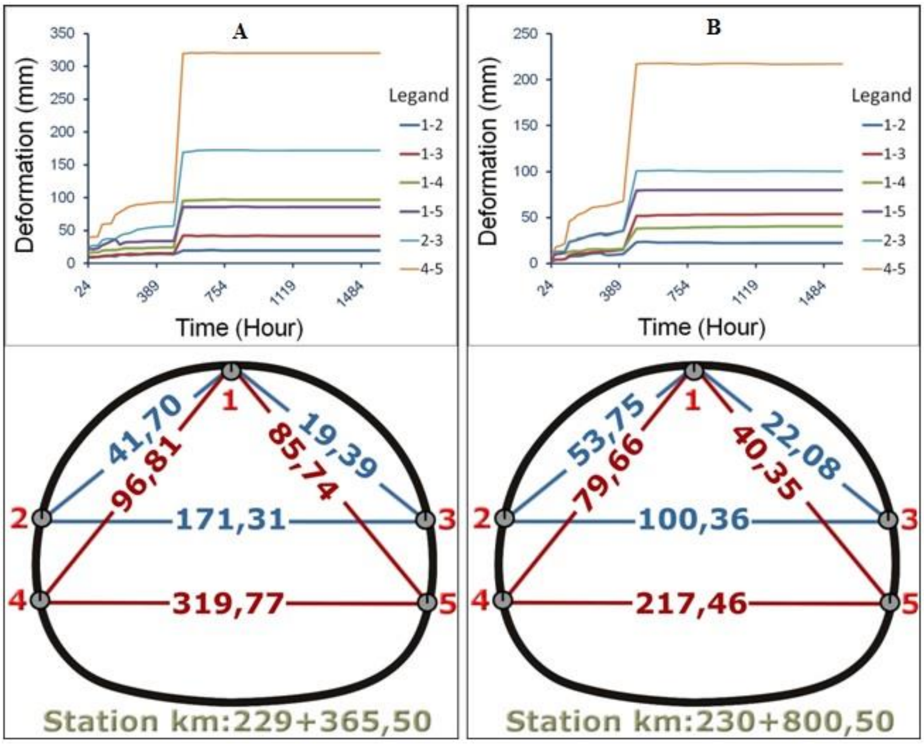


Fig. 7 Tape ekstensometer deformation measurements to determine horizontal and vertical closure of tunnel sections ((A) KM:229+365.50; (B) KM:230+800.50)

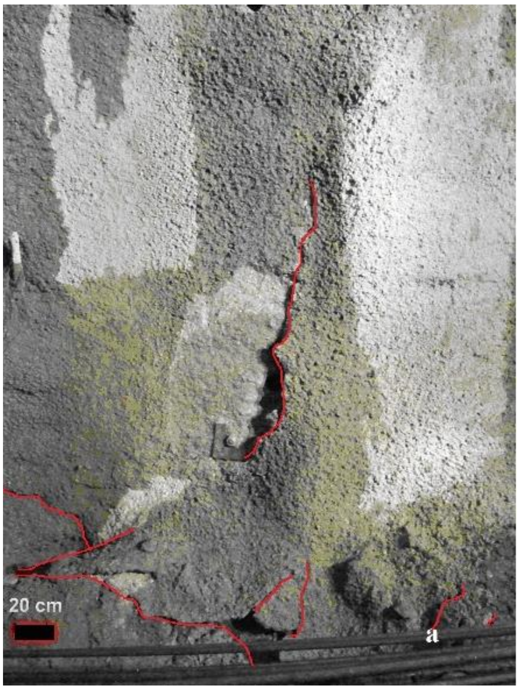


Fig. 8 The cracks of the shotcrete due to the high deformation

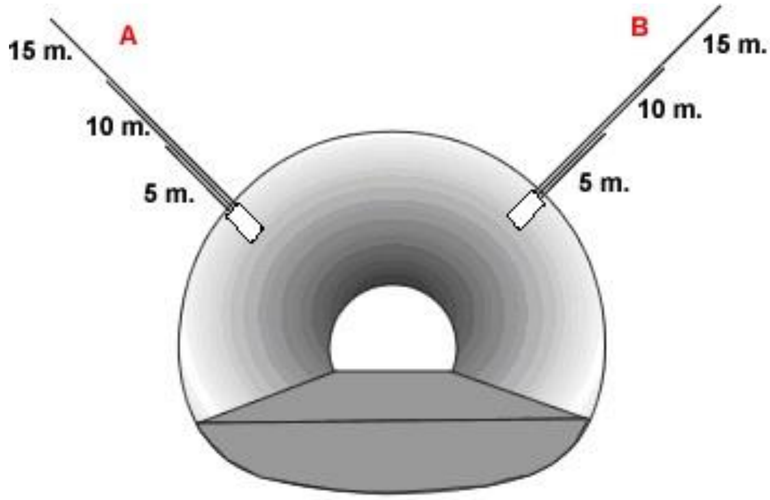


Fig. 9 The schematic views of the rod-extensometer application at Tunnel 34

It is understood from the displacement data of cumulative +11,5 mm in the left side and +6,4 mm in the right side that, 5 m long rod of classical extensometer located in the shoulder cut of the tunnel has moved 46.5 days with the tunnel plastic zone. In contrast, 10 and 15 m length rods are within the elastic zone and protect thier places due to tunnel convergence. Because, there are -15.3 mm displacement in the 10 m long rod and -17.8 mm in the 15 m long rod in the left part; there are -3.2 mm displacement in the 10m long rod and -11 mm in the 15 m long rod in the right part. It is understood that, when the readings on the extensometer, +, the rods are moving with the tunnel; when the readings on the rods, -, the rods remain constant.

Different movement values in the arms of the rod extensometer were detected while tunnel in advance. Especially, a positive value, showing that 5 m arms were moved with the rock, can be observed. After completing the tunnel excavation steps, the movement values measured from the arms were fixed. In light of these data, it is concluded that plastic zone in the tunnel is more than 5 m and less than 10 m (Fig. 10).

4.2 Design of new support system by NDSS

To solve the extreme deformation problem in Tunnel 34, in-situ measurements and analyses, 3D simple numerical models, rock-support interaction analyses were applied and as a result of those works, it is concluded that more rigid support system should be performed in Tunnel 34. Since rock-support interaction analysis and simple numerical models are guiding works in preparing final support system, they are not explained in this paper. NDSS is based primarily on 3D numerical model and also it evaluates time dependent deformations on rock mass and tunnel. As a result, the behavior of excavated area is intended to be in specified deformation tolerances. When the time dependent deformation behavior of the tunnel is investigated (Figs. 7, 9 and 10), it is seen that deformation increase in tunnel walls end within 390 hours. However, rod-extensometer measurements show that movements of the rock continue 1656 hours. For this reason, time dependent 3D numerical modeling works were planned for 2016 (first 15 days included for initial conditions) hours. In this process, excavation and support stages in the tunnel were integrated into

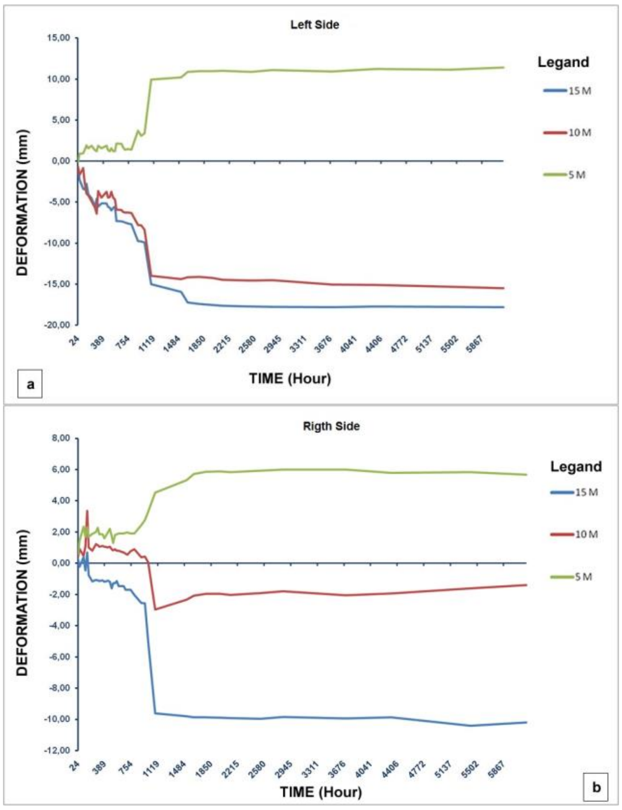


Fig. 10 The graph of rod-extensometer measurements taken from KM: 229+400.20 ((a) Left Side rod-extensometer, (b) Right Side rod-extensometer)

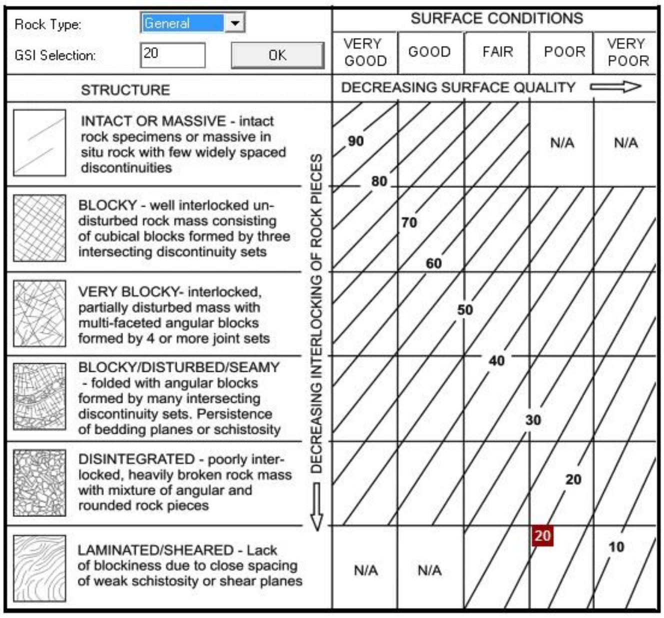


Fig. 11 GSI value of rock mass located Tunnel 34

the model in real time. Thinking of 2 m daily advance rate, 138 m tunnel excavation and support for 69 day (without first 15 days for the model initial condition) were integrated into the model.

The first step of the revision work is to redefine rock mass parameters again to be used in NDSS. New geotechnical parameters for rock mass have been determined by RockLab.

Taken into account of the Marinos and Hoek (2000) criterias of GSI value of the rock mass unit, it has been accepted as a formation with curved and very often under the influence of high tectonism with schistosity and lithologic structure rich in terms of clay and GSI value has been taken as 20 (Fig. 11).

As a result of the preliminary research (in-situ measurement, rock support interaction analyses etc.), a new support system was proposed by the project contractor (Table 4) and this support system was integrated into the numerical model. Excavation and support stages and time period applied in the numerical model are given in Table 5.

Analysis to determine the performances of the new support system was done by applying above mentioned parameters. PLAXIS 3D Tunnel V2 software is used in this analysis (Plaxis 2008). Vertical in situ stress ($\sigma_{v,0}$) applied to the model boundaries was assumed to be equal to

Table 4 Newly designed support system elements and characteristics

Supporting Components	Characteristics
Steel Support	I 200 Profile
Shotcrete	30 cm (C20)
Bolt	IBO Type (L=8 m and 1,20×1,20 m)
Temporary invert	30 cm
Face Support (as Shotcrete)	5 cm
Face Advance	1 m

Table 5 Stage Construction of Tunnel 34 used in 3 dimensional time dependent numerical modeling

Construction Stage	Application	Time (Total time)
1	30 m top excavation-support, 20 m bench excavation-support and 15 m invert excavation-support	360 h (360 h)
2	1 m top excavation	2 h (362 h)
3	1 m top support	2 h (364 h)
4	1 m top excavation	2 h (366 h)
5	1 m top support	2 h (368 h)
6	1 m bench excavation	2 h (370 h)
7	1 m bench support	2 h (372 h)
8	1 m bench excavation	2 h (374 h)
9	1 m bench support	2 h (376 h)
10	1 m invert excavation	2 h (378h)
11	1 m invert support	2 h (380 h)
12	1 m invert excavation	2 h (382 h)
13	1 m invert support	2 h (384 h)
14	Repeating the steps 2-13 for 69 days	Σ= 2016 h

overburden stress. The initial horizontal stress ($\sigma_{h,0}$) is related to the initial vertical stress by the coefficient of lateral earth pressure (k), ($\sigma_{h,0}=k*\sigma_{v,0}$). As boundary condition, model size was defined three times longer than the tunnel's size to prevent tunnels from the boundary condition. Ground water condition was integrated to numerical model. The Soft Soil Creep Model (known as Modified Cam Clay Model) was used to numerical modeling. The failure criterions of model were same as Mohr-Coulomb Model's (cohesion, friction Angle and dilatancy angle). The basic stiffness parameters are Modified Swelling Index (κ^*), Modified Compression Index (λ^*) and Modified Creep Index (μ^*).

These parameters are given in Eqs. (1)-(3)

$$\lambda^* = \frac{C_c}{2.3(1+e)} \tag{1}$$

$$\kappa^* \approx \frac{2.3C_r}{2.3(1+e)} \tag{2}$$

$$\mu^* = \frac{C_\alpha}{2.3(1+e)} \tag{3}$$

Also, the Poisson of this type of analysis is an elastic constant not a pseudo-elasticity constant as used in Mohr-Coulomb.

The properties of rockmass and structural elements are give in Tables 6 and 7.

Table 6 Properties of rock mass for the numerical modeling

Rock Properties	Value
Unsaturated unit weight, γ_{unsat}, kN/m^3	25.10
Saturated unit weight, γ_{sat}, kN/m^3	22.81
Cohesion (C), kN/m^2	119.00
Internal Friction Angle (ϕ),^{0}	25.10
Modified swelling index, κ^*	0.0006375
Modified compression index, λ^*	0.006863
Modified creep index, μ^* (*)	0.00036
Poisson ratio for unloading-reloading, ϑ_{ur}	1.5
K_0^{NC}, $\frac{\sigma_{xx}'}{\sigma_{yy}'}$ stress ratio in a state of normal consolidation	0.611
M, K_0^{NC}-related parameter (**)	2.74

*λ^*/μ^* is in the range between 15 to 25

$$**M = 3x\sqrt{\frac{(1+K_0^{NC})^2}{(1+2K_0^{NC})^2} + \frac{(1-K_0^{NC})(1-2\vartheta_{ur})(\lambda^*/\kappa^*-1)}{(1+2K_0^{NC})(1-2\vartheta_{ur})\lambda^*/\kappa^* - (1-K_0^{NC})(1+\vartheta_{ur})}}$$

Table 7 Properties of structural elements

Structural Elements	EA (kN/m)	EI (kNm^2/m)	w (kN/m^2)	ϑ (-)
Shotcrete and top heading temporary invert	1.328x10^7	177066	9.6	0.15
Bolts	5.67 x10^4	-	-	-

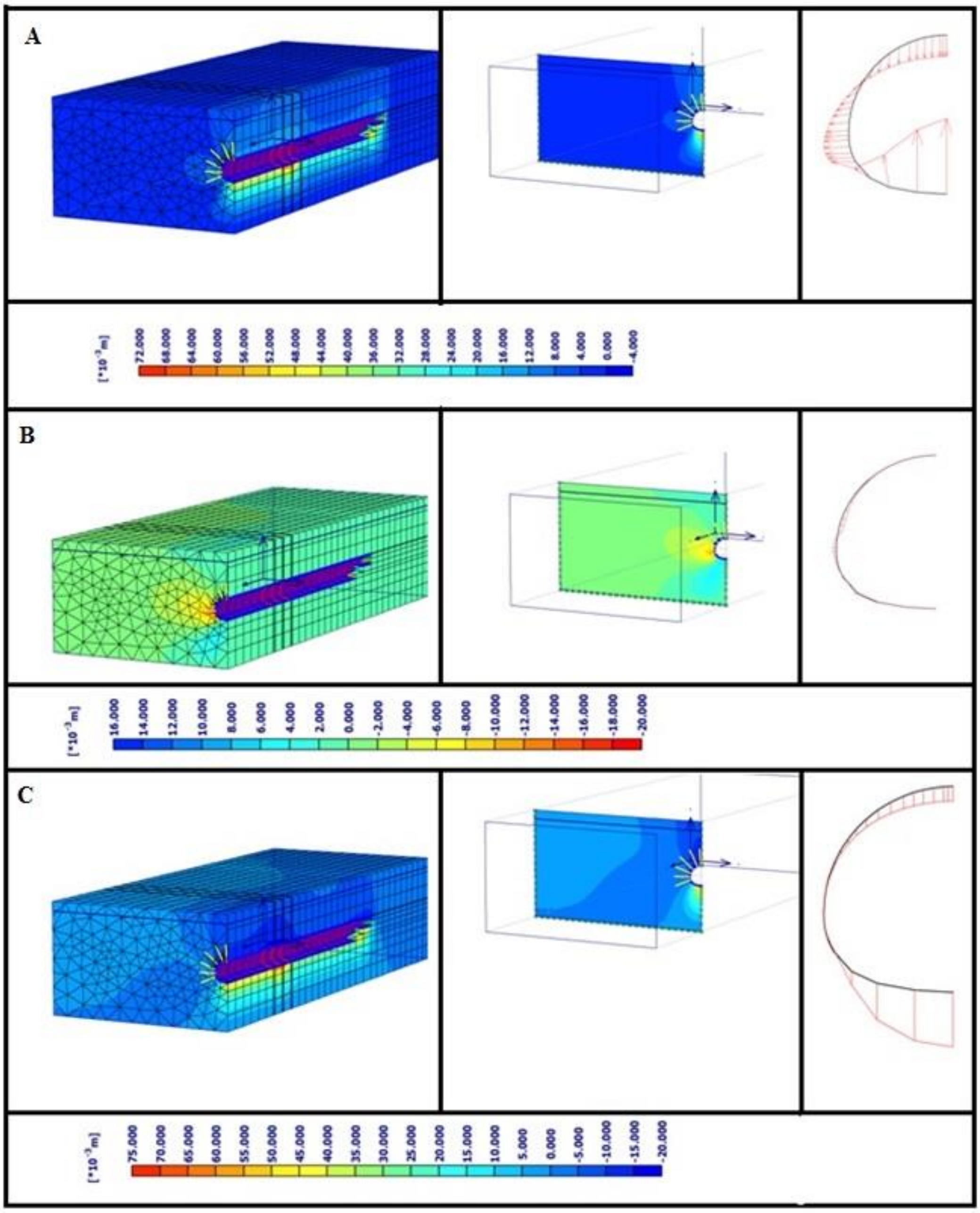


Fig. 12 Results of the time dependent numerical analysis for NDSS ((A) Total deformation (in mm), (B) Axial deformation in (mm), (C) Vertical deformation (in mm))

5. Results and discussions

The results obtained from 3D numerical model for determining the NDSS are given in Fig. 12. According to the results, 64 mm total deformation, 18 mm axial deformation and 62 mm vertical

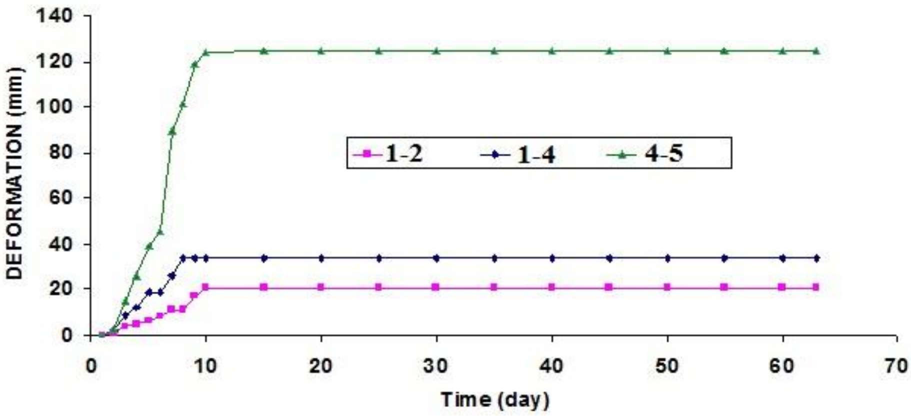


Fig. 13 Predicted convergence amount on the measurement points on Tunnel 34

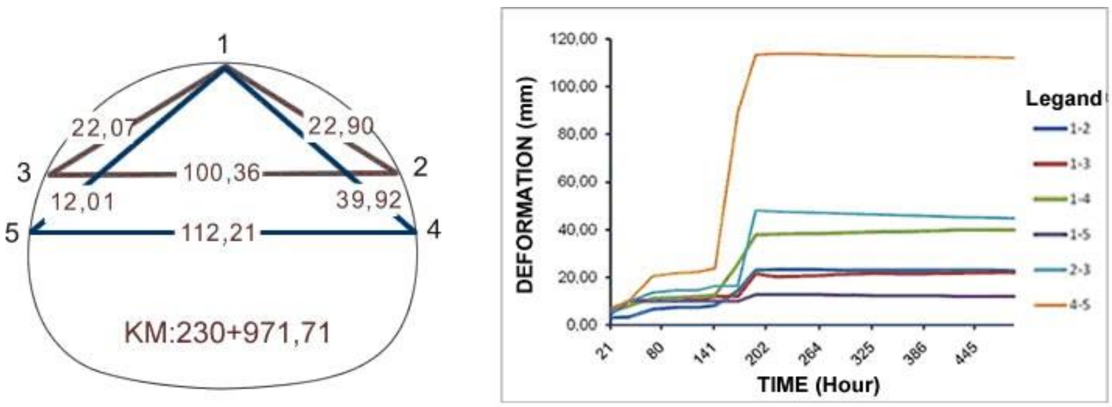


Fig. 14 Results of the tape extensometer measurements after NDSS application at KM: 230+971.71 (Left side-Cross-section of tunnel, Right side-Time-dependent measurements between points)

deformation in the tunnel are expected (Fig. 12 and 13).

Axial forces, shear forces and bending moment affecting to tunnel support are predicted as 346.25 kN/m, 61.13 kN/m and 178.49 kN/m, respectively. It is thought that this situation is enough for tunnel stability and deformation amounts expected in convergence measurement locations are determined in the model. The convergence amounts determined by time dependent numerical model based on 84 day period are given in Fig. 13.

The convergences in the tunnel support diminish approximately in 10 days according to the time dependent numerical model. The convergences after this process are negligible. Fig. 13 illustrates that 24.70 mm, 41.76 mm and 124.71 mm closures between 1-2, 1-4 and 4-5 convergence measurement points are predicted, respectively. These measurements are not deformation. These are closure between the represented measurement points.

According to the new designed support system (as non-deformable support system), the tunnel excavation and the deformation have been monitored in the tunnel (Fig. 14).

Extensometer measurements completed between convergence measurement points in Fig. 14 shows that, 22.90 mm, 39.92 mm and 118.21 mm closures are observed in point 1-2, 1-4 and 4-5, respectively. In the light of new support system, Tunnel 34 was excavated without any problem. It is seen that in situ measurements and time dependent numerical model is compatible with each other. Barla (1995) states that deformation in squeezing rocks is due to creep behavior. In Eurock 2012, Barton (2012) made a comment to a presentation about a tunnel design methodology due to its deficiency of time parameters. Indeed, evaluation of the time dependent parameters gives more accurate results in numerical models.

6. Conclusions

Prediction of deformation while tunnel support design works is very important regarding to tunnel safety. Unpredicted deformations may damage the support. It should be kept in mind that time-dependent deformations may also develop. NDSS presented in this paper can give time-dependent solutions. It allows some deformations within the calculated deformation limits rather than zero-deformation. NDSS not only used in shallow and deep tunnels, but also used in deep mine galleries satisfactorily.

To repeat the following issue will be right. NDSS means that the system does not allow the deformation more than the predicted deformation obtained from the numerical analyses. Brief conclusion of this research, there are some support design methodology similar to NDSS. However, the main principle of the NDSS is analyzing all the support system elements by using the right materials-failure models and time to together.

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