REHABILITATION OF MALATYA-NARLI NO:7 RAILROAD TUNNEL

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Approval of the Graduate School of Informatics

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ABSTRACT

REHABILITATION OF MALATYA-NARLI NO:7
RAILROAD TUNNEL

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In thesis thesis, studies associated with the rehabilitation of the Malatya-Narlı No: 7 Railroad Tunnel are presented.

The rehabilitation work includes cleaning of two collapses and stopping of deformations occurring in the tunnel as well as characterizing the rock-mass by evaluating the cores obtained from 67 drill holes. Due to two collapses ocurred in the tunnel a large sinkhole (15x10x20 meters) was developed at the surface and the tunnel closed to train traffic for 10 months (September, 2002-July, 2003) covering the initiation of Iraq War.

Originally, the tunnel had been opened into the paleo-landslide material in 1930. The rock-mass surrounding the tunnel consists of limestones, metavolcanics, and schists.

Although the main problem in the tunnel is the reduced tunnel span caused by displacements triggerred by underground water, poor rock mass and time dependent deformations, from engineering point of view the other problems can be sited as collapses occurred in the tunnel, sinkhole devoloped at the surface and unstable sections existing in the tunnel.

During the field studies, 15 deformation monitoring stations were installed aimed at determining the deviation from tunnel alignment. In order to provide stability of the tunnel Self Drilling Anchors (Mai bolts) were installed systematically around the tunnel. The details of the rock reinforcement design was presented in this thesis.

Key words: Rehabilitation, collapse, sinkhole, deformation, convergence measurement, Mai bolts

ÖZ

MALATYA-NARLI HATTI 7 NUMARALI DEMİRYOLU TÜNELİNİN ISLAH ETME ÇALIŞMALARI

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Tez Yöneticisi: Prof. Dr. Erdal Ünal

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Bu tezde, Malatya-Narlı hattı 7 numaralı Demiryolu Tünelinin ıslah etme çalışmaları sunulmuştur.

Islah etme çalışmaları iki göçüğün temizlenmesini, tünelde oluşan deformasyonların durdurulmasını ve bunlara ilaveten 67 adet sondaj karotlarının değerlendirilerek kaya kütlesinin tanımlanmasını içermektedir. Tünelde meydana gelen iki göçükden dolayı yüzeyde 15x10x20 metre ebatlarında çukur oluşmuş ve tünel ırak savaşının başlangıcı döneminde 10 ay (Eylül, 2002-Temmuz, 2003) boyunca trafiğe kapatılmıştır.

Başlangıçta, tünel 1930 yılında eski (paleo) heyelan kütlesi içerisinde açılmıştır. Tüneli çevreleyen kaya kütlesi kireçtaşları, metavolkanitler ve şistlerden oluşmaktadır.

Tüneldeki esas sorun yeraltı suyunun, zayıf kaya kütlesinin ve zamana bağlı hareketlerin tetiklediği yer değiştirmeden dolayı oluşan tünel gaberisindeki azalma olmasına rağmen, mühendislik bakış açısına göre diğer sorunlar göçükler, yüzeyde oluşmuş çukur, tünel içersinde sağlam olmayan bölgeler olarak söylenebilir.

Arazi çalışmaları sırasında tünel ekseninde oluşan kaymayı saptamak amacıyla 15 adet deformasyon gözlem istasyonu oluşturulmuştur. Tünelin sağlamlığını arttırmak amacıyla Mai bulonları belirli bir düzen içersinde tünelde uygulanmıştır. Bu konuyla ilgili detaylar bu tez çalışmasında sunulmaktadır.

Anahtar kelimeler: Islah, göçük, çukur, yer değiştirme, kapanma-açılma (konverjans) ölçümü, Mai bulon

To My Family

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CHAPTER 1

INTRODUCTION

1.1 General Remarks

In the last decade, there is a considerable increase in tunnel design and construction facilities in Turkey as stated by Turkish Road Association in 2002.

Many of old tunnels in Turkey and in the world are approaching their design life expectations. However, due to high cost of replacing these facilities, many tunnel systems are being rehabilitated to extend their useful lives into the next century.

According to studies carried out by Maintenance Department of General Directorate of Highways (KGM) approximately 84 percent of tunnels has been constructed concrete lining, 16 percent of them are unlined tunnels under high water effect and constructed between 1954-1965. There is high segregation and spalling in these tunnels in which concrete lining was used. Reason for this type of defects is the quality of concrete. Concrete class for the concrete lining was BS 16 until 1980 and increased to BS 20 from this date on. Based on the studies of KGM there are 6 tunnels with total length of 1033 meters, where increasing weathering of rock behind concrete lining causes additional loading onto lining and high water effect reduce quality of concrete. All those effects bring dangerous conditions for structural stability of tunnel.

1.2 Statement of the Problem

Poor rock mass condition, water problem and structural instability were observed in No:7 Malatya-Narlı Railroad Tunnel. As a result of groundwater effect and time dependent deformations (creep) the tunnel walls between km 155+550 and 655 was displaced 1.67 m towards the creek side, resulting in an undulating tunnel shape, a reduced roof-span, distorted steel-arches and cracked concrete-lining. Consequently, there were serious problems occuring in the tunnel namely, collapses, reducing tunnel span and unstable opening.

1.3 Objectives of the Thesis

In this thesis a comprehensive literature survey has been presented related to rehabilitation work in unstable tunnels. In addition, details of the rehabilitation work and convergence measurements carried out in Malatya-Narlı line “No: 7 Tunnel” has been given.

1.4 Thesis Outline

Following the introduction in Chapter 1, case studies related to the factors affecting the instability, collapses, improvement work and stability measurements are reviewed in Chapter 2, as part of a comprehensive literature survey. Chapter 3 includes information about the location of No:7 Malatya-Narlı Railroad Tunnel and material pertinent to the study. The problems related with the stability of the No:7 Railroad Tunnel i.e., collapses occurred during rehabilitation work, sinkhole developed at the surface due to roof collapse, are presented in Chapter 4. Chapter 5 includes MAI Self Drilling Anchors application patterns and Chapter 6 explains the convergence measurement results. Finally, conclusions and recommendations pertinent to this study are presented in Chapter 7.

CHAPTER 2

LITERATURE SURVEY

2.1 General

In this chapter, an extensive literature survey carried out and state of the art on stability and rehabilitation of tunnels is presented. Basically, the following topics are considered;

i. Factors affecting the stability of tunnels
ii. Collapses in tunnels
iii. Rehabilitation and improvements of tunnels
iv. Stability measurements

In addition to general information given, examples taken from case studies are included in the literature survey.

2.2 Factors Affecting the Stability of Tunnels

The main factors affecting the stability of tunnels are underground water, discontinuities, rock mass surrounding the tunnel, geometry of the opening, and earthquake.

2.2.1 Underground Water

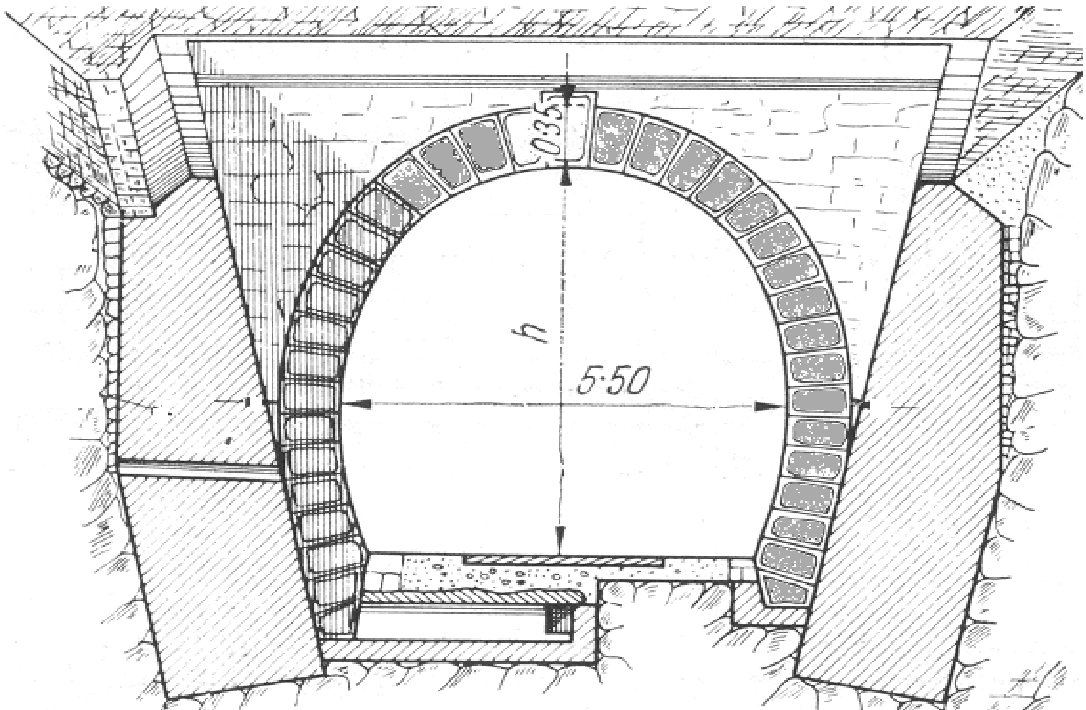
Water is a problem in any tunneller's life. It causes problems during excavation and support of the ground, can give rise to more expensive linings. Water leakage is the most common maintenance hazard, causing problems during the working life of a tunnel, not only to the tunnel lining, but also to the fittings within the tunnel (Richards, 1998). Generally talking some precaution or countermeasures against water leakage should be taken in order (Asakura and Kojima, 2003);

i. to maintain the function of the lining as a structure against the deterioration of the lining materials due to water leakage, unbalanced pressure and ground surface settlement due to the formation of voids behind the lining, resulting from flowing soil into the tunnel and the occurrence of a collapse;
ii. to prevent the corrosion of track materials such as rails, and tie-plates, the malfunction of signal communication systems, electric power facilities and so on;
iii. to prevent harmful influences to railway service, and to passing trains;
iv. to secure the safety of work and workers inside the tunnel; and
v. to maintain the appearance of the tunnel.

Problems are sometimes encountered with incoming water in tunnels in conjunction with frost, giving rise to: i. reduction of the size of the opening by the formation of ice barriers; ii. icing of the pavement in road tunnels; iii. obstruction of ventilation and other service ducts and shafts; and iv. hazards from icicles forming in the tunnel roof (Richards, 1998).

Water leakage damages the tunnel lining, the type of damage varying with the type of lining and also the method/integrity of the construction. Two main types of linings are reviewed: i. brick and masonry tunnels; ii. concrete tunnels; Examples of these two tunnel types are shown in Figure 2.1.

(a)



(b)

Figure 2.1 (a) Examples of Concrete Tunnels (after www.pghbridges.com)
(b) An Example of an Old Masonry Tunnel (after Szechy, 1973)

Brickwork and Masonry Tunnels (Richards, 1998)

In many of the older tunnels, the lining is made up of 5 or more courses of brickwork. In areas where the ground support was required during construction of the lining, this was provided by wooden beams and uprights which were often left in place. The inevitable voids between the lining and the excavated ground were filled by packed tunnel debris, generally comprising a high proportion of stones and boulders. The latter provided a good drainage medium behind the lining. These type of tunnels are very laborious and expensive.

Bricks are sensitive to water, even if it is not aggressive, depending on their firing temperature and chemical nature. Water leakage encouraged by the drainage medium inadvertently formed behind the lining leads over a period of time to decomposition or rotting of the timber supports, washing out of the ground and backfill material behind the lining causing large voids or hollow areas to form and weathering and deterioration of the brickwork and the mortar of the lining. The effects of water on mortars, especially those made with lime, are well known: the mortar loses its strength and becomes brittle. In some cases, chemical reaction from sulphates in the seeping water causes swelling.

The forming of voids can lead to increased water leakage and collapsing of ground which, together with water filled voids, may cause structural distress to the lining.

Concrete Tunnels (Richards, 1998)

These take the form of tunnels with unreinforced and reinforced cast-in situ concrete linings or tunnels with segmental concrete lining. Water leakage is most commonly the cause of giving rise to transportation of fines, the formation of voids, settlement of ground, and eventually eccentric loading and distress of the lining as described previously for the brickwork and masonry tunnels.

Inadequate sealing of the joints between concrete segments and poorly constructed joints in cast in situ concrete linings can lead to water leakage. Water leakage through porous or cracked concrete will result in loss of cement, making the lining more pervious and less strong.

The corrosion of reinforcement in both reinforced cast in situ and segmented concrete linings is a principal concern. Corrosion is caused by pervious concrete or inadequate concrete cover and will inevitably lead to cracking and spalling of the concrete and loss of structural capacity. The presence of soft water or chlorides in the water will accelerate the effects.

Repairs to the Drainage System (Szechy, 1973)

According to Szechy (1973) the most severe damage is caused by water and, thus reconstruction and repair work, in general, should be aimed primarily at protection against water. There are two alternatives for such protection: the water can be kept out of the tunnel altogether in which case the infiltration of groundwater should be prevented and it should be sealed out completely, or the water may be allowed to penetrate the tunnel but in a controlled manner through special subdrains so that it can be intercepted and removed regularly. A disadvantage of the first method is that the tunnel lining has to carry the hydrostatic pressure as well as the overburden. It has to be constructed in a box-like form; in addition, waterproofing can be also rather complicated and expensive to construct, particularly if it is done on the extrados. On the other hand, there is no danger of disturbing the equilibrium of the surrounding water-bearing ground, no continuous underground streams and no danger of wash-outs accompanied by a loosening and saturation of the surrounding ground. The second method is not without drawback, either, because the construction on an interceptor drainage system behind the tunnel lining is a complicated and expensive proposition. The construction of special drainage galleries and adits is also expensive, and it relieves the immediate vicinity of the tunnel from wash-outs and loosening only to a limited extent. Some case studies are given in the following.

Dorukhan Tunnel (Akçelik et al., 2002)

Dorukhan Tunnel, with 903 m length, was completed by conventional methods in difficult geological conditions and opened to traffic in 1976. Due to lack of water insulation, there was water inflow towards the tunnel especially at the conjunction of the main tunnel and cut and cover section of the tunnel. Water flow through the tunnel was reduced by establishing surface drainage with concrete lining trenches in portals of the tunnel (Figure 2.2).

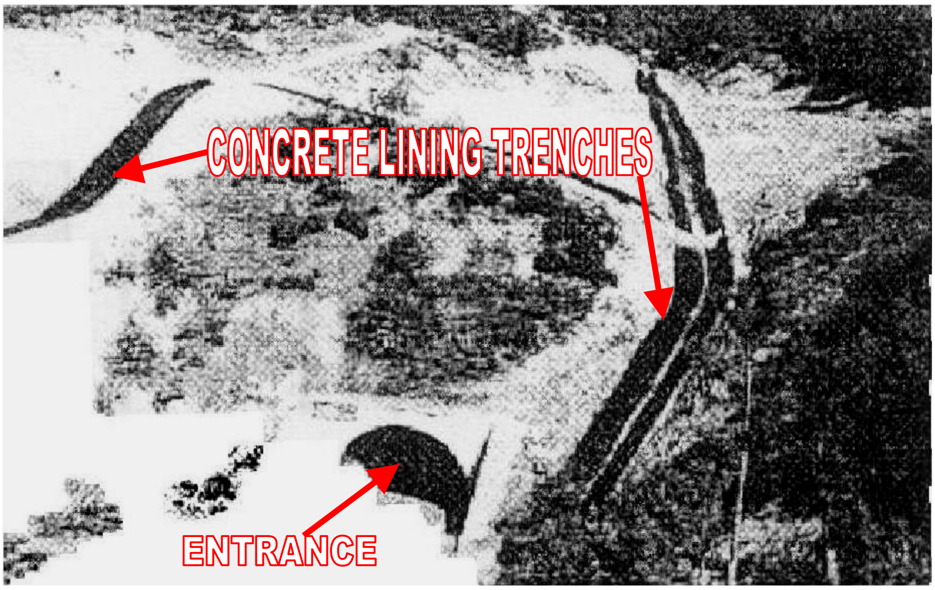


Figure 2.2 Surface Drainage on the Mengen Portal
(after Akçelik et al., 2002)

Repair on Shindagha Tunnel (Duddeck, 1989)

The repair of the Shindagha Tunnel, which runs underneath Dubai Creek in the United Arab Emirates, is relying on an epoxy resin system developed by Cormix Middle East, the Dubai-Based associate of Cormix, British specialist in concrete chemicals.

The problem with the tunnel was the leaking construction and expansion joints let in sea water, leading to severe attack on both concrete and steel reinforcement. Sea water and seeping through the water stops in the joints and permeating through the concrete lining caused the most damage. Alternate wetting and drying inside the tunnel left a concentration of salt on the surface concrete which penetrated and attacked the reinforcing steel.

Repairing started in 1986 and completed in 1989. The remedial work necessitated the waterproofing of all construction and expansion joints, which were placed at 9 m intervals along the tunnel. The technique included the injection of epoxy resin into cracks in the concrete of the tunnel lining to restore structural integrity; and into the joints in the lining where faults were found to exist.

The epoxy resin proved highly successful in practice. It was normally hand mixed prior to injection and every drum was tested prior to being issued for use. Two hardening tests were carried out each day for every injection machine and gang working.

<u>Furka Base Tunnel</u> (Amberg and Sala, 1984)

During the construction of Furka Base Tunnel, the numerous water inflows were a considerable problem. The usual method of sealing by means of plastic drains consisting of tubes cut in half and a rapid hardening mortar proved to be too time-consuming and complicated. The area of rock surface requiring treatment was more than 75000 m^2. With a special grout, which hardens within seconds and can be applied using a machine, an adequate solution could be found. Both bond and compressive strength tests gave adequate results so that the new technique could be adopted. The average compressive strength amounted to 12 N/mm^2 after 1 hour and increased to 45 N/mm^2 in 28 days. The special grout was applied in layers of 1.5-2.5 cm thickness. Although the latter was relatively thin, an excellent sealing effect was achieved. In areas of diffuse water inflow the surfaces affected could be sealed systematically. The technique employed was firstly to fix the plastic drains at equal distances (0.50-0.60 m) by spraying over them. Then these plastic drains and the intermediate rock surfaces were covered with about 2.5 cm of grout. By sealing in this way the water was forced through the joints and the cracks developed by blasting into the plastic drains. Thus an absolutely dry surface was obtained.

Kızlaç Tunnel (Etkesen et al., 2002)

Permanent drainage measures were applied. The water was collected outside grouting ring by using perforated drainage pipes, which are 50 mm and 8 m in length. Pipes were installed slightly (5°-10°) upwards in the bench and radial in the top heading. The spacing of rows was 3 m.

Kızılay Tunnel (Özaslan et al., 2002)

The periphery of the tunnel was stabilized and groundwater leakage into the tunnel was minimized by forming grout columns in the crown and sidewalls of the tunnel. It has been decided to perform this method from the ground surface considering the difficulties in execution, time factor and water leakage that may originate from the probable deviation in the horizontal drilling in case of implementing this method inside the tunnel. Figure 2.3 shows the applied jet grouting columns.

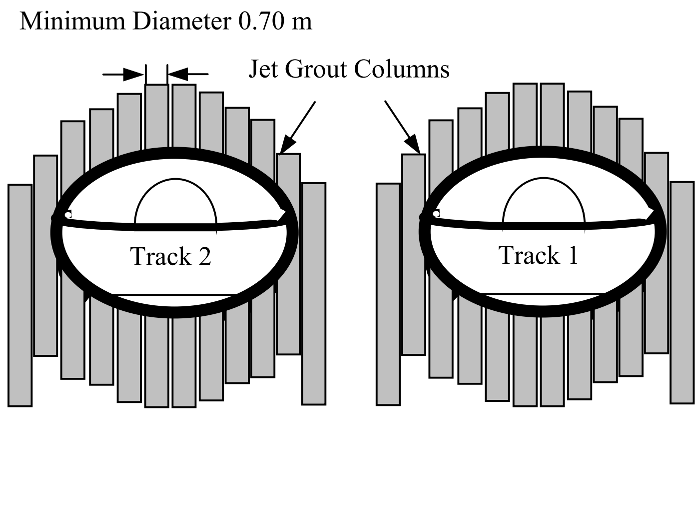


Figure 2.3 Construction of Impermeable Curtain with Jet Grouting
(after Özaslan et al., 2002)

Giswil Highway Tunnel (Meier et al., 2005)

A successful waterproofing was achieved in Giswil highway tunnel in Switzerland by a sprayable membrane system, consisting of a minimum 3 mm thick spray applied membrane which is located between two layers of sprayed concrete. The membrane provided bonding to the rock support sprayed concrete substrate, as well as to inner layer of sprayed concrete which was applied onto the membrane after the initial curing. The membrane was applied to the full tunnel profile including the invert, to provide an undrained design solution. The double sided bonding, which the system provides, ensured no migration of water along the membrane-concrete interfaces. Hence, a continuous waterproof structure was achieved.

In a project work carried out by Park et al., (2005), the N.T.R (New Tabular Roof) method was modified and utilized for solving those problems occuring because of the lack of stability due to the water leakage from walls and roof. Park et al., (2005) explains the features and the procedure of N.T.R method and mentions the applicability of the method for underground development in Korea using the data obtained from a construction site for underground drainage system and the results obtained from numerical analysis for the stability of that system.

Lambert and Hoek (2005) presents a new method for leakage repair called Bio Sealing. The new method causes bio-clogging at the leakage and has been tested in laboratory experiments and recently, also in a field-experiment. The method was developed according to following principles: i. the natural chemical, physical and biological properties of soil and groundwater, ii. the groundwater flow through the leak. Furthermore bio-sealing is based on the following assumptions: i. the location of the leak is known within 5 to 10 m distance, ii. the natural direction of groundwater flow is known, iii. bacteries are available at any place in the soil, they only have to be activated. Purposes of the repair are: i. the hydraulic conductivity of the leak has to be decreased, at least, five times ii. injections at a certain distance of the leak can be regulated such that they cause clogging at the desired location, in and nearby the leak. After small series of survey experiments, biological clogging was identified to be the most promising mechanism. The injection of even small quantities bio-activators caused a decrease of permeability with a factor 5-20.

2.2.2 Discontinuities

The effect of mountain (rock mass) structure on tunneling is quite obvious. Tunnel construction is simplified, accelerated and made less expensive by the uniformity and soundness of rock and the greater the variation and fracturization of layers, the more involved, expensive and time consuming the tunneling methods will be. Mountain formations, devoid of stratification are much more favorable for tunneling than mountains composed of layers, or shales, or granular masses of varying degrees of solidification. In the selection of the location and depth of the tunnel axis its position relative to the stratification should be thoroughly studied (Szechy, 1973).

Where the tunnel axis is perpendicular to the strike of a steeply dipping rock stratum, the excavation of the tunnel is likely to succeed under favorable rock pressure conditions. However, where the tunnel axis is parallel to the strike higher rock pressure may be expected to occur (Szechy, 1973).

Tunnel passing through mountains involves comparatively complicated geological conditions. Geological investigation for such tunnels is very important. Aerial surveying and remote sensing have been used for decades in geological investigations for some tunnels, with satisfactory results (Mi and Shiting, 1989).

The Da Yao Shan Tunnel has been driven through the mountain. Thirteen of the linear tectonic structures predicted on the basis of the remote sensing pictures were in good agreement with field verification; five of the predictions were somewhat off. Construction also proved that not all of the linear tectonic structures distinguished by remote sensing were fault structures. Rather, they included compressed broken belts, concentrated rifts and weak falling. These geological weaknesses indicate areas where collapse and water leakage may occur (Mi and Shiting, 1989).

Kızlaç tunnel, divided into two sections as T3A to the west and T3B to the east, is the longest tunnel on Tarsus-Adana-Gaziantep (TAG) motorway, located in a region nearby the South Eastern Anatolian Thrust Fault Zone and is intensively under the effect of secondary faults of different strike and dip angles with respect to the tunnel axis. The tunnel has been excavated in rock units consisting of sandstone, siltstone and shale with zones crushed and weathered material. During excavation a lot of fault zones varying thickness (5-30 m) have been encountered and collapses occurred at some these zones, which could not be predetermined in the field work and design phase (Etkesen et al., 2002).

The determination of the condition of rocks along the tunnel axis is one of the primary tasks of geological investigation. Investigations should be extended to the possible physical, chemical or biological action to which the rocks may have been exposed during their geological history and which may have influenced their strength to a certain extent and in a certain location (Szechy, 1973).

2.2.3 Rock Mass

Squeezing Rock

Squeezing ground or rock behavior is characterized by the occurrence of large rock pressure which may lead to the failure of the lining. As a consequence of insufficient or vanishing lining resistance, large rock deformations may develop. Rock pressure and rock displacements generally occur around the whole cross section frequently involving the invert as well. Especially in larger excavation sections the face is also subjected to large deformations or instabilities. It is an established empirical fact that low strength and high deformability of the rock as well as the presence of porewater pressure facilitate squeezing (Kovari and Status, 1996).

The expression ”squeezing rock” seems to be very familiar to the tunnel engineer and it is assumed that squeezing rock conditions implies that:

i. A process of failure of the rock mass occurs at the walls of the tunnel,
ii. The rock mass deforms plastically in the yielding zone around the tunnel
iii. The yield zone has a rather large extent.

All these conditions bring about large convergences that may be considered as the main feature of squeezing rock conditions (Panet, 1996).

Austrian Road Tunnel (Ayaydin and Huber, 2005)

Ayaydın and Huber (2005) explains that in the western part of Austria for an Expressway a road tunnel with two tubes each 5,8 km are under construction. On a length of approximately 2500 m heavily squeezing rock mass behaviour were encountered. Large deformations up to 0.80 m were to be mastered with continuous adjustment of rock bolt lenghts and numbers as well as with the installation of yielding steel elements in slots of shotcrete lining. The optimization of the support measures were based on the interpretation of the geotechnical measurements and on geological documentation of the tunnel face. The adaptability and economic application of NATM has been proven in the project. In this paper Ayaydın and Huber (2005) explains some cases of measurement results, geological conditions and the geomechanical models built up from these data and the measures, which are applied to manage the difficult conditions.

Swelling Rock

If the squeeze of a decomposed rock is chiefly due to expansion, the rock is referred to as a swelling rock. Swelling clays and swelling rocks are likely to exert much heavier pressure on tunnel supports than clays and rocks without any marked swelling tendency (Terzaghi et al., 1977).

Rocks containing clay minerals and anhydrite increase in volume when they come into contact with water; this phenomenon is referred to as the swelling of these rocks. In tunnel construction, swelling of rocks manifests itself as a heave of the tunnel floor, or as pressure on the invert arch. When the lining remains intact, a heave of the entire opening can occur, where the crown as well as the floor experiences an upward displacement. In many cases, the pressure resulting from the swelling of the surrounding rock leads to a failure of the invert arch (Kovari, 1988).

The consequences of such movements range from problems associated with broken drainage channels and interference with vehicular traffic during construction, to destruction of initial and final supports as well as to deformation of the railway or highway roadbeds impeding traffic (Einstein, 1996).

The analyses given by Gysel (1987) provide closed form solutions for circular tunnels in swelling rock of reversible type (swelling of clay minerals). Using the method suggested by Gysel (1987) it is possible to analyze the forces which are transmitted from the swelling rock to the tunnel lining in a given case. In principle, the method could be transformed into an elasto-plastic approach. For complex cases, the geometrical boundary conditions may not allow the use of a circular tunnel model. In such cases, Gysel (1987) suggests to use finite element solutions which are available for swelling calculations in the elasto-plastic formulation straight away.

Influence of Rock Mass Type

The support pressure is directly proportional to the size of the tunnel opening in the case of weak or poor rock masses, whereas in good rock masses the situation is reversed. Hence, it can be inferred that the applicability of an approach developed for weak or poor rock masses is doubtful in good rock masses (Goel et al., 1996).

The approach of Unal (Unal 1984, in Goel et al., 1996) has been evaluated for Indian rock tunnels having an arched roof. The results show that in good rock masses experiencing non-squeezing ground conditions, the predicted support pressure by Unal is safe for medium size (tunnel diameter 6-9 m) tunnels, but it is unsafe for small tunnels (3-6 m) and conservative for large tunnels (9-12 m). It shows that the correlation of Unal (1984) would not be applicable in rock tunnels through hard rocks with an arched roof, as more emphasis on tunnel size has been given.

Goel et al., (1996) have evaluated the approaches of Barton et al., (1974) and Singh et al., (1992) using the measured tunnel support pressures from 25 tunnel sections. They found that the approach of Barton may need to be modified in squeezing ground conditions and the reliability of the approaches of Singh and that of Barton depend upon the rating of Barton's Stress Reduction Factor (SRF). It has also been found that the approach of Singh may be inappropriate for larger tunnels in squeezing ground conditions (Goel et al., 1996).

2.2.4 Geometry of the Opening

Prediction of support pressure in tunnels and the effect of tunnel size on support pressure are two important problems in tunnel mechanics. Various empirical approaches for predicting support pressures have been developed in the recent past. Some demonstrated that the support pressure is independent of tunnel size (Daemen 1975 and Singh et al., 1992, in Goel et al., 1996), whereas others advocated that the support pressure is directly dependent on tunnel size (Terzaghi 1946 and Unal 1984, in Goel et al., 1996).

Influence of Shape of the Opening

Some empirical approaches have been developed for flat shaped roofs and some for arched roofs. In the case of an underground opening with a flat roof, the support pressure is generally found to vary with the width or size of the opening; whereas, in arched roof tunnels, the support pressure is found to be independent of tunnel size. The RSR system of Wickham (Wickham et al., 1972, in Goel et al., 1996) is an exception in this regard probably because the system, being conservative, was not backed by actual field measurement. Figure 2.4 shows the stresses for arch roof and flat roof.

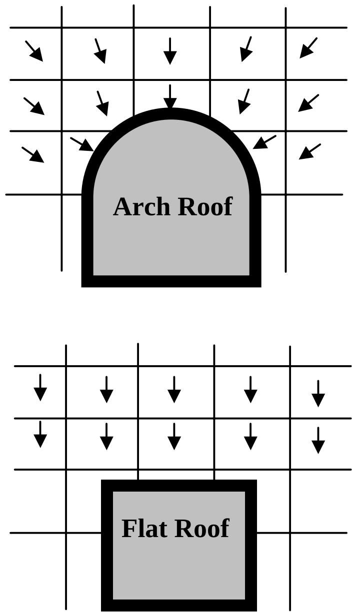


Figure 2.4 The Condition of Stresses for Arch Roof and Flat Roof
(after Goel et al., 1996)

The mechanics suggests that the normal forces will be greater in the case of a rectangular opening with flat roof by virtue of the weight of detached block of rock which is free to fall. It is highlighted that the detached blocks become interlocked on displacement in an arched roof because of the dilatant behavior (Goel et al., 1996).

Some underground excavation designers have concluded that, because the stresses induced in the rock around an excavation are independent of the size of the excavation, the stability of the excavation is also independent of the size. If the rock mass is perfectly elastic and completely free of defects this conclusion would reasonably correct, but it is not valid for real rock masses which are already fractured. Even if the stresses are the same, the stability of an excavation in a fractured and jointed rock mass will be controlled by the ratio of excavation size to the size of the blocks in the rock mass (Hoek and Brown 1980, in Goel et al., 1996).

Consequently, increasing the size of an excavation in a typical jointed rock mass may not cause an increase in stress, but it will almost certainly give rise to a decrease in stability (Hoek and Brown 1980, in Goel et al., 1996).

2.2.5 Earthquake

Earthquakes are hazardous to all surface and underground structures. They arise typically from sudden release of stresses at depths ranging up to several hundred kilometers, often by movement in the plane of a fault (Megaw and Bartlett, 1981).

Substantial surface structures are principally damaged by the horizontal component of acceleration in the vibrations at the surface, particularly in unconsolidated strata. No structure can, of course, resist a shearing movement in the plane of a fault if the structure is continuous across that plane. Tunnels in solid ground are unlikely to be severely damaged by vibrations unless the ground itself is sheared. There is, however, obvious danger of landslips at tunnel portals or in the sides of a mountain valley (Megaw and Bartlett, 1981).

The behavior of an underground linear structure such as a tunnel in the event of earthquake is independent of deformation of the adjacent ground. This is very much different from a structure on the ground such as a bridge which is greatly affected by its own inertia force. Even if a large amount of reinforcement bars are provided in the axial direction, cracks may happen on the tunnel body in the event of earthquake and strain may be intensified there. When the tunnel suffers from the forced displacement from the adjacent ground in the event of earthquake, deformation of tunnel and sectional force are small if the adjacent ground of tunnels is soft. In case the tunnel ground is soft, the sectional force becomes small because the tunnel can follow the deformation of the ground without repulsion. On this account, in the case of a tunnel with soft ground such as a submerged tunnel, distance of installation of joints can be greater since the deformation of ground can be mitigated. On the other hand, in the case of a shield tunnel with a small stiffness, there is no need of joints because the tunnel can follow the deformation. However, in the case of the present tunnel with a high degree of stiffness, which is very much affected by the effects of hard ground, the tunnel is deformed to almost a close shape displacement of ground. And, a large sectional force occurs to resist this. Therefore, the distance of joints must be smaller than in the case of a submerged tunnel. Thus, it is advisable that in the case of a tunnel, which is very much affected by the effect of hard ground, some of the construction joint of concrete must be flexible structure with as much cut-off ability as possible (Nakagawa et al., 1992).

2.3 Collapses in Tunnels

2.3.1 Collapses

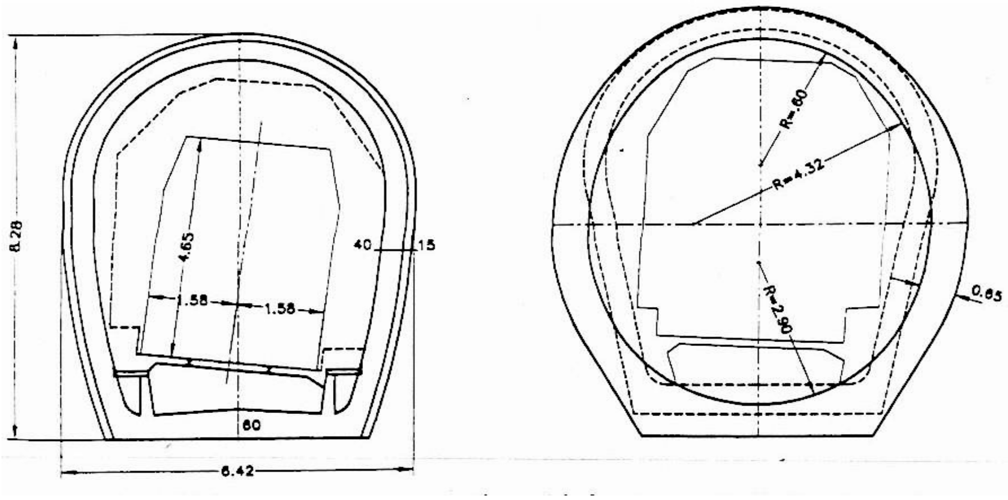
Typical examples of collapses in tunnels are presented in following subsections.

<u>The Sidi Mezghiche Tunnel</u> (Panet, 1996)

The 990 m long T8 tunnel is situated close to the city of Sidi Mezghiche, Algeria. It is a one-way tunnel with a horse shoe-shaped cross-section of 46 square meters. The maximum overburden is about 65 m.

The tunnel was driven in a mass of flyschs belonging to an overthrust with a complex geology. The rock consists of laminated argilites with a great number of lustrous shear surfaces; some parts are more sandy with a limy cementation. An interbedded stratification may be observed and the whole mass is intensively tectonised with sharp warping of the beds.

A collapse of the supported tunnel occurred over a length of 100 m with a large deformation of the support on 100 meters more. The failure went up to the ground surface. The deformed cross section had the shape of a keyhole. Figure 2.5 shows the condition of the tunnel before and after cave-in or collapse.



(a) (b)

Figure 2.5 Sidi Mezghiche Tunnel-Cross Sections,
a) Before the Collapse, b) After the Collapse
(after Panet, 1996)

<u>Kızlaç Tunnel</u> (Etkesen et al., 2002)

The tunnels advanced properly without any delay until September 30 in 1995, ten days before the first collapse. At round 156 (km:41+836) after an excavation, water inflow of about half liter per-second and a deformation of 250-300 mm on the steel rib at the left side were observed. At first, support class was not changed, but round length was reduced to 1.7 m for Rounds 158 and 159, 1.5 m for Rounds 160-163 respectively, and then rock support class was changed to C3 and round length was reduced to 1.2 m. About a week later, when the face was at Round 163, in Rounds 153 to 159 settlements, break in rock bolts and concave shape deformations on several rockbolt washers were observed. Additional bolts were provided along the right side between Round 152 to 160 as a remedy. Water flow declined and it was decided to apply extra support to Rounds 163 to 169.

On October 11, 1995; from the right side of the crown of Round 165, the ground fell down and an overbreak chimney of 12 m^3, occurred. Overbreak volume stabilized within three meters above the tunnel crown. The collapse occurred between km 41+831 and 41+840. Figure 2.6 shows the first collapse.

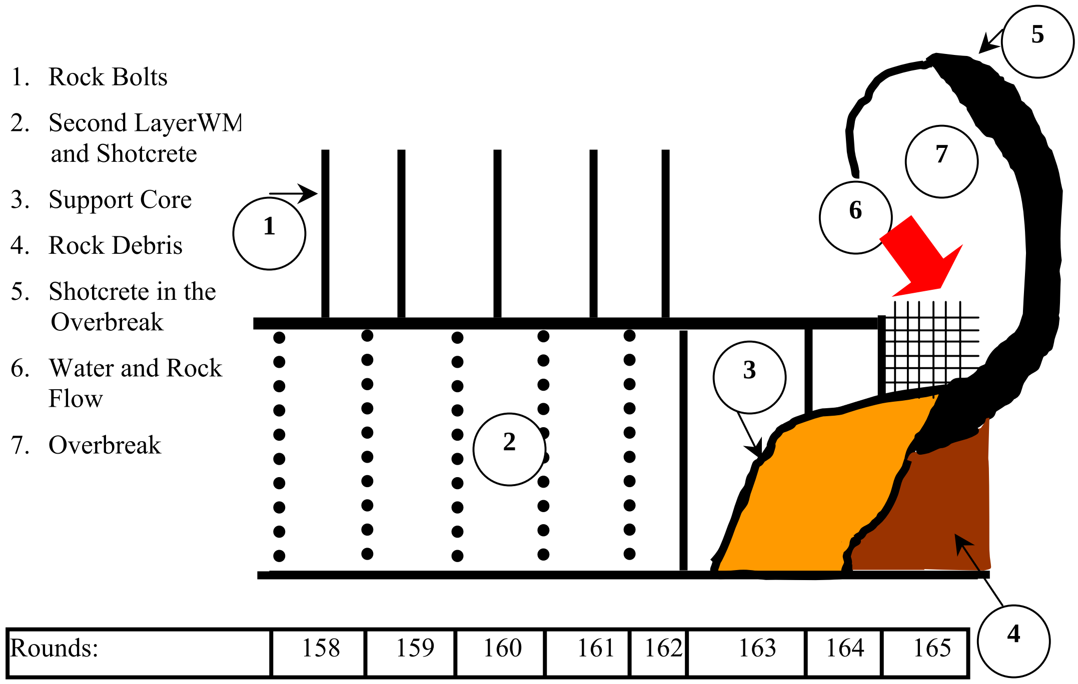


Figure 2.6 First Collapse in Round 165 (after Etkesen et al., 2002)

During removal of the rebound material, small but steady trickle of loose material began flowing into the tunnel from the right shoulder and from the back face of the cavity. Finally, there occurred a flood with rapidly increasing volume. Tunnel collapsed (between rounds 163-169). The volume of flowing material was 650 m^3, the flow rate of water was 25 lit/sec, respectively.

On 16 th March 1996, shotcrete was cracked in Rounds 138 to 146 in drived sections of tunnel. Until 19 th March, deformations had spread to Rounds 136 and 137. The second collapse occurred at the face Round 179 (at km: 41+845, while tunnel was approaching to second fault zone intersecting the tunnel axis with 70°-75° angles).

During setting out the ribs the surveyors observed unusual noise and movements. Radial movements were first measured at rib 174 in the right abutment in the order of 0.22 m, later increasing to 0.27 m. The shotcrete became increasingly cracked on the right wall of the top heading. Water relief holes were drilled, but operation stopped when it became obvious that a collapse is unavoidable. Instead, two trucks of excavation material were brought into the face area to reduce the extent of the collapse. The final collapse started from the face at the right side of the heading and then extending to the full face. The initial water inflow was about 25 lit/s, decreased to less than 3 lit/s within 4 days and less than 0.5 lit/s within 10 days.

Selatin Tunnel (Bizden and Murat, 2002)

On the date 25 th August 1991 there occurred a big water attack in the tunnel (km 88+600). On the date 15 th August 1991, 852 m excavation was completed in tube. A few days ago, at the last 15-20 m of this excavated tube, huge amount of deformations were measured.

In 11 days, deformations reached to 0.372 m. On the date 25 th August 1991 at hour 23.30, from the upper half face, 250-300 lit/sec water started to flush causing of about 19.5 m collapse. So it became clear that the reason of big deformations were water.

Emptying of water started on 26 th August 1991. When the inflush decreased to 3 lit/sec, excavation of collapsed part started on 9 th September 1991. Pumping has taken of about 1 month and 30000 m^3 of water has been piped out.

Samanalawewa Tunnel (Hagenhofer, 1990)

Samanalawewa is one of the largest single projects ever undertaken in Sri lanka. The headrace tunnel is 5.35 km long and 4.5 m in diameter.

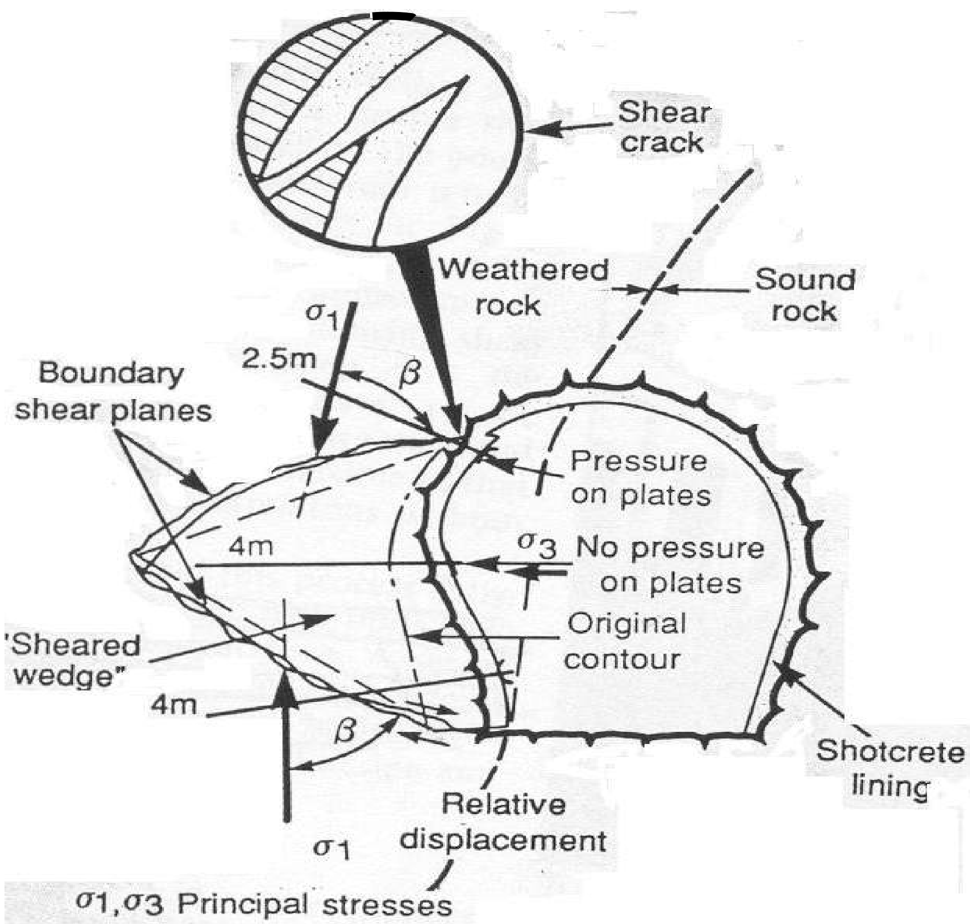


Figure 2.7 Side Wall Failure Showing Configurations of Shear Planes (after Hagenhofer, 1990)

The collapse happened on March 10, 1989. The left side wall failed within chainage 1175 to 1185, where the tunnel ran along a sharp boundary of completely different ground. The right side was within strong rock while the rock adjacent to the tunnel on the left was completely weathered and totally disintegrated to loose ground comprising clay silt and fine sand. Figure 2.7 shows the side wall failure within chainage 1175 to 1185.

The failing tunnel side stabilized after attaining a maximum convergence of about 0.60 m measured at the invert.

Kurtkulağı Tunnel (Türkmen and Özgüzel, 2003)

Kurtkulağı irrigation tunnel which is 1255 m in length and 3.50 m in diameter, is an important part of the Yumurtalık Plain Irrigation Project. The tunnel was excavated using conventional drill and blast, but some tunneling problems occurred as illustrated in Figure 2.8.

(a)

(b)

Figure 2.8 (a) View of the Collapse in the Tunnel Face
(b) View of the Sinkhole Form at the Surface
(after Türkmen and Özgüzel, 2003)

During the tunnel advance a fault zone of approximately 150 m was encountered. During excavation two major collapses occurred between chainage 744-810 m in a fault zone even reaching the ground surface where sinkholes were formed as demonstrated in Figure 2.9. Tunnelling was being performed at both sides of the fault zone, the tunnel faces collapsed. Collapse of the tunnel face continued during removal of the muck material from the tunnel. Consolidation grouting was applied to prevent collapsing, but it failed because this zone consists of the clayey material. During the removal of the collapsed material, the tunnel arch collapsed and two major sinkholes at the surface and several collapses in the tunnel formed.

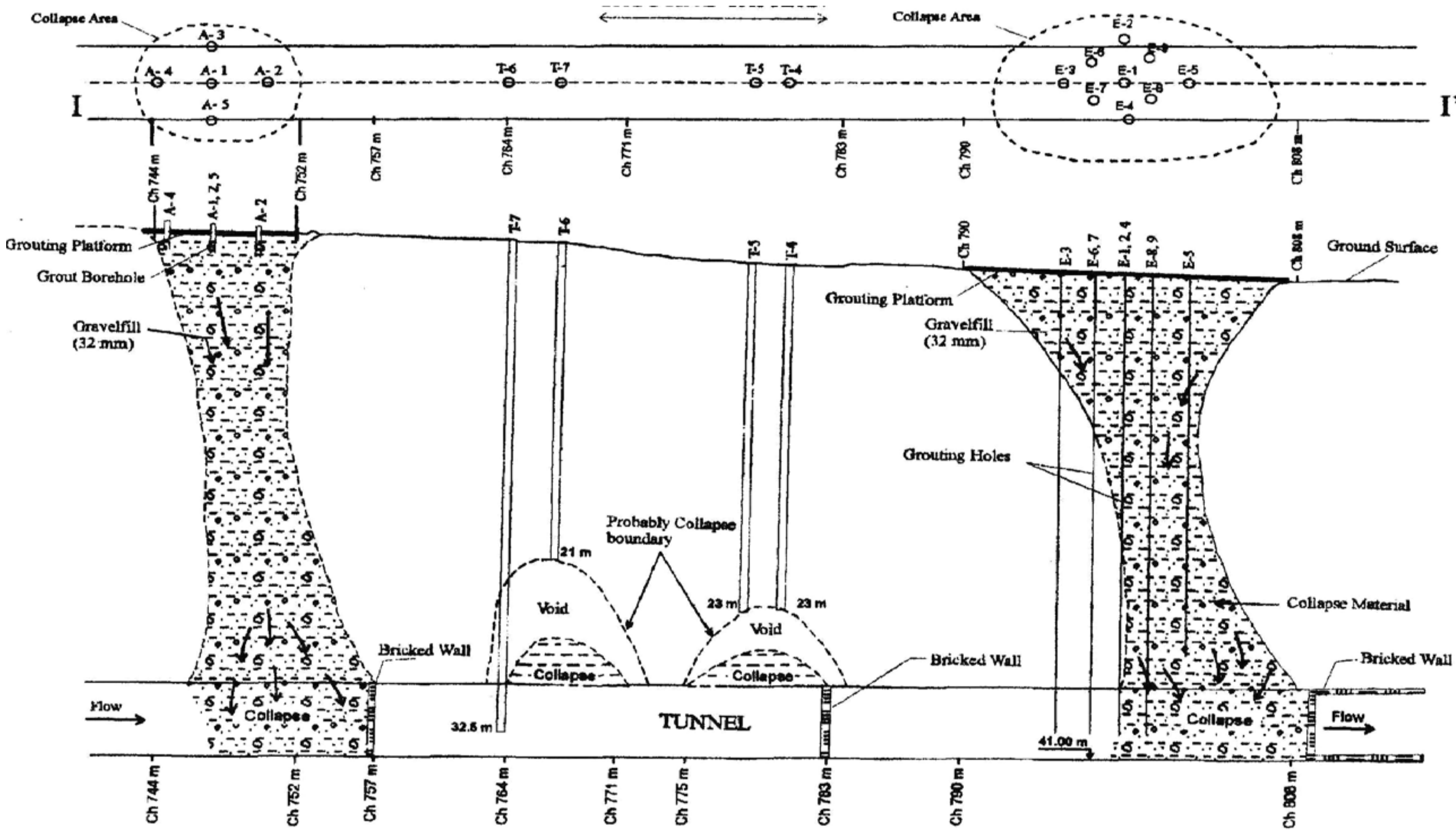


Figure 2.9 Cross-Section of the Collapsed Area and Grouting Pattern in Kurtkulağı Irrigation Tunnel (after Turkmen and Özgüzel, 2003)

Kinoura Tunnel (Kunita et al., 1994)

The Noto Offshore Earthquake, which occurred off the coast of Suzu City, Japan, on February 7, 1993, damaged the crown arch of the tunnel over a length of approximately 10 m, resulting in the formation of a gap when the ground soil collapsed about 10 m above the arch (Figure 2.10). In addition, cracks on the secondary lining and tunnel entrance and small collapses on the entrance slope face were observed.

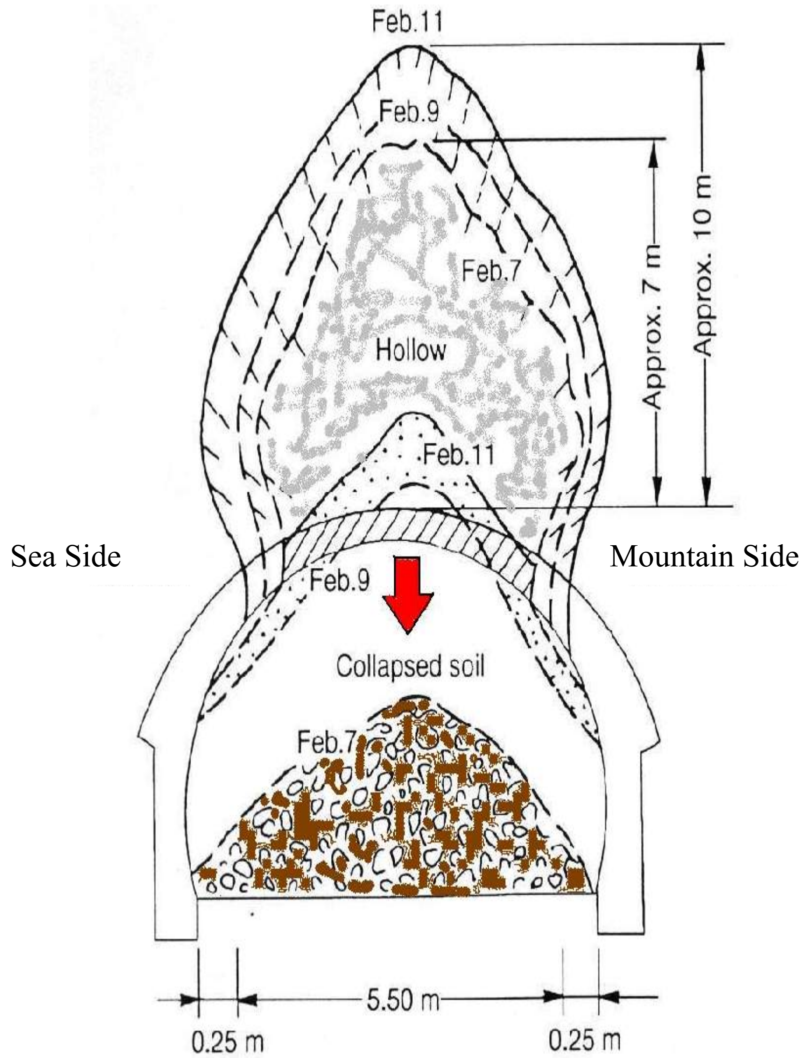


Figure 2.10 Section of the Collapsed Zone in the Kinoura Tunnel (after Kunita et al., 1994)

2.3.2 Sinkhole

In formation of the common and particularly hazardous cover-collapse sinkhole, a subsurface void forms in soil above an opening in the underlying rock. The void ultimately propagates to the surface, resulting in subsidence and generally, formation of a surface pit (Tharp, 1999). Typical examples of sinkholes have been shown in previous sections (See Figure 2.9 and 2.10).

Factors Associated with Sinkhole Formation (Tharp, 1999)

In some areas most induced sinkholes results from lowering of the water table, generally to below the soil-rock contact. This condition is conducive to sinkhole formation as a result of loss of buoyant support, increased pore pressure gradients and increased amplitude of water level variations. The soil may be exposed to repeated saturation and drying and the soil may become dry and friable. Desiccation of this kind may be important in formation of natural sinkholes associated with natural regional lowering of the water table over geologic time, but it probably does not occur in many cases of anthropogenic water table lowering. It is significant that sinkholes can appear very quickly in response to groundwater table lowering. According to Tharp (1999), Bengston (1987) observed that almost all sinkholes associated with a 3-day drawdown episode occurred within a day to a few days. Formation of sinkholes during and after rains is common, often in association with other causative factors. According to Tharp (1999), Gertje and Jeremias (1989) describe sinkhole formation associated with a modest rain following a prolonged drought. Leakage from water supply or drainage pipes is also a common sinkhole cause.

It is significant that sinkholes form after relatively brief exposure to changes in hydraulic regime, but it is interesting that they also are precipitated by loading events of even shorter duration or smaller magnitude. Dynamic loading commonly causes sinkholes. Blasting is a common trigger for sinkhole formation. Vibrating equipment and vibro-compaction cause sinkholes, as can even small earthquakes.

2.3.3 Subsidence

Nature and Occurrence of Subsidence due to Tunneling

The occurrence of subsidence during the driving of near-surface tunnels, especially those connected with urban transport schemes is fairly common feature. The construction of subway schemes in major cities world-wide has resulted in measurable subsidence, some of which has been sufficient to cause appreciable damage to surface structures (Whittaker and Reddish, 1989).

Subsidence above such tunnels is related to the following main factors:

i. The type of ground between the tunnel and the surface
ii. The method of tunnel drivage, together with phasing and degree of permanent support.
iii. The dimensions of the tunnel
iv. The depth of the tunnel below the surface
v. The stress conditions

Ground Conditions

Shallow tunnels are usually driven in soft ground conditions that necessitate special considerations to be given to temporary support. The generally weak character of the tunnel gives rise to proneness for roof falls to occur, where a highly effective and efficient means of temporary support is employed and even where such well-designed support schemes exist, the occurrence of significant quantities of water can lead to cavities forming which could result in subsidence (Whittaker and Reddish, 1989).

According to Whittaker and Reddish (1989), Donovan Jacobs (1985) has discussed tunnel failures particularly in respect of ground collapses that resulted in subsidence holes appearing at the surface, and draws attention to experiences with a serious collapse encountered in the construction phase of a highway tunnel in Hawaii. In this latter case it was demonstrated that tunnel excavation in soft ground demanded substantially increased attention to safety precautions as compared to tunnels of equivalent size in solid rock. In the Wilson Tunnel's collapse water appeared to play a role in causing the tunnel roof instability to occur which led to fairly rapid appearance of these sinkholes at the surface and is a particular characteristic of collapse behavior of unconsolidated clay materials (Whittaker and Reddish, 1989).

Drivage Method

The method of tunnel drivage whether hand excavated, drill and blast or machine driven usually results in some delay between excavation and support application unless a shield allows complete elimination of this excavation-support delay phase of the drivage operation. Irrespective of how well tunnel drivages are planned and carried out, there is always a strong likehood that the operations themselves will allow the opportunity for some subsidence to occur if the other factors are favorable to its development. However in the majority of the situations the accompanying subsidence is likely to be of no significance in respect of its effect on the surface, where a major roof fall occurs in a shallow tunnel, there is a risk of collapse propagating and a conical depression or even a well-defined subsidence hole appearing at the surface (Whittaker and Reddish, 1989).

Excavation Size

Tunnel excavated dimensions play a major role in influencing the likehood of subsidence developing at the surface. The greater the tunnel width, the greater the imposition placed on the immediate roof to offer self-supporting ability. Since the tunnel experiences its largest convergence control becomes more difficult as the tunnel width increases. Effective application of temporary support measures decreases with increasing size of tunnel, and consequently calls for special care during the drivage of such tunnels in weak ground (Whittaker and Reddish, 1989).

Tunnel Depth

The depth below surface is a major influencing factor on the likehood of subsidence occurring at the surface above a tunnel. Clearly, if the tunnel is sufficiently deep, then even the effects of major roof falls may peter out before reaching the surface (Whittaker and Reddish, 1989).

Stress Field

The nature of the in-situ stress field can play a role in influencing the stability of near-surface tunnels and consequently their potential for the occurrence of subsidence. In most situations, however, the state of stress will be governed by the overburden loading and groundwater conditions thereby giving rise to a predominantly vertical stress field. In some circumstances high lateral stresses can exist by virtue of the geological history of the rocks through which the tunnel is driven and could give rise to stability problems in high tunnels. The conditions favoring high lateral stress fields are generally encountered in semi-competent and competent rock situations. High lateral stresses in shallow tunnels can result in premature failure of natural beams of rock spanning recently excavated tunnels; this can lead to surface subsidence occurring unless tunnel support is effective (Whittaker and Reddish, 1989).

2.3.4 Slope Tunnels

Precautions and supporting techniques of tunnels, excavated in soil or rock formations, differs due to the stipulations of the surrounding. How inconvenient the stipulations are, so influential would be the problems faced both in designing and construction stages. A tunnel passing through a slope of a hill, or with it is known name slope tunnel is one of these conditions presented by nature (Vardar et al., 2002).

It is a known fact that the stability of the tunnel would be affected in a negative manner if the ground thickness over it were not enough. Similarly, if the thickness of rock cover is not enough on one side, slope tunnels cannot maintain their stability anymore and unload their inner energy, which will cause large deformations. This condition necessitates additional supporting and improvement works. To prevent excessive deformation of the excavation hole, special supporting elements should be used and a detailed monitoring program must be used to secure stability (Vardar et al., 2002).

The most of the important theme of slope tunnels is the thickness of the pillar between the tunnel and slope of the hill. As the pillar gets thicker, the safer the tunnel be excavated (Vardar et al., 2002).

2.4 Rehabilitation and Improvements of the Tunnels

Rehabilitation and improvement of the tunnels during the work of the stabilization of the problematic tunnels, some kinds of techniques are used. In this part of the literature survey these kinds of case studies are studied and explained in the following order, namely;

2.4.1 Shotcrete, Bolting and Mesh

Remodeling and Reconstruction of Tunnels

Because of additional clearance requirements of electrification of railway lines and the construction of additional tracks, a tunnel carrying such lines has to be reconstructed. Although less frequently, a tunnel may also have to be reconstructed because of damage caused by subsidence, breakdown and excessive blasting (Szechy, 1973).

Reconstruction for Operational Demands

In order to accommodate a second track, the tunnel section may be widened on both sides, in which case the existing lining is not to be demolished until the new one has been completed. This method is most practicable when working under poor soil conditions and considerable pressures. Work can be started from a top heading above the crown line, with excavation proceeding from the top and then constructing the lining in an uninterrupted sequence from the bottom or from drifts at the bottom of the section. In the latter case the new lining must be constructed in lifts following the excavated height of the drifts. The first method is limited to firm soil conditions, the second can be used in poor soil as well (Szechy, 1973).

It is also possible to widen the tunnel section to one side. This is feasible only in grounds having considerable strength; the existing arch has to be supported temporarily at the joint with the new lining (Szechy, 1973).

The enlargement and remodeling of underground tunnel profiles is a much more difficult task owing to usually difficult groundwater and soil conditions and to the restricted area in which traffic is to be maintained without interruption (Szechy, 1973).

Old double track tunnel of the City and South London was enlarged to a width of 9.0 meters from 7.5 meters, new tunnel in the relatively reliable London-clay where no ground water was encountered. The annular space around the circumference was excavated by hand and the new lining segments could be placed one by one through the opening left by the demolished and removed elements of the tunnel while the track was supported step by step by the new concrete pillars placed on the new invert. Rigid ties offer a great help not only in assuring rigidity of the partly demolished and incomplete lining rings, in offering supports for various working platforms (Szechy, 1973).

A shotcrete lining provides a surface, the steel ribs acts as linewise support, whereas rock bolts are effective in a pointwise manner. Steel ribs combined with a shotcrete lining seem to be attractive for two main reasons:

i. The erection of a steel rib requires only 15 to 20 minutes. The space between the flange and the rock is wedged or shotcreted immediately

ii. The spraying of the shotcrete lining takes more time and the curing of the concrete also needs 5 to 8 hours to attain sufficient compressive strength.

If the shotcrete lining is acting only as a membrane (theoretically), i.e. subjected only to normal forces and not to bending moments, its load-bearing capacity is extremely high. Unfortunately, two factors are in play rendering the shotcrete lining almost useless under the conditions of squeezing ground behavior (Kovari and Status, 1996):

i. Due to its high stiffness shotcrete cannot accommodate rock convergence to obtain relief of rock pressure

ii. Due to its lack of tensile strength the shotcrete lining fails for even small excentricity of the normal force, i.e. in the presence of bending moments

While special types of steel ribs can be connected with each other to form a yielding joint and despite retaining a controlled load bearing capacity the shotcrete will just fail. In order to overcome this difficulty it was proposed to include longitudinal slots in the shotcrete at the location of the yielding connections of the steel ribs. The results of this measure are three-fold (Kovari and Status, 1996):

i. No fracture of the shotcrete lining takes place.

ii. The shotcrete will bridge the short distance between the ribs.

iii. The shotcrete lining entirely loses its lining resistance contributing nothing to stabilize the rock

Shotcreting should be performed as promptly as possible as the preliminary support, the principal purpose of which is to extent the stand-up time of the ambient rock itself, thereby permitting the construction of the performing, i.e. secondary, support (Zhen-Yu, 1986).

2.4.1.1 Examples of Rehabilitated Tunnels

Malatya-Narlı No:7 Railroad Tunnel (Divleli and Unal, 2005)

Divleli and Unal (2005) presents a case study associated with the rehabilitation of the Malatya-Narlı No:7 Railroad Tunnel. The rehabilitation work includes cleaning of two collapses and stopping of deformations occurring in the tunnel as well as characterizing the rock-mass by evaluating the cores obtained from 50 drill holes.

Tsukayama Tunnel (Asakura and Kojima, 2003)

Tsukayama tunnel was constructed in 1967, with a total length of 1766 m, as a double-track railway tunnel. The thickness of lining is 0.50 m and invert concrete was not installed.

Shortly after completion, the tunnel deformed in such a way that both sidewalls were pushed into the tunnel space accompanied by mud-pumping at the tracks and heaving at the base. The arch crown was pushed upwards and failed in a bending and compressive manner. As a countermeasure against this deformation, invert concrete was installed to close the tunnel section.

In 1990 a wide area of the crown settled where compressive failure had occurred and many shear cracks developed in a radial manner with their center at the settled part. The possible collapse of the area was a serious concern. Thus, preventative work against a collapse of the area was taken immediately as a temporary measure. As permanent countermeasures, backfilling, rock bolting, reinforcement and shotcreting with steel fibers were applied, as shown in the Figure 2.11.

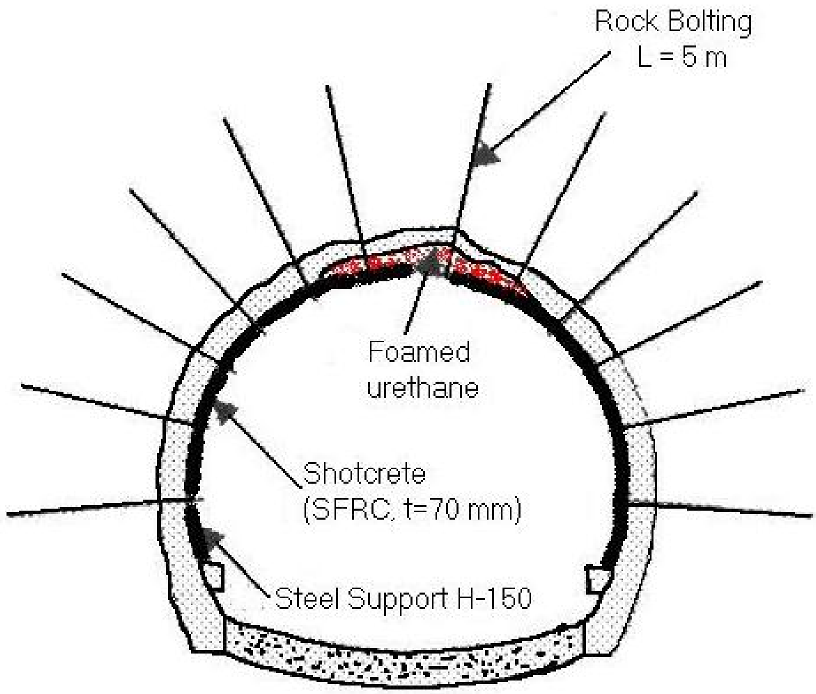


Figure 2.11 Outline of the Countermeasures in Tsukayama Tunnel
(after Asakura and Kojima, 2003)

<u>La Nerthe Railway Tunnel</u> (Nasri and Winum 2005)

Nasri and Winum (2005) explain rehabilitation of the La Nerthe Tunnel on the Paris-Maseille High-Speed Railway Line. This 4638 m long double track tunnel was built at the middle of 19 th century under a maximum cover of 180 m and currently is used as part of the high speed railway line (TGV). This masonry tunnel with hard limestone blocks at the sidewalls and bricks at the crown passes through marl, gypsum and limestone zones. A major fault with accompanying extensive tectonic deformations cuts across the tunnel axis. The in situ stress tests performed on the new high-speed rail line in the zone with intense tectonic fractures, tectonic residual stresses with very high lateral earth pressure coefficients. Nerthe tunnel underwent ovalization deformation with excessive displacement at the springline and pinching and bursting of the bricks at the crown. Nasri and Winum (2005) describe the geological context, damage to the tunnel lining as well as the rehabilitation system and the constraints of work.

The Sidi Mezghiche Tunnel (Panet, 1996)

During the excavation of the tunnel, large convergence of the walls of the tunnel were measured; displacemensts were typically around 0.10 m and reached up to 0.25 m in the most unfavorable sections. The convergence of the section was measured in an unstable section and they increased almost linearly with time. And finally rock-fall occurred. Before the collapse, the support of the tunnel consisted of H steel sets (HEB 160 according to ANFOR code) at 0.80 m spacing with a theoretical 0.15 m thickness of shotcrete; in fact the ring of shotcrete was thicker. The invert was reinforced by steel sets (TH 36/48 according to the ANFOR code). The final 0.40 m thick concrete lining was not installed. After the collapse, several finite element models based on the convergence-confinement method were studied in order to explain the instability. Some shear tests were carried out to try to determine the residual shear strengths. From these back-analyses the geotechnical characteristics in Table 2.1 were obtained.

Table 2.1 Geotechnical Characteristics of the Material

Geotechnical Characteristics	Short term	Long term
Deformation modulus (MPa)	100	50
Cohesion (kPa)	90	0
Angle of internal friction	17	10

For the natural state of stress, horizontal to vertical ratio (K_o) value was taken equal to 1.0. Then, the shape of the cross-section was modified. The support consisted of H steel sets (HEB 200 according to AFNOR code) at 0.80 m to 1.0 m spacing and 0.40 m of shotcrete. The invert was closed immediately after the excavation. The thickness of the reinforced concrete lining was 0.65 m.

2.4.2 Rock Mass Improvement

According to Pelizza and Danielle (1993) soil and rock improvement and reinforcement techniques are applied to permit safe tunnel excavations in difficult geological conditions. These techniques are used to modify the stresses around the underground works and the geotechnical parameters of the soil. The main methods used in Italy and abroad are:

i. Ground reinforcement
ii. Grouting
iii. Jet grouting
iv. Ground freezing
v. Mechanical precutting

Each intervention involves a different approach to the weak soil tunneling problem, because each modifies one of the different parameters involved within different technological solutions. The choice of the technique depends on the soil geotechnical characteristics and on the economical balance between the cost of the intervention and the benefits it can offer. With regard to their effect on the soil, the interventions can be divided into two groups:

1-Interventions that improve the soil
2-Interventions that preserve the soil.

2.4.2.1 Soil Improving Intervention

The following interventions produce an improvement in the geotechnical characteristics of the soil around the excavations (Pelizza and Danielle, 1993).

Ground Reinforcement

It is a lasting intervention that is carried out by introducing structural elements more resistant and rigid than the soil (e.g., bolts, steel cables and steel or fiberglass pipes) into the ground itself (Pelizza and Danielle, 1993).

Grouting

A lasting intervention that produces its effect by injection of grout mixes into the soil, at a 20-40 bar pressure. The mixes fill the voids of the soil, thereby reducing the permeability and improving the soil geotechnical parameters. The grout mixes can be cement or chemically based (Pelizza and Danielle, 1993).

Jet Grouting

It is a lasting intervention that is carried out by injecting cement-based grout mix into the soil at a high pressure (over 200 bars). Different injection layouts have been developed. The final result is a column of treated with better geotechnical characteristics than the ground itself (Lunardi 1992, in Pelizza and Danielle 1993).

Drainage

It is a lasting intervention that can be applied when water is the main problem. Drainage system is carried out by reducing the water pressure and controlling the water inflow with drains of different shafts and topologies (Vielmo 1991, in Pelizza and Danielle 1993).

Freezing

It is a timely intervention that can be applied when the soil is saturated with soil. The aim of this technique is to excavate the tunnel in frozen ground, which is stable. When the final support is in place, the freezing action is stopped (Gallavresi 1991, in Pelizza and Danielle 1993).

2.4.2.2 Soil Preserving Intervention

These interventions prevent the parameters of the soil from falling to the residual values; they allow little disturbance to the stresses around the excavation. The technological aim of these interventions is to reduce the deformation as much as possible. Some preserving interventions are briefly described below.

Cellular arch

It is used for short and large underground excavations. A supporting structure is made before the excavation is carried out. This technique can be successfully applied in cases where the coverage is so thin that it does not permit the use of other supporting techniques.

The principal structure support of this system is composed of large diameter pipes inserted into the ground around the crown and parallel to the longitudinal axis of the tunnel. The pipes are then filled with concrete and joined with transversal elements (Lunardi et al., 1991; Focaracci 1991, and Pelizza 1991, in Pelizza and Danielle, 1993).

Advance Precutting

It is a lasting intervention that is carried out with the excavation of a preventative “tile cut” around the tunnel, using a chain-saw blade machine. The cut is filled with a steel–fiber-reinforced concrete lining of high mechanical characteristics (Lunardi 1991, Arsena et al., 1991, in Pelizza and Danielle, 1993).

Jet Grouting Arch

An arch of sub-horizontal columns is carried out at the crown of the future tunnel. Subvertical columns are often placed in the walls of the tunnel. The arch acts by supporting the soil during the excavation and by homogenizing the stresses on the final supports (Pelizza and Danielle, 1993).

Steel Pipe Umbrella

An umbrella of steel pipes with a truncated cone shape is constructed above the crown of the future tunnel (Carrieri et al., 1991, in Pelizza and Danielle, 1993).

Preconsolidation of the Excavation Face

A timely intervention carried out by inserting fiberglass pipes on the face. This method is often coupled with a precut or pipe umbrella. This intervention acts both by reinforcing the ground to be excavated and by preventing movement of the face. These last two interventions have both an improving and a preserving action (Pelizza and Danielle, 1993).

2.4.2.3 Examples of Soil and Rock Improvement Techniques in Tunnels

Grouting

In Pelizza and Danielle's paper (1993), information is given on grouting application. A double-track tunnel with low overburden was excavated having a length of 430 meters and excavation diameter of 8.8 meters. The rock mass consisted of silty sand and sandy gravel. Problems experienced included incohesive soil requiring prevention in order to stop the movements of the foundations of the buildings at the surface. Intervention made in the tunnel includes grout injection carried out from the pilot tunnel. The grout mixture consists of both cement and suitable chemicals. Figure 2.12 shows the grout application pattern.

The following steps were adopted in the operative scheme:

i. The pilot tunnel was bored using a 3.2 m. diameter TBM.
ii. The grouting was carried out by radial injection
iii. The upper half section was excavated and supported with steel arches
iv. (1 m spacing) and shotcrete (0.20 m thick) with wire mesh.
v. The lower half section of the tunnel was excavated.
vi. The invert arch was carried out and the final lining was cast.

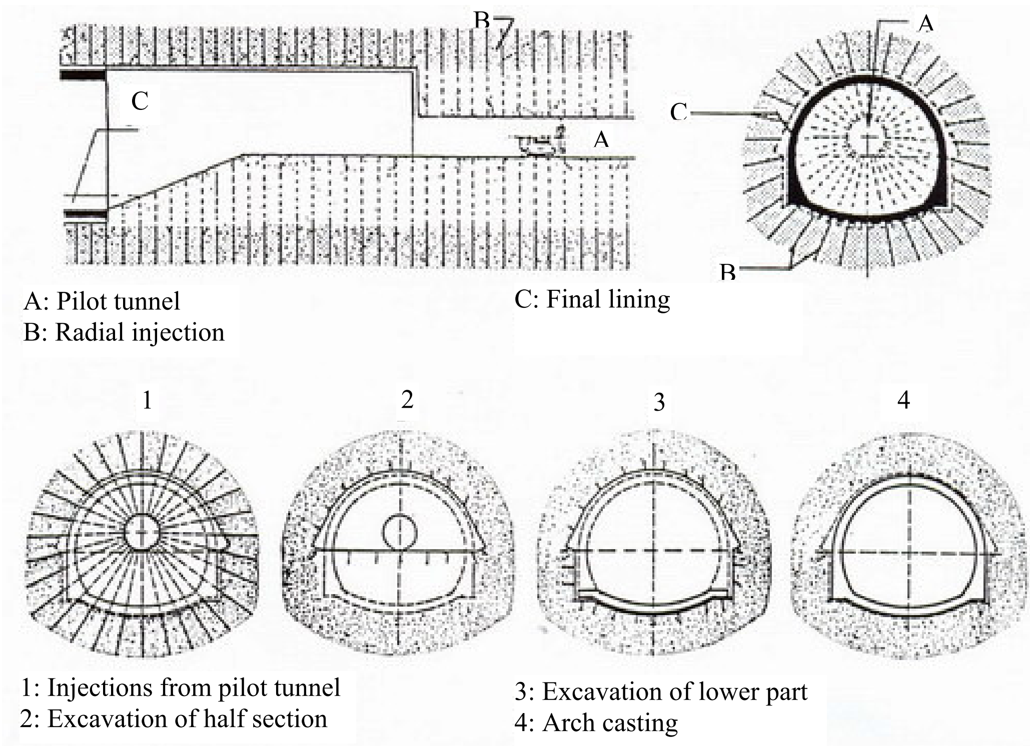


Figure 2.12 Example of Grout Injection Carried Out from the Pilot Tunnel (after Pelizza and Danielle, 1993)

Mechanical Precutting

In Pelizza and Danielle's paper (1993), it is described that a railway double track tunnel having a maximum overburden thickness of 50 meters and diameter of 12 meters was excavated. The rock mass consisted of clay, sand and sandy clay. Problems experienced during excavation included large deformations occurring due to swelling of clay as well as large face deformations. Intervention made in the tunnel included a 3.5 meters long precut, using steel-fiber-reinforced concrete in a tile shape. The face was reinforced with fiberglass pipes. The operative scheme adopted involved the following steps:

i. The precut was carried out.
ii. The previously excavated tile was supported with a steel arch.
iii. The tunnel was excavated (3 m of advance)
iv. After a face advance of 10 m, the inverted arch and the face reinforcement were carried out.

Pipe Umbrella

In Pelizza and Danielle (1993), it is also described that in a road tunnel a pipe-umbrella intervention was used to improve the stability of the tunnel. The tunnel was excavated at a depth ranging from 2 to 70 meters from the surface. The cross sectional area of the tunnel was 90 m^2. The rock mass consisted of chaotically mixed moronic and detrital deposits. Problems experienced included instability of both the tunnel and the face. Intervention made in the tunnel included an umbrella of steel pipes with a truncated conic surface 12 meters long. The excavated length per step is only 9 meters. Thus, there is a 3-meters-long superposition between two consecutive umbrellas. This intervention provides a guarantee against collapse during the excavation, and homogenizes the deformative behaviour of the tunnel by redisturbing the stresses of the supports. Figure 2.13 shows the example of pipe umbrella intervention to improve the stability of the tunnel and the face of a road tunnel.

The operative scheme consisted of:

i. Placing and injecting the pipes
ii. Reinforcing the face with fiber glass pipes
iii. Excavating the upper half section
iv. Putting in place the steel arches and shotcrete (repeated for 9 m)
v. Supporting the steel arch feet with micropiles
vi. 6-Excavating the lower part of the tunnel
vii. Casting the inverted arch and the final lining

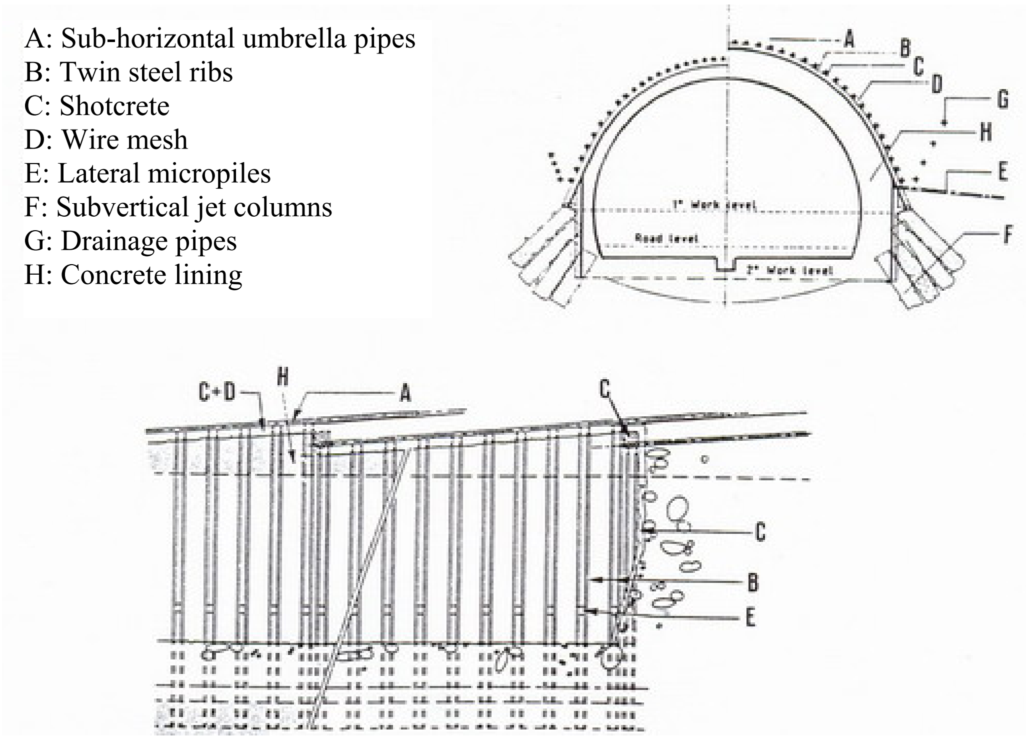


Figure 2.13 Example of Pipe Umbrella Intervention to Improve the Stability of the Tunnel and the Face of a Road Tunnel (after Pelizza and Danielle, 1993)

Kızlaç Tunnel (Etkesen et al., 2002)

Initially, capacity of support applied previously for crown and walls of first collapsed area was controlled and the cavities above the tunnel verified. Grout is applied around first collapse area.

At the area affected by the collapse, a self-supporting arch with thickness of minimum 6 m was established by grouting. The length of arch along the tunnel profile was 18 m.

The area above the tunnel crown affected by the first and second collapse were investigated by core drilling (vertically from ground surface) for voids. Open voids found with in a maximum height of 30 m above the tunnel were filled.

Because rock mass was weathered sandstone, siltstone and shale, grouting achieved success at the fault zone and outside fault zone.

The aim of grouting was to form a consolidated arch in the loose material above the tunnel crown with a minimum thickness of 6 m, in order to be able to advance through the collapsed area. The first grouting sequence covered a length of 18 m. But according to data obtained from forepoling and bore drillings some additional grout works were done where required.

Grouting created and injected area to prevent excessive flow in the tunnel. Middle of the tunnel was grouted from side drifts in order to form a grouted body.

Quality and efficiency of treatment were investigated. For this purpose, core drilling, check of grout intake and rock bolt pullout tests were carried out. Controls were done to see if there exist any cavities or loosened ground laterally or vertically.

The Kızılay Tunnel (Özaslan et al., 2002)

A 120 m long stretch of the Kızılay tunnel is formed of alluvial deposits represented by the mixture of clay, silt, sand and gravel. Soil improvement was recommended over this section in order to contribute to the tunnel stability through stabilizing the poor soil and to maintain safe working conditions. Jet grouting was employed for soil treatment since this method was relatively rapid and practical.

Soil improvement was made in a 120 m long stretch of the Kızılay tunnel passing through the alluvial deposits and existing below the groundwater table.

The crown and sidewalls of the tunnel are stabilized by means of grout columns. Curtain walls intersecting the tunnel route at right angles were constructed at an interval of 25 m in order to minimize the amount of water leaking form the space between the grout columns and to maintain the stability of the tunnel. Grout columns were made about 0.80 to 0.85 m in diameter.

The most significant problem encountered in the course of the jet grouting application is the displacement-taking place in the neighboring buildings as a result of the high pressure grouting since the tunnel route passes through densely populated areas.

These displacements take place when the grout pressure finds a suitable path and spreads through the sandy levels within the alluvium and finally reaches some locations of porous medium, which can be controlled by no means. The jet grouting is applied through one of the three holes drilled at the corners of an equilateral triangle with sides 0.80 m long. In order to avoid the displacements, two holes were drilled and left as relief holes prior to the grouting procedure.

Selatin Tunnel (Bizden and Murat, 2002)

After pumping out of the water from the collapsed part of the tunnel, face covered with wire mesh and shotcrete applied. Later on rock bolts were placed. The roof of the tunnel, emptied as a result of the collapse, was filled with concrete and cement injection. Perforated pipes were driven into periphery and cement was pumped in. Thus, the peripheral rock mass were stabilized. As a result, it became possible to have a peripheral ground of about 10-15 m stabilized with concrete and cement around the tunnel. The distance between supports decreased to 0.75 m and twin W4x13 profiles have been used as supports. In one round 8 m, for the following round 12 m 15 PG rock bolts were used.

Samanalawewa Tunnel (Hagenhofer, 1990)

The geology of the Samanalawewa site consists of entirely of metamorphosed crystalline precambrian rocks. Sandy/gravelly areas were densely compacted, the clayey/sandy silt was generally stiff to firm, with uniaxial compressive strengths estimated in the range of 60-80 kPa. However, in contact with water the soil underwent a rapid decrease in material strength, becoming soft or very soft, flowing ground. There was no support class defined to cope with soft ground conditions. Therefore, the work cycle had to be changed completely to implement the new soft ground tunneling concept. In this method, regular holes were drilled to relieve water pressure in cases of excessive pore water pressure. Weak and running rock masses were stabilized by applying shotcrete. The convergence quickly reduced with the application of the new system. No further collapses occurred after the introduction of the new system.

Kurtkulağı Tunnel (Türkmen and Özgüzel, 2003)

In the Kurtkulağı irrigation tunnel several problems occurred during the tunneling. Within 70 m long section of the tunnel, collapses occurred at two locations both occurred along the fault zone in tunnel alignment. A sinkhole was formed at the surface as a result of collapse. Collapses in the tunnel filled by concrete and the sinkholes were also filled with gravel as the muck was being removed.

Filling was continued until the sinkhole was completely filled with gravel. Then, the two sides of the collapsed area were closed with brick walls and grouting was carried out from the surface. Long (L=1.5 m) grouting pipes were installed in gravel. The grouting mixture is made of 7/5 (cement/water)+50% sand +2-4% bentonite.

Jiulongkou Coal Mine (Song and Lu, 2001)

Some of the supports in the permanent access tunnel of the Jiulongkou Coal Mine had failed and large deformations were observed as shown in Figure 2.14.

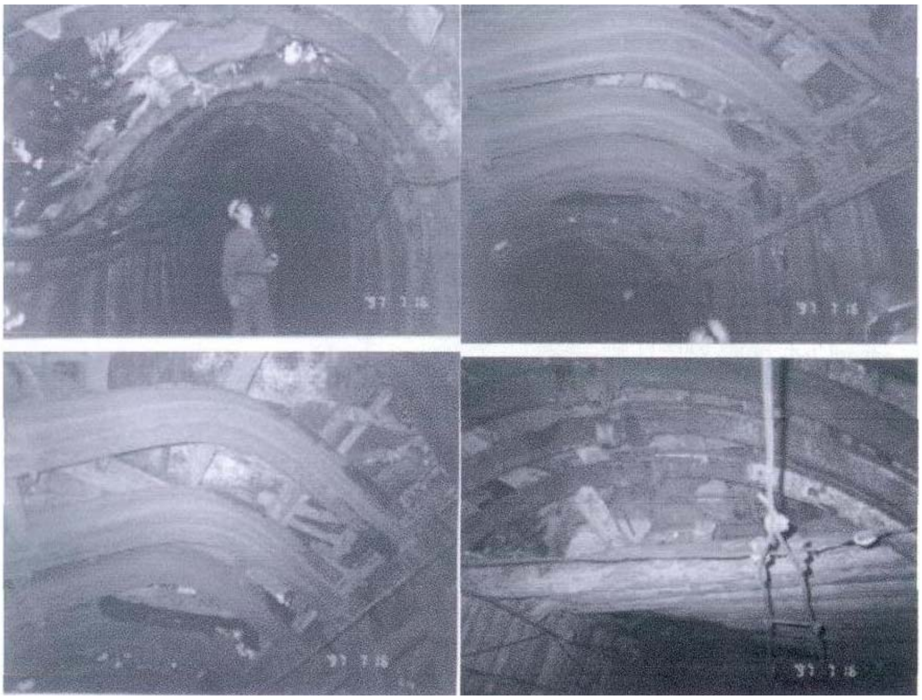


Figure 2.14 Some Failed Tunnels in Jiulongkou Mine (after Song and Lue, 2001)

In recent years, the supports of the southern main access tunnels were repaired with steel sets and timbers. The standing duration was only approximately 12 months for steel sets and 3 months for timbers after each repair. Therefore, it was decided to perform repair of the deformed tunnels with a new approach.

According to the studies, it was concluded that the larger the enlargement, the greater the further dilatancy deformation, and the more difficult stabilizing the tunnel will be. The support measure, such as rock bolt, mesh and shotcrete are adopted based on above concept in repair of the deformed tunnel.

At the Jiulongkou Mine, the rock masses are generally soft rocks, and therefore development of the loosening zone takes a considerable time. Therefore, supporting the deformed tunnel was implemented in 2 stages. In the first stage, flexible support was applied to accommodate the dilatancy deformation during development of the loosening zone. In the second stage, stiff support was applied to provide a strong supporting reaction and to maintain the long term stability of the tunnel.

Based on their engineering practice for the soft rock at the project site, when the advance distance is 0.6 and 1.2 m roof collapse can be avoided during enlarging and repairing.

Steel bolts, shotcrete and mesh were used as the supporting measures. Split-tube steel bolts were applied on the roof and sidewalls. The bolt spacing was 0.5 m and the bolt length was 1.8 m.

The total thickness of shotcrete applied was 120 mm on average and was sprayed as 3 layers. The final layer was 50 mm in thickness and was sprayed after the surrounding rock mass became stable.

The total thickness was used together with shotcrete to increase the tensile and bending strengths of the shotcrete.

Dorukhan Tunnel (Akçelik et al., 2002)

Dorukhan Tunnel's concrete lining and pavement were heavily damaged; illumination and ventilation systems could not be established due to the lack of waterproofing. Vertical area also was not enough.

Tunnel location is near North Anatolian Fault Zone and was driven in phyllite-metasiltstone and granodiorite which are poor and weathered rock.

Boreholes were driven at the bottom of tunnel to check the rock conditions during the rehabilitation. Since rock at the tunnel bottom was highly weathered it was decided to form a grouted zone, as shown in Figure 2.15 with a depth of approximately 2.5 m at tunnel bottom along the tunnel.

In order to evaluate the performance of grouting, trial of grouting was performed in 3 sections having different characteristics.

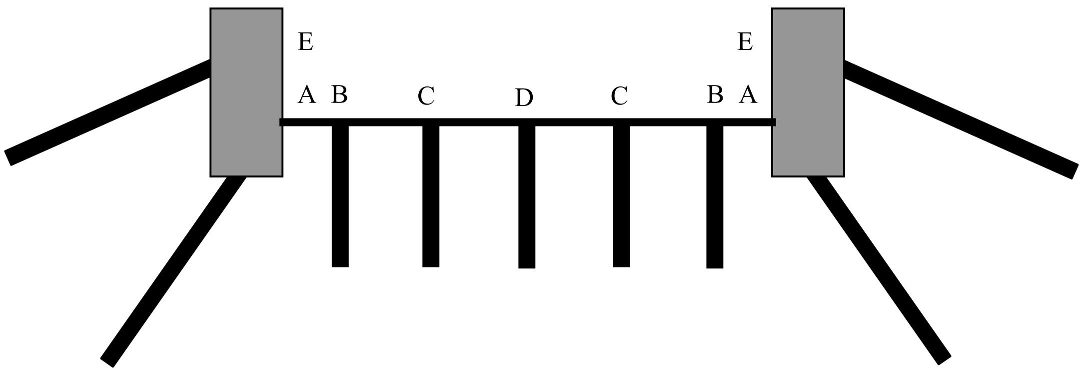


Figure 2.15 Section of Grouting Holes at the Bottom
(after Akçelik et al., 2002)

A: Grouting inclined from vertical, located at the bottom of the tunnel (L=4.83)

B: Grouting perpendicular to the bottom of the tunnel, located 1.0 m apart from the wall of the tunnel

C: Grouting perpendicular to the bottom of the tunnel, located 2.6 m apart from the wall of the tunnel

D: Grouting perpendicular to the bottom of the tunnel, located at the tunnel axis (L=3.1 m)

E: Grouting 65° inclined from vertical, located 0.50 m above the tunnel bottom (L=3.1 m)

It was concluded that by grouting a reasonable improvement at the tunnel bottom was maintained. The depth of grouting at the tunnel bottom was increased in order to obtain a deeper grouted ring in highly weathered rock conditions.

Waterproofing and shotcrete application were performed along the tunnel. Shotcrete was applied on steel mesh which was reinforced with ø12 mm bars bent in radial direction with 0.2 m spacing.

<u>Kinoura Tunnel</u> (Kunita et al., 1994)

The Noto Peninsular Offshore Earthquake occurred off the coast of Suzu City, Japan. As a result of the earthquake, the Kinoura Tunnel was critically damaged by a collapse of the arch and forced out of service. Because the width of the excavation in the collapsed zone was greater than the width of the existing tunnel, excavation was carried out in the sequence as shown in Figure 2.16. During the excavation of the enlargement zone it was found that the rock mass deteriorated considerably.

The original lining was removed with a large breaker in 1-m steps, leaving center core at the face. During the excavation, the ground above the arch fell and the face in the fallen material collapsed. Therefore, forepoling (l=3 m), shotcreting were adopted for face stabilization.

Mortar grouted dowel rockbolts were mainly used, although self-drilling bolts also were used where ground condition were bad.

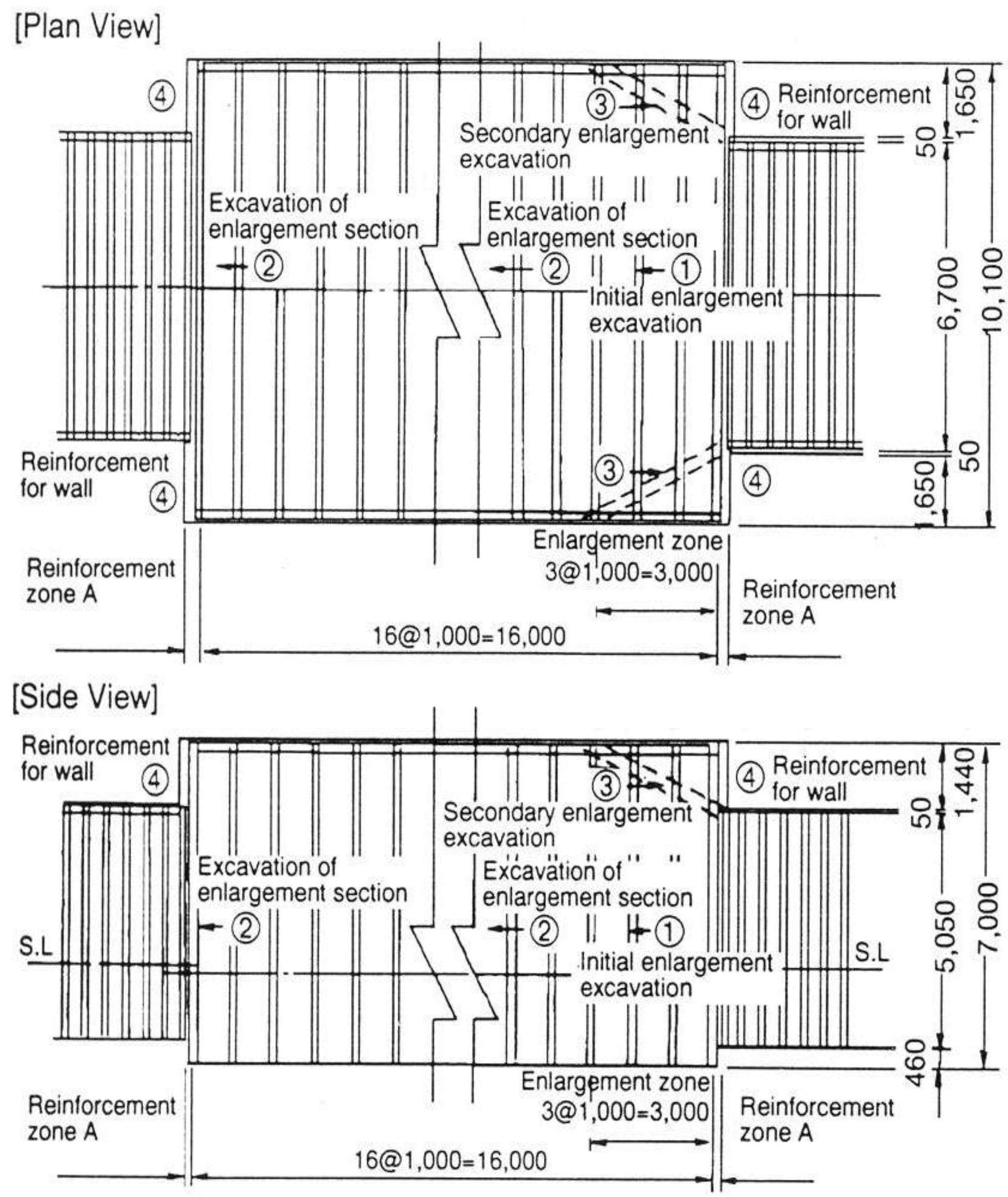


Figure 2.16 Phased Excavation of the Collapsed Zone
(after Kunita et al., 1994)

2.4.3 Regeneration of Lining

The deterioration of the tunnel lining is generally considered to be an effect of aging. In fact, however, it is often related to water leakage. External factors such as toxic water, frost damage, salt damage and smoke pollution are also operative in some cases. Internal factors include the deficiency of the materials, such as the lack of cement in the concrete, the insufficient filling of the joints, or in special cases, deterioration due to an alkali-aggregate reaction (Asakura and Kojima, 2003).

In repairing a deteriorating lining, countermeasures are selected based on the degree and the causes of the deterioration. It is also necessary, however, to take into account the locations of the deterioration, the likehood of the drop of lining tips, and the degree of the critical margin (Asakura and Kojima, 2003).

After an extensive survey of the Negresse railroad tunnel it was decided that the arch, uprights and floor would have to be repaired in addition to a straightening of the track and an increase in the gauge (Andrews, 1982).

Work on the tunnel commenced with a preliminary lining of shotcrete 0.07 m deep. This would afford general protection of the site. The shotcrete was reinforced with a sheet of welded mesh. Once this had been completed systematic injections of the extrados took place. The injections consisted of a mixture of cement, fine sand, bentonite and water. It was hoped that they would generate the lining of the masonry as well as ensuring water tightness. The entire length of the tunnel was treated through drill holes at a pressure of only 2 bars. A drainage system was also incorporated into the walls of the tunnel. This consisted of a number of aureoles in the walls of the tunnel which drained into a common central canal in the floor (Andrews, 1982).

The roof of the tunnel was reinforced with conventional cast-iron arches. The cast iron arches were placed in groups of four or five pre-assembled elements provisionally maintained in place by straps and expanding anchor pins. The completely assembled ring is finally anchored by 1.5 m long pins and secured by injections of mortar after plugging (Andrews, 1982).

The arches were placed in conjunction with the reinforced concrete uprights. The operation was split into two parts, each upright was put in place together with half of the arch, the work was then switched to the other side of the tunnel and the same repeated. Once this had been achieved the arch was finally anchored. The arches were placed in groups of four or five pre-assembled elements, provisionally maintained in place by straps and expanding anchor pins. The completely assembled ring was finally anchored by pins 1.5 m long and fixed injections of mortar after plugging. Once this had been achieved the new floor was placed with a new track (Andrews, 1982).

During the 90 years, various sections of the tunnel had been repaired with a cast in place concrete arch. In most sections this appeared to be over existing timber sets. Most of the concrete arches had visible voids and punky areas where the concrete had decayed and now showed the remains of corroded rebar. These concrete arched sections were repaired in three steps. First, steel c-channel arches were bolted into place on 1.2 m centers. Next the steel arches and concrete were covered with 0.08 m of shotcrete. Finally the voids behind the concrete were filled with cellular concrete grout. Blocky areas were bolted. The more sheared areas were supported with a combination of rockbolts, mine straps and shotcrete (Goss, 2002).

The detailed inspection of the tunnels revealed weaknesses in places in the arching and behind the linings. Some pressure resulted in distortion occurring to some of the masonry arches within the tunnels. Cement grout was injected under low pressure to fill the voids and weep holes were made. The old mortar was blown out by compressed air and the new forced in under high pressure (Goss, 2002).

After the void was concreted, the underlying muckpile was grouted and stabilized. Finally the remaining adjacent lining in both tunnels was cavity grouted and fissure grouted and 7 m long rock bolts supplemented with fully grouted dowels installed between both tunnels (Goss, 2002).

Soft Ground

According to Szechy (1973) the rock mass can be divided into four classes namely: running rock mass, soft rock mass, firm rock mass and self supporting rock mass. The running rock mass may consists of dry sand or gravel and should be supported instantly. The soft-rock mass roof should be supported instantly however the walls could be stable for a short period of time. In the firm rock mass the roof could stay unsupported for a short time however the side walls and the face could be stable for a longer period of time. Finally, the self-supporting rock mass can stand unsupported while the entire tunnel is driven a few meter ahead of the supports.

The standard methods of driving through soft ground are forepoling with wood or steel, or working in a shield. Plenum method is keeping out soil and water with air pressure, with either forepoling or shield (Szechy, 1973).

Forepoling: The use of plank forepoles was formerly the standard method of driving a tunnel through soft ground. While this technique has been largely replaced by steel liner and poling plates, it is still widely on jobs too small to justify obtaining steel (Szechy, 1973).

In forepoling, the tunnel is protected by timbering and by breast board set against the face. Planks are driven through slots cut in the breast board and supported cantilever fashion to make a temporary roof, under which dirt can be dug and permanent supports installed (Szechy, 1973).

2.4.4 Tunneling Through Running Ground by Forepoling Method

Material with no cohesion, such as clean sand or gravel is commonly referred to as running ground regardless of whether it is located below or above the water table. In running ground, excavation or opening of collapsed part of the tunnel requires use of the ancient forepoling or one of its modern equivalents (Szechy, 1973).

The boards which are driven ahead to support the grounds ahead of the last rib are known as spiles. They act as cantilevers which carry the weight of the ground until their forward ends are supported by installing the next rib (Szechy, 1973).

The spiles are installed as far down around the sides of the tunnel as necessary. Then the top breast boards are removed, the exposed ground is excavated and the breast boards are reinstalled ahead. This process is continued until the excavation arrives at floor level, where upon the next rib can be installed. At some time afterwards the tails of the spiles are cut off (Szechy, 1973).

Tunneling in Raveling Ground

The term raveling ground indicates a ground which can stand up for at least two or three minutes when an opening approximately 0.91 m wide by 0.41 m forward is made. However, after this period, it may start to scale off. Slightly cohesive sand belongs in this category. In the past tunneling through this kind of ground was done by means of the forepoling method, which is slow, difficult and expensive. Today, the more expeditious and more economical liner plate method is generally used (Szechy, 1973).

The installation of liner plates starts at the top center. The top breast board is removed and an opening just large enough to admit liner plate, usually 0.41 m is scraped out and the liner plate immediately showed up against the ground. Its rear flange is bolted to the preceeding course and a block is placed under its leading edge. The breast board is reinstalled 0.50 m ahead (Szechy, 1973).

Umbrella Arch

Umbrella Arch is usually obtained by means of a series of holes, hole diameter ranges between 100 and 180 mm, that are drilled, even spaced, along the contour of the tunnel vault in such a way to form half cone. Similar holes can be drilled, if needed, also along the contour of the sides of the tunnel cross section (Kadkade, 2002).

In the holes are then placed steel pipes of suitable diameter, perforated along the lower side; grouting agents are pressure injected in the pipes (Kadkade, 2002).

The umbrella combines the advantageous of modern forepoling system and of a series of grouting injections. It allows the excavation to progress with a full control of subsidences through heterogeneous rocks having poor geomechanical properties as alluvial deposits, moraines, mylonites. A limit to the useful application of the method is posed only by groundwater under high pressure (Kadkade, 2002).

2.5 Stability Measurements of the Tunnels

Convergence Measurement

The usual laboratory and in-situ tests proved to be insufficient to provide reliable data at the scale of the rock mass. Limited technique is available to determine the natural state of stress, the knowledge of which would be essential for any model. The monitoring of convergences during face advance and the back-analysis of the measurements appear to be the most reliable approach to provide an overall assessment of the behavior of the rock mass (Panet, 1996).

Frejus Tunnel (Panet, 1996)

During the excavations of Frejus tunnel, large convergences occurred in the direction orthogonal to the plane of schistosity. These convergences were measured with a distometer on about 150 profiles. For each profile, the initial measurement was carried out close to the face (from 2 m to 5 m), and the convergences were measured regularly during 3 to 4 months until placing of the concrete lining. The convergences measured were over 0.10 m on many sections; the maximum value measured for the final convergence was about 0.50 m.

The convergences of a tunnel are to be analyzed taking into account the immediate convergence due to the advance of the face and the time-dependent convergence due to the rheological behavior of the rock mass. From the detailed analysis of the convergences, the following formula was established for the convergence law in terms of the distance x between the section and the face and time t:

$$C(x,t) = A_1 f(x)(1+A_2\, g(t)) \qquad [2.1]$$

For various tunnels, a good agreement is obtained with the following functions:

$$f(x)=1-(X/(X+x))^2 \qquad [2.2]$$
$$g(t)=1-(T/(T+t))^n \qquad [2.3]$$

Where x is convergence and t is time. The parameters of this convergence law are:

i. X which characterizes the distance of influence of the face of the tunnel; in the elastic domain, X is equal to 0,375B, B being the width of the tunnel section and when there is a plastic zone around the tunnel, X is proportional to the extent of the plastic zone.

ii. T and n are constants depending only on the rheological behavior of the rock mass.

iii. A_1 is the convergence observed for the initial rate of excavation

iv. $(1+A_2)$ is the ratio between the final convergence and A_1 ; it characterizes the importance of the time-dependent deformations.

In order to stabilize the convergences, the tunnel was supported by rockbolting and a strong mesh to confine the loose rock between the bolts. The most extensively used type of rockbolt was ϕ 20 mm, 4.65 m long anchored rockbolts. In the most critical areas, the density of bolting was 1.2 bolt per square meter and various devices were used to increase the yield strain of the bolts and to prevent their failure. The rate of convergence varied between 0.1 mm/day and 0.5 mm/day after installation of concrete lining (Panet, 1996).

Significant studies have also been carried out on convergence measurements and interpretation of the results by Unal et al., in 2001.

In Jiulongkou Coal Mine the first phase of repair work began in mid-1997, without interfering with normal coal mine production activity, and ended in early 1998. After the first phase repair work repair work and evidence of the effective results, more repair work has been carried out in other tunnels (Song and Lu, 2001).

A monitoring station was set up during test-phase repair work. Up until August 1998, the data on tunnel deformation had been taken for over 370 days. It is evident from the convergence curves of the tunnel that the period of stress redistribution was approximately 30 days. After the stress had stabilized, measurements over a further 300 days were taken. These measurements showed that the maximum convergence was less than 14 mm, and indicate that repair of the tunnel were successful (Song and Lu, 2001).

For railway tunnels in China, primary stress field and common physical and mechanical parameters of surrounding rocks are determined by FEM back analysis on the basis of peripheral displacements measured in-situ. Viscous parameters of rock mass can be ascertained from further back analysis. Much work also has been done in investigating and improving measurement instruments. Peripheral displacement measurement instruments of types SWJ-81, SWJ-II, and SWJ-III were developed in the China Railway research Institute, with nominal mean square error 0.01-0.04. Type SWJ-III instruments also can be used to measure oblique convergences in the extent of 70° angles (Mi and Shiting, 1989).

A type HW slide resistance displacement measurement instrument; an HJ-1-type receiver with system sensitivity 0.1 mm/graduation; and a three point differential transfer type displacement measurement instrument that can be used for remote measurement of displacements (Mi and Shiting, 1989).

In order to ensure the safety of the supports in the collapsed zone and the safety of the tunnel during the restoration work and temporary service, series of measurements were carried out in the reinforcement zone as shown in Figure 2.17 (Kunita et al., 1994).

Inward displacement measured above 5 mm during excavation, but convergence in several days. Ground displacement was 6-7 mm on the mountain side and approximated 3 mm on the sea side (Kunita et al., 1994).

The overall results indicated a sufficient level of safety of both the ground and the supports (Kunita et al., 1994).

Brunswick street rail tunnel was monitored for convergence at the cross sections below lines of settlement monitoring points. The convergence was measured using an interfels tape that could be read to 0.01 mm precision. The cross-sections were read daily, when the face was still close to the monitoring cross-section, and then at longer intervals as the face moved further away (Asche and Baxter, 1994).

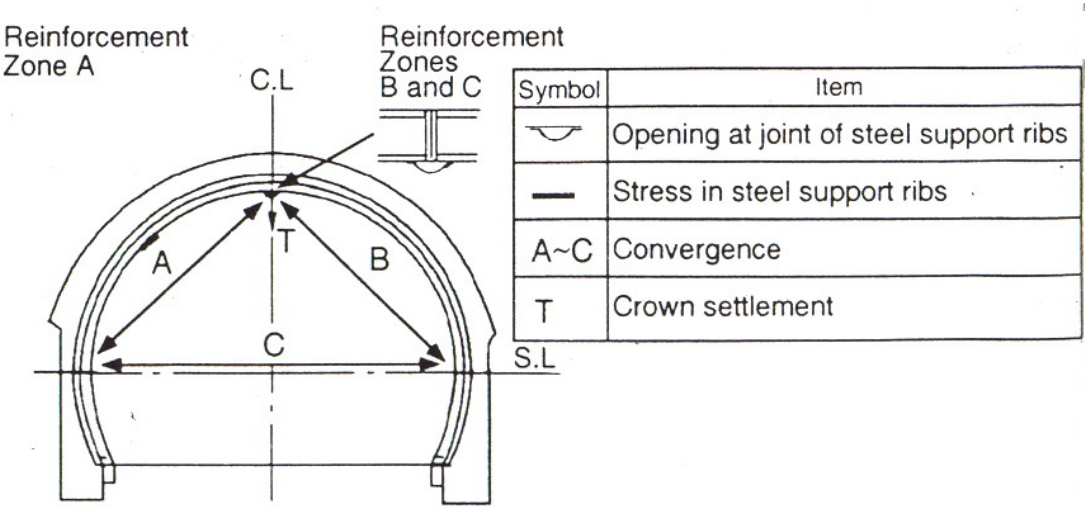


Figure 2.17 Measurements in the Reinforcement Zone
(after Kunita et al.,1994)

The results of the convergence monitoring appeared to show some slight movement initially and then to show virtually no movement after the face had moved 10 m away. Some long term movement was measured, although it was small (Asche and Baxter, 1994).

CHAPTER 3

GENERAL INFORMATION ON NO: 7 MALATYA-NARLI RAILROAD TUNNEL

3.1 General

In this chapter, general information about No: 7 Tunnel, geological and hydrogeological conditon in the vicinity of the tunnel and stability problems will be summarized.

3.2 General Information About No: 7 Malatya-Narlı Railroad Tunnel

3.2.1 Location of the Tunnel

The nearest town to the tunnel is called Gölbaşı, which is about 7 km away from the tunnel, and it is located in the west part of Adıyaman. Gölbaşı is surrounded by Malatya in the north, Besni and Tut in the east, Gaziantep in the south and Kahramanmaraş in the west. The altitude of Gölbaşı is 862 m. Figure 3.1 shows the location of Gölbasi.

Gölbaşı district is located on the route connecting Blacksea and South Eastern Anatolia to Mediterranean. It is also located on the route connecting Malatya, Adıyaman, Gaziantep and Kahramanmaraş to each other. The distrcit's economy has been founded based on the advantages of being on the railway and highway as it is shown in Figure 3.2 and 3.3.

Figure 3.1 The Location of Gölbaşı (after www.adiyaman.gov.tr, 2003)

NETWORK OF TCDD

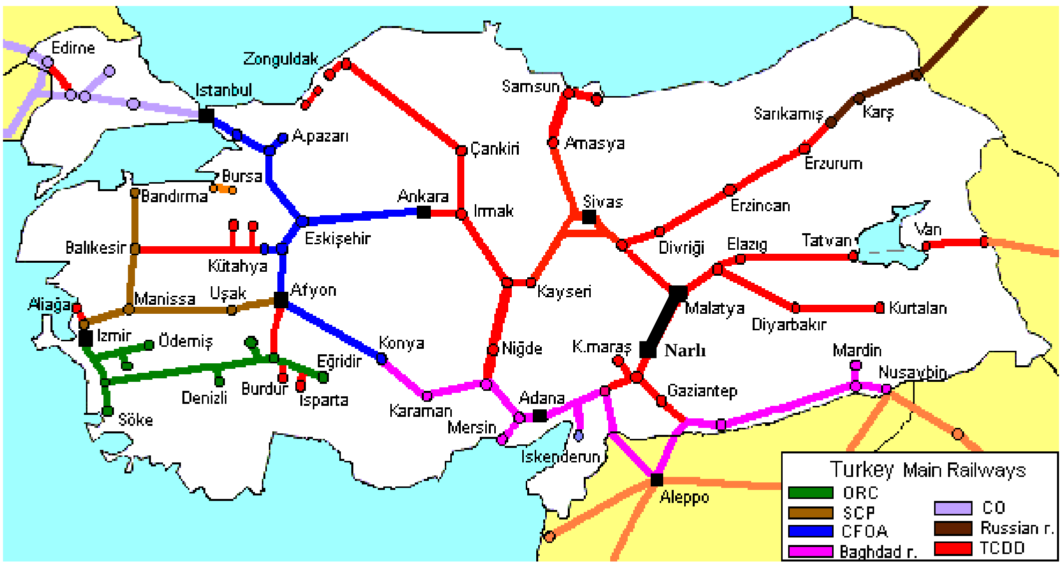


Figure 3.2 Turkish Railroad Map (after www.trainsofturkey.com, 2004)

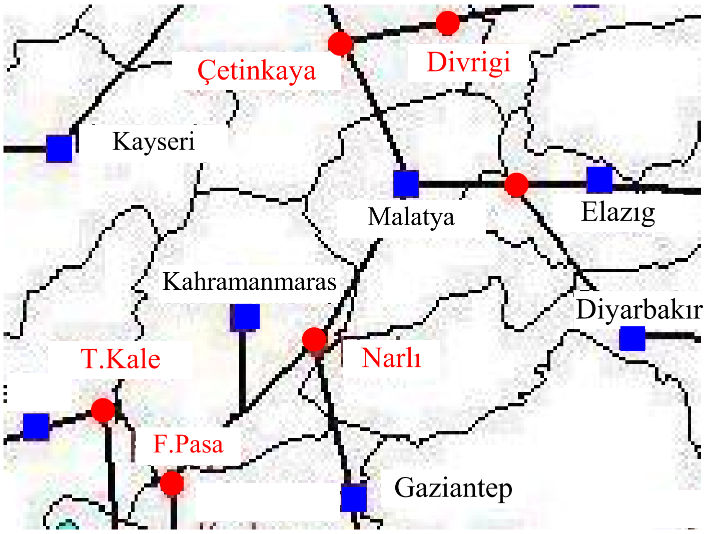


Figure 3.3 Malatya-Narlı Railroad Map (after www.tcdd.gov.tr, 2004)

3.2.2 Details Related to Excavation and Dimensions

No:7 Railroad Tunnel was constructed in 1930 by German engineers using drilling and blasting method and have a length of 357. Originally, the tunnel was excavated in horse-shoe-shape about having a width of 5.5 meters and height of 5.25 meters. On account to the fact that the mountains existing on both sides of the Kapıdere valley, the area has a potential of landslides. As a matter of fact, the tunnel was excavated inside a paleolandslide. Figure 3.4 shows the detailed information related to No: 7 Tunnel and its surroundings.

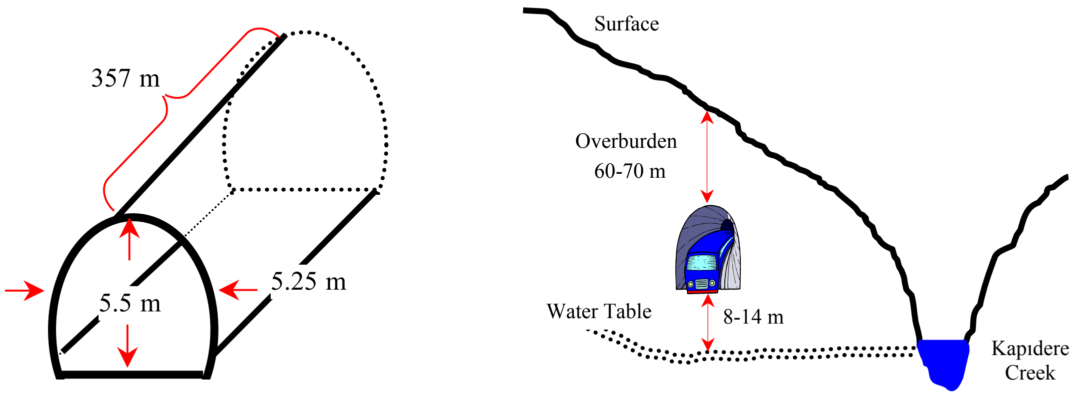


Figure 3.4 Detailed Information Related to No: 7 Tunnel and its Surroundings

No: 7 Tunnel is located at the back of Alisliseki which is the part of Kapıdere creek. Kapıdere stretches from North to South direction and flows along quite rough topography with approximately 250 m wide flood area. But, the creek passes through bottleneck at the location of No: 7 Tunnel. In this part of the valley, there is no clear lithological control to prevent occurrence of the flood area.

As shown in Figure 3.5 and 3.6, the riverbed takes place between the right and left bank of the creek and has been displaced towards left (west) due to Alisliseki landslide.

The tunnel in question have had some temporary rehabilitation work in order to reduce or stop deformations and sliding taking place in the tunnel. However, during the rehabilitation attempts and providing alignment to the slided tunnel, collapses occurred accompanied with a sinkhole at the surface. Hence, the tunnel closed to the railroad traffic for about 10 months (September, 2002-July, 2003) during the initiation of Iraq War.

Due to the stopped railroad traffic over Adana and Malatya line the material transportation was realized over Kayseri, Ulukısla and Adana route. That compulsory route changing increased the operational expenses in terms of money, energy and time. The money lost by TCDD was considerable. In addition, the money lost by steel factory İsdemir, reached to high levels because the iron ore extracted from Divrigi and Hekimhan was transported through a longer route.

3.3 Geology of the Site

No:7 Railroad Tunnel was excavated within the heel material of a paleolandslide being accumulated over the old flood area of Kapıdere and eroded from Alisliseki ridge. The rock mass around the tunnel is melange type having a complex geology. The rock mass around the tunnel consists of metavolcanics, schists, fractured limestone blocks, antigorite and radyolite in patches. The matrix material consists of clay and schist having low swelling potential. Limestone blocks are generally closely jointed and highly fractured. Pelitic schists around the tunnel have distinct deformations traces and shear deformations around them (Ergun et al., 2002).

During field studies it was determined that the landslide, having a crest in some 1140 m. elevation, is a paleolandslide (Ergun et al., 2002). The rock mass surrounding the tunnel consists of limestones and crystallized limestone pieces between kilometers 155+453 and 155+663, formed at the crest of this landslide, while low grade metamorphic rocks and metavolcanics possibly developed in small blocks at lower parts. However, the rock mass surrounding the tunnel between kilometers 155+663 and 155+753 mainly consists of metavolcanics and pelitic schists.

Figure 3.5 The Creek Flowing Parallel to the Tunnel

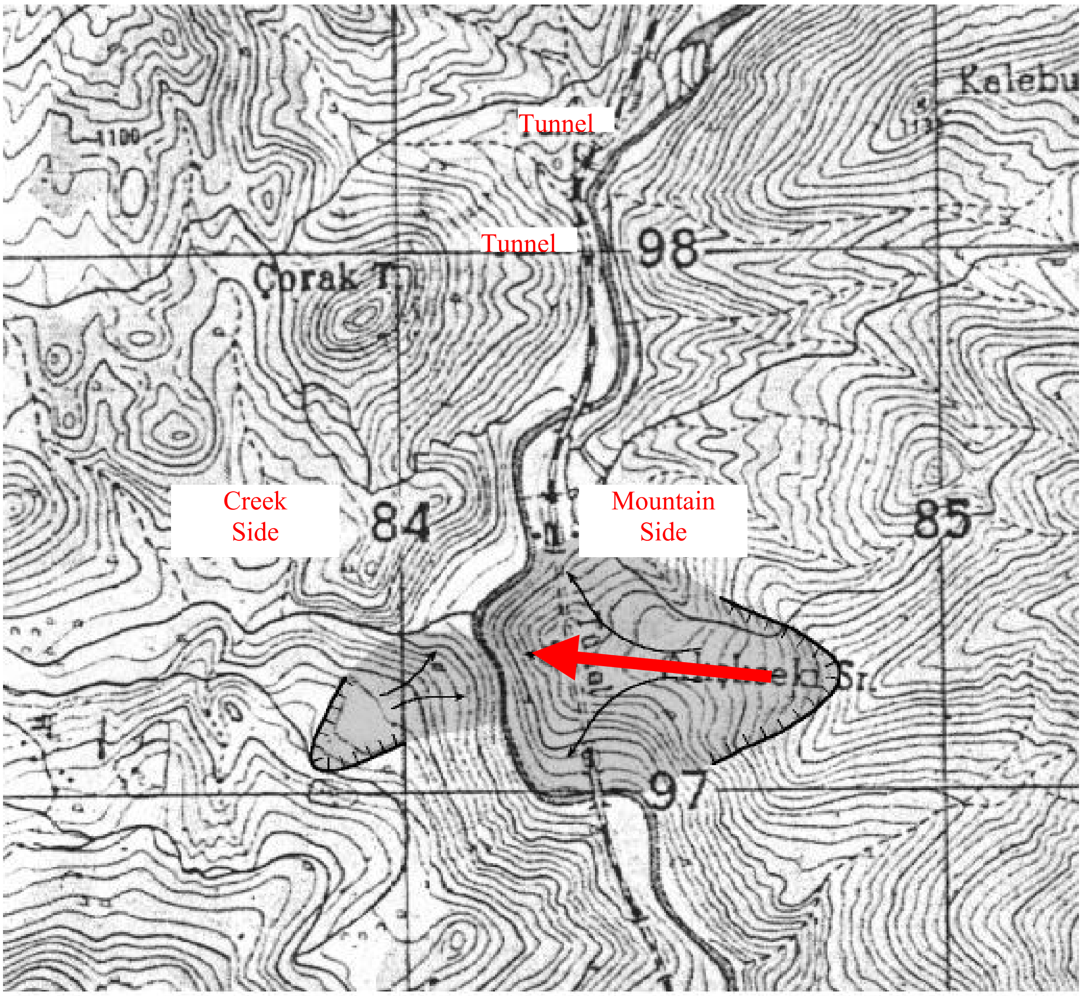


Figure 3.6 The Location of the Landslides at Right and Left Banks at Kapıdere (after Ergun et al., 2002)

Metavolcanics and schists taking water from limestones turn into smectite type of clay, having low potential swelling type, by means of hidrotation. The tunnel takes part in second seismic zone.

3.4 Hydrogeology of the Site

The complex lithological units surrounding the tunnel where very low permeability schist and metavolcanics taking part as blocks in the matrix material indicate a medium permeability. However, in the existence of limestone blocks the permeability level increases due to existence of dense cracks and fractures.

3.5 Stability Problems in the Tunnel

No: 7 Railroad Tunnel experienced stability problems ever since the opening of the tunnel in 1930. The main reason was excavation of the tunnel inside the toe of a landslide material. In addition, TCDD struggled with the stability problems created due to two roof-falls, tunnel width decreased due to squeezing, failure of steel-arches and concrete lining and effect of groundwater.

Morphology of the surrounding of the tunnel clarifies that Kapıdere riverbed was squeezed by the hillside. The landslide morphology observed along the Alisliseki ridge and other bank of the creek confirmed the existence of the landslide as shown in Figure 3.7 and 3.8.

According to geological observations, landslide scarp (crown) takes part in the east part of the Alisliseki back. Sliding surface ends at the bottom level of Kapıdere. Limestones taking part in the crown of the landslide with coarse to fine blocks spreading all the area are observed as floating blocks as shown in Figure 3.9 and 3.10. In addition to limestone blocks, scattered schist and metavolcanic blocks are observed.

Figure 3.7 Alıslıseki Landslide scarp, Swellings and Settlements at the Mountain Side (after Ergun et al., 2002)

Figure 3.8 Landslide Materials at the Mountain Side (after Ergun et al., 2002)

Figure 3.9 Landslide Materials at the Mountain Side (after Ergun et al., 2002)

Figure 3.10 Landslide Materials at the Mountain Side (after Ergun et al., 2002)

CHAPTER 4

PROBLEMS RELATED TO THE STABILITY OF THE NO: 7 RAILROAD TUNNEL

4.1 Problems Related to the Stability of the Tunnel

The stability problems observed in and around No: 7 Railroad Tunnel can be categorized into seven main groups, namely;

i. Excessive displacement occurred in the tunnel
ii. Collapses occurred during rehabilitation work
iii. Sinkhole developed on the surface due to roof collapse, and flow of rock material into the tunnel
iv. Reduced tunnel span and height preventing free trains pass inside the tunnel
v. Poor rock mass causing instability in the tunnel
vi. Surface and underground water reducing tunnel stability
vii. Unstable regions requiring urgent repair

4.2 Excessive Displacements Occurred in the Tunnel

An obvious misalignment was observed within 105 meters of the tunnel between kilometers 155+550 and 155+655 and 35 meter section of it (between kilometers 155+569.15 and 155+604.15) was displaced about 1.70 meters over the years. Figure 4.1 demonstrates excessive displacements occurred in the tunnel.

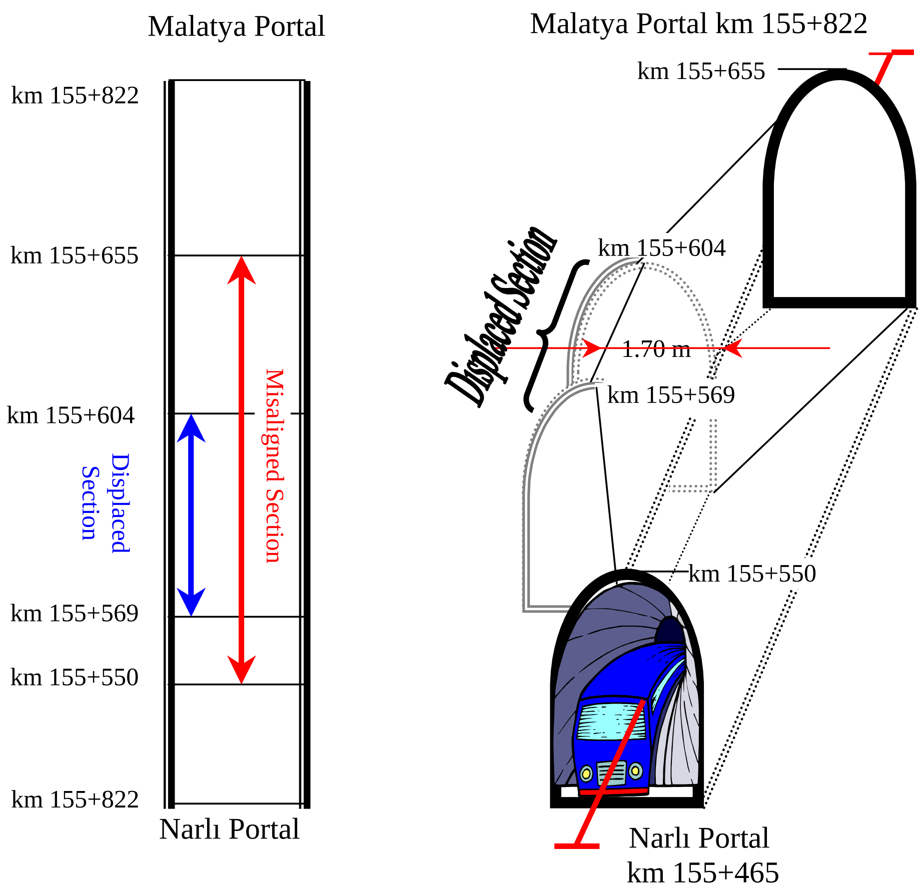


Figure 4.1 Excessive Displacements Occurred in the Tunnel

4.3 Collapses Occurred During Rehabilitation Work

The problems in the past include surface and underground water, deformation, roof rock fall, clay bearing strata and voids existing in the rock mass. The first collapse in the tunnel (roof-fall) occurred on July 30, 2002 due to breakage of the existing concrete-lining during enlargement of No: 7 Railroad Tunnel. This collapsed material flowed from the roof filled a 21-m long section of the tunnel between km 155+666 and 687 as shown in Figure 4.2. The volume of the roof-fall material flowed into the tunnel was estimated to be 500 m^3. Because of this roof-fall the tunnel stayed close to traffic for about 10 months.

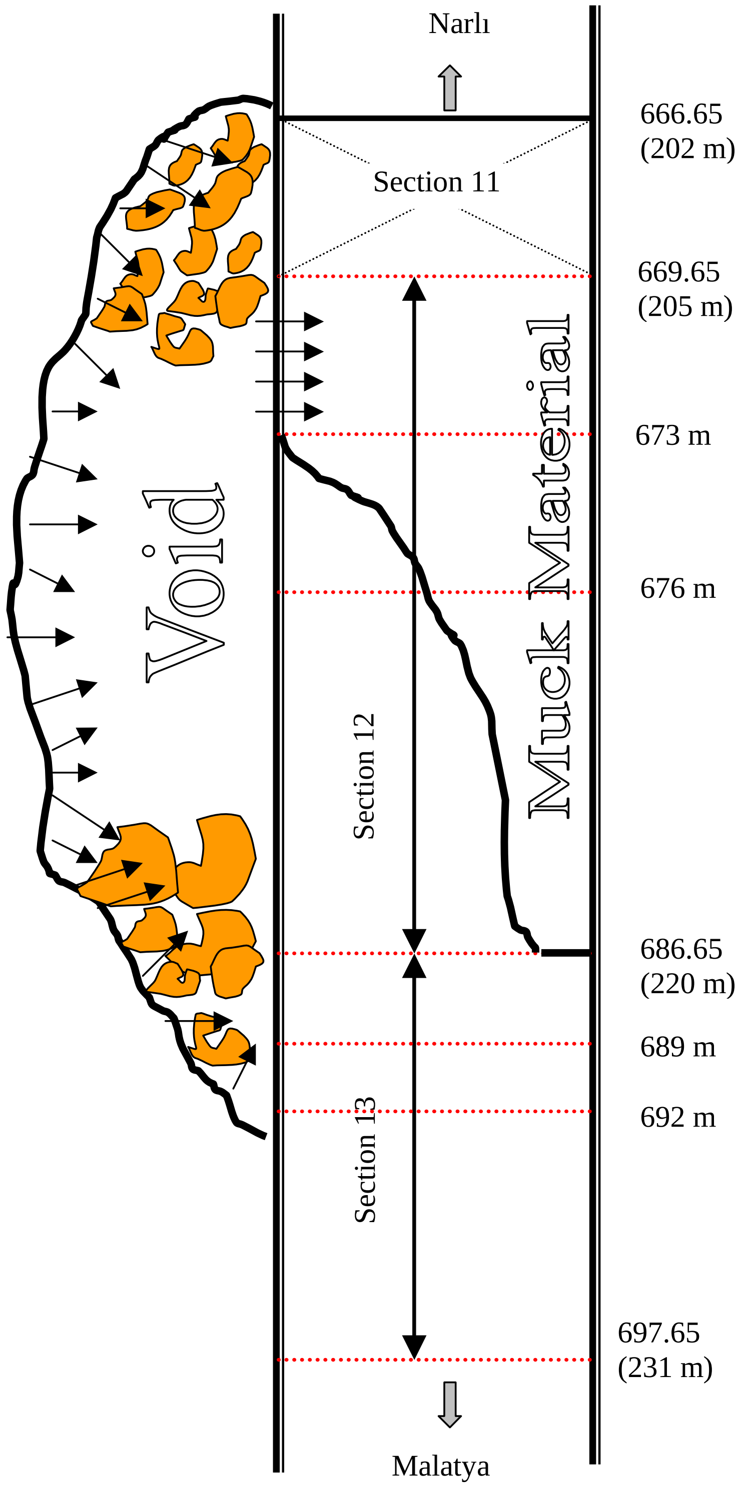
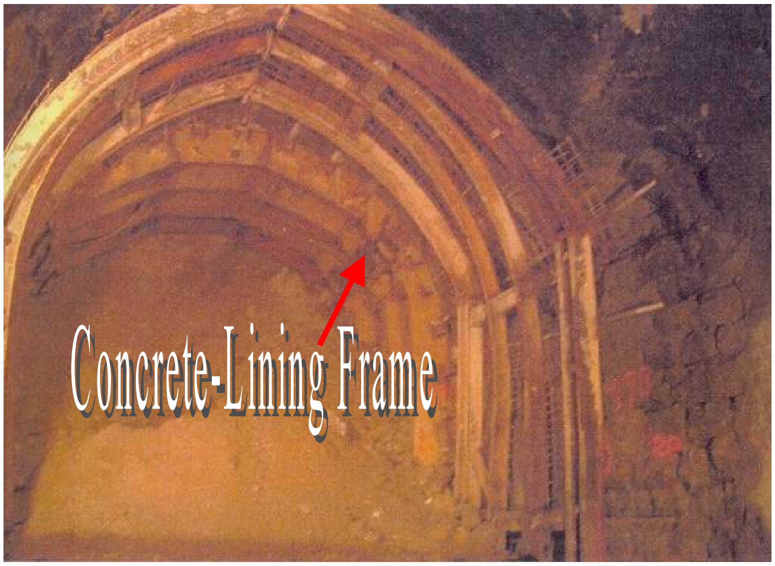


Figure 4.2 Collapse Area and the Void Developed above the Tunnel

While cleaning up of the roof-fall material in the tunnel, a new roof-fall occurred at a 3-meter long section, on February 7, 2003 in such a manner that the loose material at the roof poured into the tunnel's cavity as shown in Figure 4.3.

(a)



(b)

Figure 4.3 Appearance of the Roof-Fall, (a) from Malatya Side and (b) from Narlı Side

4.3.1 Clearance of the Roof-Fall

Before removal of the muck-material a cement injection was made into the loose material existing in the roof. Initially, 5.5 m^3 of cement was grouted between km 155+660 and 669 (195-204 m from the Narlı entrance) and then 47 m^3 of cement between km 155+669 and 676 (204-211 m) as demonstrated in Figure 4.4.

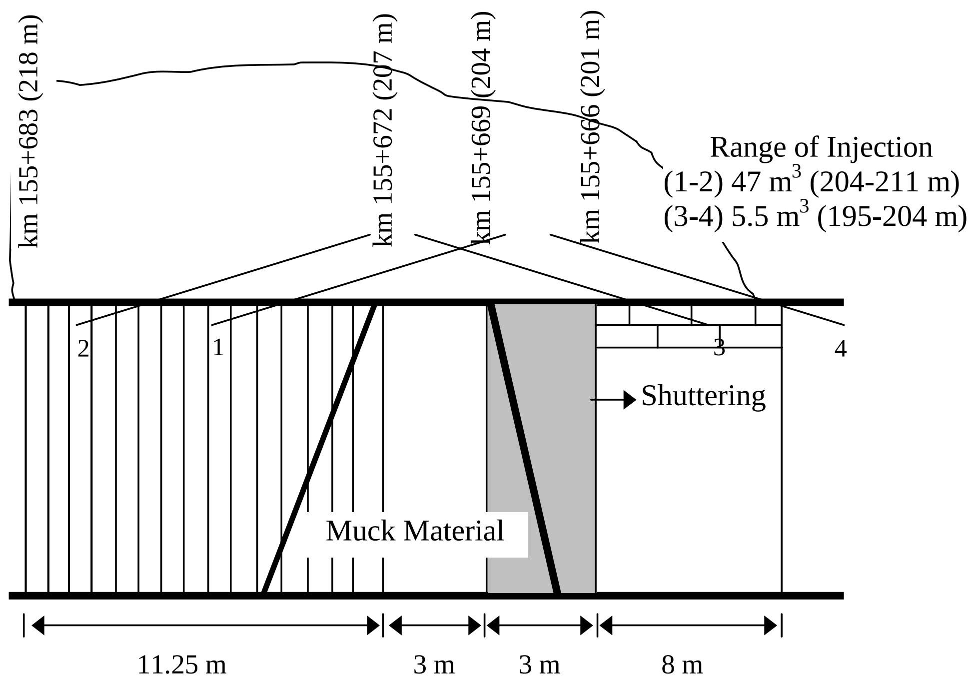


Figure 4.4 Range of Injection Made above the Roof of the Collapsed Area

The forepoles, made of I-120 and I-160 steel profiles and having a length of 3 meters were hammered into the loose material existing in the tunnel roof. Out of 28 forepoles, 6 were made of I-120, and the remaining be made of I-160. A general view of the support system used during the clearance of the collapse is shown in Figure 4.5.

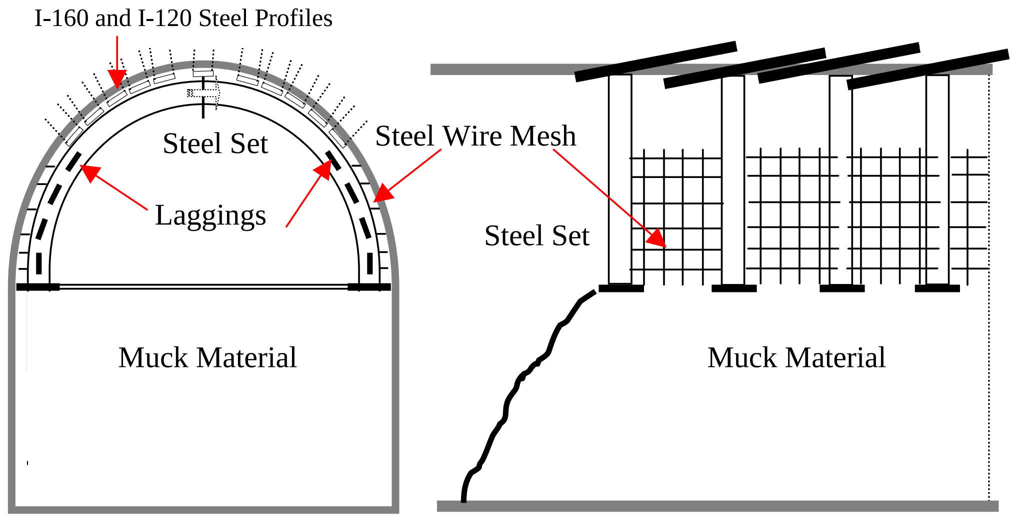


Figure 4.5 Views of the Support System Used During Clearance of the Collapse

In order to provide an initial stability and a safe working place, 8 steel-sets, two of which were twin, were installed shoulder to shoulder on the muck material located between km 155+670 and km 155+676. For this reason, first of all, the muck material in the both sides of the tunnel walls was removed and cleaned providing a space of approximately 0.5 m. The top two pieces of the steel-sets forming the arch were joined to each other from their peak points by using M20x60 mm type bolts. After this step, the arch parts of the steel-set were inserted into the cavity so opened. A total of six and four joints or bolts were used in the double and single steel-sets, respectively.

Furthermore, the steel-sets installed while clearing the bench of the roof-fall or muck material were connected to each other by using a number of laggings (or steel-profile brushes). Finally, a steel wire-mesh was installed between the steel-sets and then the total structure was shotcreted.

The steel-arch profiles were connected to each other by welding with a horizontal I profile connecting two arch profiles. In addition, two diagonal I profiles were welded between the horizontal profile and the steel-arch as shown in Figure 4.6 and 4.7.

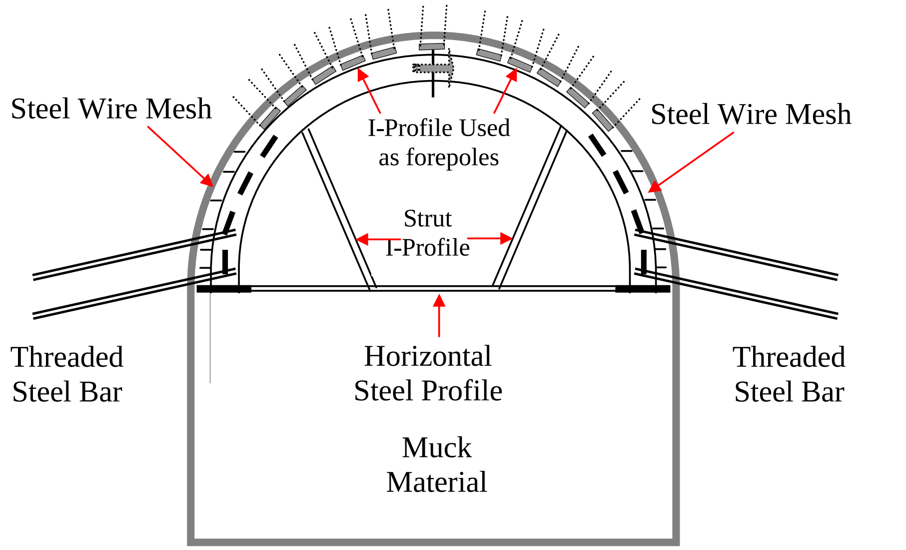


Figure 4.6 Bench Support Used During Clearance of the Collapsed Material

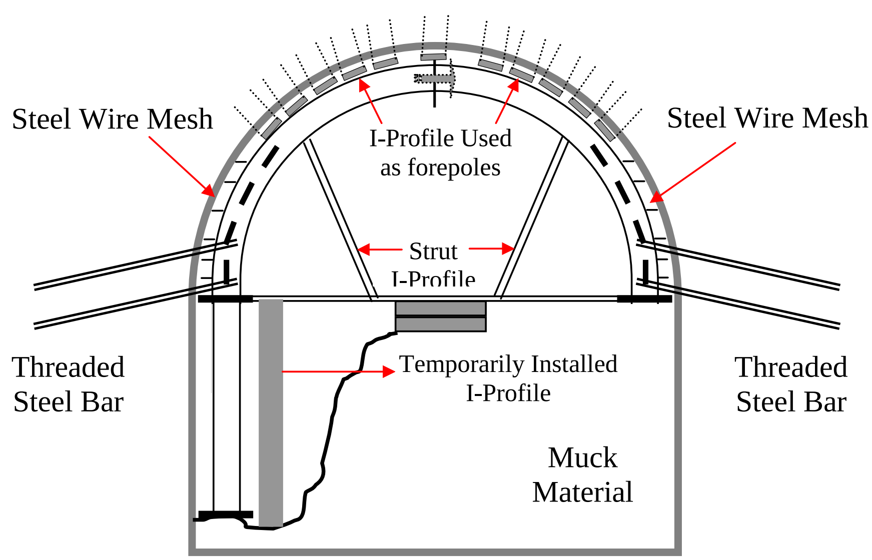


Figure 4.7 Installation of the Limp (Short) Leg of the Steel-Arch and of the Temporary I-Profiles

The arch part of the steel-set was consolidated in the tunnel wall by hammering the threaded steel rebars having a length of 2.0 m and diameter of 32 mm. Each rebar was hammered underneath the laggings between two consecutive steel-sets for the creek and mountain side walls. At this stage, the height and width of the muck material were 2.0 meters and 4.50 meters, respectively. However, the height of the muck material to be cleaned at the invert was 3.60 meters.

As a result of the supporting work described in this section, the stability of the caved area provided as follows:

i. A reaction of reinforcement was provided against lateral loads

ii. The steel-sets were strengthened against sudden loads that might be exerted from the roof

iii. Measures were taken against the possibility of the steel-arch falling down. In addition, the legs of the steel-sets were mounted while the muck material on the floor of the tunnel was removed by using back-hoe type equipment.

The bottom leg of the steel-arch was completed by welding an additional I-profile piece beneath the steel-arch.

Using this procedure explained above, the installation of 8 steel-sets was completed. Initially, it was not possible to put the legs of the steel-sets on the ground level (floor of the tunnel) as shown in Figure 4.8. In the meantime, two vertical steel I-profiles were welded temporarily to the horizontally welded I-profile at the shoulder level. Starting from one side, the muck material on the bottom of the tunnel was removed.

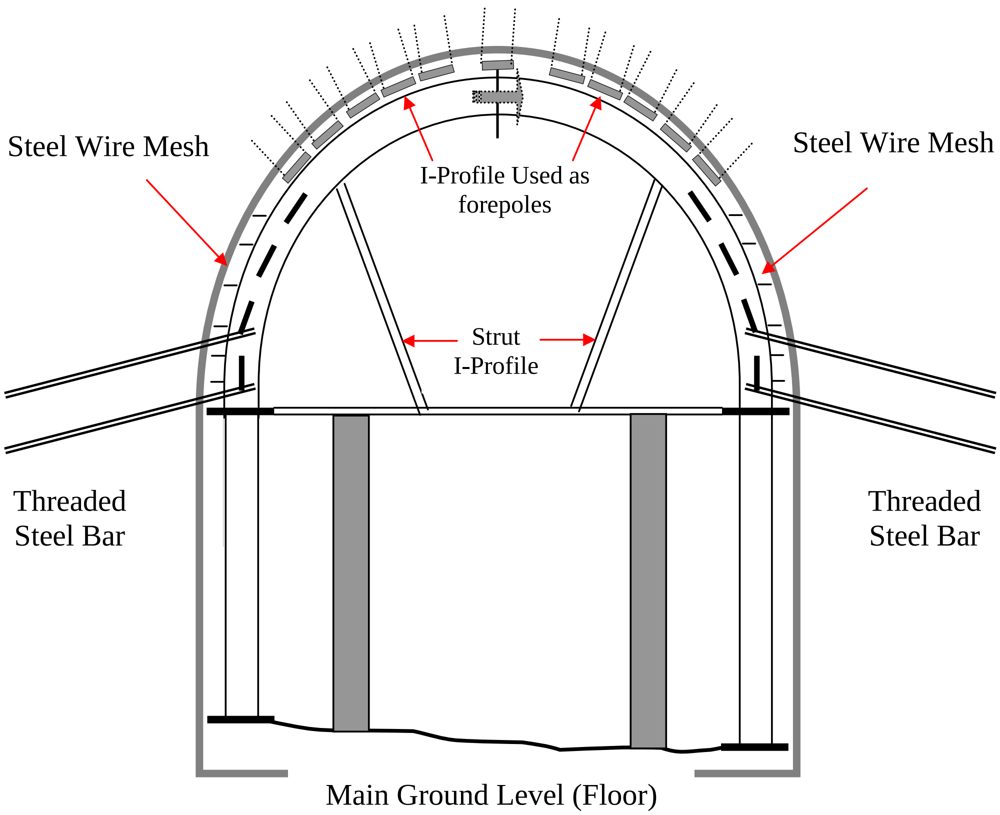


Figure 4.8 Support Used in No: 7 Tunnel During Clearance of the Caved Material

At the end of the work explained in above paragraphs the legs of the steel-sets were connected with the arch at the top side and located on floor at the lower side. Consequently, the roof-fall zone of about 5.0 meters in length was cleaned up, and a tunnel clearance of about 4.50 m. in width and 5.0 meters in height was obtained.

Bench support during clearance of the collapsed material and the last stage of the clearance of the collapse are shown in photographs presented in Figures 4.9 and 4.10.

Figure 4.9 Bench Support Used During Clearance of the Collapsed Material

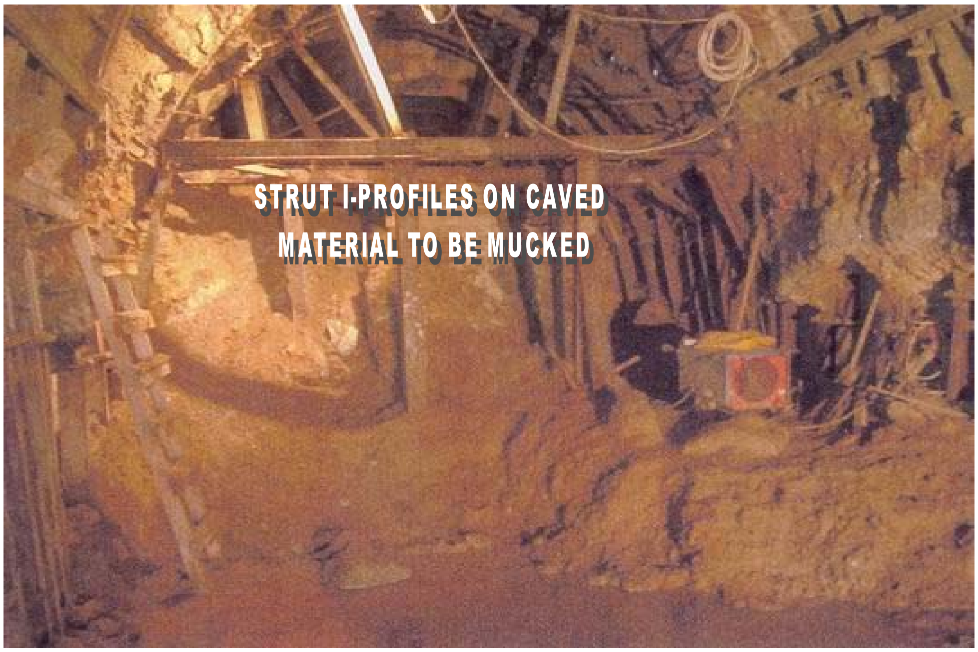


Figure 4.10 Last Stage of the Clearance of the Collapse

4.4 Sinkhole Developed at the Surface due to Roof Collapse and Rock Mass Flow into the Tunnel

As a result of the two collapses occurred in the tunnel a sinkhole was developed on the ground surface as a result flow of 500 m^3 material into the tunnel from the tunnel roof. The dimensions of the sinkhole were about 15x10x20 meters. The sinkhole developed at the ground surface and a sketch of the collapsed area with the sinkhole developed at the surface are shown in Figures 4.11 and 4.12, respectively.

(a)

(b)

Figure 4.11 Sinkhole Developed at the Surface as a Result of the Collapse

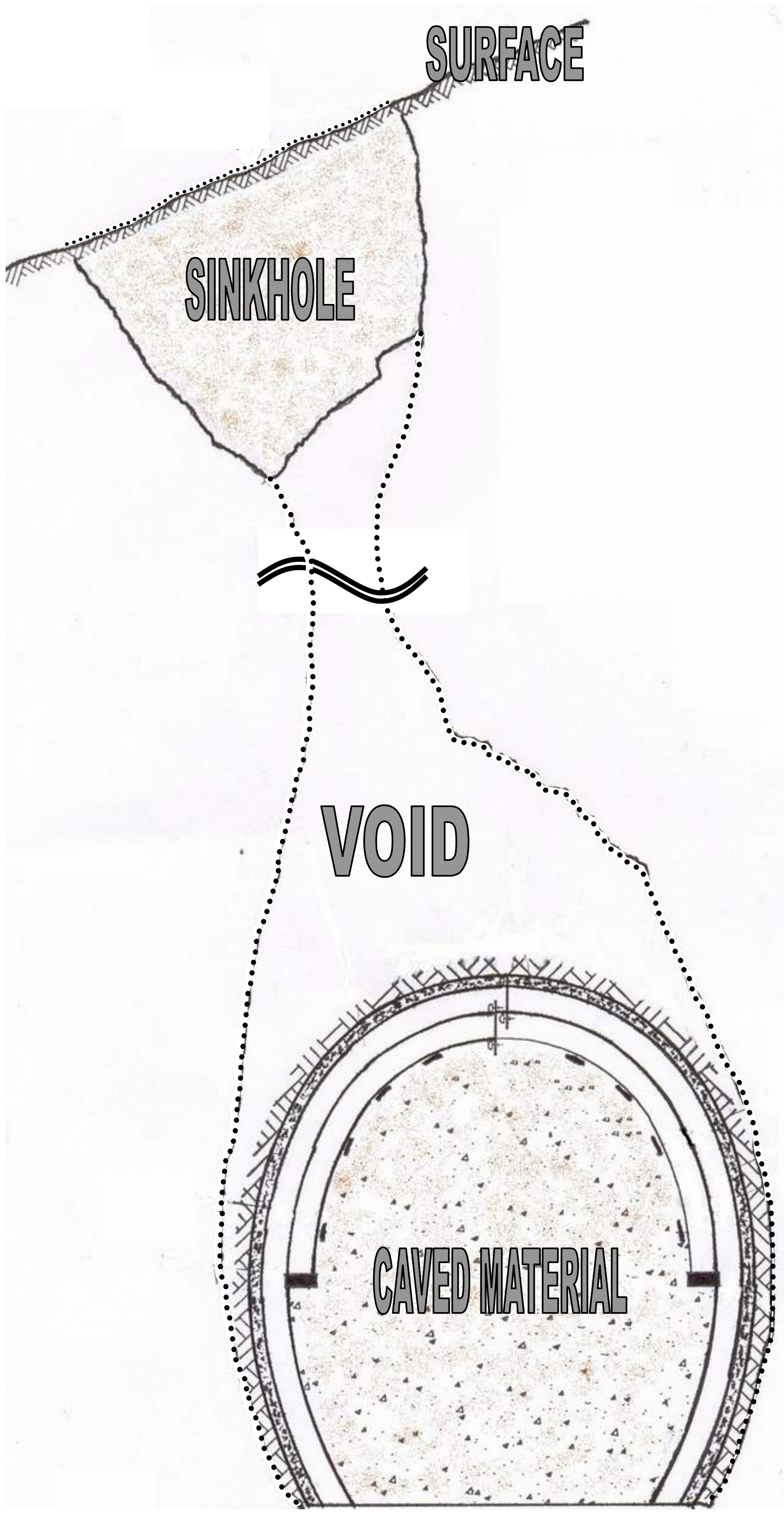


Figure 4.12 Estimated Geometry of the Subsidence above the Collapsed Tunnel

4.5 Reduced Tunnel Span and Height Preventing Trains to Pass Freely

As a result of ground water effect and time dependent deformations (Creep) the tunnel walls between km 155+550 and 655 have displaced through the creek side resulting in; i. an ondulating shape of the tunnel, ii. a reduced roof-span as shown in Figure 4.13, iii. deformed steel-arches, and iv. cracked concrete-lining. Consequently, there was a serious problem namely, a reduced span and unstable tunnel through which trains could not pass.

For determining such problematic sections, the cross-sections at 0.50 m intervals were drawn and the existing dimensions of the tunnel were indicated. A 1:50 scale map of the tunnel was also drawn. By using a scaled template of the tunnel, the sections where the train touches to the tunnel walls and the roof were marked up. It was realized that there were a number of sections that would create very serious problem due to the reduced span and height of the tunnel.

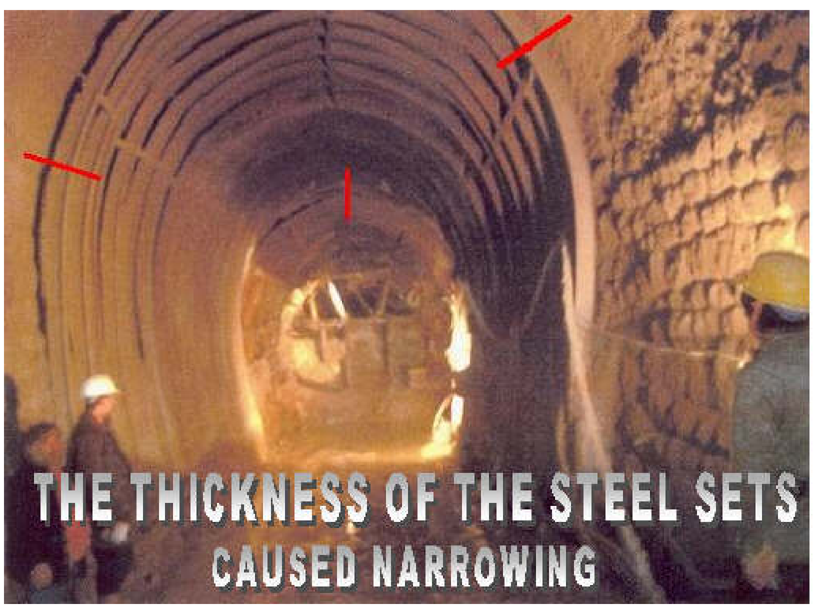


Figure 4.13 Photograph Showing a Reduced Span Close to the Collapsed Area

4.6 Poor Rock Mass Causing Instability

As a result of 17 cores drilled from inside the tunnel at various locations (Figure 4.17, in page 89) it was determined that the rock mass around the tunnel was weak and had flowing characteristic which created some problems during repairment of the lining.

4.6.1 Condition of the Tunnel Roof

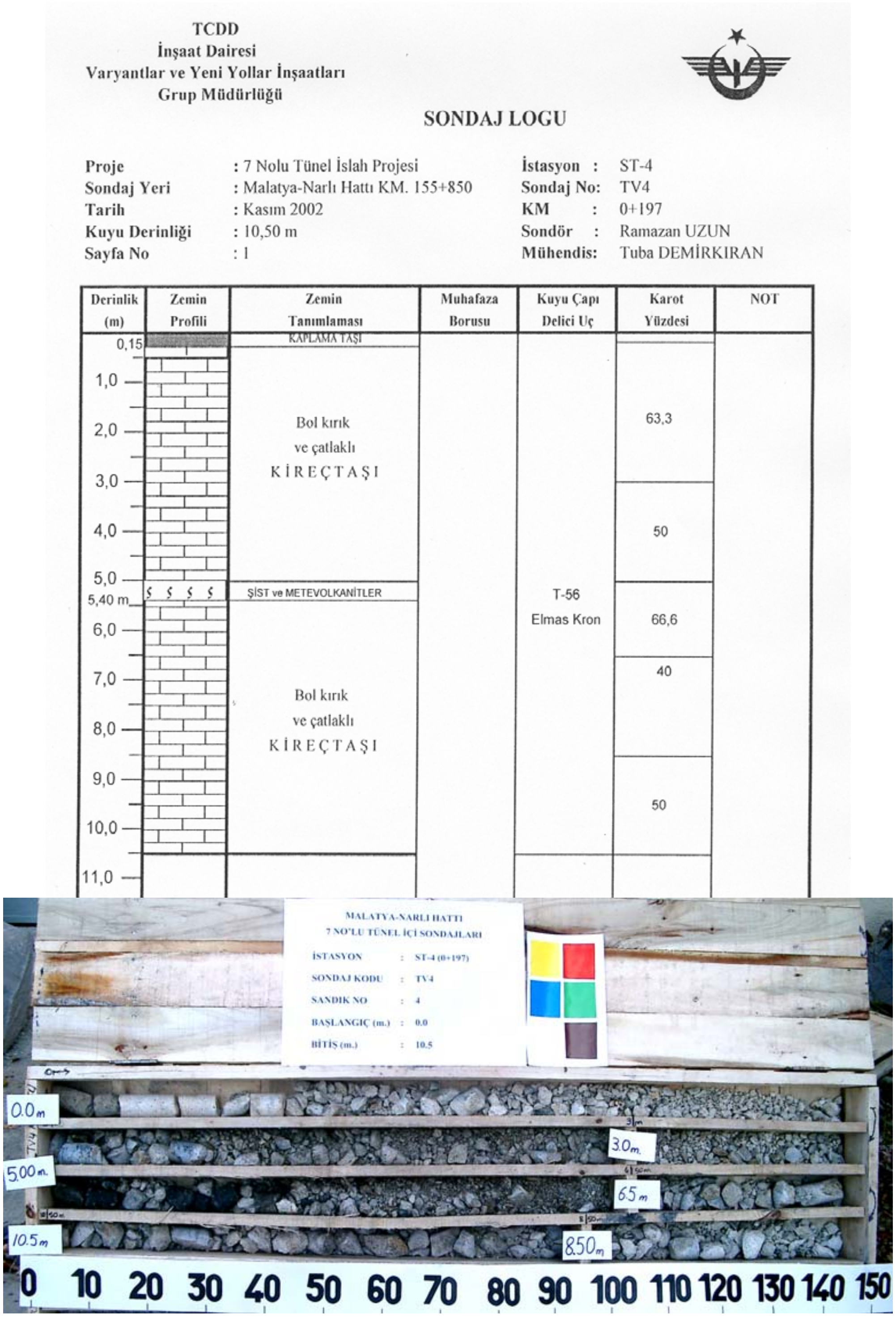
Between 90 m (km 155+555/ST-1) and 152.5 (km 155+617.5/ST-3) of the tunnel, some small parted schist and metavolcanics were encountered in the tunnel roof. In front of the caving zone located 199 meters from Narlı portal (km 155+666/ST-4) partially fractured limestone blocks were observed.

The total core recovery (TCR) obtained from drill cores made between 90-152.5 meters of the tunnel was very low and varied between 17 % and 48 %. These values reached the lowest values in the displaced section (km 155+587.5/ ST2) (17-28%). An increase at TCR values were observed at the 199 th meters located in front of the roof-fall.

The thickness of roof lining between 90 and 152.5 meters was 1.0 m. At the 199 th meters located in front of the roof-fall, there was a 0.15 m thick stone-arch instead of concrete-lining. The cores taken from station No: 4 (ST-4) is shown in Figure 4.14.

4.6.2 Condition of the Wall at the Mountain Side

The cores taken from the wall at the mountain side at the 152.5 th meters (ST-3) of the tunnel indicated the existence of small parted schist and metavolcanics within the first 7 meters in this location. And at the other drilling stations (ST-1, ST-2 and ST-4), small parted limestone blocks were encountered. Thickness of the wall at the mountain side of the tunnel varied between 0.40 and 0.80 meters.



TCDD
İnşaat Dairesi
Varyantlar ve Yeni Yollar İnşaatları
Grup Müdürlüğü

SONDAJ LOGU

Proje	: 7 Nolu Tünel İslah Projesi	**İstasyon :**	ST-4
Sondaj Yeri	: Malatya-Narlı Hattı KM. 155+850	**Sondaj No:**	TV4
Tarih	: Kasım 2002	**KM :**	0+197
Kuyu Derinliği	: 10,50 m	**Sondör :**	Ramazan UZUN
Sayfa No	: 1	**Mühendis:**	Tuba DEMİRKIRAN

Derinlik (m)	Zemin Profili	Zemin Tanımlaması	Muhafaza Borusu	Kuyu Çapı Delici Uç	Karot Yüzdesi	NOT
0,15		KAPLAMA TAŞI				
1,0 – 5,0		Bol kırık ve çatlaklı KİREÇTAŞI			63,3	
					50	
5,40 m		ŞİST ve METEVOLKANİTLER		T-56 Elmas Kron	66,6	
6,0 – 10,0		Bol kırık ve çatlaklı KİREÇTAŞI			40	
					50	
11,0						

Figure 4.14 Cores Taken from the Roof at Station No: 4

Just behind the concrete, there was a weathered clayey matrix with a thickness of which varied between 0.25-0.60 m. Based on the evaluation of the cores taken from boreholes drilled in the tunnel it was concluded that rock mass at the mountain side of the tunnel exhibits of a highly complex structure. At locations between 90 and 122.5 meters, as from the Narlı entrance, rock mass exhibited relatively better quality. Its TCR values reached up to 100 % at several locations. There were only two sections where RQD values were 45 and 55, respectively. The RQD values were zero in all other locations except these two sections. Typical cores obtained from different borehole stations are shown in Figure 4.15.

In one of the boreholes drilled at displaced section (122.5 m), it was observed that especially between the first 1.2-7.5 meters of the drilling, TCR and Intact Core Recovery (ICR) values were respectively high. These results indicated the existence of a stronger rock mass behind the tunnel wall exposed to deformation, which did not exist in the other sections. Rock mass at the 152.5 th meters of the tunnel was very weak (8<TRC<30; RQD=0) as was the case in most of the other locations.

4.6.3 Condition of the Wall at the Creek Side

The drillings made towards the wall at creek side, between locations 122.5 and 152.5 meters of the tunnel, showed the availability of small pieces of schists and metavolcanics within the first 7 meters from the tunnel wall. In the boreholes drilled at the 90 th m (ST-1), 122.5 th m (ST-2) and 199 th m (ST-4) of the tunnel small blocks of limestone scattered in the matrix were observed (Figure 4.16) as in the case of the mountain side.

Thickness of the concrete-lining at the tunnel wall at creek side varied between 0.40 and 0.90 m. Just behind the tunnel-lining, there was a clayey strata or cavity having a thickness at 0.30-0.40 m at the displaced section, and 0.70 m at the 90 th meters. Fractured and scattered limestone blocks were seen at locations 152.5 th (ST-3) and 199 th (ST-4) meters. The ICR values at these locations were 30-45%.

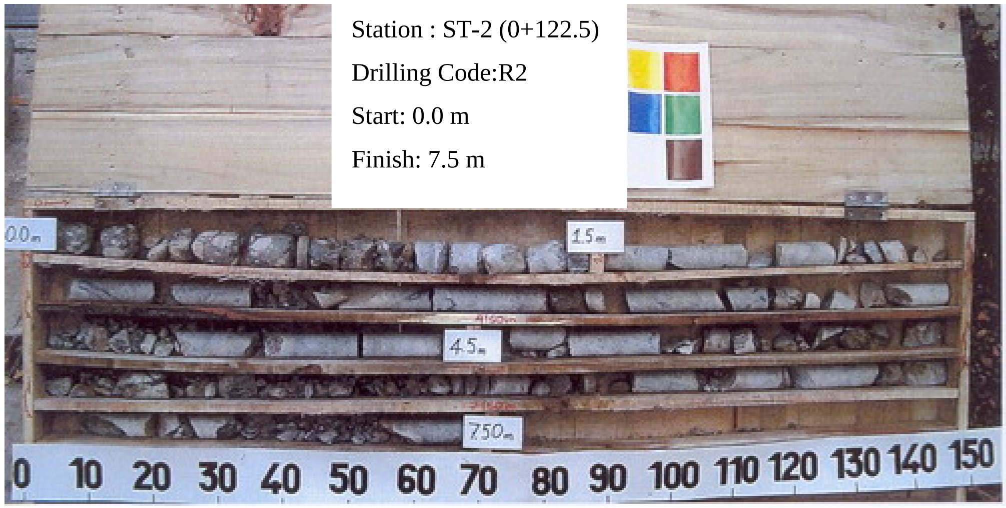


Figure 4.15 Typical Rock Mass at 122.5 m (ST-2) on the Mountain Side of the No: 7 Tunnel

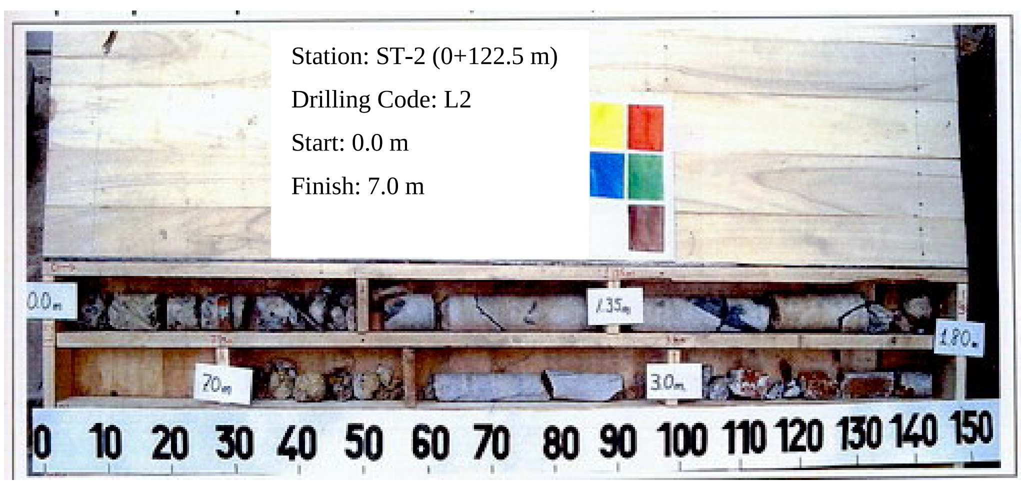


Figure 4.16 Typical Rock Mass at 122.5 m (ST-2) on the Creek Side of the No: 7 Tunnel

4.6.4 Condition of the Tunnel Floor

As part of the project work carried out by METU Mining Engineering Department for TCDD (Unal, 2003), through the drillings made in the tunnel floor it was understood that concrete-lining invert thickness was 0.5 meter at the 90 th and 122.5 th meters, and 0.75 meter at the 152.5 th meters from the Narlı entrance. TB-2 borehole drilling made at the 122.5 meters from the Narlı entrance progressed in alluvial sand at depth between 11.5 m and 21.00 m from the tunnel floor and terminated within ophiolite.

4.6.5 Results of Borehole Investigation after Roof-Fall

A second series of studies were carried out in the tunnel in September-December, 2003 in order to obtain information from the existing conditions of the tunnel after cleaning of the collapse and repair of the tunnel (Unal, 2003).

During the studies mentioned above (Unal, 2003) 50 coreholes at different sections of the tunnel were drilled aimed at investigating the rock mass and inspecting the possible voids existing around the tunnel, obtaining information necessary to decide whether the tunnel needs extra support, and lastly determining thickness of the concrete grout injected in Sections of 2 and 3. To answer all these questions, 10 boring stations were set as shown in Figure 4.17. Borings were applied to roof, floor and side walls. Roof boreholes were 10 meters in length, while the side wall and floor boreholes were 4 meters in length.

The cores obtained from the holes drilled inside the tunnel were evaluated to obtain following information:

i. Voids around the tunnel
ii. Washed soil during drilling
iii. Existence of clay
iv. Low Total Core Recovery (TCR)

v. RQD
vi. Water-loss during drilling
vii. Type of rock mass

Typical coreboxes obtained at station S-2 are shown in Figure 4.18 and 4.19. The borehole information obtained at station-2, color classification and color codes are shown in Figure 4.20 and Figure 4.21, respectively. The information obtained from all of the stations are presented in Table 4.1.

The details of the investigations can be found in a report prepared by Unal Consulting Report, 2003.

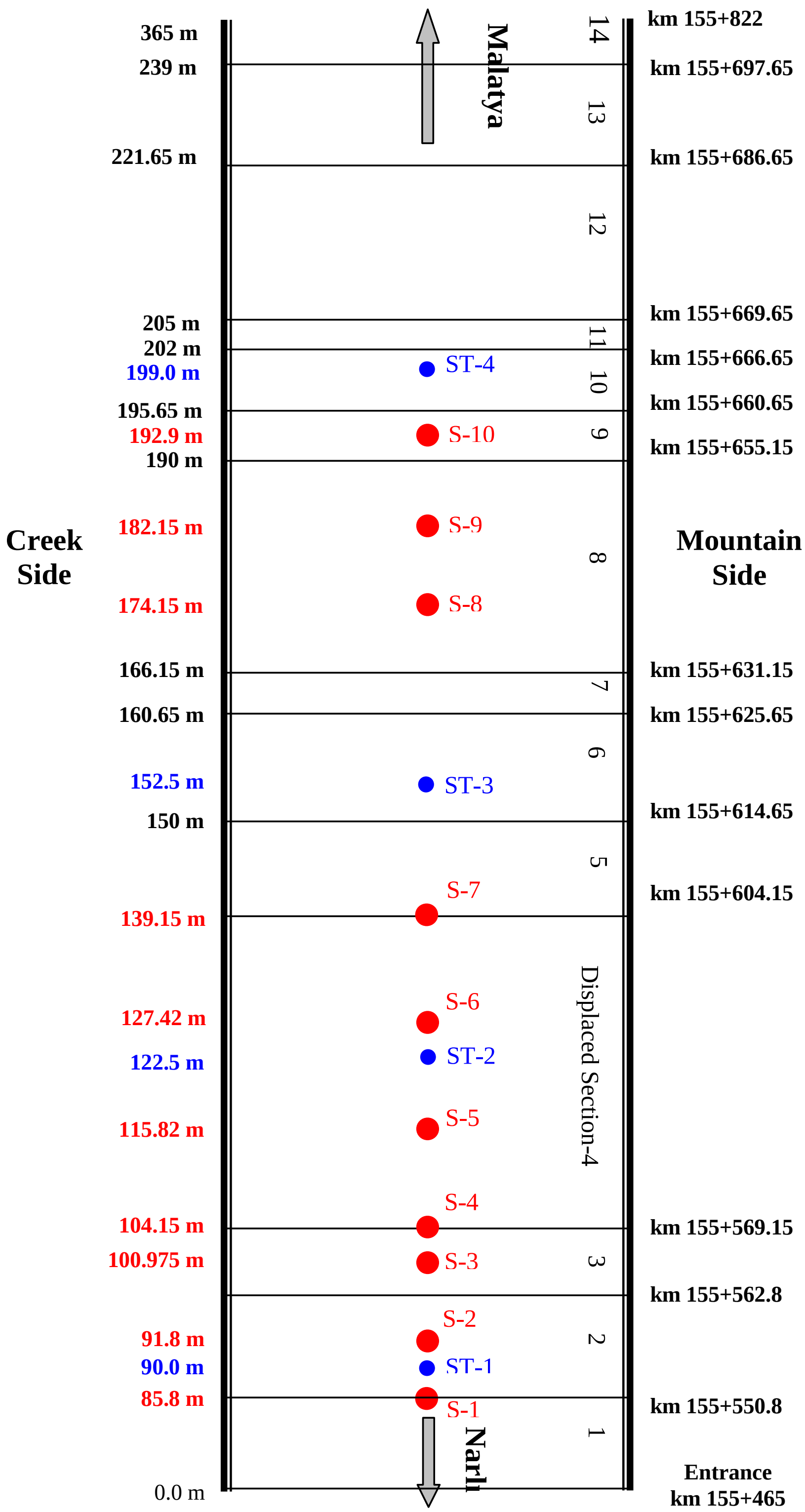


Figure 4.17 Drilling Stations Before (ST) & After (S) Collapse

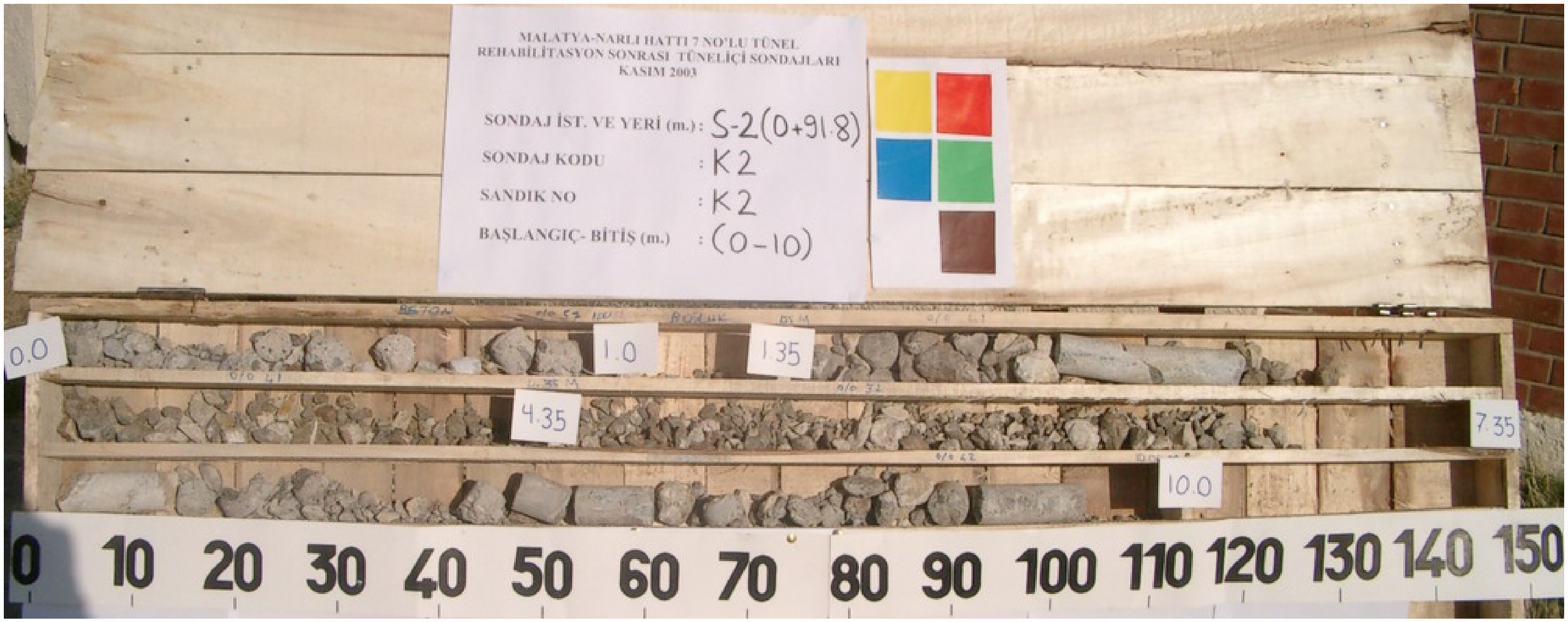


Figure 4.18 Cores Obtained from the Roof in Section 2

Figure 4.19 Cores Obtained from the Side Walls in Section 2

Figure 4.20 Core Classification and Color Codes (after Unal et al., 2003)

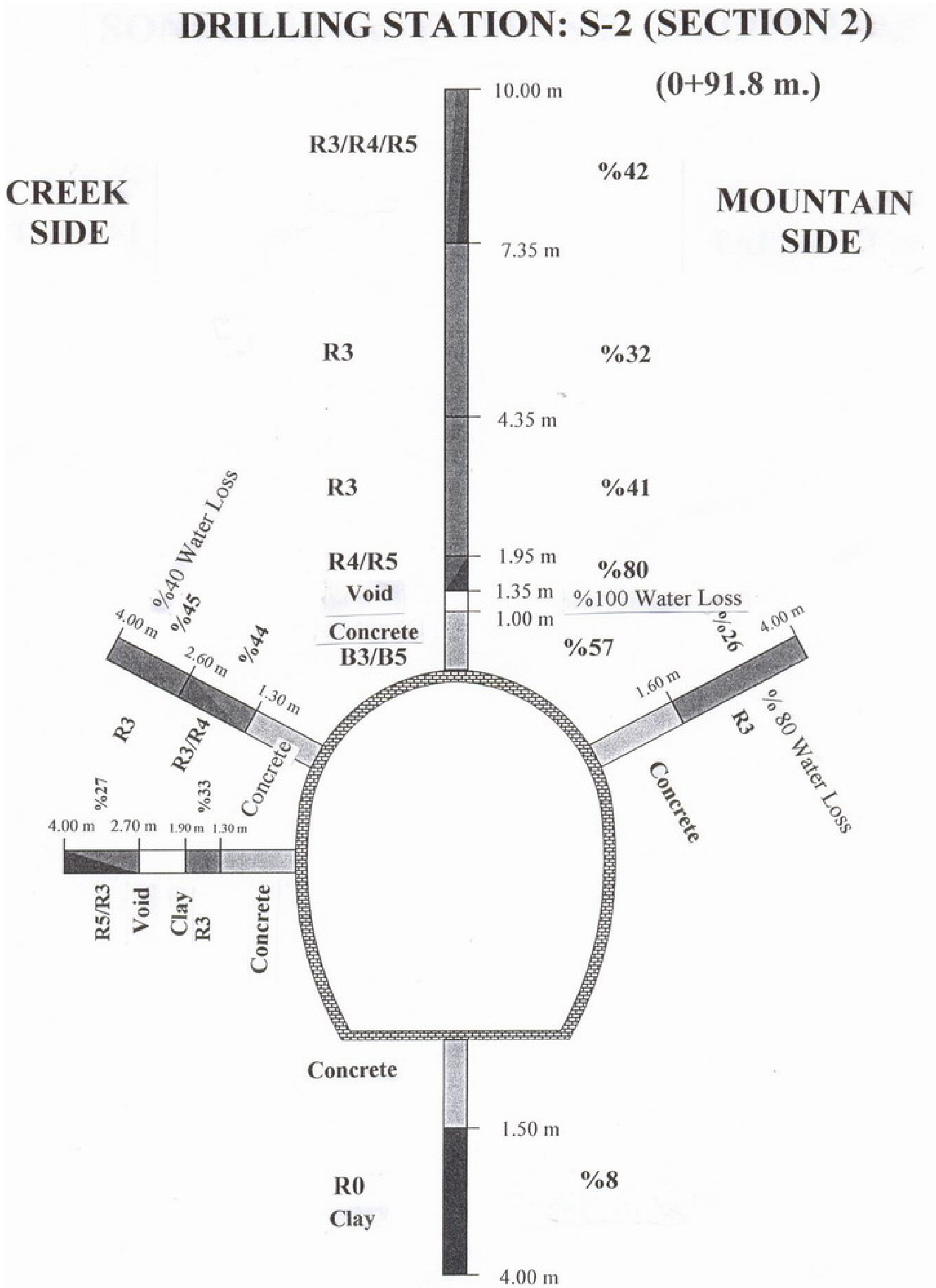


Figure 4.21 Coreholes Showing Rock Mass Conditions Surrounding Station 2

Table 4.1 The Information Obtained from Cores about the Stability of the Tunnel (1)

Drilling Station	Section	Distance from Narlı Side (m)	The Thickness of the Concrete Lining (m) (Total Core Recovery)	Void and washed Material Behind the Concrete Lining and Notes
S-1	1/2 Border	85.8	TV: 0.80 (%78)	Void 0.10 m; between 0.9-10 m R3 TCR=%57
			TB: 0.90	No;between 0.9-4 m R3 limestone; %49
			DÜ: 0.80	No;between 0.8-2.2 m R1/R2; %60
			DA: 0.90	No;between 0.9-4 m R3 limestone; %49
			NÜ: 070	No;between 0.7-4 m R2/R3; %69
			NA: -	-
S-2	2	91.8	TV: 1.00 (%57)	Void 0.35 m water loss %100; R3/(R4+R5)+above 7 m inj.mat.%41
			TB: 1.50	Void 2.5 m; between 1.5-4 m TCR %8; clay
			DÜ:0.80	No; between 1.6-4.0 m; TCR %26, water loss %80
			DA: -	-
			NÜ: 1.30	No; between 1.3-4.0 m R3/R4; %45
			NA: 1.30	Washed mat. between 1.90-2.70 m; R3/R5; %30
S-3	3	101.0	TV: 1.00 (%60)	No: between 1-7 m R3, %46; between 8.70-8.90 void; %100 water loss
			TB: 1.20	Void, between 1.2-4 m; TCR %5; clay
			DÜ: 1.60	No; between 1.6-4.0 m R3/R4, %70
			DA: 2.00 (%?)	No; between 2-4 m. R3/R4; %54, water loss %40 (?)
			NÜ: 1.50	No; between 1.5-4.0 m. R2/R3, water loss %30
			NA: -	-

TV: Roof
TB: Bottom
DÜ: Mountain Side Up
DA: Mountain Side Down

NÜ: Creek Side Up
NA: Creek Side Down
DD: Mountain Side Wall
ND: Creek Side Wall

TCR: Total Core Recovery (%)
Narlı Entrance: Km 155+465 m.

Table 4.1 The Information Obtained from Cores about the Stability of the Tunnel (2)

Drilling Station	Section	Distance from Narlı Side (m)	The Thickness of the Concrete Lining (m) (Total Core Recovery)	Void and washed Material Behind the Concrete Lining and Notes
S-4	3/4 Border	104.15	TV: 1.30 (%43)	No; between 1.3-10 m R2/R3 %63
			TB: 1,35	Washed mat. between 1.20/1.35-4.0 m. TCR= %51; clay / marn
			DÜ: 1.30	No; between 1.3-4.0 m R3/R4, %48
			DA: -	-
			NÜ: 0.80	No; between 0.8-4.0 m R3/R4; %49
			NA: 0.85	No;between 0.85-4.0 m R2/R3; %58
S-5	4	115.82	TV: 1.30 (%69)	No;bet. 1.3-10 m. weak roof, R1/R2, upper side of conc.%45 gen.%53
			TB: 1.50	Washed mat.2.8 m, remaining very disintegrated, gravel, %35
			DÜ: 1.50	between 1.5-4.0 m %40 water loss, TCR= %49, (R2/R3)
			DA: 1.50	At the end of the drilling %60 water loss, TCR=%47, (R2/R3)
			NÜ: 1.30	between 1.3-4.0 m water lost %30, (R1)
			NA: -	-
S-6	4	127.42	TV: 1.00 (%100)	No; The last 1 m during drilling TCR %35 (R3)
			TB: 1.00	Between 1-2 m timber/sand/clay;between 2-4 water loss %50
			DÜ: 0.80	3.6 m upper part of the tunnel, 0.40 m void and %100 water loss, R2
			DA: -	-
			NÜ: 1.00	3-4 m upper part of the tunnel %70 water loss, R2/R3; TCR %68
			NA: 0.80	No;between 0.8-4.0 m R1/R2/R3, TCR=%73

TV: Roof
TB: Bottom
DÜ: Mountain Side Up
DA: Mountain Side Down

NÜ: Creek Side Up
NA: Creek Side Down
DD: Mountain Side Wall
ND: Creek Side Wall

TCR: Total Core Recovery (%)
Narlı Entrance: Km 155+465 m.

Table 4.1: The Information Obtained from Cores about the Stability of the Tunnel (3)

Drilling Station	Section	Distance from Narlı Side (m)	The Thickness of the Concrete Lining (m) (Total Core Recovery)	Void and washed Material Behind the Concrete Lining and Notes
S-7	4/5 Sınırı	139.15	TV: 1.30 (%86) TB: 1.00 DÜ: 1.00 DA: 0.60 NÜ: 1.05 NA: -	No; Sound roof No; water loss behind the concrete (%50-100), (R3/R4/R5) Void is very low; R2/R3 Void, 1.20 m upper of the concrete is R1/R2; TCR %56; with clayey No;between 1.05-4.0 m is R2/R3/R4; TCR %39 -
S-8	8	174.15	TV: 1.50 (%98) TB: 1.20 DÜ: 0.80 DA: - NÜ: 1.0 Y+0.65 NA: 1.00	No, between 4-10 m %30 water loss, R2/R3; TCR=%67 No, R2/R4; TCR=%47 No, (btw. 0.8-2.1 m R2/R3;%71); up. 0.6 m clay; 2.70-4 m R1/R2 clayey - Void 2.70-3.0 m; %50 water loss; TCR= %90 No; 1-2.5 m R4; %72; 2.5-4.0 m clay; %63
S-9	8	182.15	TV: 1.00 (%100) TB: 1.50 DÜ: 0.85 DA: 0.90 NÜ: 1.50 (%?) NA: -	No; R2/R3/R4; %51;between 7-10 m %40 water loss No; 0.1 m clay; between 1.60-4 m R2/R3; %65 No;between 0.8-4 m R3/R4/R5; %54 No;between 0.9-4 m R2/R3; %61 Void between 2.5-3 m and %100 water loss; between 1.5-2.0 m %72 -

TV: Roof
TB: Bottom
DÜ: Mountain Side Up
DA: Mountain Side Down

NÜ: Creek Side Up
NA: Creek Side Down
DD: Mountain Side Wall
ND: Creek Side Wall

TCR: Total Core Recovery (%)
Narlı Entrance: Km 155+465 m.

Table 4.1 The Information Obtained from Cores about the Stability of the Tunnel (4)

Drilling Station	Section	Distance from Narlı Side (m)	The Thickness of the Concrete Lining (m) (Total Core Recovery)	Void and washed Material Behind the Concrete Lining and Notes
S-10	9	192.9	TV: 1.30 (%85)	4.20-5 m void and %100 water loss; 1.3-4.2 m R4; %32. washed mat.
			TB: 1.50	No; between 1.5-3.0 m material with sand (R1); %36
			DÜ: 1.00	No;between 1.0-4.0 m R2/R3/R4/R5; %58
			DA: -	-
			NÜ: 1.00	No; btw. 1.0-4.0 m R2/R3/R4/R5; %57; 4 m Surr. %50 water loss
			NA: 1.20	No;btw. 1.20-2.5 m R2/R3 clayey %46; upper part R5; %66
ST-Extra	10		ND: 0.30	btw. 0.3-1.5 m R2/R3; %60; btw. 1.5-4.0 m R2/R3 with injection; %65

TV: Roof
TB: Bottom
DÜ: Mountain Side Up
DA: Mountain Side Down

NÜ: Creek Side Up
NA: Creek Side Down
DD: Mountain Side Wall
ND: Creek Side Wall

TCR: Total Core Recovery (%)
Narlı Entrance: Km 155+465 m.

4.7 Surface and Underground Water Reducing the Tunnel Stability

4.7.1 History

The reports prepared earlier by TCDD engineers before the beginning of the rehabilitation work of the tunnel indicates the existence of a serious water problem in the tunnel.

The water-table is about 8-14 meters below the tunnel floor and it is located at the top of an alluvial strata. It is probable that this water-table on alluvial strata triggers the tunnel to be displaced from the mountain side towards the creek side. On the other hand, the water precipitation leaking into the rock-mass around the tunnel has always given rise to stability problems. The water always weakens the disintegrated rock-mass and exerts pressure on concrete-lining thus reduce the stability.

During the geological studies carried out before the cleaning of the collapse, it was comprehended that the tunnel had been driven in paleo-landslide of melange type. Consequently, it is always expected to have underground water inflow at different sections of the tunnel, especially from the mountain side. During investigations it was also seen that a water drainage gallery at a distance of 40 m to the tunnel on the mountain side had been opened parallel to No:7 Tunnel aimed at relieving the effect of water. Nonetheless, this precaution worked partially, and the tunnel continued to deform together with the drainage gallery through the years.

4.7.2 Work Performed Inside the Tunnel

As stated in section 4.7.1 in order to reduce the effect of water to a minimum inside the tunnel, a drainage system was accomplished to reduce the water pressure on concrete walls of the tunnel, to prevent the effect of water on poor rock-mass and reduce the swelling effect of water on smectite type of clay.

Along the both sides of the tunnel drainage channels were constructed inside the invert at the floor. The drainage pipes having a diameter of 200 mm and surrounded by geotextile were inserted along the tunnel bottom as demonstrated in Figure 4.22.

Figure 4.22 Water Drainage Ditches before and after Inserting the Tunnel Type Drainage Pipes

Along the mountain side of the tunnel, galvanized and perforated drainage pipes having a diameter of 75 mm and length of 10 m and spacing of 5.0 meters were inserted into the tunnel wall with a staggered pattern as shown in Figure 4.23 and 4.24. The location of the upper holes of the drainage pipes were 2.0 meters and the lower holes were 0.5 m from tunnel bottom.

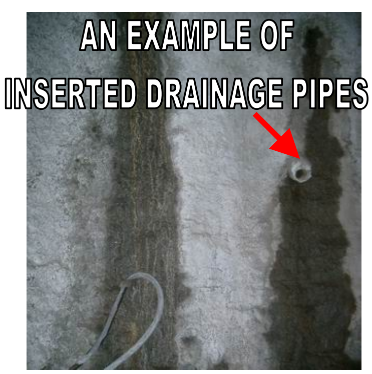


Figure 4.23 Perforated Drainage Pipes at the Walls of the Tunnel

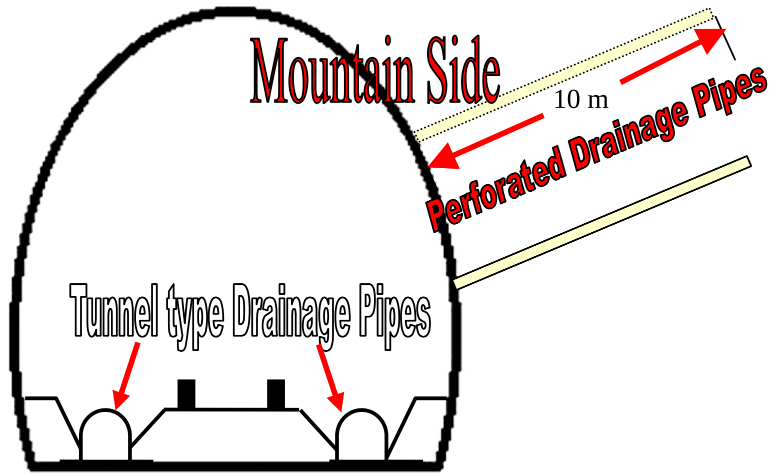


Figure 4.24 The Drainage System Along the Tunnel

At the creek side of the tunnel, in Sections 2, 3, 3/4 border, 4 and 8, drainage pipes having a length of 3.0 meters were inserted.

4.7.3 Work Performed on the Surface

On the earth surface above the tunnel, a total of 11 drainage wells, spaced 15 meters to the each other and having a diameter of 300 mm were sank vertically from the surface to the drainage gallery, in order to collect the surface and underground water inside of the drainage gallery as demonstrated in Figure 4.25.

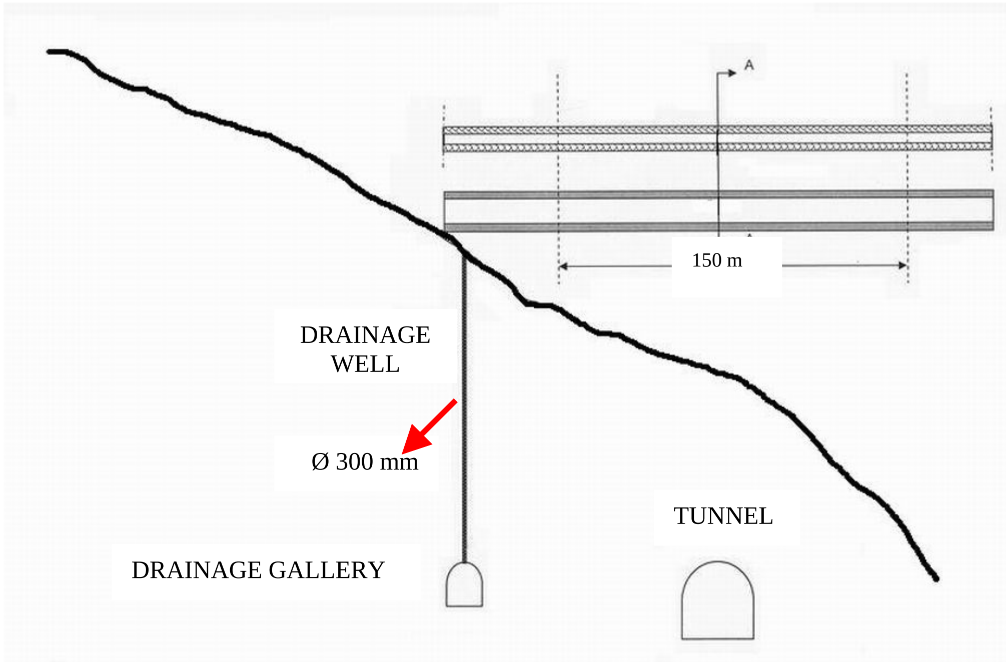


Figure 4.25 The Drainage System above the Tunnel

During rehabilitation work the earlier collapse in the drainage gallery was cleaned up. Drainage pipes having spacing of 1.0 meter, a diameter of 100 mm, and a length of 0.50 meter were inserted into the walls as shown in Figure 4.26.

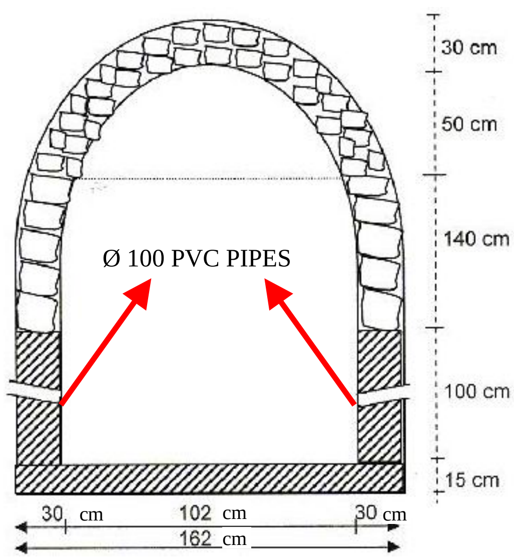


Figure 4.26 The Condition of Drainage Gallery after Rehabilitation

4.8 Unstable Sections Requiring Urgent Repair

Section 13 (The section between km 155-686.65 and 697.65)

This section was shotcreted and supported by 17 temporary steel-arches to protect existing stone lining. The condition of the old tunnel support (walls made up of) were in good condition.

Section 11 (The sections between km 155+666.65 and 669.95)

In this section, reinforced concrete-lining was completed and the steel-frame for making concrete-lining was removed after providing the stability of the adjacent sections.

Section 10 (The section between km 155+660.65 and 666.65)

There was a 0.60 m cavity between the steel-frame, used for making concrete-lining, and the tunnel's stone wall and the existing roof was unstable. Hence, by applying a grout injection to the roof, a concrete arch having a thickness of 3.0-4.0 m was formed

Section 8 (The section, 25 meters between km 155+631.15 and 655.15)

The concrete sleeves inserted on the original tunnel walls were broken. Moreover, axial cracks were observed on the wall at the mountain side. During the rehabilitation work, the suspended and peeled-off concrete sleeves were broken and dropped down by workers.

Section 7 (The section, 5.5 meters between km 155+625.65 and 631.15)

This is the section where a relatively small roof-fall occurred earlier. The concrete-lining between the mountain side and the roof arch was broken. Also, the concrete-lining in the roof was broken. During the rehabilitation works 10 new steel-sets were installed in this section and shotcreted and the existing cavities in the roof were back-filled by grouting. In addition, the concrete sleeve that was peeled off at the creek side was broken and dropped down.

Section 5/6 Boundary
(The 2 meters section between km 155+614.65 and 616.65)

There were some peeled-off rocks in the roof. The old steel-sets could be visibly seen in the wall at the mountain side. During the rehabilitation work the concrete sleeves on the creek side were poured, and coated by shotcrete.

Section 2 (The section 12 meters between km 155+550.80 and 562.80)

The sleeved concrete-lining on the creek side was fractured and suspended. In addition, there were deep cracks especially on the mountain side and arching towards the inner parts. During the rehabilitation works the concrete sleeves were peeled off and dropped down. Afterwards, 0.20 m thick shotcrete was applied over the steel- arches. Typical photographs showing the tunnel Regions requiring immediate repair and support are shown in Figures 4.27, 28, 29 and 30.

Figure 4.27 The Condition of Section 7 before the Rehabilitation of the Tunnel

Figure 4.28 The Condition of Section 7 after the Rehabilitation of the Tunnel

Figure 4.29 Examples of Spalling at the Tunnel before Rehabilitation Work

Figure 4.30 The Condition of the Sections 7 and 8 at the Mountain Side after Rehabilitation works

In Sections 2, 3, 4 (last 5 m to the Section 5 border), 7, 8, 10 and 12 NPI-160 steel-sets were installed with an interval of 0.50 m. Steel-sets were connected to each other by using I-120 steel profiles. The legs of the steel-sets were nailed to the concrete-lining by using 3 m rebars. This application is shown in Figure 4.31.

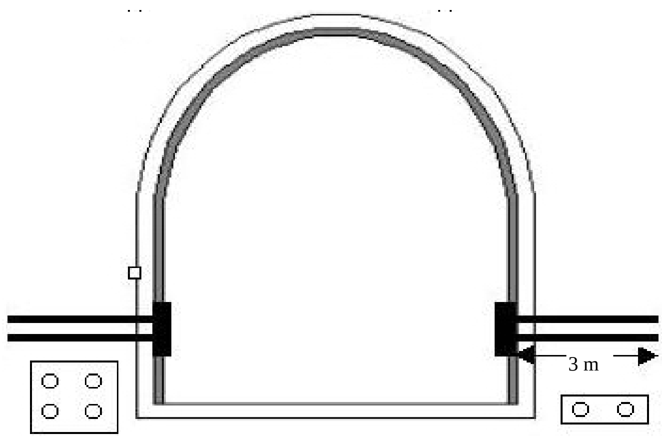


Figure 4.31 Nailing of the Steel-Sets by Using 3 m bolts

After injection of grout having a thickness of 8.0 m at the arch part of Sections 2, 3, 7 and 10, these sections were reinforced with ribbed or threaded bolts as shown in Figure 4.32. However, the thickness of the injection was 5.0 meters at the border of 3 and 4. And also, the quantities of the applied injection in different sections are given in Table 4.2.

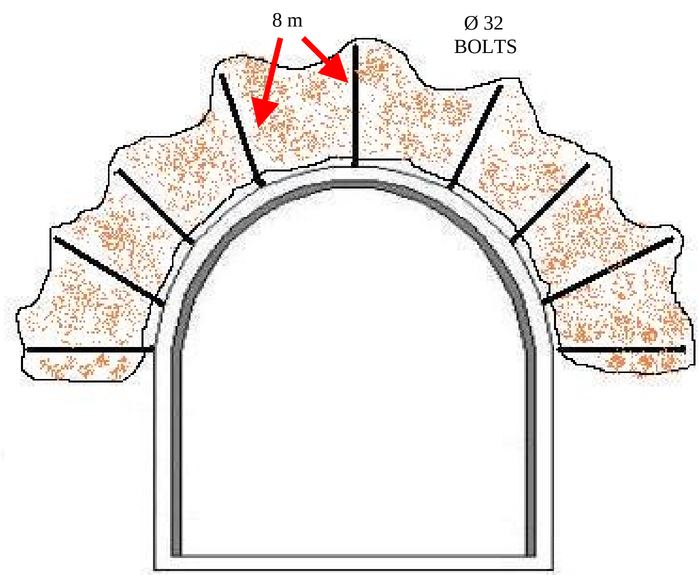


Figure 4.32 Concrete Injection Covering the Roof

Figure 4.2 The Quantity of the Applied Injection in Different Sections

Tunnel Sections	The Amount of the Injection Applied (m^3)	The Length of the Sections (m)	Unit Area (m^2)	Expected Thickness of the Grout Arch	
				Assuming a Ground Porozity of 100%	Assuming a Ground Porozity of 20%
2	243.144	12.00	20.262	1.730	5.67
3	58.716	6.35	9.247	0.89	3.27
3/4	22.032	5.00	4.406	0.42	1.86
7	112.308	5.50	20.420	1.74	5.70
10	110.424	6.00	18.404	1.59	5.31
12	48.000	17.00	2.824	0.27	1.27

CHAPTER 5

ROCK REINFORCEMENT IN NO: 7 RAILROAD TUNNEL

5.1 General

During rehabilitation work, the collapses were cleaned, adequate span was provided for normal tunnel traffic, voids were tried to be filled with grout injection and repair of the support was completed. Consequently, the tunnel was put into service on August 3, 2003.

After completion of the rehabilitation work, a total of fifty cores were drilled inside the tunnel in order to assess the rock mass conditions. In addition, to decide on the short-term behaviour of the tunnel and to evaluate the stability, deformation measurements were carried out.

Based on existing rock mass conditions and the numerical studies carried out using Phase 2D software, the lowest safety factor in the roof was found as 1.20 (Unal, 2003). Consequently, Self Drilling Anchors (SDA) and ground injection were decided to be carried out.

5.2 General Remarks about Rock Reinforcement

Tunnel Sections 2, 3, 4, 5, 6 and 8, located between km 155+548 and km 155+658 (Figure 5.1), have been reinforced systematically by using MAI type self-drilling anchors (SDA) and ground injection was applied in these sections.

5.3 Reinforced Sections (at the Walls of the Tunnel)

Before application of the bolting system, the tunnel has been divided into different support sections according to the bolting pattern used as A1, B and A2 as shown in Figure 5.1. The bolt length used in the tunnel was 6.0 meters.

Section B was the most problematic section of the tunnel affected by excessive deformation and displaced towards the creek side as much as 1.70 m. This section taking place between km 155+563-610 was reinforced with self-drilling anchors (SDA) as shown pattern in Figure 5.2. The other two sections, namely, A1 and A2 have been reinforced by the same type of bolts having different pattern as shown in Figure 5.3.

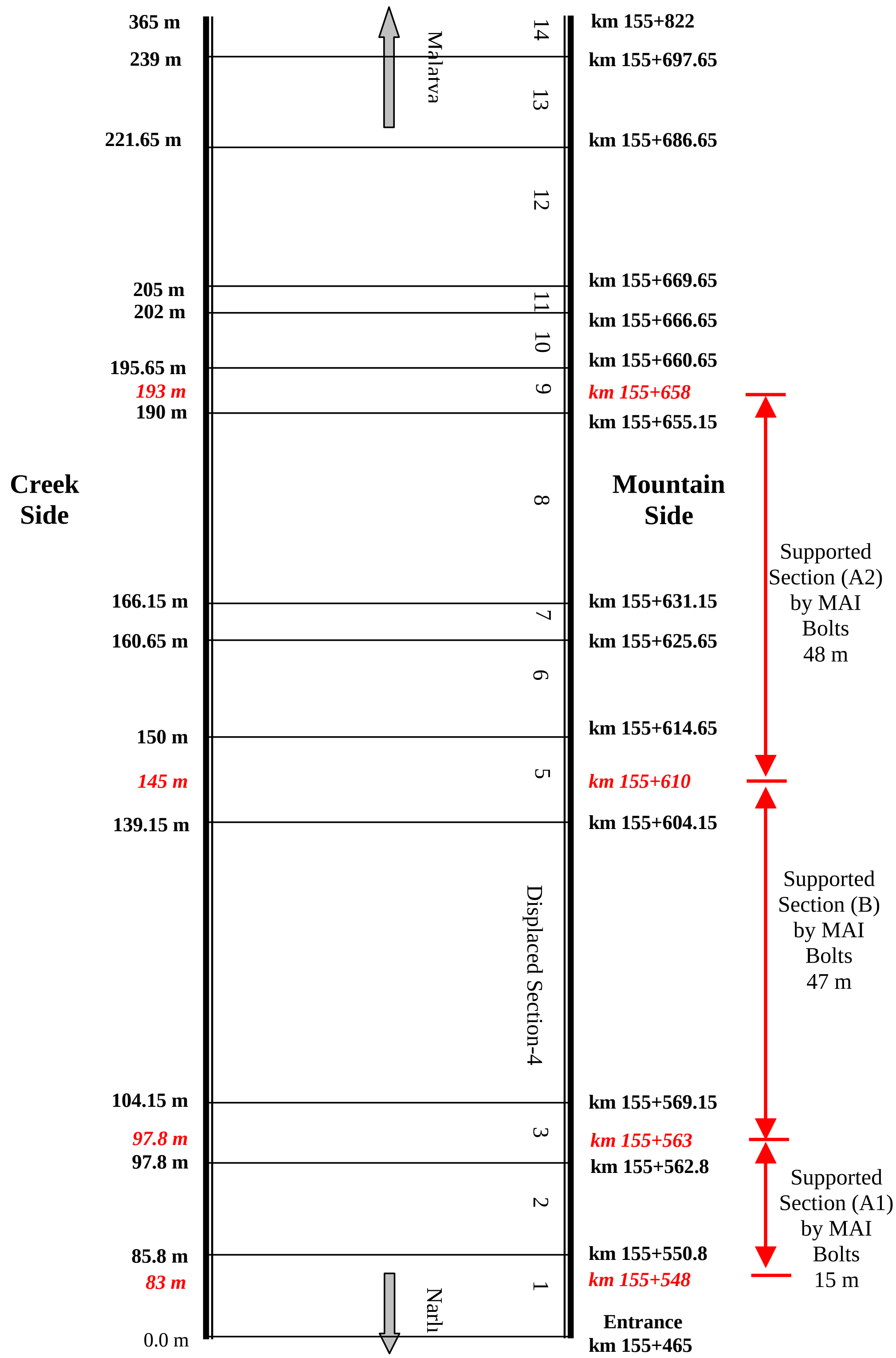


Figure 5.1 SDA Applied Sections for the Tunnel Walls

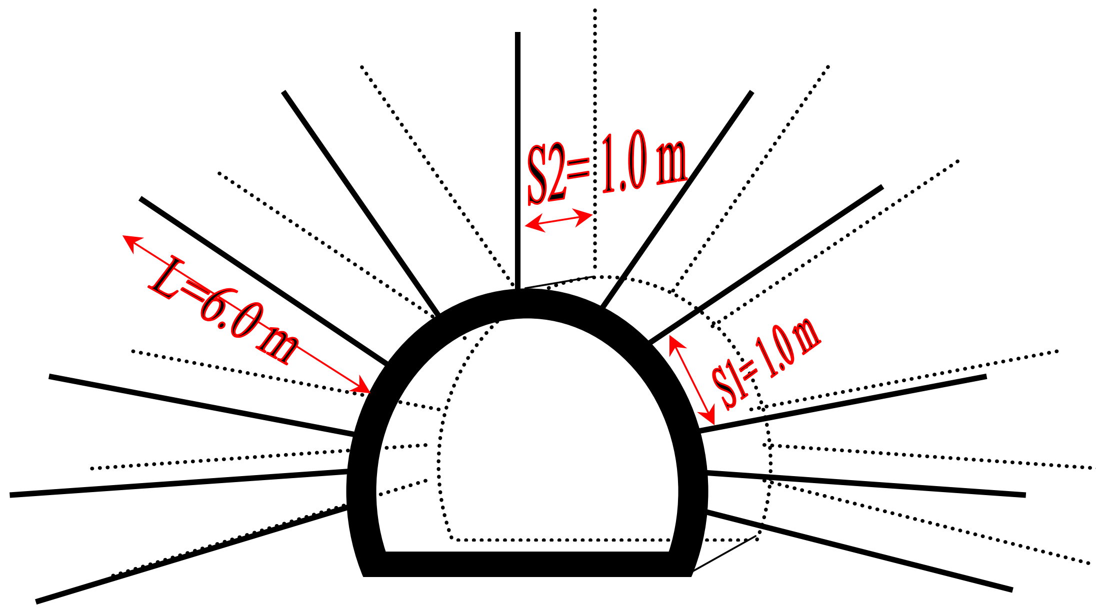


Figure 5.2 Mai Bolt (SDA) Installation Pattern for Section B

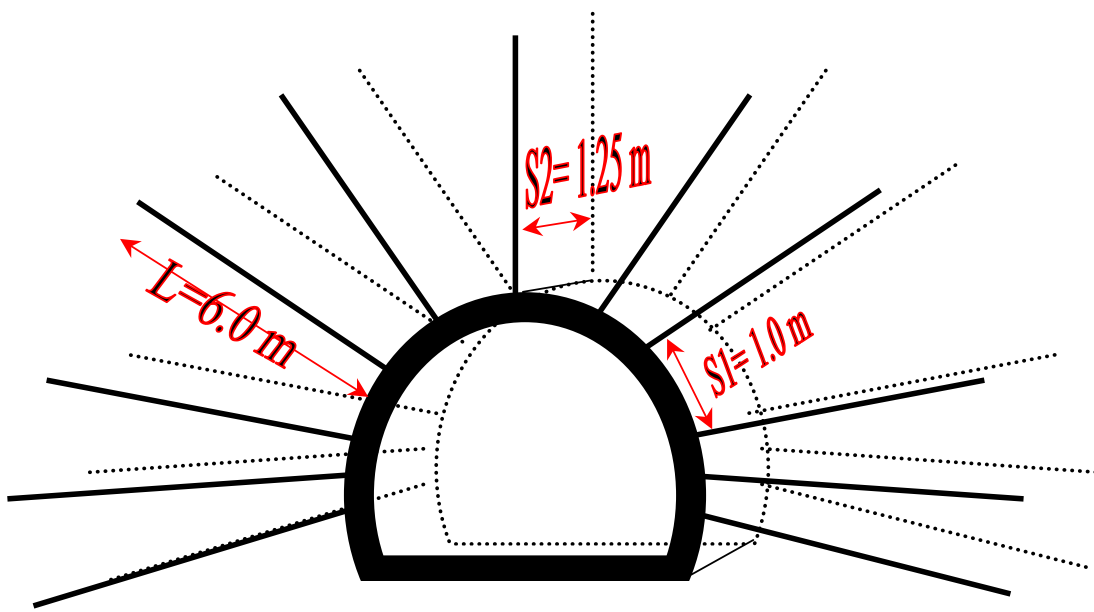


Figure 5.3 Mai Bolt (SDA) Installation Pattern for Sections A1 and A2

5.4 Stability of the Floor

After examining the cores drilled in different sections of the tunnel, the necessity to improve the stability of the tunnel bottom appeared especially at Sections 2, 3, 4 and 9 owing to the voids under the bottom of the tunnel.

There were neither concrete and invert nor support in the tunnel bottom at locations between km 155+545-631 and km 155+631-661 before the MAI bolt application has been started. There were just ballast, concrete invert having a thickness of 0.15 m and other fittings under the rail as shown in Figure 5.4. The suggested rock reinforcement application for the tunnel floor is also shown in Figure 5.5.

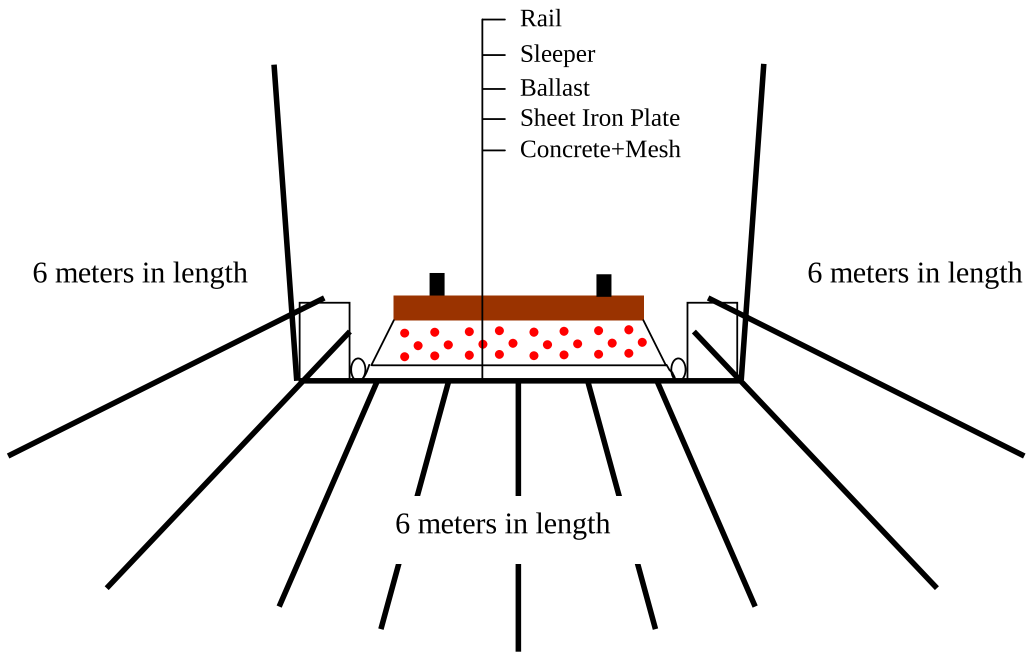


Figure 5.4 Rock Reinforcement Application for the Floor

Typical rock reinforcement work carried out in No: 7 Tunnel are shown in Figures, 5.6, 5,7 and 5,8.

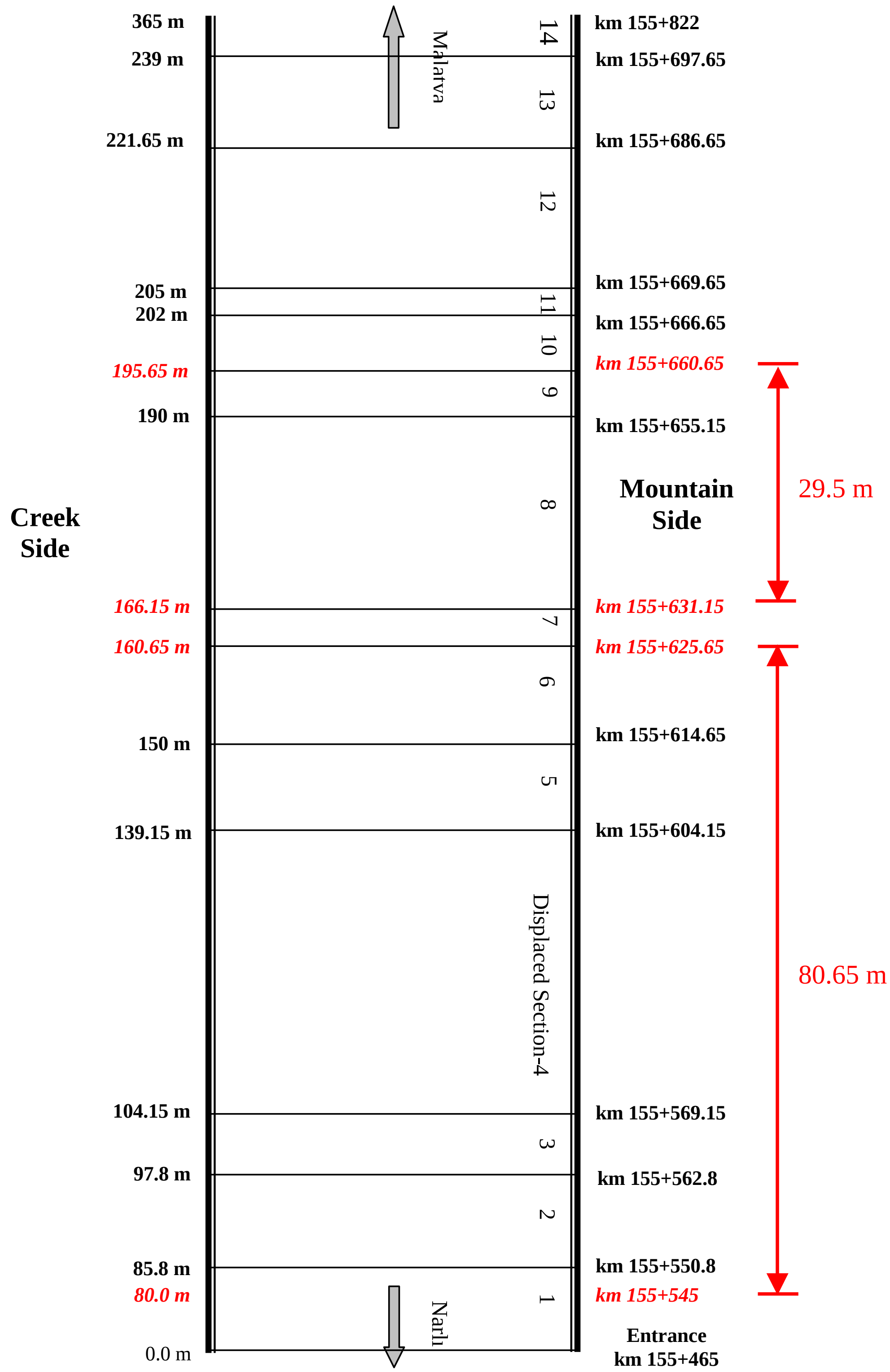


Figure 5.5 The Mai Bolt SDA Applied Sections for the Tunnel Floor

Figure 5.6 Jumbo Used in No: 7 Tunnel for Installation of MAI SDA

Figure 5.7 Coupling and Drill Bits Used for MAI Self Drilling Installation

(a) (b)

(c)

Figure 5.8 (a) Grout Injection, (b) Grout Coming Out of the Drill-Hole
(c) Rock Bolt Plate

5.5 Self Drilling Anchorage (MAI Bolt)

MAI Self Drilling Anchoring System (SDA) is a fully threaded steel bar which can be drilled and grouted into loose or collapsing soils without the use of a casing. The bar, or SDA, features a hollow bore for flushing, or simultaneous drilling and grouting and has a left-hand rope thread for connection to standard drill tooling.

The system can be installed in a variety of different soils and ground conditions ranging from sand and gravel to inconsistent fill, boulders, rubble and weathered rock as well as through footings and base slabs.

Self Drilling Anchors are installed with air driven or hydraulic rotary percussion drilling equipment, using a borehole flush medium suitable for the specific ground conditions.

There are three types of borehole flush: i) water flush for long boreholes in dense sand, gravel formations or rock conditions, for a better transportation of large cuttings and cooling of the drill bit; This procedure has been applied in Malatya-Narlı No: 7 Railroad Tunnel ii) air flush for short boreholes in soft soil, such as chalk and clay, where water spillage is to be avoided; iii) simultaneous drilling and grouting for all lengths of boreholes in all unconsolidated soil conditions.

Underground applications of the Mai Bolts include:

i. Radial anchoring for stabilization of tunnel circumference during excavation
ii. As forepoles, spiles or as an umbrella for advance protection of the excavation
iii. As roof piles for reaction load of steel support arches
iv. Slope stabilization of a tunnel portal

The system has six main components: bar, NG-coupler, hexagonal nut, bearing plate, drill bit and grouting MAI Pump as shown in Figure 5.9.

The MAI bar is manufactured based on API standard heavy walling steel tubing, cold rolled to form a rope thread profile (Atlas Copco Catalog, 2003). The rolling process refines the grain structure of the steel, increasing the yield strength, producing a durable drill rod suitable for a range of applications. The anchor bar's full-length left-hand rope thread gives the flexibility to adjust the bar length to the actual requirement. So, it is produced in 12 m lengths and then cut to size depending on requirements.

The NG-coupler enables direct end to end bearing between each rod, reducing energy loss and ensuring maximum percussive energy at the drill bit. It has a thread arrangement in which the top half of the thread is rotated against that of the lower, providing a centre stop for each bar. All couplers exceed the ultimate strength of the bar by 20%.

The hexagonal nut, which is machined with right-angled edges on both ends from high precision steel, is tempered to meet any stringent specifications and the daily operations of underground works. All nuts exceed the ultimate strength of the bar by 20%.

The bearing plate is a formed steel plate with a centre hole, allowing articulation of seven degrees in all directions.

The MAI Pump had a flexible adjustment of the water/cement ratio depending on the geology and positioning of the bolt which ensured that grout wastage could be reduced to a minimum (Figure 5.10).

Technical specifications of MAI SDA and MAI Pump used in Malatya-Narlı No: 7 Tunnel are shown in Table 5.1 and 5.2 respectively.

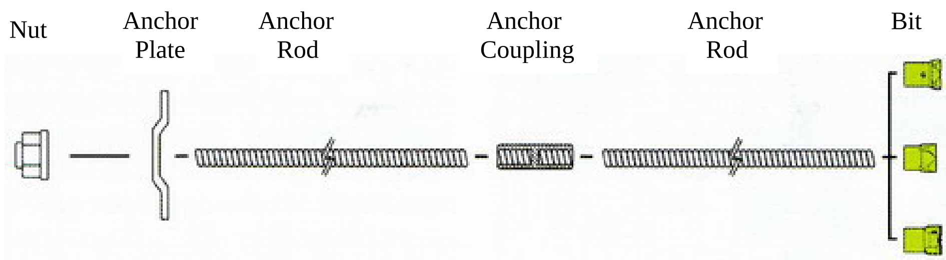


Figure 5.9 Components of MAI Bolt SDA (after Atlas Copco, 2003)

Figure 5.10 M400 MAI Pump Used During Pumping Grout into the Hole (after Atlas Copco, 2003)

Table 5.1 Technical Specifications of Mai SDA

ANCHOR ROD		
Technical details	Unit	R32N
Outside dia.	mm	32
Ave. internal dia.	mm	18.5
Eff. external dia.	mm	29.1
Ave. eff.cross-sect. area	mm^2	396
Ultimate load capacity	kN	280
Yield load capacity	kN	230
Ave. tensile strength Rm	MPa	720
Ave. yield strength Rp 0,2	MPa	560
Weight	kg	3.4
Length	m	3.0

COUPLING		
Size	Unit	R32N
Diameter	mm	42
Length	mm	160
Weight	kg	0.85

NUT		
Size	Unit	R32N
Key Size	mm	46
Length	mm	45
Weight	kg	0.35

PLATE		
Size	Unit	R32N
Dimension	mm	200x200
Thickness	mm	10
Hole-diameter	mm	35
Weight	kg	3.0

DRILL BIT	
Type	R32N
EX	R32/51

Table 5.2 Technical Specifications of Mai Pump

MAI PUMP		
Technical details	Unit	m400
Delivery rate	l/h	400-2400
Delivery distance	m	60
Delivery pressure	bar	60
Total length	mm	1800
Total width	mm	790
Total height	mm	1050
Total weight	kg	235
Electric supply	V	400
Amperage	A	13.7
Injection pressure	bar	4-7
Water/cement ratio		0.4-0.7

CHAPTER 6

FIELD MEASUREMENTS IN NO:7 MALATYA-NARLI RAILROAD TUNNEL

6.1 Introduction

In this chapter, details associated with convergence measurements will be given. In addition, information on convergence recorder “DISTOMETER”, locations of convergence stations, and installations of stations will be explained. Finally, the results of convergence measurements will be presented in detail considering the relation between “convergence” vs. “time”.

6.2 Convergence Measurement

In order to determine the short term behaviour of the tunnel and to evaluate the stability of the tunnel before rock reinforcement applications, a series of measurements were carried out at 15 deformation stations. These stations were installed through the tunnel. In each station, deformation between roof and floor, and two walls of the tunnel were measured.

6.3 Distometer

The Kern Distometer ISETH (a development by members of the Institute for Road and Rock Engineering of the Federal Institute of Technology, Zurich, Switzerland) is a mechanical precision instrument determining the differences in distances with the aid of invar wires. The system combines in one single instrument, a means for adjustment of tension force with a means for measurement of length both necessary or measuring with invar wires.

The required tension force is produced by a precision steel spring whose extension is measured by a dial gauge. The required tensile force of 8 kgf (78.85 N) is reached as soon as the dial gauge indicates the corresponding elongation which can be checked by a standard weight.

By means of a shiftable pull rod the Distometer can be varied in length between its connections at the setting bolt and at the measuring wire. Therefore, a strechted wire connected with the Distometer is subject to a varying tension force. The length of the Distometer necessary to maintain a tensional force of 8 kp yields the measured distance-except for an unimportant constant. A variation in the length of the observed line is thus indicated by a variation in length of the Distometer which is equal to the positional variation of the pull rod displayed by a dial gauge. The difference between two measurements values therefore represents the variation in length of the measured line.

A calibration gauge for monitoring the length of the Distometer and a standard weight of 8 kg for calibrating the dial gauge for measuring spring extension, are part of the Distometer equipment. A general view of Distometer used during convergence measurements is demonstrated in Figure 6.1.

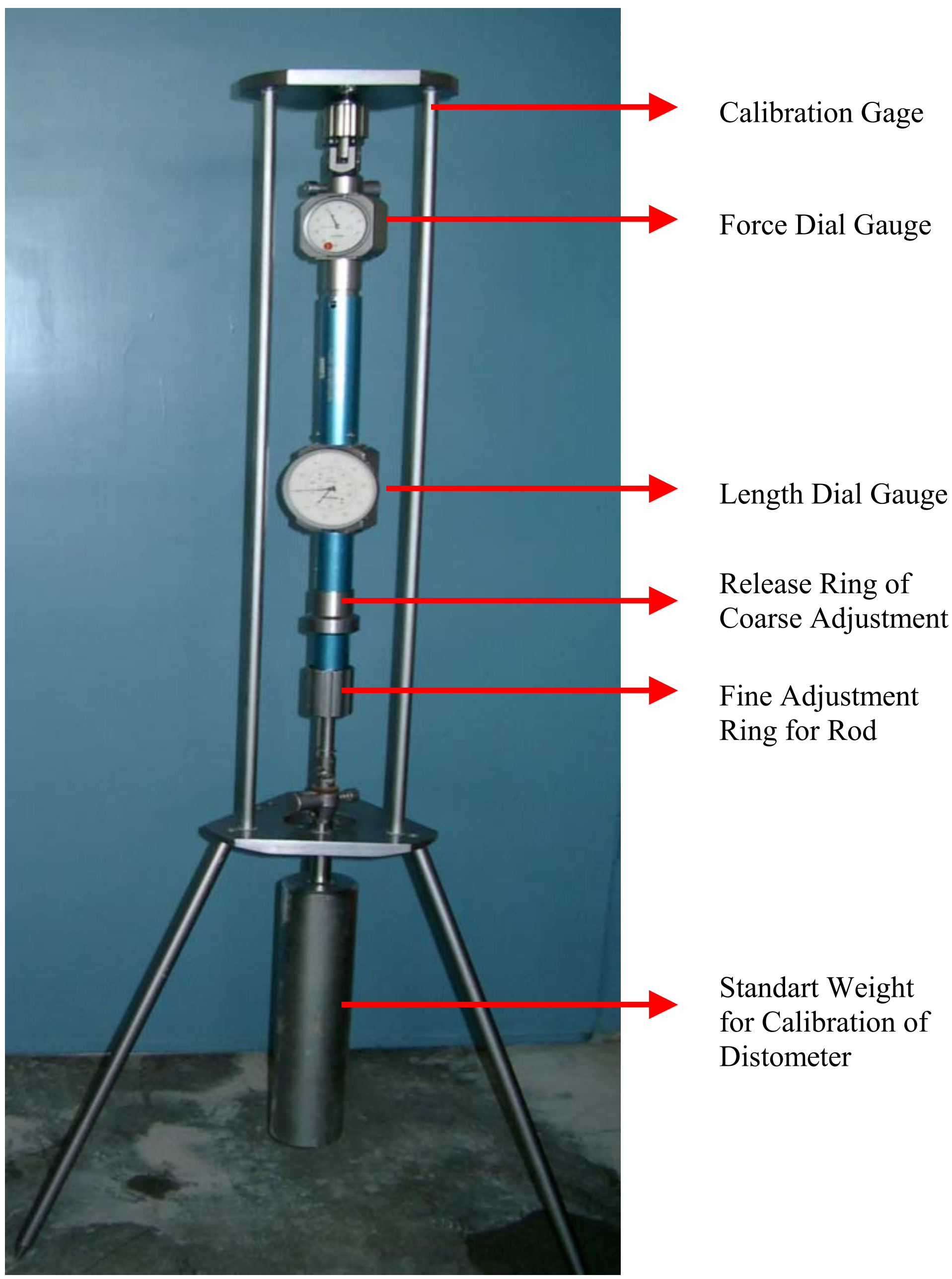


Figure 6.1 Distometer and its Basic Calibration Equipment

6.4 Locations of Convergence Stations

During field studies a total of 15 deformation stations were installed in the tunnel. 11 of these stations (Y1, Y2,.....Y10) were located in the deformed part, and 5 of them (Y11, Y12,......,Y15) were located in the non-deformed part of the tunnel. A general view of a convergence station is shown in Figure 6.2. The locations of the stations through the tunnel are shown in Figure 6.3.

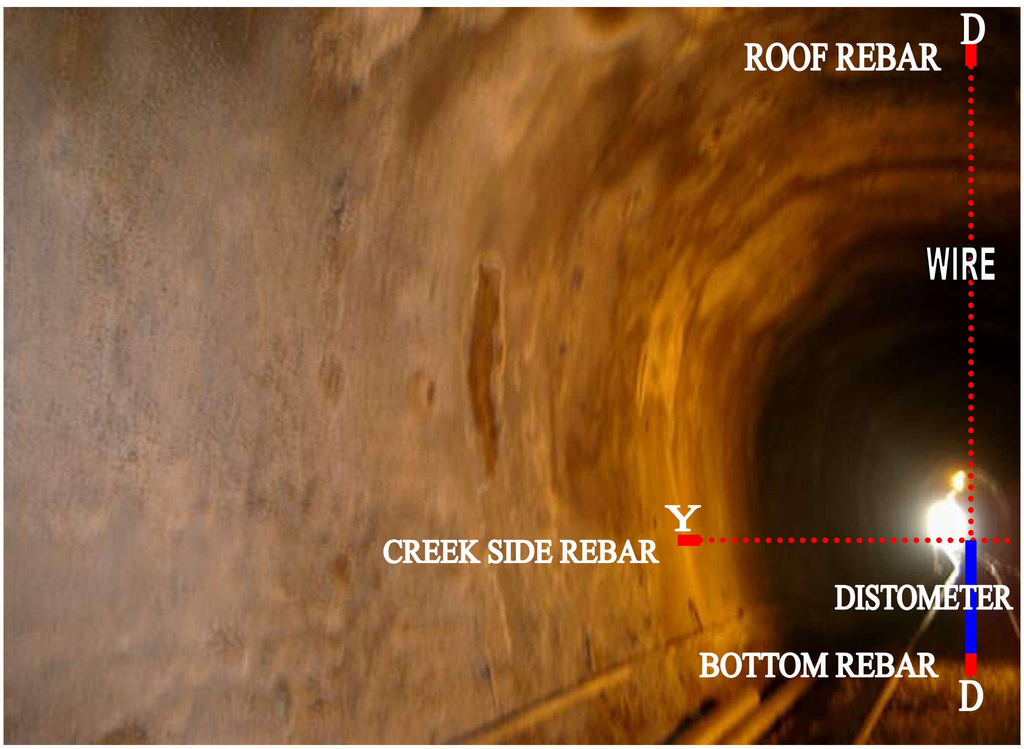


Figure 6.2 General View of a Convergence Station where Horizontal and Vertical Displacements are Measured

6.5 Installation of Convergence Stations

At each convergence station four steel rebars were fixed into the rock-mass after drilling holes in the both walls, the roof and the bottom, as illustrated in Figure 6.4.

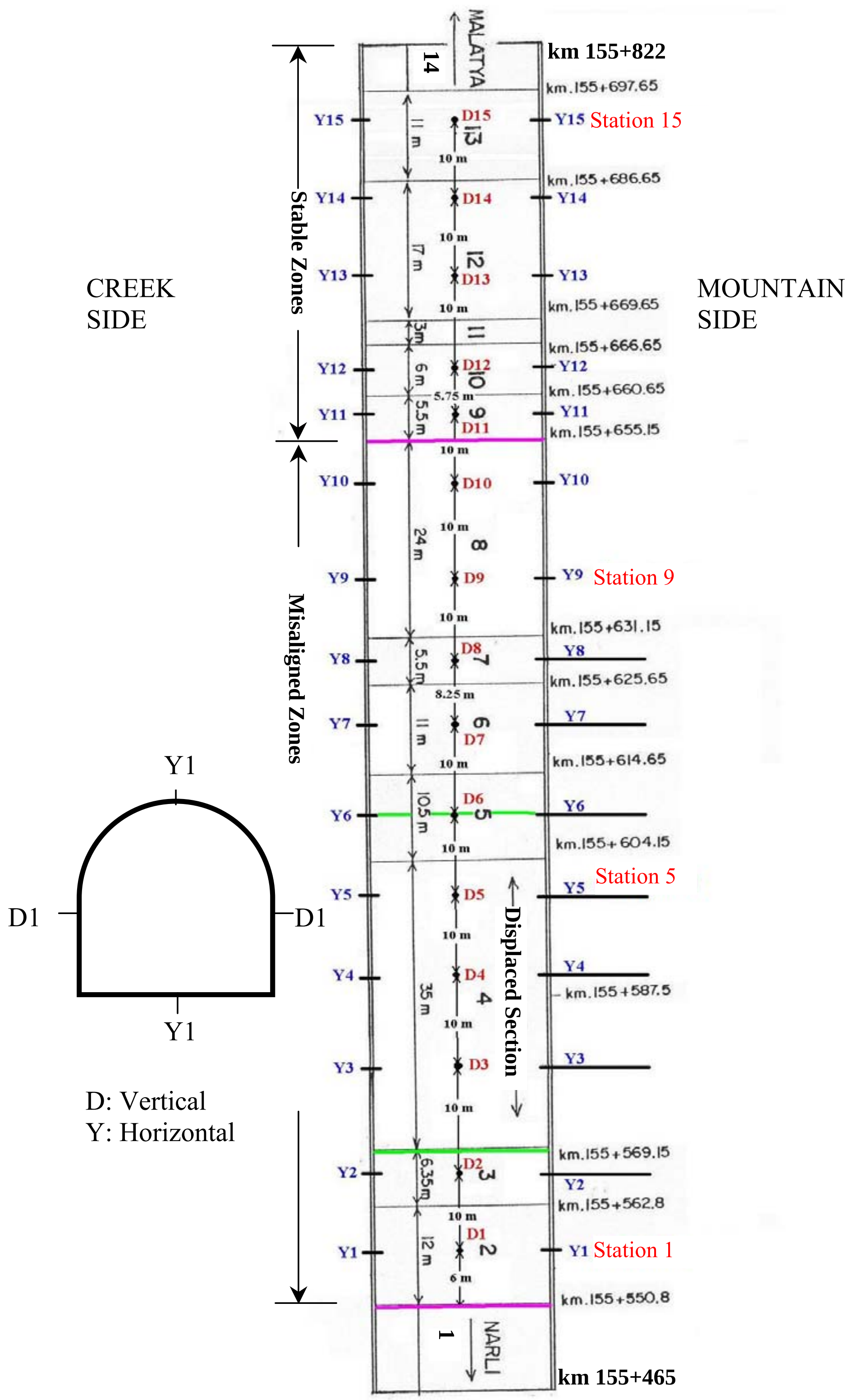


Figure 6.3 Locations of the Stations Through the Tunnel

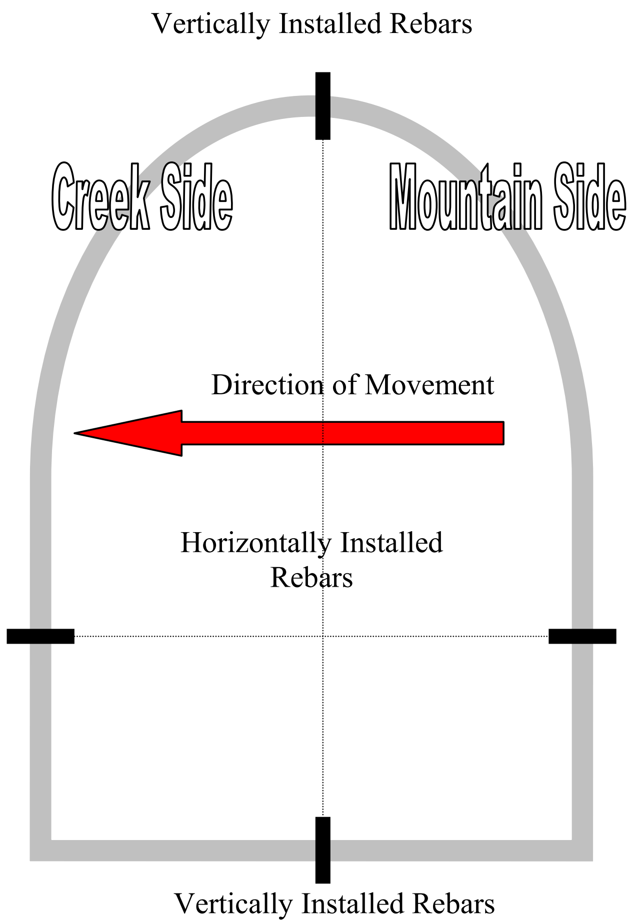


Figure 6.4 Locations of the Rebars Used as Measuring Points

Each station consisted of 4 rebars. The rebars fixed at the mountain side, between Stations 2 and 8 were 8 m in length. Whereas the remaining rebars were 0.50 m. The reason why 8.0 meters long rebars were fixed at the mountain side was because of providing a relatively stationary (not moving) anchorage inside the rock mass at the mountain side moving towards the creek side.

The convergence stations were installed by the help of the members of TCDD and contractor firm, Tokacar İnşaat.

6.6 Frequency of Convergence Measurements

Convergence measurements were taken as three times between the dates January 22 and April 30, 2004. Consequently, the movement occuring due to displacement of the tunnel from the mountain side towards the creek side was tried to be determined. The measuring period was about 100 days. The measurement could not be taken after this period because rock reinforcement was started in the tunnel.

Figure 6.5 Convergence Measurement at a Station

“Convergence” versus “Time” relationships were plotted for each station by using the data obtained from convergence measurements. Considering the total convergence occured in the tunnel maximum vertical convergence was 4.78 mm at Station 2 and maximum horizontal movement was 8.78 mm at Station 4. At this stage, it was concluded that rehabilitation work had not been adequate enough to provide stability of the tunnel and additional support would be required. Interpretations of the other measurements are presented in the next section.

6.7 Relations Between "Convergence" and "Time"

In this section the relation between "convergence and time" is evaluated. A total of 90 convergence measurements were taken in the tunnel in 100 days. Typical plots obtained from the measurements are shown in Figure 6.6. Considering the maximum vertical convergence at Stations 2 and 6 occured in 100 days, they were 1.15 mm and 4.78 mm respectively. The maximum horizontal convergence occured at Stations 1 and 8.0 were 2.40 mm and 6.66 mm respectively. Figures showing vertical and horizontal convergences obtained from different stations are presented in proceeding pages in this section. The measurements could not be continued due to initiation of the rock reinforcement work. However, it can be concluded with the available data that although small, that are still movements in the tunnel that may cause unstability.

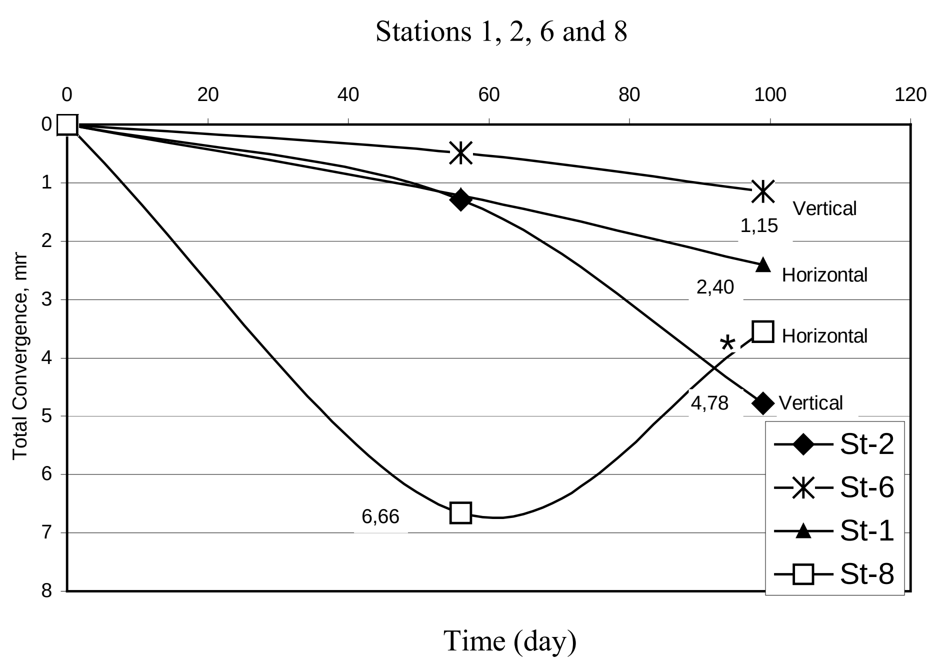


Figure 6.6 "Convergence" vs. "Time" Relation for Stations 1, 2, 6 and 8

(*) Fixed rebar was unstable so the measured value is not correct

The convergence results obtained from Stations 9, 10, 11, 12 and 13, installed in tunnel Sections 8, 9, 10 and 12, are less than 1.0 mm (see Table 6.1 and Figures 6.9 to 6.12). Although more data is needed to be able to interpret the results, it can be concluded that 167 meters of the tunnel is stable (including Section 14) as seen in Figure 6.3.

Due to the compression, closure was observed in the tunnel at Stations 4 and 5 installed in Section 4. The maximum horizontal movement was 8.78 mm at Station 4 probably due to compressive stresses acting from the mountain side. Consequently, it was concluded that there were still movement in Section 4.

Sections 2 and 3 were the most problematic sections in the tunnel in terms of water effect, especially in rainy seasons. At Station 1, installed in Section 2 horizontal closure and vertical dilation were observed as shown in Figure 6.7.

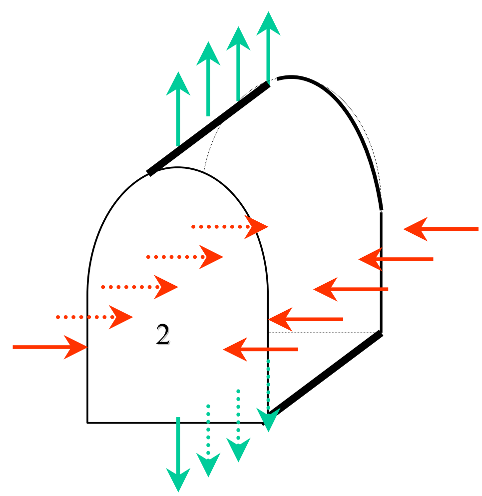


Figure 6.7 Horizontal Closure and Vertical Dilation in Section 2

The last kind of movement of the tunnel occurred at Stations 2, 3, 6, 7 and 8 installed in Sections 3, 4, 5, 6 and 7 respectively. Especially Sections 3 and 4, which are approximately 40 m in length, have a tendency of displacement from mountain side towards creek side. Eight meters long measuring pins or rebars were fixed in the rock-mass at the mountain side between Stations 2 and 8, in order to obtain a relatively stable point and observe the movement of the creek side wall. Based on the measurements in these sections, it can be concluded that the tunnel has a tendency of shifting from mountain side towards creek side as shown in Figure 6.8. Consequently, a reinforcement application is necessary to stabilize the tunnel.

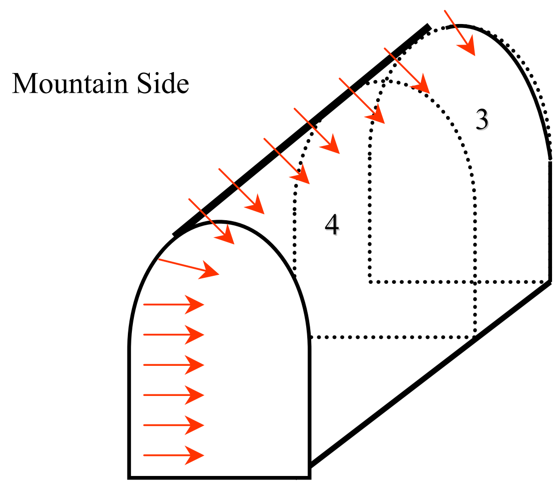


Figure 6.8 The Behaviour of Tunnel Sections 3 and 4

It is very difficult to interpret the results of convergence because only three measurements (including, “0”) had been taken in 100 days and then, the rock reinforcement process was started and caused measurements to be stopped. However, considering the horizontal and vertical convergences larger than 1.0 mm (please see, Table 6.1 and Figures 6.9, 6.10, 6.11 and 6.12). It can be concluded that:

i. In tunnel Sections 2, 3, 4 and 7 (Stations 1, 2, 4, 5 and 8) there were both horizontal and vertical movements.

ii. In tunnel Sections 6 (Station 7) there was horizontal movement only.

iii. These results show that Sections 2 to 9 No: 7 Tunnel are still instable and requires rock reinforcement.

Table 6.1 Measured Convergence Values

Station No	Section No	Distance from Narlı Side, m	Deformation (Vertical), mm		Deformation (Horizontal), mm	
			18.03.2004	30.04.2004	18.03.2004	30.04.2004
1	2	92	-1.98	-3.67	1.22	2.40
2	3	102	1.29	4.78	-1.87	1.22
3	4	115	0.84	0.31	-1.03	0.19
4	4	125	1.54	-0.21	7.91	-8.78
5	4	135	2.88	0.94	1.68	-0.48
6	5	146	0.49	1.15	-0.23	1.38
7	6	156	-0.19	0.30	1.48	-0.72
8	7	164	-1.26	-1.06	6.66	3.55
9	8	175	-0.74	-2.19	1.84	0.34
10	8	186	-0.78	-1.80	0.45	-0.08
11	9	195	-0.47	-0.82	0.07	0.19
12	10	200	-0.62	-0.06	0.27	1.25
13	12	210	0.53	-1.04	0.26	-0.10
14	12	221	0.43	-0.21	0.21	-0.56
15	13	231	0.27	-0.14	0.14	0.13

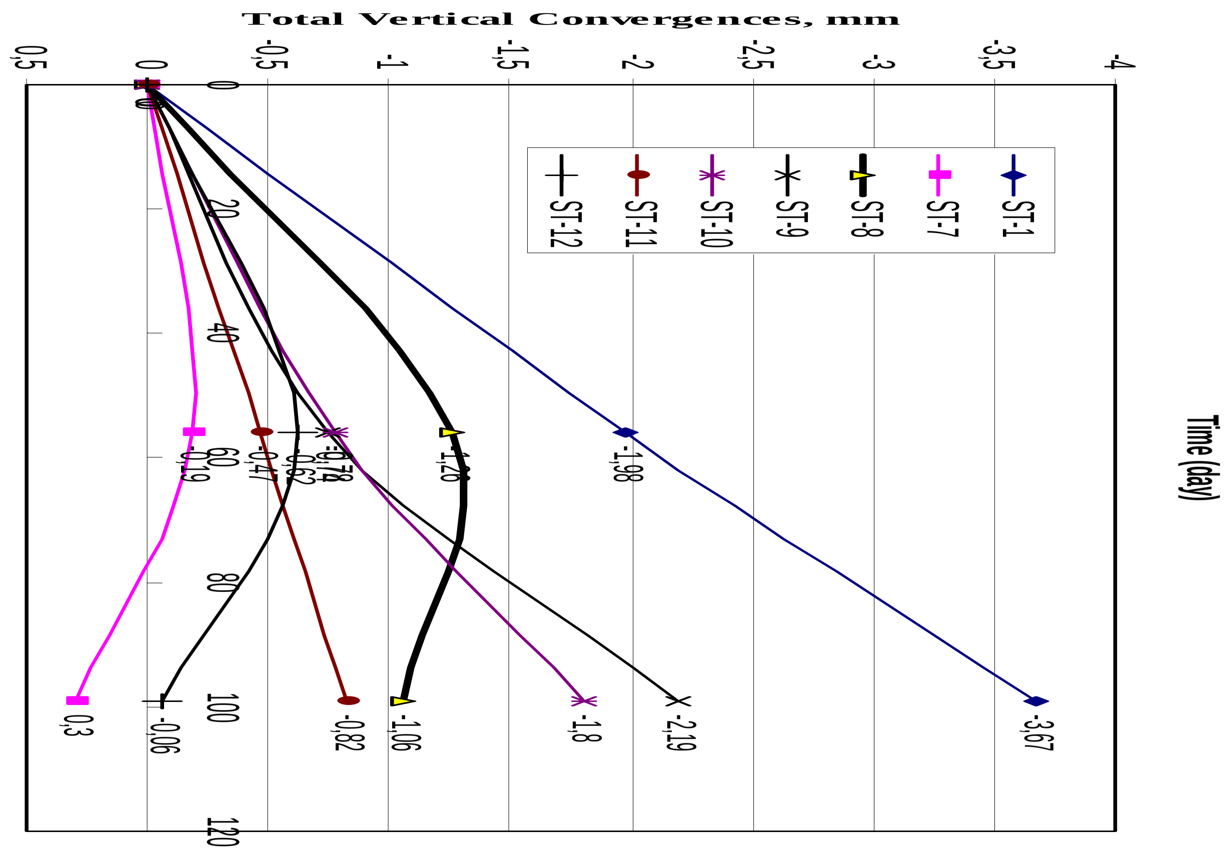


Figure 6.9 Relationship Between Convergence (Dilation) and Time for Vertical Stations

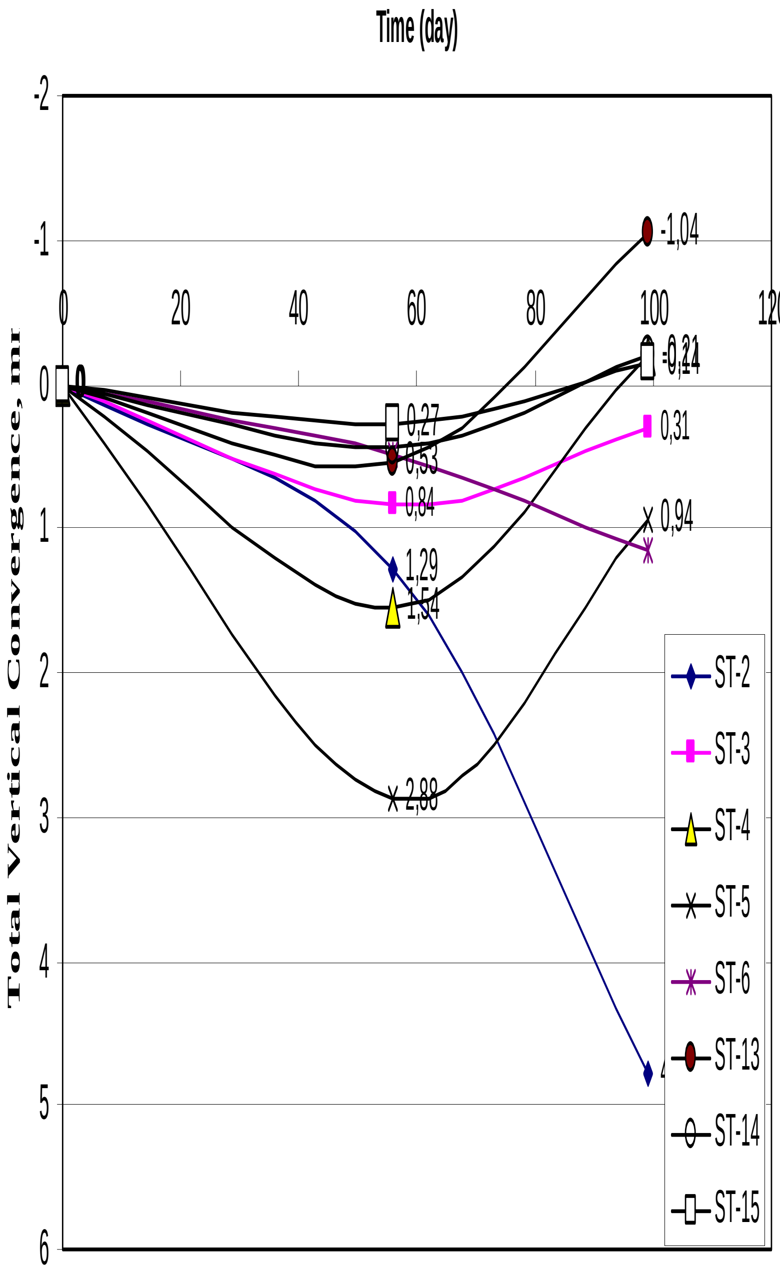


Figure 6.10 Relationship Between Convergence (Closure) and Time for Vertical Stations

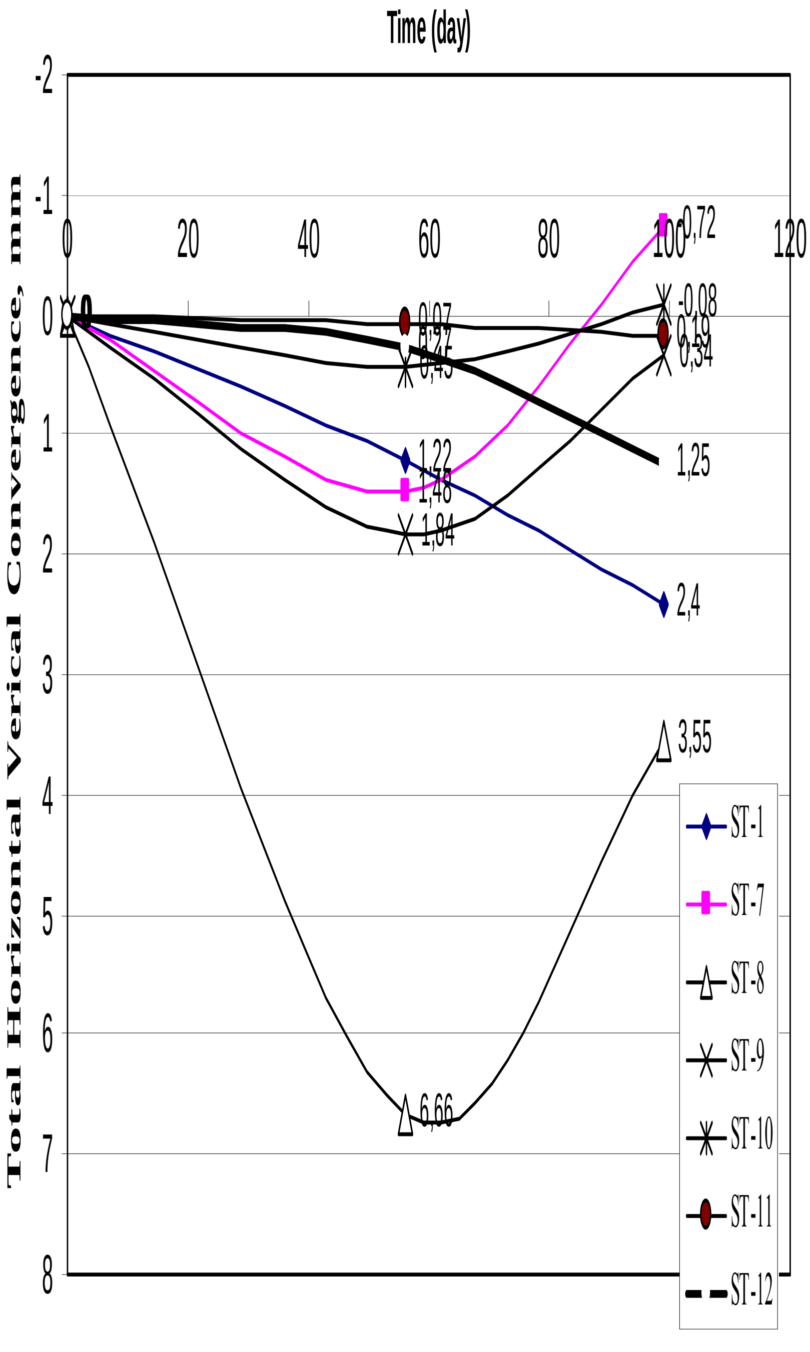


Figure 6.11 Relationship Between Convergence (Closure) and Time for Horizontal Stations

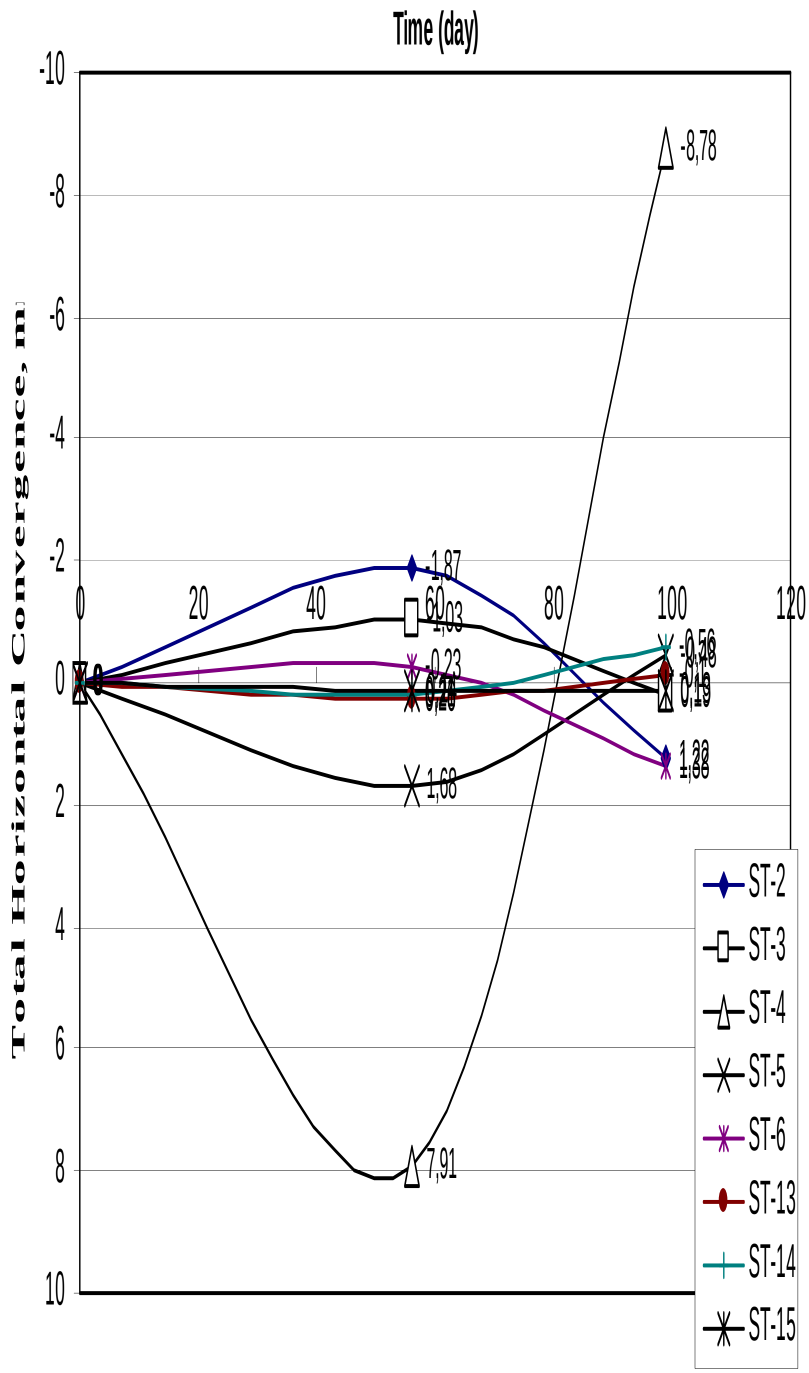


Figure 6.12 Relationship Between Convergence (Dilation) and Time for Horizontal Stations

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

Based on the studies carried out within the scope of this M.Sc. Thesis, the following conclusions can be drawn:

1) The extensive literature survey carried out in this study identifies the problems of the excavated tunnels and typical excavation work necessary for unstable tunnels.

2) The No: 7 tunnel located between Malatya-Narlı was excavated within the heel material of a paleo-landslide located over the old flood area in 1930. This was a big mistake in selecting a tunnel location on the first place. No: 7 Tunnel continuously faced with serious stability problems ever since it has been constructed.

3) Based on the field studies, the problems related to instability of No:7 Railroad Tunnel were categorized into five main groups:

   i. Collapse: Due to breakage of the existing concrete-lining, carried out in order to extend the span and to provide alignment, the first collapse had occurred on July 30, 2002. The second collapse occurred on February 7, 2003 while clearing up the roof-fall material in the tunnel. The reason of the collapse was a large unsupported roof span opened in very poor rock mass.

ii. Development of a Sinkhole: As a result of two collapses occurred in the tunnel a sinkhole having dimensions of 15x10x20 meters was developed at the ground surface.

iii. Reduced Tunnel Span: Owing to ground water effect and probably time dependent deformations (Creep), the tunnel between km 155+550 and 655 (105 meters) was misaligned and the tunnel between kilometers 155+569 and 155+604 (35 meters) was displaced towards the creek side resulting in an undulating shape of the tunnel, a reduced roof-span, distorted steel arches and cracked concrete-lining.

iv. Poor Rock Mass: As a result of cores drilled inside the tunnel at various locations it was determined that the rock mass around the tunnel was extremely weak having the following general characteristics; RMR=0, RQD=0 and 30<TCR<60.

v. Water: Underground water-table is located below the tunnel floor. On the other hand, the water leaking into the rock mass around the tunnel due to precipitation always creates problem. The water which could not be effectively drained disintegrates the extremely poor rock-mass and exerts pressure on tunnel lining. In addition, the water-table triggers the movement of the tunnel towards the creek side. Consequently, the water is the main source of instability of the tunnel.

4) During rehabilitation work two collapses occurred in different sections of the tunnel have been cleared up. In addition, a serious

problem, namely the reduced tunnel span was extended again to its normal width thus allowing normal train traffic-run again. However, so as to comprehend the short and long-term behavior of the tunnel 15 convergence stations were installed at different sections of the tunnel. The results of measurements indicated the possibility of tunnel shifting in particular locations.

5) It is very difficult to interpret the results of convergence because only three measurements (including, 0) had been taken in 100 days and then, the rock reinforcement process was started and caused measurements to be stopped. However, considering the horizontal and vertical convergences larger than 1.0 mm. It can be concluded that:

i. In tunnel Sections 2, 3, 4 and 7 (Stations 1, 2, 4, 5 and 8) there were both horizontal and vertical movements.

ii. In tunnel Sections 6 (Station 7) there was horizontal movement only.

iii. These results show that Sections 2 to 9 No: 7 Tunnel are still instable and requires rock reinforcement.

5) By evaluating the cores drilled at different sections of the tunnel it was detected that there were voids in the roof and side-walls of the tunnel. Especially in sections 2, 4, 5 and 6. It was planned to fill these voids by cement grouting during MAI bolt application. In addition, the voids the floor of the tunnel at sections 2, 3, 4 and 9 were decided to be filled with grouting during application of MAI Self Drilling Anchors.

7.2 Recommendations

1) A detailed study and application of a new technology in design of surface and underground water-drainage systems is necessary.

2) Studies related to improvement of the passing standards of the tunnel are necessary. This study requires enlargement of the tunnel size and locating larger railroad curves in the tunnel.

3) A detailed support design and stability studies for the tunnel should be carried out.

4) 3-D convergence measurements should be realized after all kind of work is completed in the tunnel.

5) Slope stability studies should be realized.

6) Inclonometers should be replaced in order to determine the slope movement.

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