EFFECTS OF GEOLOGICAL STRUCTURES ON TUNNEL EXCAVATION AND IMPROVEMENT WORKS AT KIZLAÇ TUNNEL ON TARSUS-ADANA-GAZIANTEP MOTORWAY

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ABSTRACT

Kızlaç Tunnel is the longest tunnel (L= 2800 m) 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. During the excavation a lot of fault zones of varying thickness (5-30 m) have been encountered and collapses occurred at some of the zones, which were not predetermined in field works and design phase. These collapses were eliminated by continuous evaluation of the data obtained from the tunnel face recordings, geotechnical monitoring and additional research works. This paper discusses the effects of geological structures on two consecutive collapses within 25 m (km: 14+822-41+847), the mislead tunnel excavation and support design due to the deceptive positive ground conditions between the fault zones, the control of the improvement works and basically the data about behind the tunnel face that can be obtained by geotechnical measurements in coordination with other recordings.

1. INTRODUCTION

Tarsus-Adana-Gaziantep section is on the southeastern part of the Trans European Motorway (TEM). Length of this section is 258 km; 40 km of the route reaches up to 940 m above sea level and the route crosses the steeply undulating terrain by a combination of tunnels, cuts, embankments and viaducts .

Four tunnels (Taşoluk, Ayran, Kızlaç, Aslanlı) existing in this part are horse shaped twin tube with three lanes and there are 30 m between centerlines of each tube. Kızlaç tunnel, divided into two sections as T3A to the west and T3B to the east, is the longest tunnel on 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

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excavated in rock formations consisting sandstone, siltstone and shale with zones of crushed and weathered material (Figure 1). During excavation a lot of fault zones of varying thickness (5-30 m) have been encountered and collapses occurred at some of these zones, which were not predetermined in the field works and design phase.

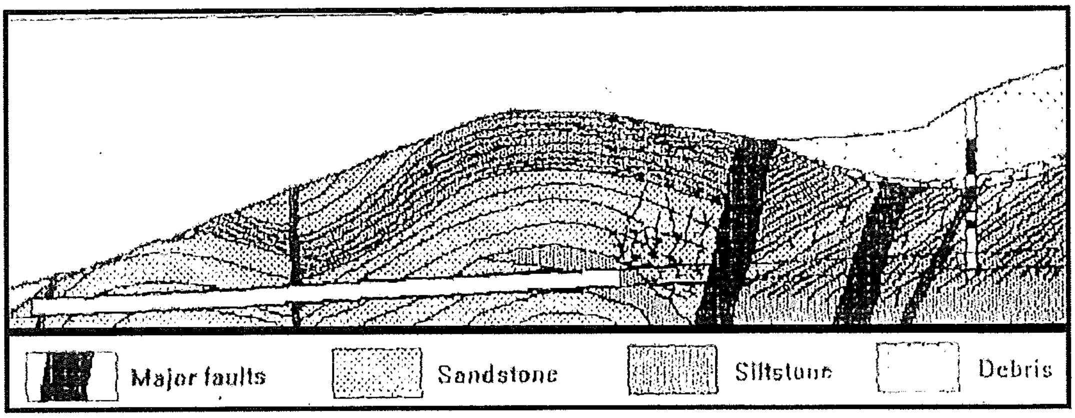

Figure 1: Longitudinal section showing geology

Tunnel designs and construction were performed according to NATM. The tunnels were driven by multistage drill and blast excavation. In the better ground conditions there is top heading followed by bench excavation, while in the worst ground conditions an invert may is provided for ring closure and top heading may is divided into two side drifts. In order to provide support to the face, support core at top heading excavation has been used (Figure 2).

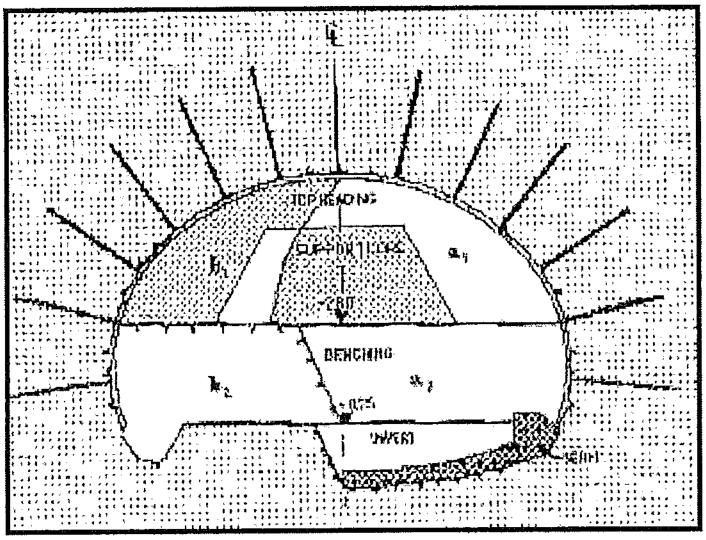

Figure 2: Excavation cross-section

While crossing fault zones, excavation and support of rock class C2 as; 0.7-1.2 m round length, 25 cm shotcrete thickness, forepoling pipes at crown with a spacing of 40-50 cm and length 2.5-3 m, wire mesh consisting two layers, steel rib–HEB 100, SN bolts of L=6-9 m and invert arch concrete had been used.

2. *COLLAPSES IN KIZLAÇ TUNNEL*

2.1. *FIRST COLLAPSE*

The tunnels advanced properly without any delay up to 30th September 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 meters for Round 158 and 159, 1.5 meters for Rounds 160-163 respectively,. and then rock support class was changed to C3 (highly squeezing) and round length was reduced to 1.2 meters.

About a week later, when the face was at Round 163, in Rounds 153 to 159 settlements, break in rockbolts 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 (Fig.3). Overbreak volume stabilized within three meters above the tunnel Crown. The collapse occurred while approaching the fault gouge with C2 rock class between Km: 41+831 and 41+840.

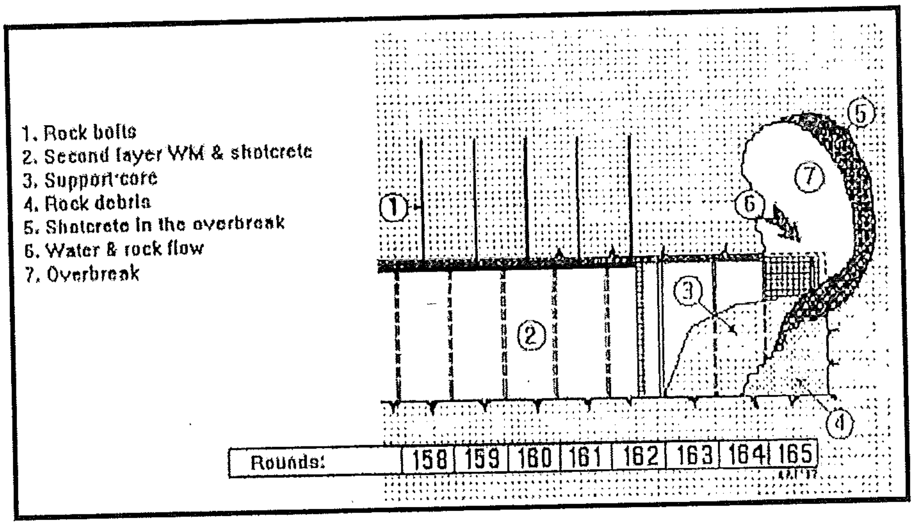

Figure 3: First collapse in Round 165

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 at the area indicated by 6 in the above figure (between rounds 163-169). The volume of flowing material was 650 m^3, the flow rate of water was 25 lit/sec, respectively.

As an immediate measure for this situation, support was reinforced with extra mesh and shotcrete and longer rockbolts were installed on the left side. The face of flowing material was given steeper slope, supported with shotcrete, mesh and face bolts, while a shotcrete arch rib was formed. Additional and continuous deformation and water flow measurements were done.

For determination of the extent of the area affected by the collapse, two boreholes fifteen meters each vertically, and three boreholes in Round 154 and 155 at inclinations of 30, 45, 60 degrees from horizontal longitudinally were drilled in the crown and both shoulders extending to eight meters above the tunnel crown. Water was drained and flow was measured twice a day.

Two months after collapse, a three layer umbrella made of perforated pipes drilled and installed at varying angles from flat, to 25^0 and 50^0 to has been formed as a curtain so as to prevent flow into tunnel, over the collapsed part of tunnel. Then more holes were drilled in the crown and collapsed zone, and grouted with cement grout, while extra rockbolts of six meter long were installed between the grout holes. Excavation was made by two side-drifts at the top heading and the middle top heading. The position of old face was recovered on 5 th March 1996 five months after collapse. By this time North Tube, thirty meters away, had advanced past the South Tube without any problem.

2.2. *SECOND COLLAPSE*

On 16 th March 1996, shotcrete was cracked in Rounds 138 to 146 in drived sections of tunnel. Up to 19 th March deformations had spread to Round 136 and 137. The second collapse occurred at the face at Round 179 (at Km: 41+845, while tunnel was approaching to second fault zone intersecting the tunnel axis with 70^0-75^0 angle). Before the second collapse, excavation and support class of C2 was used due to positively improving rock conditions between two faults, as observed at the face.

At the time of second collapse, C3 rock class for first collapsed area and C2 rock class for the face of secondly collapsed area was applying. After observing deceptive positive ground conditions between two faults, while approaching the second fault C3 rock class had not been used.

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 22 cm, later increasing to 27 cm. 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 (estimated), decreased to less than 3 lit/s within 4 days and less than 0.5 L/s within 10 days. (Fig.4).

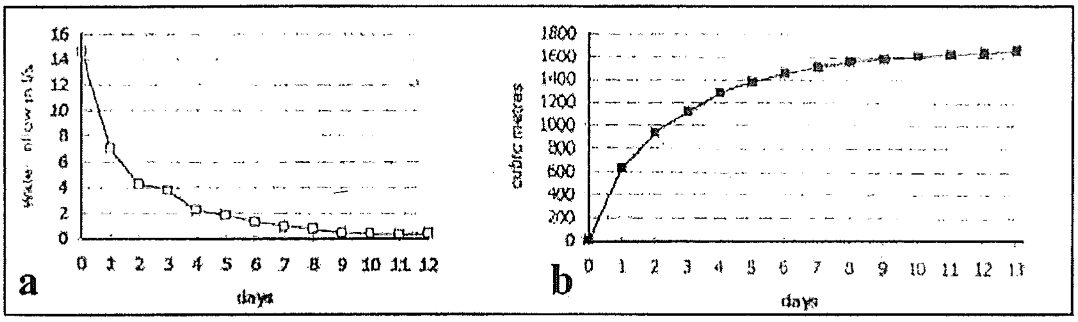

Figure 4: Second collapse-a: water inflow rate, b:total water intake

2.3. INVESTIGATIONS AFTER SECOND COLLAPSE AND MECHANISM

After second collapse data provided by following investigations was used; surface geology surveys, deformation recordings, tunnel face and both walls recordings, visual observations from tunnel engineers, measurement and analysis of inflowing water.

The excavation recordings in the same section of north tube corresponding to the same interval within south tube were as follows; the face of north tube advanced 30 m in front of the face of the south tube. Two fault zones corresponding to the collapses in south tube had been drived in the south tube properly by using C2-C3 rock support system and the distance between two faults was more than the distance in the south tube. During second collapse, the fault from South Tube had just been past between Km: 31+896-31+902, by a shortening in length.

Depending on data obtained, boreholes for further investigations were drilled on collapsed area as:

- S1 and S2 boreholes vertically from ground surface
- S3 inclined trough the fault zone from right wall of South Tube at Km: 41+827
- S4 horizontally trough the second fault from the face
- S5 perpendicular to the right wall in front of the collapse area, Km: 41+825, with a length of 30 m
- S7 with an azimuth of 30^0-40^0 from right wall near to the face, after cleaning the collapse material

- S8 straight ahead in the right of face, with 50 m in length.
- N1, N2, N3, N4 and N5 boreholes in North Tube.

The core logging of boreholes S1 and S2 showed that, grouting at the area of the first collapse in the south tube extended up 6-16 meters above the tunnel crown. Fault zone thickness ranges between 5 meters to 8 meters.

The collapses happened in an area where two secondary faults, crossing the tunnel nearly perpendicular, were joining each other and where a fault was striking parallel to the southern side wall (Fig.5). After second collapse the tunnel will be driven in fractured sandstone, and shale/siltstone and abundant fault gouge. According to boreholes in north tube, there is another fault dipping towards southern tube, in a manner to intersect the tunnel axis nearly horizontally (10^0-30^0) trending southeast.

At the right side, where two faults were joining asymmetric stresses occurred laterally. Seeping water accumulated in pressurized fault zone and was flowing out from weathered sandstone. The area in front of the face was not supported (only sealing shotcrete was applied). This area was also affected by additional stresses from remaining loose zone of first collapse. The rock around the tunnel and behind face (fault zone) could not sustain these stresses and failed subsequently.

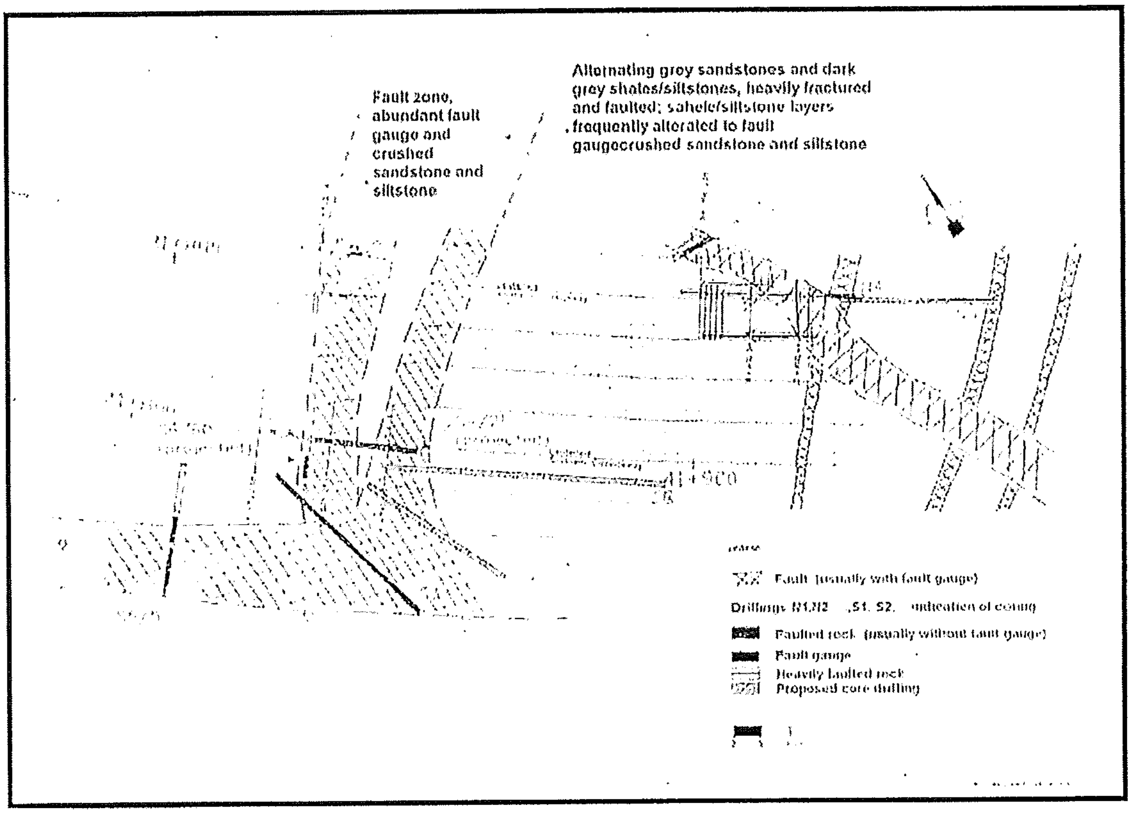

Figure 5: Geological horizontal section

3. TREATMENT AND RE-EXCAVATION PROJECT OF COLLAPSED AREA

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

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

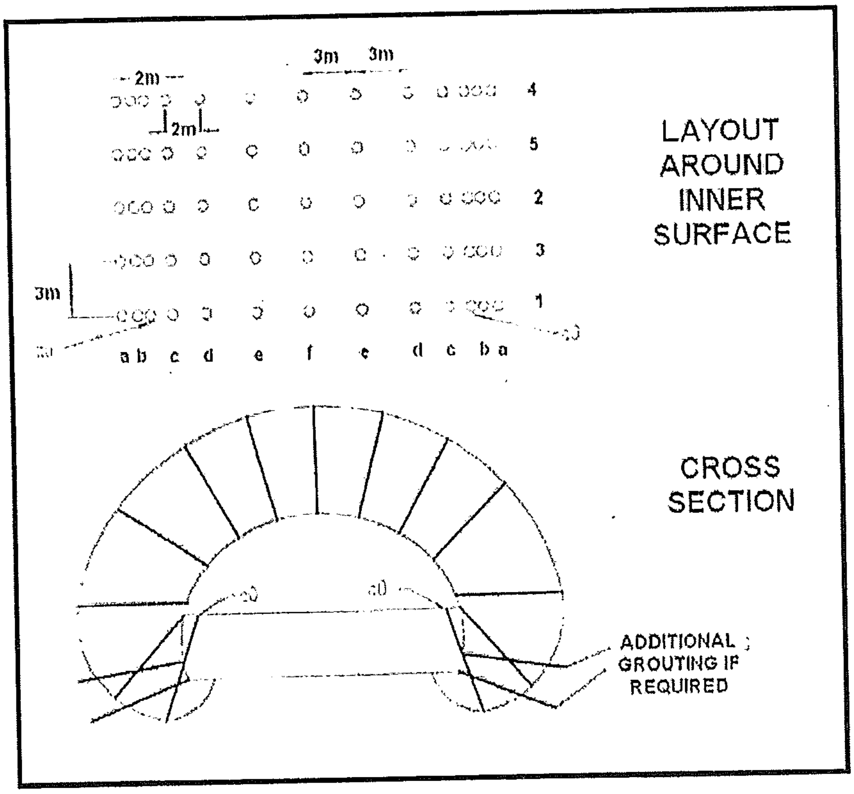

Figure 6: Layout of final grouting.

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.

Permanent drainage measures were applied. The water was collected outside the grouting ring by using perforated drainage pipes (diameter 50 mm, length 8 m). Pipes were installed slightly (5^0-10^o) upwards in the bench and radial in the top heading. The spacing of rows was 3 m.

Because rock mass was weathered sandstone, siltstone and shale, grouting achieved success at the fault zone and out side 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 meters. But according to data obtained from forepoling and bolt drillings some additional grout works were done where required. Perforated pipes around the periphery in the flattest possible angle at 30 m spacing were installed. Forepoling pipes diameter 2", for a length of 18 m was used.

Grouting created and injected area to prevent excessive flow in the tunnel (w/c ratio 0.3 to 0.4). Grouting began at the shoulder with a pressure of 3 bars. After grouting, the pipes were cleaned by means of pressurized air inserting a blowpipe. Middle of the tunnel was grouted from side drifts in order to form a grouted body .

The loose material in the tunnel was stabilized, a new face was cut and the face bolts were installed. Advance in the collapsed area is provided by application of two side drifts based on rock class 6, till undisturbed rock mass ahead of the previous face. (Figure 7)

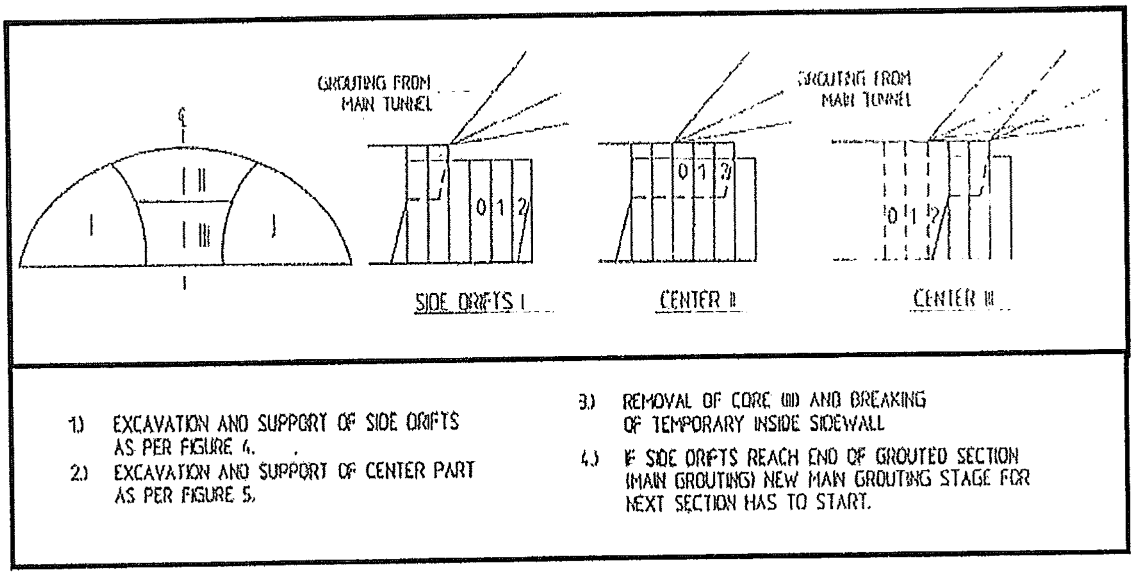

Figure 7: Excavation sequence; side drifts and center

When the center (middle) part of the side drift advance reached 3-4 rounds in undisturbed rock mass, excavation was stopped and shotcrete and rock bolts were completed. (Figure 8,9)

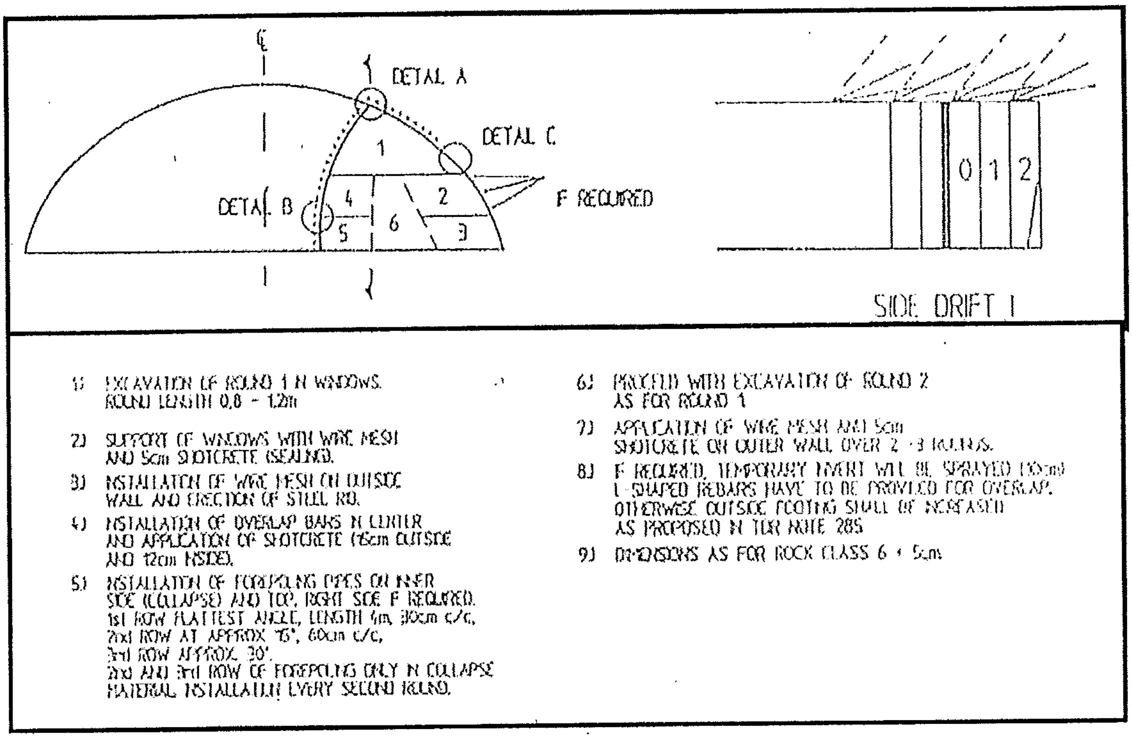

Figure 8: Excavation sequence; side drifts

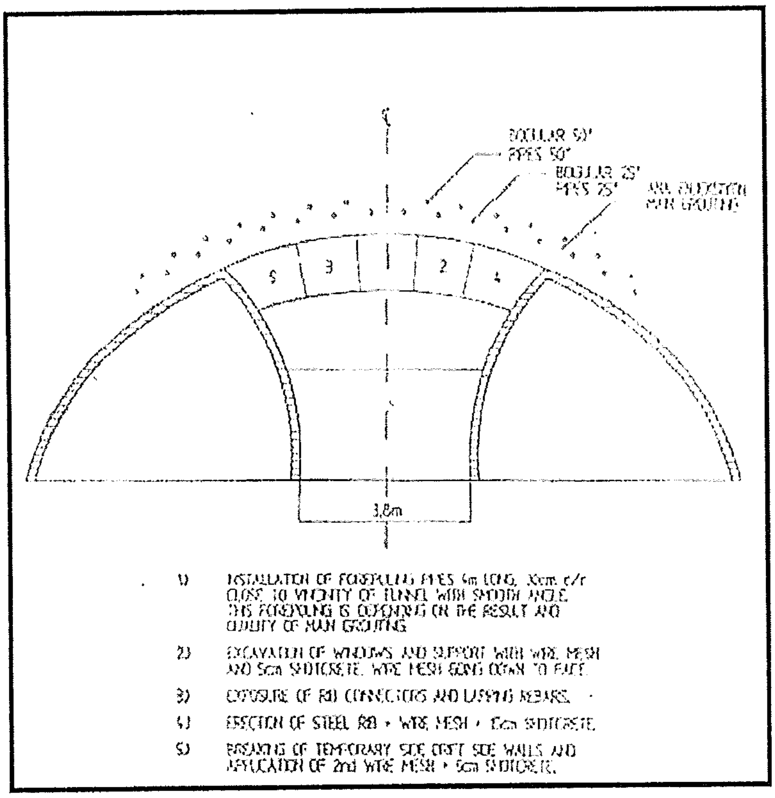

Figure 9: Excavation of center

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

Top heading advance in the south tube was min. 30 m ahead of the current bench in north tube. Carefully and controlled advance was done. Foundation beams and invert arch were provided as a monolithic structure.

4. CONCLUSION

The most dangerous risks in tunneling are generally those associated with inflows of ground, of water or of gas, from the zone around the tunnel or ahead of the face. Hence, the lesser knowledge on the coming ground conditions there exists a lower risk for problems that one may encounter during the tunnel drive. The knowledge to minimize such risks can easily be eliminated with detailed geological predictions before and during the tunnel construction. It is obvious from the problems that were encountered within Kızlaç Tunnel that, although there has been somehow very accurate data about the ground, due to the chaotic behavior of the nature, there is always a possibility of different ground conditions that cannot be easily predicted. In the case of Kızlaç Tunnel, although the geological predictions were accurate at the start of tunnel driving, the ground gave deceptive data after some two hundred meters resulting in mislead tunnel excavation and support design and collapses. The point here is that, it is a game played with nature and the rules of winning this game before loosing some scores, passes through not only relying on the predicted geological and geotechnical evaluations but also on continuous monitoring of changing site conditions, accurate analysis of deformation data, re-evaluation of predicted data, and the ability to apply flexible and immediate measures during unexpected situations in every phase of an excavation that is to be made under ground.

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