# Bolu NATM Tunnels, Changes to the Support System due to Soft Ground Conditions and Deformation

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ABSTRACT: This article makes a contribution to the data-base that illustrated our experience with displacement, which far exceeded the deformation tolerances anticipated in areas of tunnel excavations conducted in soft ground, and on the changes it created in the tunnel entrenchment. The subject of this text is an investigation into the deformation effect of soft grounds that cause unavoidable high deformations in the worst ground conditions, with high plasticity, and with a length reaching up to 50 meters in the tunnel alignment and vertically proceeded up to surface level, and additionally did not contain any hard ground, but consisted of faulting layers throughout, and the changes that this type of ground conditions inevitably caused in the tunnel supporting system.One of the biggest challenges during tunnel excavations in "Weak Flyschoid Series", with a high clay ratio and low geo-mechanical parameters, was the high risk of damage caused by induced deformations to the open tunnel geometry. Therefore, in order to prevent unforeseen and rapidly progressing deformations in the tunnel and any resulting faults, important changes were made and applied to the NATM supporting system.

## 1 INTRODUCTION

The Anatolian Motorway Gümüşova-Gerede Section is a portion of the Trans-European Motorway (TEM) network that links Ankara to Istanbul and Turkey to Europe. The Motorway Owner and Operator is Karayolları Genel Müdürlüğü (The General Directorate of Highways) who act on behalf of the Turkish State. The Engineer was a joint venture between Yüksel Proje A.Ş. of Turkey and High Point Rendel Ltd. of UK, with the Geotechnical Consulting Group of UK providing specialist advice on the Tunneling related matters, up to July 2001. The 115 km long Gümüşova-Gerede motorway was fully opened to service in 2006, and has a twin tunnel with average length of 2926 m. The twin tunnels were excavated in a faulted and heavily tectoniced sequence.

The Bolu mountain crossing twin highway tunnels, each 18 m in excavated diameter and each tube 2,963 and 2,790m long according to the new alignment. The ground pillar between the two tunnels was about 50m, with overburden cover above the tunnels varying up to maximum 250m, and with the majority of the tunnels under a cover of 80-150m, and alignment ground water levels at 45% - 95% of the overburden cover above the tunnel crown. The face excavations area of twin tunnels varied between 133m² - 260m², depending on ground conditions, lining thickness and deformations. Each tube comprised 13.00-18.2m excavation diameter for an equivalent circle, and the twin tunnels were excavated in a faulted and heavily tectonicsed sequence. The Tunnel designs were prepared based on the standard Austrian rock classification system, with the original tunnel design performed according to NATM principles, with shotcrete, rock bolts and light steel sets. The project layout and location are shown in Figure 1.

As already noted, the original tunnel design was based on the principles of NATM tunnels. However, as major problems were encountered during the tunnel excavations where deformations occurred due to radial convergent movement reaching up to 1-meter, extensive changes were made in 1998 to the original design. Consequently, a different tunnel design system and methodology was developed based on the geo-mechanical characteristics of the ground that necessitated 'stepping outside' the NATM principles.

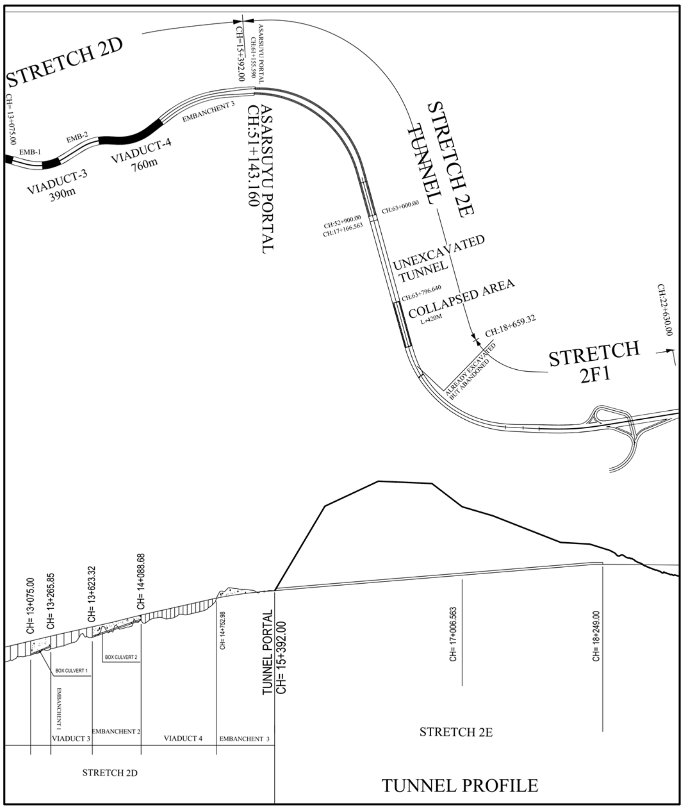


Figure 1. Project layout (plan and profile) and location of Bolu Tunnels.

## 2 TUNNELING GROUND CONDITIONS

### 2.1 *General Geology*

The tunnels required boreing through highly tectonised and faulted sequences of rock. The fault systems within this region are classified as seismically active at the first degree. As noted, the geology at and near the fault zone consisted of highly tectonised, intermixed with a series of mudstones, siltstones and limestones, with stiff heavily slicken-sided highly plastic pure fault gouge clay (Geoconsult, May 28, 1998). The boring of tunnels were therefore highly influenced by the above referred conditions, where the Low angle Contact Fault Gouge and Fault Breccia constituted of low to medium, and partly high plastic, stiff, extremely slicken-sided silty-clay, coupled with low residual shear strength parameters. The proportion within the clayey matrix by volume varied from between 30% to 100%. In some zones the ground consisted of uniform fault gouge without hard inclusions, comprising the minimum favorable conditions. Of these, such zones had been encountered in thickness of up to 50m along the tunnel alignment. And equally, it also extended vertically up to ground level (with an overburden cover of 80m to 120m of poor material). Along some sections of the tunnel the dip of the slicken-sided surface had been towards the face with the potential for large blocks sliding on such surfaces. Face bolting was used to reduce this risk. This fault gouge material was at the interface between Asarsuyu – Elmalık geological formation. The proportion of the clayey matrix varied substantially between the different geotechnical units, such that the worst ground comprised of wide zones of uniform pure fault gouge clay. Fig.2 shows the location of some of the more extensive of such zones.

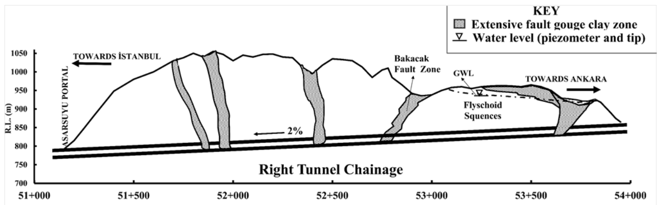


Figure 2. Simplified geological longitudinal tunnel profile and measured pre-tunneling ground water levels.

### 2.2 *Geo-Mechanical Properties of Soil Conditions*

Generally, the existing ground consisted of sub-angular blocks of hard material within a clayey matrix. The proportion of the clayey matrix varied substantially between the different geotechnical units, such that the worst ground comprised of wide zones of uniform pure fault gouge clay. In some zones the ground consisted of uniform fault gouge with no hard inclusions, comprising the least favorable conditions. The mixed ground conditions made tunneling conditions difficult. Along some sections of tunnel, the depth of the slicken-sided surface had been towards the face with the potential for large blocks sliding on such surfaces. In 1998/99, a detailed characterization of the ground ahead of the tunnel faces was implemented via a pilot tunnel test program (Geoconsult, May 28, 1998). The mechanical properties are given in Table 1. The following tests were used to quantify the geotechnical design parameters, which are summarized in Tables 1 and 2 (Menkiti, C.O. & Işık, S. (June 8, 2000).

Table 1. Measured strength and stiffness parameters

| **Unit** | **Peak** | | **Residual** | | $G_o/s'_v$[7] |
|---|---|---|---|---|---|
| | **φ[6]** | **c[6](kPa)** | **φ[6]** | **c[6](kPa)** | |
| High PI[5] flysch clay (clayey matrix 80% to 100%) | 15°-17° | 100 | 9°-12° | 50 | 500[1] |
| Blocky flysch clay (clayey matrix 60% to 65%) | 20°-25° | 100 | 13°-17° | 50 | 650[1] |
| Area 3 FG[3] clay | 13°-16° | 100 | 9°-12° | 50 | 700[1] |
| AS/EL FG[3] clay | 18°-24° | 100 | 6°-12° | 50 | NA[4] |
| Metasediments | 25°-30° | 50 | 20°-25° | 25 | 825[1] |
| Crushed MCB | 20°-25° | 50 | 15°-20° | 25 | 950[1] |
| Sound MCB[2] | 55° | 1500 | NA[4] | NA[4] | High |

1: From high quality pressure meter tests, 2: Metacrystalline basement rock, 3: Fault gouge, 4: Not available, 5: Plasticity index; 6:$\phi$, c' = effective stress friction angle and cohesion, respectively, 7: $G_o/\sigma'_v$ = ratio of max shear modulus to initial vertical effective stress.

Table 2: Summary of additional ground investigation carried out for design review in 1998/99.

| Investigation method | No | Meterage | No.of tested samples | | | |
|---|---|---|---|---|---|---|
| | | | Block | Core | Pressure meter | Piezometer |
| Exploratory pilot tunnel | 2 | 829 | 40 | 11 | 2 | 9 |
| Sub-surface boreholes | 22 | 531 | - | 24 | 9 | 5 |
| Surface boreholes | 3 | 348 | - | 11 | - | 3 |

## 3 DEVELOPMENT OF DEFORMATIONS AND EFFECTS OF SOFT SOILS (INSTRUMENTATION RESULTS)

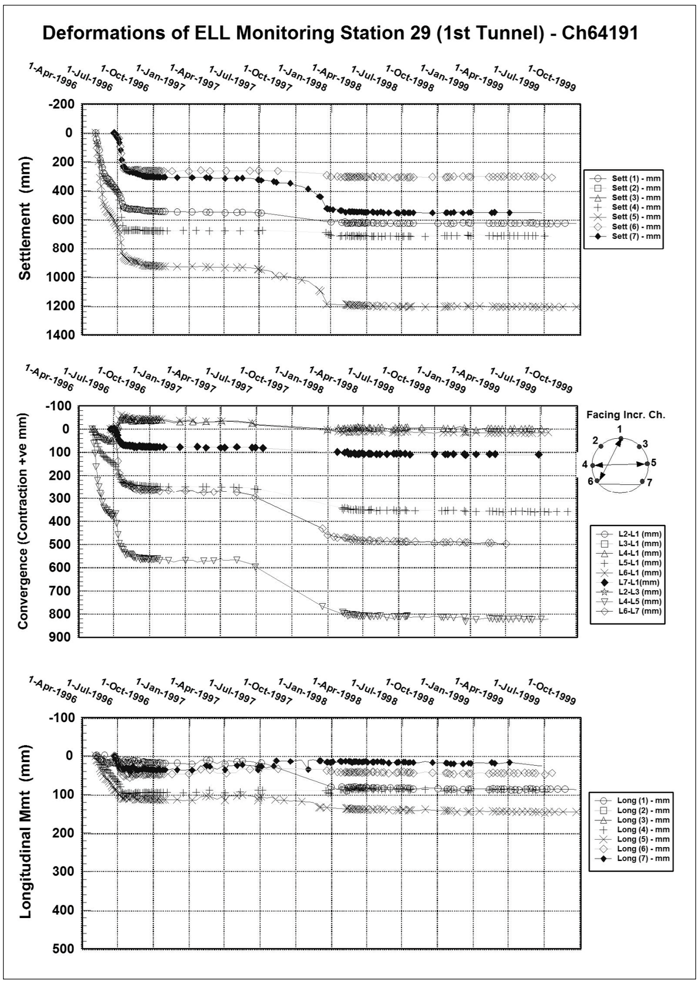


Figure 3. Typically, 3D deformation behavior of tunnel section in fault gouge that contains clay.

The poor ground conditions resulted in large deformations of the primary shotcrete lining arising in heavily squeezing clay zones, where deformations in excess of 1m occurred. However, when the tunnel excavation (in both tubes) reached a large fault line, a significant amount of radial deformations were recorded, that measured in excess of 1000mm, which were characterized by an initial large excavation induced movement followed by slower but continuous creep movements. Additionally, movements were observed due to the interaction between the two tunnels. Such deformations continued to be observed in both tubes on the Elmalık side, where large deformations in excess of 1m were measured, and which equally contributed to 800mm average settlement of the top heading, as illustrated 3D in figure 3. The overburden thickness in this section was 105m where we observed significant movements in the low-strength rock formation. The overall vertical displacement was 1200mm with an overall horizontal displacement of 813mm. Flyschoid series and extremely faulted slicken-sided, crushed highly to completely altered metacrystalline basement contact zone (compressible soil). Figure 3 shows the 3-D optical deformation reading for the first supporting system (s/c, steel ribs, and bolt) ((Menkiti, C.O. & Mair, R.J. & Miles, R. (April 2001), "Tunnelling in Complex Ground Conditions in Bolu, Turkey" for Publication in the Proceedings of Underground Construction 2001)).

An inspection of the tunnel was carried out after these unpreventable deformations occurred, and the following damage patterns were noted.


Figure 4. Photograph for tunnel profile at the fault gauge clay where deformations were excessive (Elmalık Left Tube, HEB-100, 20.4kg).

In the Elmalık right tunnel within the Flyschoid and Fault Gauge damage was observed as slabbing and spalling of shotcrete and circumferential cracking and serious deformation of potentially weak zone TH-Bench joint. In the Elmalık left tunnel, in fault gouge clay, moderate-severe damage was observed to the shotrete, where the shotcrete arch lining comprising S/C slabbing / spalling, S/C compression crushing and associated steel rib (HEB-100 20.4kg) buckling at crown, shoulder and knee areas (Figure 4).

The Elmalık tunnels were excavated in the main in fault gouge clay, at a center-to-center separation of 54m and supported by 45cm-75cm of shotcrete ($f_{cu}$=20/30 MPa). Figure 5 shows the severe damage in the completely closed top heading face that occurred during the excessive deformations. As noted, such large deformations necessitated re-profiling works in this fault gauge area at the Elmalık tunnels.

It is acknowledged that the accuracy of this system is limited to several millimeters. It can be seen that a settlement of about 1000mm and a convergence of about 800mm is indicated by the excessive deformations.

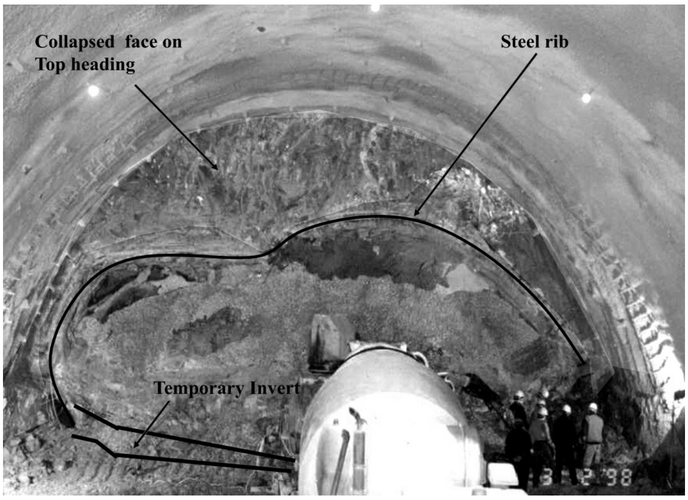


Figure 5. Photograph for tunnel profile at the main fault area after 41 day from first settlement (Elmalık Right Tube).

## 4 TUNNEL DESIGN AND PHILOSOPHY

### 4.1 *Tunnel Original Design and Construction Details*

As previously alluded to the original design was based on the New Austrian Tunneling Method (NATM), which philosophy was adopted with the primary support just carrying the immediate ground loads and providing a stable environment within which the final lining is cast to complete the tunnel.

This NATM method, otherwise known as the Sprayed Concrete Lining (SCL) method was being utilized initially, with the primary support consisting of lining of sprayed shotcrete (sometimes with rock bolts), and light steel ribs as the primary support and a cast in-situ concrete inner lining. This design philosophy considers that the primary lining supports the immediate ground loads providing a stable environment in which the secondary lining (or inner lining) is cast to give the necessary long-term safety margin. The Secondary support consists of a cast in-situ concrete lining. The original support systems proposed by the Designer comprised Options 1 and 2, as shown in Figure 6 and 7 (Geoconsult (July 15, 1998), Design Methodology Un-excavated Sections Gümüşova-Gerede Motorway, project report, No.45110/R/2149). Using these support systems deformations of the primary lining increased from about 250mm near the portals to over 500mm further into the mountain. A collapse ultimately occurred in the Elmalik Right Tube (as shown in Figure 5.). This section of tunnel had been back-filled with concrete (check) and re-excavated with a very robust support system. It was thought that the (poor) ground conditions associated with the collapsed region was of a limited extent, and another face was opened in the Elmalik Right Tube by passing the failure zone in order to maintain production. However, large deformations of the primary lining continued to be observed in both tubes on the Elmalik side where deformations increased to over 1000mm as the overburden increased to 150m as shown in Figures 3 (which are typical). These deformations were characterized by large excavation induced movements (initially occurring very rapidly but later stabilizing) followed by slower but continuous creep movements. Furthermore, in poor ground, additional movements were observed due to the interaction between the two tunnels.

In other words, the construction of the second tunnel (ELR) induced significant deformations in the first tunnel (ELL) as indicated in figure 3. For these reasons, in a large portion of the tunnels constructed with the old design the shotcrete lining infringed significantly into the required profile for the inner lining. As such, these zones then had to be re-profiled, a process involving excavation of the existing shotcrete shell and surrounding ground and the installation of a new shotcrete lining outside the required profile, with account being taken of the expected deformations of the new shotcrete shell. This revised re-profiling was required for about 70% of the portion of the first tunnel and 20% of the portion of the second tunnel that were built using the old design.

In some tunnel sections, re-profiling works had to be carried out twice since the new shotcrete shell continued to creep at such a rate that prevented the inner lining installation, as it again infringed into the required profile. (The project criteria for inner lining installation is that the shotcrete lining must have stabilized to a convergence rate of 2mm/ month or less.)

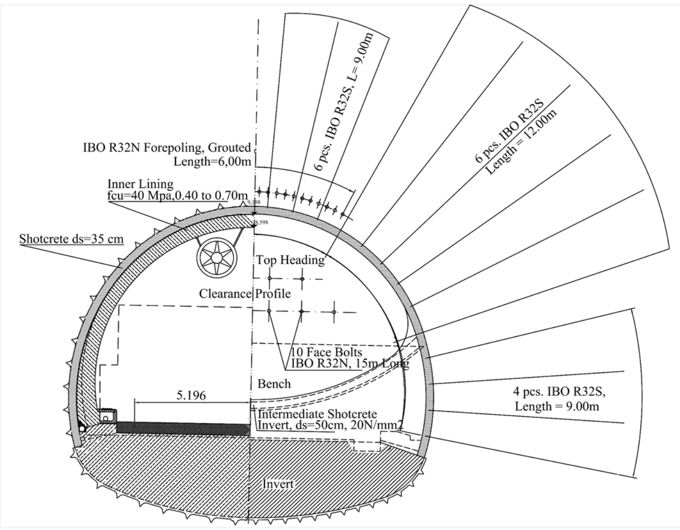


Figure 6. Typical CM support class typical cross section (original design). Excavation area 204m². Option-1(without temprorary invert) and Option-2 (with temporary invert) for high and low plastic Flyschoid series.

## 4.2 *Design Change Caused by Excessive Deformations*

On that basis new designs were developed for clayey ground to augment the old NATM designs, which were satisfactory for good to moderate ground. In 1998/99 a re-design was instigated for the worst ground under a new philosophy. With these it was recognized that the new designs must provide a stiff support close to the tunnel face to limit deformations to acceptable levels. The new designs achieved this balance by retaining the staggered excavation sequence of top heading bench and invert, to allow staged deformation of the ground and lining before full ring closure. Two new designs were developed termed Option 3 & Option 4(Refer figs. 8 & 9). The Option 3 design (Fig.8) was developed for blocky flyschoid ground (Menkiti et al. 2001). It could also be used to cross thin layers of fault gouge clay.

## 4.3 *Tackling the Challenges of Deformations and revised Design Options adopted*

One of the biggest challenges during tunnel excavations in "Weak Flyschoid Series" (with high clay ratio and a low geo-mechanical parameters) was the high risk of damage caused by the induced deformations to the open tunnel geometry. Therefore, and as referred to previously, in order to prevent unforeseen and rapidly progressing deformations in the tunnel (and any resulting faults), important changes were made and applied to the NATM supporting system. As mentioned above the original tunnel design was based on the principles of NATM tunnels. However, when major problems arise during the tunnel excavations a different tunnel design system and methodology was developed based on the geo-mechanical characteristics encountered during actual ground excavations necessitating reprofiling work.

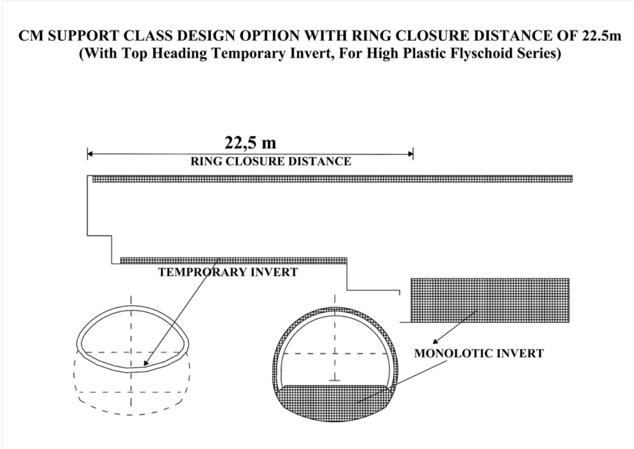


Figure 7. Typical CM support clas longitudinal section (original design). Option-1(without temprorary invert) and Option-2 (with temporary invert) for high and low plastic Flyschoid series.

For example, when tunneling in heavily faulted rocks with NATM, roof settlement in excess of 1 m occurred. Additionally, when radial deformations at the face exceeded 1200mm at the Elmalık Side, further advancement of the two Elmalık drives were stopped, and consequential extensive review of the excavation and support methods were performed.

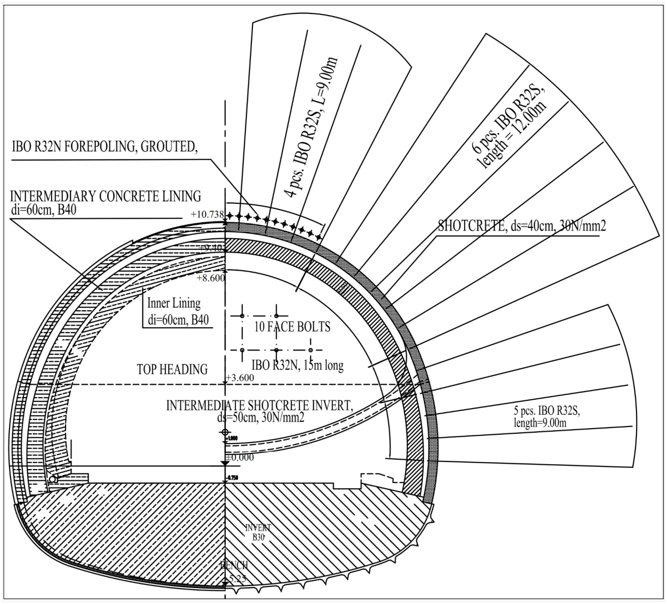


Figure 8. Option-3 cross section, support class design option with intermediate lining and in poor Flyschoid Series in clayey fault gauges. Excavation area 245m2.

Following a detailed investigation program including driving of pilot tunnels, advancement of tunnels recommenced with modifications to the original design, namely Option-3 (figure 8 & 9) and Option-4 (figure 10 & 11).

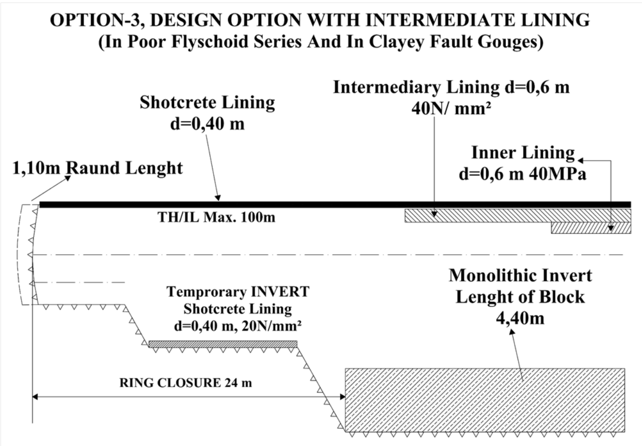


Figure 9. OP-3 support clas longitudinal section and, with temporary invert for high plastic Flyschoid series. Ring closure distance of 24m.

The Option 3 design consisted of 40cm primary shotcrete lining installed at the face with a 60cm cast in-situ concrete intermediary lining (B40) installed 28-33m behind the face. The final lining was then a 60cm B40 concrete lining installation. The deep monolithic invert was retained, but fully reinforced to British Standards. Where necessary, the top heading footings with a temporary invert installed until the bench. Support was provided by shotcrete shell reinforced with TH-29 (29kg/m) steel arches every round (generally at 1.1m spacing).

A minimum shotcrete strength of 30N/mm2 at 28 days. Alkali-free shotcrete is now being utilized with additives from Master Builders Technologies (MBT) of Switzerland.

The Option 4 design (Fig.10&11) was developed for thicker zones (≥20m) of pure fault gouge clay, representing the worst ground conditions. For such ground the top heading stability could not be achieved even with a temporary invert. To overcome this problem two pilot tunnels were first driven in the bench area and back-filled with reinforced concrete. The concrete beams also provided some lateral restraint against excessive horizontal convergence at the foot of the top heading. The primary support consisted of 40cm of MBT shotcrete with steel fiber (40kg/m³, 0.60x30mm) installed at the face. The intermediary lining installed was 80cm thick with steel fiber 30kg/m³ (wire and 0.60x30mm) (Menkiti and Isik 2001) at between 8 and 16m behind the face. The bench and deep monolithic invert were installed at between 22m and 35m behind the face to achieve ring closure.

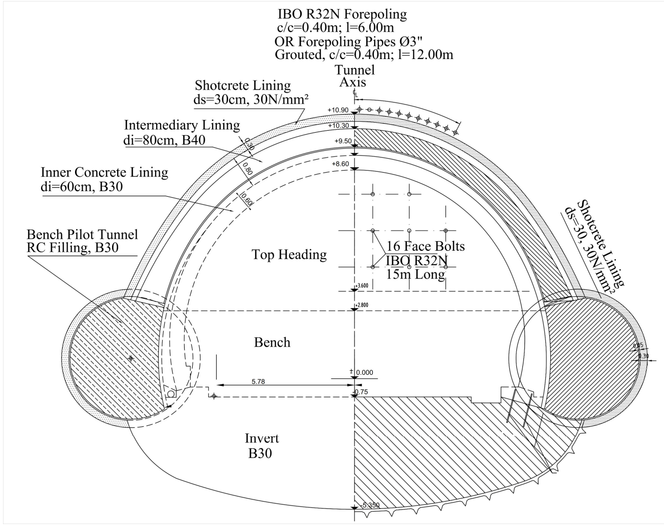


Figure 10. OP-4 support clas design option with Bench Pilot Tunnel (BPT) The BPT represent the main items/feature of Option 4 rock class. Excavation area 275m2.

The final lining installed was 60cm thick of B40 concrete lining with reinforcement ((Geoconsult, (July 15, 1998)). No rock bolts were use in fault gouge clay (figure 10). Figure 11 shows a photograph of the early stages of the Option 4 construction. Excavation was mostly by conventional excavators, although some stretches of better ground required drilling and blasting, or the use of a pneumatic hammer to loosen the material. Excavated spoil was disposed using mechanical loaders and dumpers.

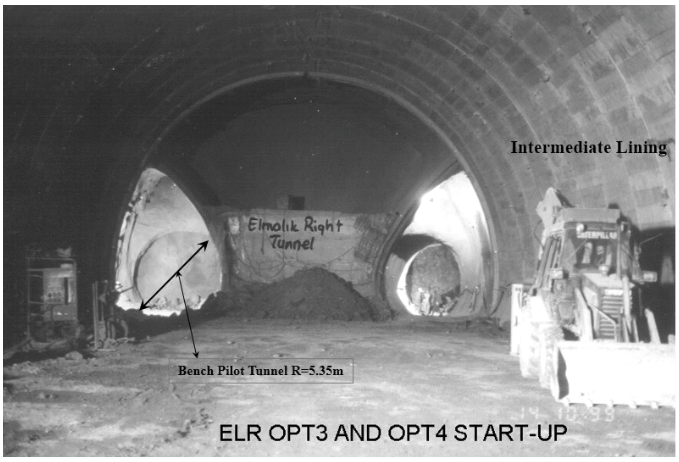


Figure 11. OP-4 support clas the regular BPT (bench Pilot Tunnels) are two tunnels having cross section of a circular shape, with a theoretical internal radius of 2,55 m. BPT are parallel driven, with about 18,50 m distance between two BPT center lines.

Excavation was in round lengths varying from 0.80m to 2.20m, with shorter round lengths being used for poorer ground conditions. Face shotcrete 5cm to 20cm, with face dowels of min 5 no of 12m length, and forepoling 6m long at 0.40m circumferential spacing were required to stabilize the advancing face in the worst ground. In poorer ground, temporary ring closure was achieved by using a TH with a temporary invert. Typically, the temporary invert was applied within 4 rounds of the face.

## 5 CONCLUSION

The ground conditions encountered in the Bolu tunnels were exceptionally challenging. When, during the Asarsuyu and Elmalık excavations a large fault line was encountered in both tunnels, a significent increase in radial deformations were observed.

In this fault zone, deformations of up to a maximum 1 m (exceeding 1 m on the Elmalık side) were encountered, and instability at the portals were observed.

It was observed that as a result of the disintegration and faulting in a wide zone, low-angle faults have an impact on the stability of tunnel excavation support systems. For this reason, as the tunnel excavation support system was markedly unstable, a new project design was needed.

With the help of a 3D monitoring system it was established that in low-angle faults the stresses formed by the interaction zone cause substantial deformations. The conclusion from all this is that strict monitoring of daily geological records and assessment of measurement readings with the support of a 3D monitoring system and geotechnical devices contribute greatly to the design of the tunnel support system.

While the original Tunnel Design had been developed according to the NATM tunneling principles, where ground conditions changed, and in particular for the poor flyschoid series and clayey fault zones of less than 20m, where unavoidable excessive deformations occurred OP-3 support system with an intermediate lining (60cm) needed to be applied. For the much more unfavorable soil conditions, in clayey fault zones of more than 20m OP-4 support system with an intermediate lining (80cm), bench pilot tunnel became applicable.

According to the above investigation results, the width of the plastic zone around the tunnel (plasticization radius) can exceed 50m, and its development has a direct influence on the radial displacement of the tunnel, axial displacement of the rock and radial pressure (stress) over the tunnel first support lining. An increase of the tunnel's peripheral pressure causes undesirable excessive deformations on the tunnels. When this plasticization radius around the perimeter of the tunnel has decreased after the precautions taken by the support systems indicated on Option-3, and Option-4.

Option-4 Project is prepared following pilot tunnel investigations for thick fault clay layers which is seen to be representing the most inconvenient soil conditions. In this case, top heading stability could not been provided even with the temporary invert. In order to overcome this problem, excavation of two pilot tunnels at the bench section was performed as a first step, then it was backfilled with reinforced concrete. This concrete also provides a longitudinal support against excessive lateral movement (convergence) at the top heading footings.

In order to ensure that the updated ground model is adapted to the geotechnical challenges, an NATM design concept was developed that will benefit from a high strength support which can reduce radial deformations, that equally has sufficient strength to withstand the circular deformations which have a breaking load higher than shotcrete.

In conclusion then, Bolu tunnels works are very important events with regard to the validity and practicality of the models developed for the deformation performance of tunnels.

Comparison of the damage observed and the geological conditions and deformation monitoring, including the design followed has indicated that these could be the basis for assessing the performance of tunnels and identifying potentially hazardous zones.

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