Design aspects of segmental tunnel lining in difficult rock conditions

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Abstract. Despite plenty of research studies in design of segmental tunnel lining, there have still been serious concerns in damage mechanism of segmental lining in weak rock masses. With significant increasing of mechanized tunnelling practically in all kind of ground, including highly weathered rocks, severely jointed rocks, shear, and fault zones, squeezing, and swelling prone conditions, the need to investigate the mechanical behaviour and the damage mechanism of segmental tunnel lining is unavoidable. Any misjudging of those adverse geo-conditions may often cause an irreversible economical loss in the tunnelling project. This paper deals with the significant design aspects of segmental lining for such difficult rock mass conditions which can cause lining deformation and failure in segment in terms of either compression or tension as well as in the segment joints. Further, some critical design aspects extracted from the tunnel T26 of Ankara – Eskisehir high speed railway are presented and analysed.

1. Introduction
With increasing mechanized tunnelling in wide range of ground qualities from soft soils to poor rocks, medium to highly jointed rock masses, the geological and geotechnical uncertainties are being amplified due to the lack of sufficient ground investigation which in most cases is unavoidable for many economic reasons and technical limits. Unfortunately, it might be too late when one of the involving parties in project such as owner, contractor, and designer recognizes a remarkable unforeseeable condition and inconsistency because both machine and lining have already been suited for the project.

Therefore, identifying and quantifying the main sources of risk in terms of geo-hazards is crucial to minimize the possible impact on machine drive and segmental tunnel lining.

In this paper, an effort is made to study the modes of segment damage due to over stressing as a consequence of prevailing adverse geo-hazards, focusing on some significant design aspects. Broadly speaking, the tunnel segments shall support appropriately all type of acting loads while crossing the difficult ground, achieving Ultimate Limit State (ULS) verification and guarantying both short and long-term stability of the tunnel. Any types of segment overloading bring about the failure and the damage in concrete segment. Some typical damages in segmental tunnel lining, resulting from geo-risks in difficult rock conditions are represented in Figure 1.

2. Adverse geo-conditions and segmental lining design

2.1. Uncertain piezometric level
In most of the TBM drive tunnels, the actual position of the groundwater level is known only in the tunnel portal areas and where the accessibility to the tunnel route is available. Seeing that the specific piezometric above tunnel in rock medium depends mainly on the degree of the jointing and permeability
of ground that are not well recognised in preliminary design stage, so the designer should be able to pre-dimension the segmental tunnel lining with regards to the maximum lining capacity against the water pressure. Even such an investigation is necessary to assess the probable water pressure if TBM crosses a fault zone.

Figure 1. Typical damages in segmental tunnel lining resulting from the manifesting of the main geological risks in difficult ground excavation due to different types of geo-hazards (Almost most of these phenomena are also presented in the case history reported in this paper).
While the weakness point of the segmental rings is attributed to the longitudinal joints in terms of compression action owing to the hoop load, so the first important step is to work out the maximum water pressure that lining withstands for joint action. Figure 2 demonstrates exemplary the results of a probabilistic analysis carried out for the following conditions: Geological Strength Index (GSI) = 20-50 [1], water table level above tunnel = 10-150m, concrete class of segment C40/50, segment thickness = 45cm, effective segment joint contact area = 70%. As can be observed the water table of 80m is threshold value in which the Factor of Safety (FoS) falls below one. The details of formulation derived for these calculations have been given elsewhere [2],[3].

2.2. Fault and shear zone

The fundamental failure mechanisms in a fault/shear zone are caving ground, ravelling ground, and flowing ground. The term “caving” identifies generic gravitational collapse of portions of highly fractured rock mass from the tunnel cavity and/or tunnel face; therefore, given their very poor self-supporting capacity or inadequate stand-up time, the highest risk of caving is associated with the most unfavourable rock mass classes. Ravelling ground is characterized as the flow of cohesionless dry or moist, intensely fractured rocks and soils even residual debris. In the same way, when such materials are accompanied with huge water content, it is called flowing ground.

![Longitudinal Joint Verification Relationship Hw-FoS-Frdu](image)

**Figure 2.** Determination of maximum water head (Hw) acting on segmental tunnel lining satisfying successful longitudinal joint verification for the concrete failure in compression at ULS [4].

The fault zone would also be cause of asymmetrical load state. Silo effect and chimney formation leading to surface sinkhole are the typical risk of fault and shear zone. Inadequate stand-up time allows material to cave (or to be eroded by water) with a faster rate than TBM advance, and void result. The void may take the form of “chimney” or “inverted trench” or may extend the diameter of the tunnel on one or both sides. Furthermore, one of the worst scenarios crossing fault zone is sudden water inflow.

In term of design of segmental lining, main strategy to overcome fault and shear zones might rely on using a special segment cage reinforced with intensely conventional steel rebars to increase the bending capacity in order to adapt the high load eccentricities and also to extend the M-N strength envelope in ULS with a reasonable safety margin. Nevertheless, in some actual cases in such fault and shear zones, the tunnel segments are equipped with drain holes to reduce the pore water pressure in highly permeable medium. It should be noted that the application of sole Steel Fiber Reinforced Concrete (SFRC) segments should be avoided in faults and shear zone due to their low bending capacity. However, SFRC segments can be used provided that additional conventional steel rebars are integrated.
2.3. Water inflow

One of the worst scenarios that during TBM drive can occur is the sudden inflow of water accompanied by debris into the tunnel. This phenomenon may result from in presence of (1) very poor ground condition (i.e. crossing faults or highly fractured rock masses), (2) extreme change in the permeability degree of different rock layers, (3) high hydraulic charge. Pore water pressure due to sudden relief of water inflow on TBM cutterhead, shield and segmental lining behind the tail-shield in the form of asymmetrical loading may cause of segment damage in ULS as well as a significantly negative impact on tunnel watertightness. Further, the presence of karst and the trapped water in a complex strata can also release a huge water flow and consequent pore water pressure on the segmental lining. Therefore, such a risk and huge water pressure should well be taken into account in tunnel segment design.

2.4. Asymmetrical loads

This hazard could affect specific sections of the segmental lined tunnel associated with low overburden, asymmetric ground pressure (i.e. when tunnel develops parallel to a slope), deep landslides and tectonically active zones, or release of suddenly water inflow.

Anisotropy and heterogeneity in filed stress, rotational unstable rock wedges, loss of rock arch effect due to poor interlocking and poor integrity of the rock fabric are the main sources of the asymmetrical loads on the segmental lining. Figure 3 shows a trial back-analyzed structural model to simulate the asymmetrical load shape for a collapsed segmental lined tunnel carried out for Tunnel T26 (see the next section). The structural model was created based on Spring Bedded Model (SBM) as introduced by Duddeck and Erdmann [5].

![Figure 3. Asymmetrical load model for segmental lining and unachievable ULS verifications even in presence of heavy reinforcement amount.](image)

Asymmetric loads can also happen in fair rock mass condition in presences of potential detached rock blocks due to poor interlocking in relatively low lateral stress condition where the response upon the excavation is dominated by the shear strength of discontinuities and a translational failure may occur. These phenomena occur in highly jointed rock mass, not necessary connected to fault zone or tectonic features, where the principal discontinuities (stratification planes, joints, fractures, schistosity planes) are characterized by low shear strength.

2.5. Squeezing ground

In TBM drive tunnelling, the severe and very severe squeezing are characterized by high shield and segmental lining loads. This phenomenon mostly occurs in weak rock masses which are prone to such a behaviour. In this condition, occasionally TBM installed thrust force would be by far insufficient for overcoming friction and the lining would be overstressed.
An extreme ground squeezing condition in Carboniferous Schistoses layers with an overburden of 300m and convergence of around 2.0m is presented in Figure 4 [6], which is quite like the geomechanical behavior of the graphitic schist of the Tunnel T26. This condition had a very bad impact on both TBM shield and segmental lining. The deduced large time-dependent deformation in associated with the progressive de-structuration of the plasticized rock mass with decay of residual properties “softening” has a significant influence on the segmental lining with regards to the progressively load increments to the extent of exceeding the limit state. Furthermore, such a mechanism provokes the rock mass permeability to attract the ground water from surrounding resulting a dramatic increase in water pressure on the lining.

Figure 5 represents the results of a Convergence-Confinement Method (CCM) implemented through probabilistic method [7],[8],[9],[10],[11], for a segmental lined tunnel of 13.4m internal diameter, like that of Tunnel T26, that shows the risk of the TBM jamming. The analyzed rock mass was given a suitable statistical distribution; namely normal, triangular, uniform, exponential, lognormal (H: Overburden =100-350m, GSI=15-35, mi=5-15, UCS: Uniaxial Compressive Strength=15-50MPa, MR: Modulus Ratio=200-600) while the TBM was given a shield conicity and overcut of 150mm on radius, friction coefficient μ=0.15-0.45 for continuous and re-start condition in rock, nominal, and exceptional thrust of 85.000kN and 120.000kN, respectively [12]. This example proves the serious risk of TBM jamming for such given geotechnical condition to be considered during feasibility study of the tunnel excavability by a TBM. The analysis of such results helps also design engineer to foresee the possibly suitable mitigation or countermeasures.

2.6. Swelling prone ground
The swelling risk should be well studied during design. The volume increases due to physical bonding of water with fine particles in the rock mass, for instance, the fixing of water molecules on clayey minerals in the rock mass. This phenomenon is like the well-known swelling of expansive clays. Such an increase in volumetric strains causes a progressively time-dependent pressure on the segmental lining
in such a way that the segmental ring joints are displaced and even it can heave up the lining. It is interesting, at this point, to note that the swelling effect can be accompanied with slaking phenomenon. Some rocks may slake (hydrate or swell, or oxidize), disintegrate or otherwise weather in response to the change in humidity and temperature consequent on excavation. For this type of hazard ground gradually breaks up into pieces, flakes, or fragments in the form of disintegration (slaking) of some moderately coherent and friable materials. The slaking is a very dangerous risk if it is accompanied by a complex mechanism where the rock intends to swell. The combination of slake and swell is one of the most dangerous hazards.

3. Studied case history: the tunnel T26 of Ankara–Eskisehir high speed railway.

The problematic tunnel T26 of Ankara-Eskisehir high speed railway has attracted interest of many researchers to investigate both machine and lining behaviour. While the geological conditions and construction problems have been presented by [13] and [14], Aydan et al. [15] developed a method based on CCM to estimate the load on the jammed TBM shield. Furthermore, Hasanpour & Rostami [16] investigated the behaviour of single EPB shield machine by means of 3D numerical modelling. In all of research studies, the adverse geological conditions were cited and taken into considerations in calculations. The tunnel T26 suffered from many instabilities problems related to adverse geo-hazards, resulting in different kind of segment failure modes.

3.1. General tunnel characteristics

The 5.4 km long tunnel T26 of Ankara- Eskisehir high speed railway route in Turkey is considered because representative of how extreme geo conditions can cause significative segmental lining deformation and damages. The T26 tunnel segmental lining was originally designed with a universal segmental ring (7+1), Reinforced Concrete (RC), internal diameter of 12,50m, thickness of 0,45m, ring width of 2,0m, concrete class of C40/50 ($f_{ck}=40$MPa) steel ratio of 65kg/m$^3$. The tunnel was being excavated by means of a single shield Herrenknecht-EPB TBM with boring diameter of 13,77m equipped with a drive system able to provide nominal and maximum thrust force of 84.500kN, 96.500kN respectively, and a torque of 16.000kNm and 25.700kNm at RPM of 1,0 - 4,0 respectively.

The geology of the project area consists of graphitic schists and chlorite schists. A major fault zone was identified in the middle of the tunnel while other minor fault zones might have been encountered over the entire tunnel alignment. The maximum overburden was around 220m and the piezometric level falls within 1m-9m below the ground level. The graphitic schists were known as potentially problematic rocks because their squeezing prone ground condition in terms of machine and lining design, such a risk might have resulted in blocking of the TBM shield and overloading of the segmental lining.

3.2. Difficulties in TBM drive and tunnel segmental lining damages

Ever since the commencement of tunnel excavation in 2012, many instability problems have occurred once the TBM started to drive in a “deep seated rotational” complex landslide. As a first evidence, the TBM induced over excavations traced by mucking monitoring systems. Such instabilities occurred with different sizes up to collapse to the surface creating sinkholes. In addition, the segmental lining suffered from many failures and damages in the form of major cracks, joint displacements and burst, extrusion of steel rebars and concrete crashes, particularly at tunnel crown and invert. The tunnel itself seemed to have a direct influence with the slope stability with measurable movements during some phases of tunnelling, particularly in case of significant over-excavation. In terms of tunnelling difficulties, three scenarios of instability degrees have been recognized in tunnel T26 (see Table 1).

3.3. Load estimation on the segmental lining

The first step in back calculating the acting load on the segmental lining behind the TBM tail-shield is to verify the in-situ stress compatibility for different weathering conditions of the graphitic schists by means of valid geotechnical models. Applying generalized Hoek & Brown failure criteria [17], it was possible to depict the strength envelopes ($\sigma_1 - \sigma_3$) for three Instability Degrees Scenarios as well
demonstrated in Figure 6. Presuming different strength parameters of rock masses associated with those of site investigation and relating to the in-situ stress state, it was verified that the strength envelope C could be used to reconstruct the limiting condition, i.e. Instability Degree Scenario II whereas the strength envelopes A and B represented the Instability Degree Scenarios I. Furthermore, the strength envelope D represented the Instability Degree III (stable condition).

Table 1. Failure mechanisms in tunnel T 26 associated with the encountered difficult ground conditions

| Instability degrees scenario | Level of acceptance | Level of threshold in terms of FoS | Evidence of hazard occurrence (TBM and lining) | Consequences |
|-----------------------------|---------------------|-----------------------------------|-----------------------------------------------|--------------|
| I (Unfavourable)            | Prohibitive         | <1.0                              | TBM jamming and/or severe failure/collapse of | Large instability of the face involves notable over-excauttaion, chimney formation, and successive collapse, with extreme deterioration of the ground shear strength parameters. |
|                            |                     |                                   | the segmental lining                           | Face instability is relatively less pronounced than instability degree I, resulting in a more contained decay of rock mass properties. |
| II (Intermediate)           | Critical            | ≈1.0 (instability threshold)      | Localized failure / damage on the segmental lining | Face instability is relatively less pronounced than instability degree I, resulting in a more contained decay of rock mass properties. |
| III (favourable)            | Relatively regular  | >1.0                              | No anomalies evidence. To be checked the FoS | Minor instability of the face and/or local more favourable geotechnical condition determine a rather regular excavation behaviour. |

Note: FoS = Factor of Safety

![Figure 6. Rock mass strength envelopes A, B, C, D [17] and compatibility check with in-situ stress for different weathering degree of the graphitic schists.](image-url)
3.3.1. Back analysis of the limiting state and reconstruction of CCM

With reference to Table 1, a back-analysis has been carried out for the Instability Degree Scenario II related to the evaluation of the equilibrium conditions by means of CCM and using strength envelope C to estimate the load acting on the lining at limiting state. Note that the extreme instability conditions referred to the Instability Degree Scenario I appear likely related to the uncontrolled excavation process by strongly reducing the geotechnical properties to produce a significantly overstressed lining.

To perform a significant back analysis, it was needed to reproduce the actual geotechnical conditions on which scenario II might be based, considering the in-situ stress state and Long Term (LT) condition to stimulate the time-dependent behavior of rock mass. Short Term (ST) mainly governs the immediate response upon excavation and may be considered as reference for the deformations occurring in the correspondence of the shield, if the TBM advancement speed is not too low, while LT characterizes the effect of decay induced by the real excavation process, as well as approximately simulates the ground time-dependent (viscous) behavior. Therefore, the LT condition shall in turn be considered for a proper tunnel lining design.

Assuming a proper geomechanical model and applying CCM, it was possible to reconstruct the ST and LT behaviors of the ground and lining (Figure 7). As observed, the ST design load (in terms of $P_i/P_0$ with $P_i=$acting load and $P_0=$geostatic load) on lining at equilibrium was obtained around $P_i/P_0=4\%$ whereas the LT load, which to be used in design, was calculated $P_i/P_0=22\%$, i.e. seven times higher.

![Ground Reaction Curve](image)

**Figure 7.** Estimating load on segmental lining in a weak rock mass prone to a complex loading behaviour through CCM method [10], adopted from studied case.

Applying LT load case in a structural model, it was possible to observe that the combination of N-M led to a limit for the integrity of the RC segment as showed in Figure 8 and Figure 9.

3.3.2. Back analysis of the damaged segments

Compared with scenario II (limiting equilibrium condition) the rock mass strength envelopes “A” and “B” given in Figure 6 were assumed to develop the CCM for the damaged segmental rings. Therefore, upon the TBM jamming condition and progressively load increment on the segmental lining behind the tail-shield with the course of time, the analysis of the stress and strain in the instrumented segments (Figure 10) help to calculate the actual ground load on the damaged lining.
Figure 8. Obtaining limit state condition for the back-analysed load case (applied load factor, $\gamma_f=1.35$, material reduction coefficient, concrete, and steel: $\gamma_c=1.5$, $\gamma_s=1.15$). N-M domain constructed for main reinforcement 10$\phi$25/m

Figure 9. The inception of first crack and subsequent crack propagation at crown of the RC segmental lining.

3.3.3. Segmental ring failure due ring ovalization

The ring ovalization may cause the cracks in the middle section of the segments at tunnel crown because of lack of prompt lateral confinement (bedding) in correspondence of the ring abutments when it exits the TBM tail shield. Ring ovalization creates the unbalanced bending moment due to the lack of axial load induced by proper arch effect. By means of theory of elasticity, Morgan [18] introduced the following formula commonly used to calculate the additional distortional bending moment. This formula is useful to quickly back-calculate the limit state condition of acting load on the lining.

$$M_{dist} = \frac{3EI_e \delta}{r_0^2}$$  \hspace{1cm} (1)

where $E$ is the elasticity modulus of the segmental lining, $I_e$ is the equivalent area-wise moment of the section defined by Muir-Wood formulation [19], $r_0$ is the lining radius, $\delta$ is the distortion.

A back-analysis was carried out for a damaged ring in tunnel T26 where the maximum stress on lining crown was 1.6MPa corresponding to a maximum deformation of 47mm. Applying Morgan formula, the flexural moment ($M_{dist}$) was calculated 1.200kN, combined with a hoop force (N) of 20.200kN for a segment of 2.0m width. Such a combination leaded to failure in segment as reproduced in Figure 11. The load factor $\gamma_f=1.35$, material reduction coefficient for concrete and steel: $\gamma_c=1.5$, $\gamma_s=1.15$, respectively were considered in compliance with Eurocode 2 [4] and the N-M domain was constructed for the main reinforcement 2$\times$20$\phi$12.
3.4. Performance capacity of longitudinal joints

The longitudinal or radial joint of a segmental ring is the weakness point of the lining because of its ability to rotate and to displace to accommodate the different load types, particularly ground load.

Just after building, the segmental ring, subjected to the ground and water load, is ovalized (a deformed form of a circular ring being shorter diameter at vertical and longer diameter at horizontal axes or vice versa, considering the feature of field stress and stress ratio “$k$ =horizontal stress/ vertical stress”). The occurrence of the ovalization causes the joint rotation, which let the joints be opened. In this case the effective area of joint contact is reduced. In addition, the deduced eccentricity due to worse combination of $N$ and $M$ (axial force and flexural moment) is another reason of reduction of effective joint area. Both cases cause consequently diminishing the actual joint performance capacity [2] and [3].

In other words, such an eccentricity reduces the contact surface area of the joint, leading to a considerable increment in acting compressive and tensile splitting stresses in concrete.

Considering the back-calculated $N$-$M$ combinations for the damaged segments carried out in tunnel T26, with a combination of $N=10100\text{kN/m}$ and $M_{\text{segment}}=600\text{kNm/m}$, and defining a variation for bending moment in joint between $M_{\text{joint}}=282\text{kNm/m} - 600\text{kNm/m}$ complying with JSCE [20], a probabilistic back-calculation has been performed for different distribution of flat joint shapes, varying dimensions of gasket groove, guiding bar, chamfers, and eccentricity, to evaluate the correlation between concrete bearing capacity and eccentricity. This helps investigating the compressive failure mode of the segment joint.

As can be inferred from Figure 12, the bearing capacity of the concrete “$F_{\text{rd}}$” begins from limit state (i.e. $F_{\text{rd}}=N_{\text{ld}}$) and its value falls always below the value of $N_{\text{ld}}$ when the ratio of critical eccentricity to thickness reaches to 0.066 ($e_{\text{critical}}/h=0.066$), resulting in segment joint failure (FoS < 1.0). This exactly demonstrates the segment damage in the form of joint burst as observed in Tunnel T26 (Figure 1.c)

It is interesting to note that despite the low amount of the eccentricity at joint (lower than middle-third), due to high value of hoop force at joint, the compressive failure took place.
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Figure 12. Probabilistic back-analysis showing segment longitudinal joint burst (compression failure of the concrete) observed in damaged segmental rings in tunnel T26.

4. Conclusions
This paper aimed at identifying the adverse geo-conditions facing up with mechanized tunnelling in difficult ground conditions where a particular attention should be given to an appropriate design of segmental tunnel lining. Tunnel segments shall support appropriately all type of acting loads while crossing the difficult ground, achieving ULS verification and guarantying both short and long-term stability of the tunnel.

Hence, some significant design aspects of segmental tunnel lining for different adverse geo-conditions have been dealt with in this paper. Several practical examples of such geo-hazards such as: uncertain piezometric level, fault and shear zones, squeezing ground, swelling-slaking prone ground, highly fractured and jointed rock mass, water inrush, asymmetrical load state in terms of segment design were presented. Different types of segment damage, due to overstressing were reported and analyzed in terms of strength N-M interaction diagram as well as ring joint capacity performance.

An effort was made to depict three scenarios in terms of in-situ stress compatibility and rock mass strength parameters for Tunnel T26 in such a way as to produce the realistic geotechnical model for each scenario. Moreover, the correlation between the load estimation on the segmental lining by means of CCM and that obtained by instrumented segmental ring were examined through back-analysis technique to reconstruct the failure mechanism of the lining for the tunnel T26, considering three main instability degree scenarios. The results of the back-analysis were in good agreement with on-site observations in terms of segment damage.

Under the light of the analyses and real case representations given in this paper, to tackle with the unfavorably asymmetrical loads on segmental lining resulted from adverse geological conditions and to increase the bending capacity of the segment the usage of well-calculated steel reinforcement is unavoidable.

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