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Some Aspects on the Design of Near Surface Tunnels - Theory and Practice

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Abstract:

At present, a variety of empirical, analytical and numerical methods are available for stability and serviceability analyses of underground structures in rock masses. In the design of a tunnel in near-surface rock, a distinction may be made between those problems that are related to the strength and deformability of the rock mass, and those that are related to the response of the rock mass to the shallow ground cover.

When designing shallow tunnels, the proximity of the ground surface and the tendency of the material ahead of and above the tunnel face to ‘cave to surface’ has to be taken into account. Near-surface rock masses tend to be more ‘mobile’ than similar rock masses in the confined conditions that exist at greater depth. These factors introduce issues that are not present in the analysis and design of deep tunnels [6].

The design problem listed above is at present largely solved based on experience and rule of thumb. However, some methods, including but not limited to the convergence confinement method, are applicable for deep tunnels in a hydrostatic stress field. Recent developments in rock mechanics and numerical methods in geotechnical engineering show promise of providing more realistic approaches to certain aspects of the design.

The focal point of this paper is to discuss some theoretical and practical design aspects of near-surface tunnels. The discussion is based on advanced numerical analyses verified using field data obtained from a constructed tunnel. This paper contributes to understanding rock mass behaviour and design of near-surface tunnels.

1 Introduction

Rock masses at shallow depths are generally known to show weak mechanical behaviour. Tunnel excavations in such conditions can cause movements along major joint sets and hence there is a tendency for the material ahead of and in front of the tunnel to ‘cave to surface’ (i.e. the rock mass is ‘more mobile’ than similar rock masses at greater depth due to the confined stress conditions). The following questions arise:

- Is there a systematic rock mass behaviour for near surface tunnels?
- What are the necessary geotechnical parameters and boundary conditions based on field data?
- What kind of calculations shall be performed for a safe design?

Figure 1: Tunnelling in near-surface rock conditions causing cracks and “cave-ins”

The focus of the present paper is on sedimentary rock with 'hard soil / soft rock' conditions, i.e.:

- effects of strength anisotropy more pronounced
- effects of deformational anisotropy of minor importance
- more dependency upon spatial distribution of discontinuities in relation to size of the tunnel opening

Figure 2: Typical sedimentary rock conditions

2 Modelling Approach

The typical rock mass behaviour (according to the Guideline for the Geotechnical Design of Underground Structures with Conventional Excavation [1]) for such a failure mechanism of near-surface tunnels is described as 'behaviour type BT 7', i.e. a crown failure (voluminous overbreaks in the crown with progressive shear failure).
Hence, it is a discontinuity dominated failure mechanism, which occurs especially in rock formations with horizontally layered and vertically jointed rock. The occurrence of progressive breakout has to be taken into account.
In such conditions rock failure is often of brittle type. The strength of the rock mass increases with depth (silo effect) and a strength reduction is observed when wet due to typical weathering effects. Due to the weakness planes in the sedimentary rock there is a marked anisotropy of the rock. A continuous stress arch mobilization takes place during tunnelling and the rupture progresses to the ground surface in case 'no stable condition' can be reached.

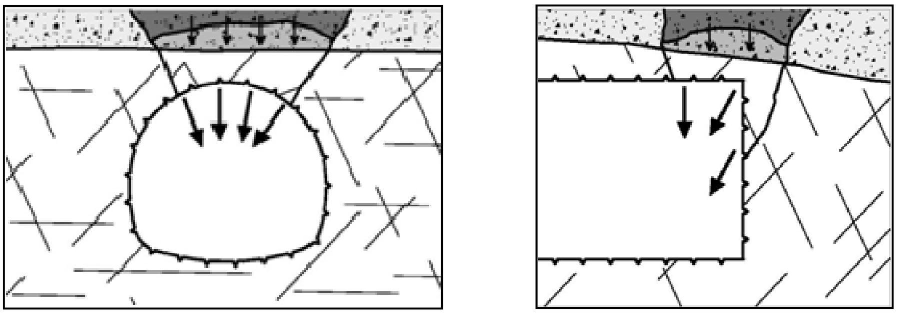

Figure 3: Ground behaviour type: ground failure

The strength and deformation characteristics of sedimentary rock (with discontinuity dominated rock behaviour) are difficult to determine; the reasons are:

- only intact rock and joint strength can be determined in lab tests
- hence only small scale specimens of the rock material are available, which is not adequate for a reliable estimate of rock mass parameters
- therefore, large scale tests would be required, but they are time-consuming, expensive and not always reliable
- rock mass characteristics to be determined using empirical classification systems/methods

In spite of advances in numerical methods and constitutive laws for weak rock, the correct prediction of the excavation behaviour in near-surface rock remains a challenge. The paper presents effects of overburden height and strength of sliding planes on the predicted settlement trough as well as the problem of evaluating correct rock mass parameters using classification systems

For discontinuity dominated failure mechanisms, the software UDEC, which permits to evaluate effects of breakout and stress arching mechanisms, can be chosen to simulate the behaviour of the jointed media. The calculation process is based on a force-displacement law in order to analyse the forces and moments between the blocks, followed by finding the force and moment acting on a individual block. From the law of motion, acceleration, velocities and displacements can be calculated [10]. An alternative is to apply a discretely fractured numerical continuum model, such as the joint network by Phase^2 [9].
Finally, the classical finite element analysis (FEM) can also be used, where different material characteristics are applied for different zones and rupture is governed by classical plasticity laws. Shear or fault zones can by modelled by applying interfaces and local continuum clusters.

3 Case Studies

Examples are used to illustrate factors influencing the ground behaviour of near-surface tunnels.

3.1 Influence of Overburden on Settlement Behaviour

The tunnel cross section of the first example is typical of a single-track high speed railway tunnel in Germany. The height is approx. 7.5 m and the width approx. 11 m. The excavation area is approx. 92 m². The overburden ranges from 5 to 16m. Hence the ratio D/H (equivalent tunnel diameter to overburden height) results in 0.5 to 1.5.
The typical geological formation is described to be “Obere und Mittlere Buntsandsteine“, which consists of interbedded sand- and siltstone layers. The characteristic E-modulus of the rock is in the range of 250 ÷ 1000 MPa, the E-modulus of the rock mass results in 100 ÷ 300 MPa. The discontinuity strength is determined with a friction angle of φ = 20 ÷ 25°, the cohesion c is assumed to be zero. The joint sets are illustrated in Fig. 4.

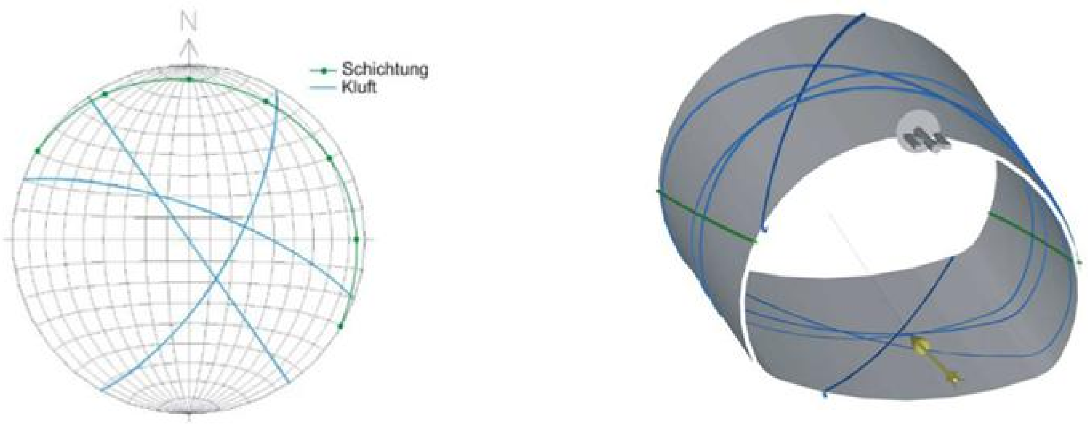

Figure 4: Tunnel geometry and joint sets – example 1

A parametric study has been undertaken (overburden 10 m versus 16 m) to show the influence of overburden on settlement behaviour.

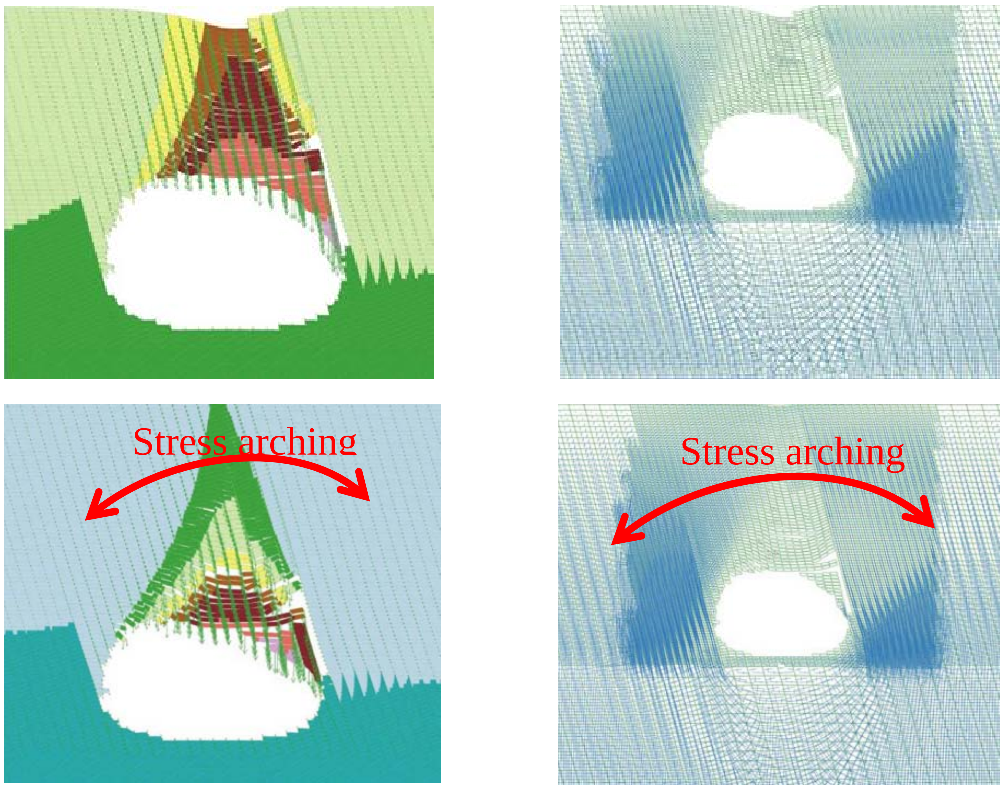

Figure 5: Influence of variation in overburden (above: 10 m overburden / below: 16m overburden)

Minor arching effects above the tunnel are already mobilized for an overburden of 10m. Due to joint movements, considerable surface settlements develop. Mean effective stresses will be more pronounced when considering an overburden of 16m, with minor settlements developing at the surface.

3.2 Influence of Strength on Sliding Planes

The second example uses a twin-track tunnel section with an excavation diameter of approx. 13 m; the tunnel is situated on ÖBB's Westbahn railway line in Austria. The ratio D/H (equivalent tunnel diameter to overburden height) results in 0.5 to 2. The tunnels are mainly located within tertiary valley fill (Oncophora layers). The major characteristic of this geological formation is an intensive alternation of sandstone with silt- and claystone.

The characteristic E-modulus of the rock is in the range of 250 ÷ 1000 MPa, the E-modulus of the rock mass results in 250 ÷ 300 Mpa. The discontinuity strength is determined with a friction angle of φ = 20 ÷ 25°, the cohesion c is assumed to range between 0 ÷ 60 kPa.

The influence of shear parameters on sliding planes, e.g. effect of existing joint strength on bedding planes is shown in Fig. 6, taking an overburden of 25 m and a joint orientation of 80° into account.

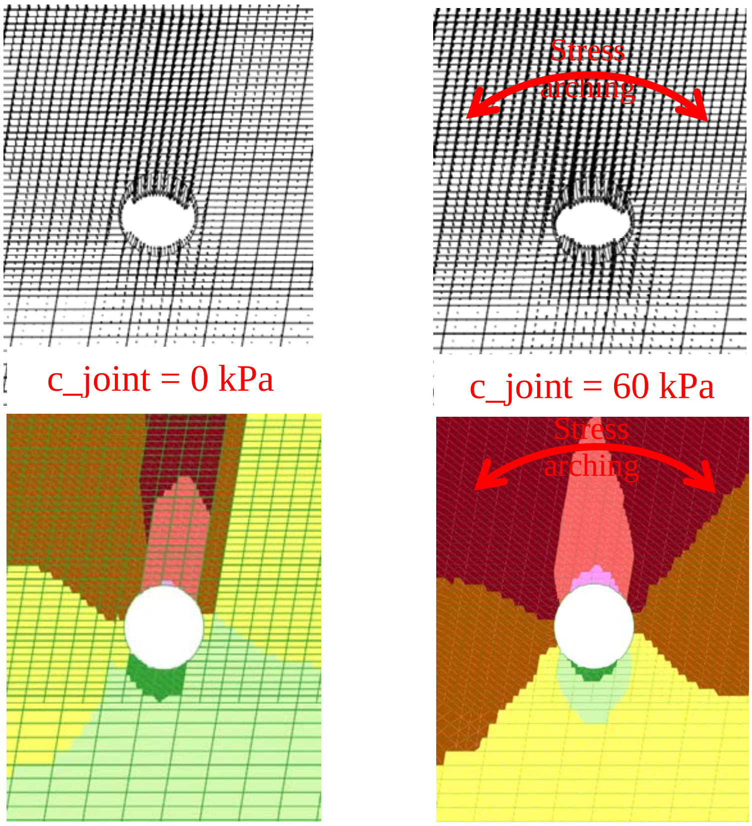

Figure 6: Influence of strength on sliding planes (left: c = 0 kPa / right: c 0 60 kPa)

Generally the pre-existing stresses in the rock mass will be relocated by the tunnel excavation and will be channelled around the tunnel. This redistribution of stresses primarily takes place as arching effect around the tunnel. Fig. 6 compares the displacement and stress state around the tunnel. While a joint cohesion of c=0 kPa only leads to little stress arching above the tunnel, this effect is much more pronounced for a cohesion c = 60 kPa at the joint discontinuities.

4 Rock Masses Classification Systems vs. Interactive Geomechanical Design

At present, a variety of numerical methods are available for stability and serviceability analyses of underground structures in rock masses. The rationality and reliability of the results from those methods depend, to a great extent, upon the appropriate selection of computational model (continuum, discontinuum, etc.), as well as mechanical and mathematical parameters. Once the computational model has been determined, the key to success hinges on the rational selection of the design parameters.

Providing reliable input data for numerical models, which are used in engineering design, is one of the most difficult tasks facing engineering geologists and geotechnical engineers. Unlike other engineering material, rock presents the designer with unique problems. First of all, rock is a complex material varying widely in properties, and in most engineering situations, not one but a number of rock mass types (RMT) and rock mass behaviour types (RMBT) will be present. The designers are confronted with rock as an assemblage of blocks of rock material separated by various types of discontinuities, such as joints, faults, bedding plane and so on. Furthermore, the behaviour of a rock mass is scale and stress-strain dependent.

Rock mass classification systems have been developed to assist in (primarily) the classification of rock into common or similar groups. The first truly organised system was proposed by Dr. Karl Terzaghi (1946) and has been followed by a number of schemes proposed by others. Specific to tunnelling applications, three such rock mass classification systems include the Q-System, Rock Mass Rating (RMR) and Geological Strength Index (GSI); e.g. [2-8].

Such rock mass classification systems provide engineers with a consistent approach to meaningfully compare the highly varied rock conditions likely to occur between respective sites.

Case studies from a shallow cover tunnel construction (tunnel opening approx. 15x8m, overburden approx. 5-10m), situated mainly in phyllite rock formation, have been found to have movements greater than the initial design predictions. As a result there have been breaches of the trigger levels. As construction proceeded and a greater knowledge of the behaviour of the ground in relation to tunnel construction was gained, it became apparent that the initial numerical analyses, where the rock mass parameters were estimated based on indirect methods, had underestimated the amount of deformation that was occurring.

Consequently, a suite of numerical sensitivity and back-analyses have been performed aiming at calibrating the rock mass strength and stiffness parameters against the field instrumentation data. As a result of this numerical study it was concluded that in the design of tunnels in near-surface rock, a distinction may be made between those problems that are related to the strength and deformability of the rock mass, and those that are related to the response of the rock mass to the shallow ground cover.

When designing very shallow tunnels, the proximity of the ground surface and the tendency of the material ahead of and above the tunnel face to 'cave to surface' have to be taken into account. Near-surface rock masses are subject to weathering and stress relief as a result of nearby excavations, e.g. road work, foundation work, etc. These processes disrupt or destroy the interlocking between rock particles that plays such an important role in determining the overall strength and deformation characteristics of rock masses. Near-surface rock masses tend to be more 'mobile' than similar rock masses in the confined conditions that exist at greater depth. This greater mobility must be recognized and allowed for in the selection of input parameters for any analysis.

Figure 7 (left) shows the contours and vectors of total displacement in the rock mass surrounding the tunnels. The displacements extend to the surface and the directions of the displacement vectors indicate the sensitivity of the above described 'caving' process. The contours of maximum shear strain and stress trajectories are shown in Figure 7 (right). The stress trajectories show a 'partial' arching above the tunnel, where the maximum shear strains are concentrated. Under these circumstances a 'complete' arching of the rock mass above the tunnel cannot be relied upon and the initial ground support will have to be designed to carry the adverse ground load.

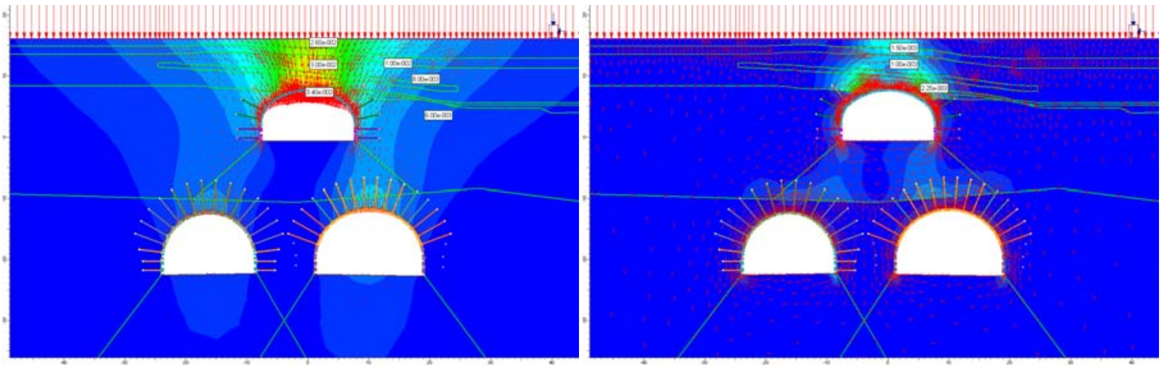

Figure 7: Contours of total displacements (left); contours of maximum shear strain and stress trajectories (right)

A safe and economical tunnel design depends on a realistic geological model, a quality rock mass characterization, and the assessment of influencing factors such as, but not limited to, primary stresses, groundwater, size of underground opening and its orientation in the rock mass complex, as well as excavation sequence and method. Knowing that no amount of investigation or analysis can precisely or fully predict the characteristics, quality or quantity of subsurface and site conditions, statistical and/or probabilistic analyses should be used to account for the variability and uncertainty in the key parameter values and influencing factors during the design phase of the project, supplemented by sound engineering judgment.

In many cases the rock mass conditions cannot be defined with the required accuracy prior to construction, despite an elaborate geotechnical site investigation and design. This is especially true for tunnels in complex geological conditions. Consequently the construction phase plays a significant role in the final determination of excavation methods, as well as support type. During construction continuous updating of the geotechnical model and an adjustment of excavation and support to the actual ground conditions is required. This design method's basic procedure is outlined in Figure 8.

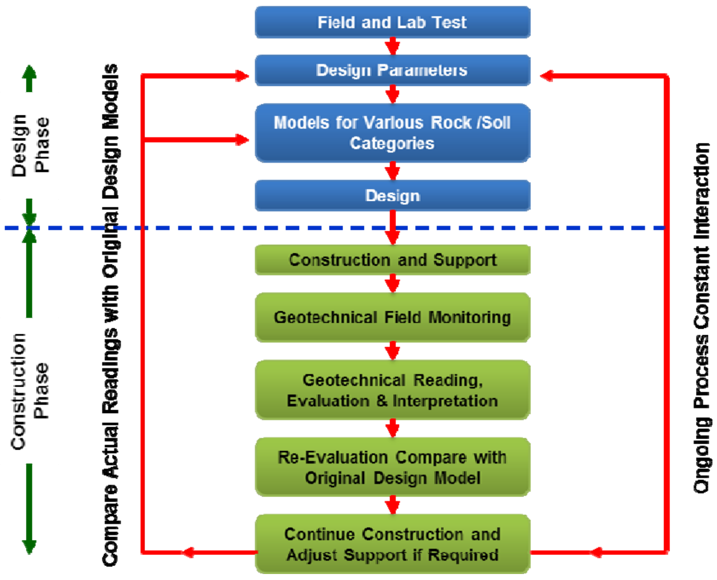

Figure 8: Flow chart of basic procedure for geomechanical design and verification of the system behaviour during construction

5 Conclusions

In shallow tunnel applications and beneath surface structures - such as buildings - that are sensitive to deformations, ground deformations and consequently surface settlements have to be kept within acceptable limits.

Each numerical prediction has advantages and disadvantages. While purely discrete models, such as UDEC, perfectly simulate the discontinuity oriented failure mechanism, several input parameters are difficult to evaluate. The weakness of a FEM approach is its continuous nature, but when the behaviour is more dominated by the stresses it is a practical design tool.

It has to be recognized that theoretical analyses are prediction tools that have to be supplemented by engineering judgment, local experience and case histories data where available. In particular, predicted displacement values should be verified against measured values obtained either from excavated tunnel sections and/or from tunnels in comparable ground condition and depth.

A continuous and sound geotechnical interpretation of monitoring results, geological mapping and observation of ground behaviour during excavation has to be performed to prove the applicability of deformations in the design phase of the project.

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