Geotechnical Demands of Investigations for Tunnelling in Soft Ground

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ABSTRACT: The report presents ground investigation programmes for 2 railway tunnel projects which include common techniques such as core-drillings, laboratory and field soil and rock tests as well as project-specific geotechnical, hydrogeological and geophysical investigation methods in particular. The sum of the results of ground investigations has to lead to a detailed prognosis of the geotechnical and hydrogeotechnical situation, including mechanical parameters of the ground and a prognosis of the ground behaviour due to tunnelling. Recommendations on practicable tunnelling and excavation techniques are to be given under respect of admissible deformations. Geodetic and geotechnical programmes have to be carried out and to be adjusted to the requests for feasible and applicable tunnelling techniques, to the expected response of the ground and to the given administrative conditions. At the example of two tunnel projects the efforts, demands and at least the success of particularly intense exploration programmes are described.

1 TUNNEL SCHULWALD

With 4.5 km length this double-tracked railway tunnel is the longest tunnel of the new high-speed railway line Cologne-Frankfurt. The ground conditions were explored in a large-scale investigation programme.

1.1 Ground Investigation

The altogether 65 up to 80 m deep investigation drillings were developed into groundwater gauges or as inclinometers in most cases. Apart from the classic soil mechanical lab- and field tests a hydrogeological investigation was carried out. Additionally, geophysical investigation methods were applied in the boreholes (Fig. 3) and from the ground surface. Electric profile sections were used on both sides of the tunnel route (Fig. 1) in the portal areas. In the area of a potable water gaining facility a geo-electric cross test for the development of a three-dimensional model with possible aquifers was carried out (Fig 2). Ground areas found to be heavily faulted in the drillings were examined in detail with the seismic method of standard-refraction. The tunnel route itself was continually explored on a length of more than 3.8 km in a procedure that combined refraction and reflection seismic methods.

![Figure 1. Results of geoelectrical measurements, portal north](image1)

![Figure 2. Results of geoelectrical cross test](image2)
1.2 Ground and Groundwater

The sum of the results of the geotechnical, tectonically, hydrogeological and geophysical exploration lead up to the predicted geological longitudinal section presented in figure 4. The Schulwald-Tunnel is situated at the southeastern edge of the Rhenish Massif within a tectonic transitional zone between the Upper Rhine Graben and the Lower Rhenian Depression. In the southern part the tectonic transition to the Upper Rhine Graben is tunnelled through. The soil is characterised by an intensive change in lithology of weak and very hard phyllites. The soil is very intensely and deeply weathered and tectonically intensely stressed within the entire area of the line. In the southern part of the Schulwald-Tunnel tertiary sands are predicted to be only a few metres apart from the tunnel roof. In the north there is a stratigraphic transition up to an entirely weathered sericite gneiss. Parallel to the line quartz-filled, water-conducting tension fractures are striking. The groundwater level is at a maximum of 51 m, the greatest covering of rock is 60 m above the tunnel roof. The ribbon in figure 4 shows the variance of the highest and lowest groundwater levels measured during the controlling period.

From laboratory testing the phyllites are to be announced as soft rock with unconfined compression strength of 5 MPa in average, but not exceeding more than 10 MPa.

1.3 Construction Techniques

The double-tracked Schulwald-Tunnel was constructed with a full section of about 156 m² in shotcrete tunnel method with partial drivings. There were 4 starting points available. The particular excavation steps are shown in a longitudinal and cross section in figure 5.

A pilot tunnel (clear section ≈ 30 m²) hurrying on ahead was constructed within difficult rock areas. From time to time this pilot tunnel had to be once more divided into an upper and a lower half due to major stability problems of the face within entirely weathered and intensely stressed rock. The crown area had to be secured by 12 m long injection fore-piles with 4 m overlapping length. In weak rock areas the immediate support was strengthened in general. The shotcrete invert lining of the pilot tunnel as well as the invert lining of the crown were rounded out deeply.

Depending on the ground the bolting of the pilot tunnel had to be stretched up to 12 m long injection bolts. These bolts were reused after widening of the crown and integrated in the systematic bolting of the vault. Due to the results of geotechnical mapping and monitoring of the pilot tunnel after widening of the crown the vault feet in some cases were constructed with a width of up to 1,4 m. However, these widened vault feet could be supplied with an invert lining, if deformation exceeds previously determined limits. The driving was carried out with continuous groundwater lowering resp. relaxation’s hurrying on ahead.
excavation class In general following major results can be noted:

- The roof settlements due to the driving of the pilot tunnel are very low
- The main deformation arises in the course of the widening of the pilot tunnel up to full section of the crown resp. due to the top heading
- The deformations resulting from bench- and invert excavation are low
- The sum of deformations resulting from all excavation steps reaches about 25 cm roof settlement at maximum (southern driving), but 12 cm to 14 cm in average.

2 TUNNEL RÜDESHEIM

For the about 2,100 m long Tunnel Rüdesheim a feasibility study is on the run in order to evaluate the construction of either two single-tracked tunnels (Fig. 7) running in a close distance or one double-tracked tunnel. Depending on the results of the geotechnical investigation the decision will be made according to the demands of the rescue concept for

Figure 5. Particular excavation steps and securing measures

1.4 Geotechnical Monitoring

Settlements and convergence within the tunnel were measured in permanent distances (25 m - 50 m) and supplied by further measurements, e. g. in the course of tunnelling through fault zones. In defined cross sections the behaviour of rock due to excavation has been measured by means of additionally installed pressure cells within the preliminary shotcrete lining as well as by extensometer monitoring.

1.5 Deformation

In figure 6 the results of roof settlements are shown for the three main construction steps. Slight differences regarding time-settlement behaviour of the mainly weathered phyllites in the northern section and the mainly tectonically stressed phyllite in the southern section of the Schulwald-tunnel are to be noted. Both drivings were excavated under equally conditions regarding overburden and

North

station

South

Figure 6. Roof settlements due to the particular excavation steps.
tunnels of the Deutsche Bahn AG under respect of the effective costs and under general aspects regarding the urbanisation of the city of Rüdesheim.

In general the results of the geotechnical and geophysical investigations outline mainly deep weathered and heavily tectonically stressed

Figure 7. Project site, planned lining

2.1 Ground Investigation

The geotechnical exploration programme consists of altogether 45 borings, which had to explore the above mentioned to slightly different linings of the planned tunnel. The investigation included geophysical exploration methods such as combined refractional and reflectional seismic methods along the tunnel line, refractional and geoelectrical profiles almost perpendicular to the tunnel axis or normal to the striking of the rocks. Within several boreholes further geophysical methods have been applied such as calibre, temperature, gamma-ray, electrical conductivity, acoustic and optic scans. Apart from that the hydrogeological investigation included single and multiple pumping tests and permeability tests as well as measurements of groundwater flow velocity. Soil mechanical lab- and field tests were carried out to define the mechanical parameters of the relevant soil and rock for stability analysis of open cuts, deep foundations and the tunnel linings and to define the behaviour of the ground during excavation. As field tests lateral pressure probing was applied in order to determine the elastic modulus of the rocks, an in-situ stress monitoring station has been installed that should give general information on the actual stresses. Additionally in debris slope areas at the planned Western portal vertical inclinometers have been installed to control any recent slope deformation.

Historical and laboratory investigations of the environmental geotechnical situation were carried out; the possibilities and costs to recycle, re-use or storage the soils do not only depend on anthropogenic but also on geogene contamination.

Devonian rocks consisting of claystones (phyllites) with varying contents of quartzite mineralizations and up to 60 m thick quartzite schist. Tertiary clays, silts, sands and gravels and Quaternary silts and sands are overlain by up to 5 m thick fillings. In both portal areas slope debris covers the weathered phyllites; in the Eastern section on a length of about 300 m these debris sediments do reach the level of the planned tunnels. The ground is heavily tectonically stressed, a pattern of probably older SW-NE striking fault zones and thrusts which are disturbed and dislocated by younger SE-NW striking faults and shear-zones shows a local tectonically regime that fits very well into the general tectonic regime of this region. These tectonic structures have been carried out by the interpretation of the core drillings, through hydrogeological tests and corresponding results of the geophysical investigation as shown exemplary in figure 8.

Figure 8. Results of seismic investigation along the tunnel line (section of 500 m), tectonic structures

In figure 9 the results of geoelectrical measurements along the tunnel line in the area of the portal West are presented; a large fault zone dislocates quartzite layers in the Western area from phyllites. A distinct thrust of slope debris as top layer characterizes this weakened zone.
Within the local tectonic regimes the groundwater level depends directly on and is influenced by these structures. Hence, regarding tunnel driving technology and design of the tunnel this means extremely varying groundwater levels at very close distances.

2.2 Construction Conditions

On demand of the railway corporation an outer diameter of 12.6 m for the double-tracked tunnel and for the single-tracked tunnels an optimised outer diameter of 9 m at minimum is given. The covering to undergo the urban area is growing from the East to West from the starting point of the driving with 6.3 m resp. 4.5 m (about half of the tunnel diameter) to 15 m at maximum. In the undeveloped area in the Western part of the tunnel project the covering is about 65 m at maximum. The groundwater level is about 25 m above track level at maximum.

In the following two characteristic schematic situations for the construction of either two single-tracked tunnels which are connected with rescue tunnels in a distance of about 600 m or one single-tracked tunnel with an emergency exit right in the middle of the tunnel are presented. The emergency exit for the double-tracked tunnel would require an up to 25 m deep, 100 m long and 40 m wide open pit in an area, where the ground is deeply weathered and the groundwater level is close to the surface.

In the area of the planned open cut at the portal East there is a major fault zone that divides the ground and the groundwater conditions significantly (fig. 10). On the one northern side of the fault zone the ground consists of intensive decomposed and faulted phyllites which are overlain by Quaternary/Tertiary sands and gravels. The groundwater was found 35 m deep under surface first but the artesian pressure head reaches up to 10 m under surface. On the southern side of the major fault the basis of dislocated Tertiary clay reaches up to 32 m under surface, overlain by Quaternary gravel and loess. There was no groundwater found after three months of monitoring.

In the area of the planned starting point for the later discussed shield drivings the ground consists of a heavily faulted phyllite. A major fault zone effects that in the area of the foundation of the tunnels semi-solid to hard phyllites border on extremely hard quartzitic rocks. Additionally the results of the overall geotechnical and geophysical investigation confirms that at least one tunnel has to be driven within an area of rock slope debris, the groundwater
level is at the top of the tunnels. Prior construction extraordinary investigations and stability considerations are to be carried out in order to guarantee a stable and secure driving.

2.3 Recommendations for the Tunnel Lining and Tunnelling Technology

On the basis of the results of the geotechnical and geophysical investigations, the facility of shallow tunnelling in the urban area under such difficult ground and groundwater conditions determines the tunnel design and the tunnelling concept. Regarding the construction of two single-tracked tunnels a universal driving (shotcrete tunnelling method) with sequential excavation and either a pilot tunnel (see chapter 1) or side wall drifts running ahead or a heading - bench - invert excavation in a very close distance is practicable. Such an universal driving demands extensive additional measures to guarantee the stability of the face, of the tunnel, the stability against ground failure under respect of the groundwater conditions as well as measures from within the tunnel and from the surface to minimize settlements in the developed urban area. It is expected that groundwater lowerings are not allowed by the authorities, besides that they would effect inadmissible settlements prior excavation.

A tunnel boring machine feasible to drive through a series of loose or semi-solid soils and semi-solid to very hard rocks is not available for the double-tracked diameter at the moment whereas the universal driving is practicable.

The recommended driving concept is to construct the two single-tracked tunnels with a tunnel boring machine, in this case an earth-pressure-balanced (EPB)-shield equipped with an hardrock boring head. This tunnelling technique includes several advantages considering the ground and groundwater conditions as described before as well as the possibilities to control respectively to minimize settlements in the suburban field.

3. REFERENCES

Deutsche Bahn AG, DBBauProjekt GmbH Frankfurt, Tunnel Schulwald and Tunnel Rüdesheim
