Design and construction of tunnels in Lot A of the highspeed rail link Cologne-Frankfurt

Planification et construction des tunnels en Los A pour le train à grande vitesse Cologne-Frankfurt

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ABSTRACT: The twin-track tunnels of the high-speed railway line between Cologne and Frankfurt on Main with their large excavation section of up to 160 m^2 and shallow cover posed great challenge for contractors and designers. The paper describes the geotechnical problems and their solutions during the design and construction of the six tunnels in the north section of the alignment. The paper also touches on issues relevant to the design-build practice in the project.

RÉSUMÉ: Les tunnels à double voie pour la ligne ferroviaire à grande vitesse entre Cologne et Francfort sur le Main avec leurs sections de larges excavations allant jusqu'à 160 m^2 et une très mince couverture ont posé de grandes difficultés pour les entrepreneurs et les projeteurs. Ce papier décrit les problèmes géotechniques rencontrés ainsi que les solutions apportées lors de la planification et la construction des six tunnels de la section nord du tracé.

1 INTRODUCTION

The new high-speed line Cologne – Rhein/Main of the German Railways will connect two major metropolitan areas in Germany: the Rhein/Ruhr area around Cologne in the north and Rhein/Main area around Frankfurt in the south. Apart from its importance for domestic rail traffic, this high-speed line is also an integral part of the planned European high-speed rail network. The alignment is about 180 km long and is divided into three sections, north, middle and south. The middle section with a length of about 130 km is further divided into three lots, A, B and C. The alignment in Lot A in the north is about 47 km long and passes through six tunnels (Table 1). From north to south they are (1) Ittenbach, (2) Aegidienberg, (3) Rottbitze, (4) Günterscheid, (5) Ammerich and (6) Fernthall. All tunnels have been awarded on a design/build basis.

Of the six tunnels the consortium Mittelstand, comprising the companies Bögl, Moll, Schmalz, Wiebe etc. was entrusted with the construction of the four tunnels in the north and the consortium ATAC, comprising the companies Alpine, Ilbau, Beton & Manierbau etc. with the two tunnels in the south. Lahmeyer International was retained by both consortia for the detailed design of the six tunnels. In addition Lahmeyer International provided geotechnical expertise and survey for the four tunnels of the Mittelstand consortium.

Table 1. Tunnels in Lot A

Tunnel	L [m]	L[illegible] [m]	L[illegible] [m]	max. Overburden [m]	max. GW o. invert [m]
Ittenbach	1145	755	390	27	15
Aegidienberg	1240	1120	120	37	30
Rottbitze	820	710	110	15	10
Günterscheid	1130	1060	70	30	17
Ammerich	755	655	100	29	8
Fernthal	1555	1382	173	45	25

2 GEOLOGICAL SETTING

The following descriptions are summarized from the Geotechnical Baseline Reports for the tunnels (GBR1 1995 and GBR2 1995). Under a thin layer of Quatenary deposit up to 5 m the bedrock along the alignment in Lot A is comprised of strongly

Figure 1. Typical alteration zone.

folded and weathered sedimentary rock of the Devonian Formation, in particular the Upper Siegner Member. This geological unit mainly consists of fine-clastic sandstone and mudstone in intervals of interbedded sequences. The sedimentation of the Devonian Formation took place some 380 millions years ago in a marine environment. During the varizic orogeny the Devonian Formation has been folded intensively and cut by numerous shear and fault zones. The orogenic activities have given rise to joints, fractures and schistocity. The two prevailing joint sets are orthogonal and normal to the bedding plane. The bedding planes and the joints reduce greatly the shear strength of the rock mass and endow the rock mass with orthogonal anisotropy. Occasionally schistocity can be observed in mudstone. Most joints are closed and free from fine filling. However, there are also open joints with fines in particular near the surface.

Between Carboniferous and Tertiary the Devonian Formation cropped out and was strongly weathered. The weathering horizon went as deeply as about 70 m under the ground surface. Weathering is the primary factor controlling the rock quality. The following rock classification was used in the Geotechnical Baseline Report: unweathered (V1), slightly weathered (V2), strongly weathered (V3) and entirely decomposed (V4).

The second factor controlling the rock quality is the presence of zones of hydrothermal alteration along the alignment. A typical alteration zone is shown in Figure 1.

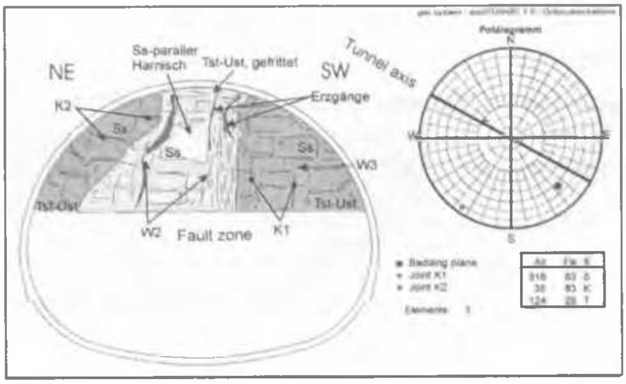

Figure 2. Digital image of geological structure at face

The alteration of the Devonian bedrock was probably associated with the intrusive activities of the former volcanic chain of the neighboring Siebengebirge to the north. The volcanic activities in the project area can be observed in form of andersite dike and basalt sill. The distinction between weathering and hydrothermal alteration is made, because the hydrothermally altered zones have generally varied engineering properties and have no distinct alteration boundaries. Weathering horizons tend to be near surface and the rock quality generally improves with depth. Alteration zones were frequently registered in the tunnels Aegidienberg and Rottbitze.

As is usual the case for sedimentary rocks the most significant structural element in the Devonian bedrock is the bedding plane. The striking and dipping of the bedding plane is determined by the geometrical structure of the folds. In the project area the fold axes strike NW/SE and are normal to the alignment with minor local deviations. The alternation of anticlines and synclines along the alignment gives rise to varying dipping of the bedding planes. Recumbent dipping can only be observed in the vicinity of crest and trough. Frequently the bedding planes are slickensides with very low shear strength. In this case the dipping of the bedding planes is particularly relevant for the face stability during tunnel driving.

The low permeability of mudstone and the relatively high permeability of sandstone characterize the hydrogeological setting of the site. The groundwater level is generally high and lies immediately under the Quantenery deposit. Frequently the different permeability of mudstone and sandstone leads to perched groundwater levels. Therefore the water head on the permanent lining is usually lower than the maximum groundwater level. Water ingress during tunnel driving was low and mostly along bedding planes and joints.

The geological structure at the face during tunnel excavation was mapped daily using digital picture and image processing (Figure 2). As compared with the conventional pencil sketch the face image is objective in color and structure. Moreover the mapping can be made available to all members involved in the project via Internet. In this way the decision making process can be accelerated.

3 EXCAVATION AND SUPPORT

In general the tunnels were started from the lower end and were driven with increasing gradient to avoid groundwater build-up at the face. In general excavation and mucking were carried out using excavator. In fresh rock formation loosening blasting or blasting were also used. The tunnel support comprises a temporary shotcrete lining and a permanent lining of cast-in-place concrete. Since the alignment runs through an environmentally sensitive area, all tunnels were undrained and waterproofed. For water head under 30 m over the invert waterproofing was provided by concrete lining with a minimum thickness of 35 cm. If the water head exceeds 30 m a waterproofing membrane was arranged between the temporary and permanent lining.

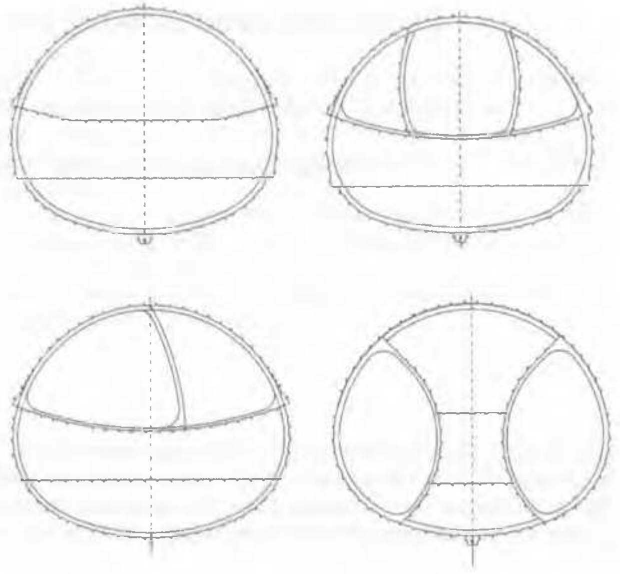

Figure 3. Excavation schemes: (1) top heading, (2) middle adit, (3) asymmetrical adit, (4) side gallery,

In order to meet with the different ground conditions the excavation sequence and the temporary support were varied according to the excavation or support classification. The excavation classification was designated based on the ground conditions as described in the Geotechnical Baseline Report. To this end, the anticipated ground conditions along the alignment were idealized into sections. In each section the ground conditions are regarded as the same with reference to difficulty for tunneling. In specifying the excavation classes the following factors were taken into consideration: excavation sequence, support at face, advancing support and time of support (ETB 1995).

Figure 3 shows the excavation sequences for the tunnels in Lot A. The top heading is the most economical and widely used excavation scheme. For adverse ground conditions with strict settlement requirement the side gallery is often used as the last resort. However the side gallery is labor and cost intensive. In particular the change from top heading to side gallery or vice versa is very cumbersome and time consuming since the entire tunnel section must be excavated and supported prior to the change. Therefore the contractors wished to have alternatives to side gallery. The two alternatives in Figure 3, both the middle adit and the asymmetrical adit, have the merit that a change to the conventional top heading scheme can be done easily. Our experience showed however these two alternatives are not to be recommended to meet strict settlement requirement.

The above excavation schemes are combined with other support measures, such as shotcrete lining, structural rib or lattice girder, radial anchor, steel mesh and spiling, to have numerous support classes commensurate with the actual ground conditions.

Based on the geological mapping the decision as to which excavation class was to be applied was jointly made by the contractor and the owner's engineer. Note that the success of a tunneling project depends to a large extent on a workmanship with hand-on experience and an experienced tunneling consultant. The increasing bureaucracy in some projects is an alarming signal for the tunneling community. Everybody is in charge of something, but nobody is responsible for anything. One often ends with endless claims and rocketing cost.

4 ANALYSIS AND MONITORING

The structural analysis of the temporary lining was based on the finite element method (FEM). The ground was treated as a per-

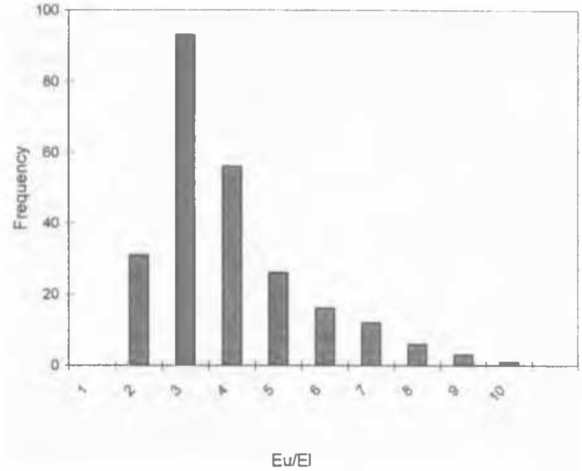

Figure 4. Frequency of ratio E_u/E_l obtained from pressuremeter tests

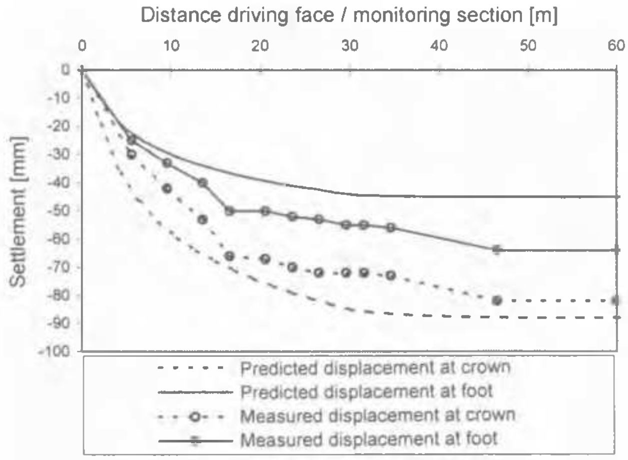

Figure 5. Comparison between prediction and measurement for top heading

fectly plastic medium with the Mohr-Coulomb failure criterion. The shotcrete lining was simulated with elastic beam elements. The three-dimensional stress and strain around the driving face was idealized into a plane strain model. The stress relief before and after the installation of the shotcrete lining was simulated by the so-called load method (Kielbassa & Duddeck 1991). The excavation sequences and the installation of support measures were simulated by removing and adding sub-structures from and to the system respectively. The permanent lining was dimensioned based on the bedded beam model. The interaction between ground and lining was modeled by a series of elastic springs connected to the beam elements.

The calculations were carried out for representative sections perpendicular to the tunnel axis. The representative sections were chosen by considering excavation scheme, support measures, ground conditions and overburden. Due to the large variation of the ground properties two sets of design parameters were proposed for the characteristic case and the worst case respectively. The dimensioning of support measures was based on the characteristic parameters. The parameters for the worst case were used to show the effect of ground variation on support. Since the worst scenario is unlikely to occur it was decided to use a reduced safety factor of 1.0 in this case.

As mentioned in the introduction the project was awarded on a design/built basis. As contractor's designer we proposed the design parameters and the dimensioning strategy aiming at an economic and safe design. An important design parameters is the elastic modulus of the ground. During the exploration campaign some 250 pressuremeter tests were carried out. The loading program of each test comprised several loading, unloading and reloading cycles. In the Geotechnical Baseline Report however only the loading modulus was considered, which gave rise too conservative design. Since the ground during tunnel driving experiences mainly unloading it was decided to use the unloading-reloading modulus. Figure 4 shows the ratio of unloading modulus E_u over loading modulus E_l. The unloading modulus is seen to be at least twice as high as the loading modulus.

The design of underground structures differs from the design of surface structures in two aspects. First, a separation of action and resistance, as is usual for surface structures, is not appropriate for tunnel design, since the ground acts both as load imparting and as load bearing. Second, tunnel design is a trial-and-error process and therefore subjected to continuous modifications, whereas the design for surface structures usually remains changed. There have been some attempts to standardize tunnel design. However these attempts were not successful, since the ground is different for every tunnel. Probably this is why tunneling is the only discipline in geotechnical engineering where the finite element method has its indispensable place in design.

In this sense tunnel design still allows us some freedom to work out our own dimensioning strategy. For shotcrete lining different safety factors were used for normal thrust and bending moment. For thrust the safety factor according to the German Concrete Code (DIN 1045) was adopted. For moment a reduced safety factor of 1.0 was used. The bending moment, which is of importance for surface structures, is less important for tunnels. While the bending moment is greatly reduced by the creep behavior of young shotcrete and by the nonlinear behavior of hardened shotcrete due to cracking, the normal thrust remains almost unchanged. Moreover failure due to excessive moment is ductile, while failure as a results of too high thrust is of brittle nature. The latter leaves less time than the former and therefore demands a larger safety reserve.

The fact that NATM is based on the so-called observational method places great emphasis on the geotechnical monitoring during tunnel driving. The geotechnical monitoring registers the ground response to tunneling excavation and support. With the help of this feed-back the designer decides whether the design parameters need be modified and the support re-calculated. This interaction was also likened to a dialog between ground and engineer (Müller 1978). In our project an extensive monitoring program was prescribed including displacement and convergence measurement in tunnel, settlement measurement on surface, extensometer, measurement of rock pressure and concrete stress, measurement of anchor force. One of the tasks of the geotechnical monitoring was to verify/disverify the predictions based on the FEM. This was furnished by the so-called calibration curve shown in Figure 5. A large discrepancy would mean that the underlying design parameters need be modified.

5 ITTENBACH TUNNEL

The 1045-m long Ittenbach tunnel consists of a NATM section of 755 m in the north and a cut and cover section of 390 m in the south. The tunnel alignment shows a downward gradient of 4 % from north to south. Tunnel driving of the NATM section started from the north in an access pit. The front side of the access pit was supported with soldier piles of structural steel rib and shotcrete lagging. The soldier piles were 2 m apart and tied back by pre-stressed anchors with length between 10 and 20 m. The side slopes of the access pit with an inclination of 1 to 1 were supported by soil nails. The support measures were dimensioned based on the Geotechnical Baseline Report. The ground was assumed to be of moderately weathered siltstone. The tunnel support immediately behind the soldier piles and shotcrete lagging consists of shotcrete lining (d=30 cm) reinforced symmetrically with two steel wire meshes of Q513 (14 kg/m^2), SN radial anchors and forepiling (reinforcing steel rod). The excavation

Figure 6. Slide along bedding planes

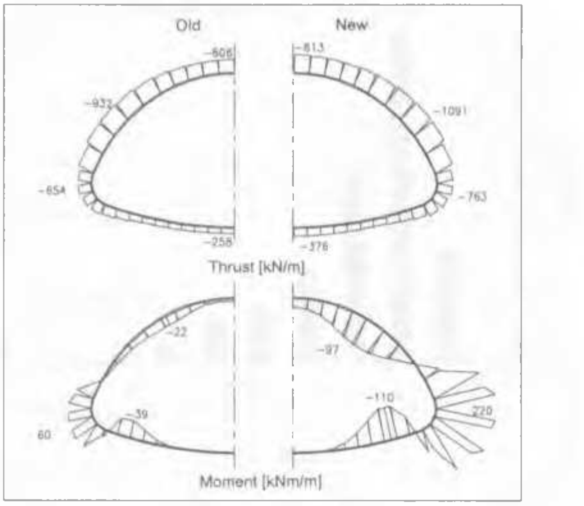

Figure 7. Moment and thrust with old and new parameters

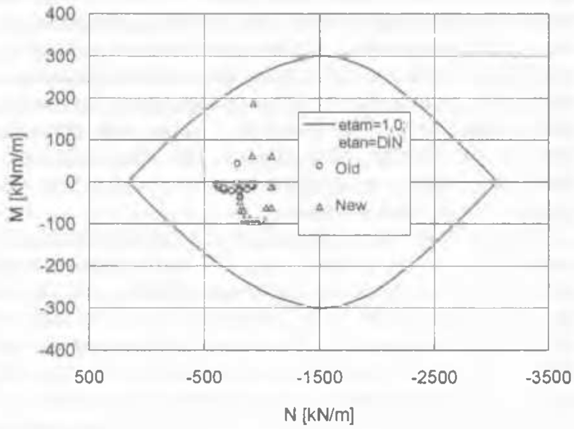

Figure 8. Bearing capacity diagram

work shall be carried out in the sequence of an advancing top heading followed by the excavation of bench and finally the invert.

The excavation and support of the 17 m deep access pit brought unexpectedly adverse ground conditions to light. The ground is composed mainly of fully decomposed mudstone with interbedded sandstone. The mudstone possesses very low permeability and is sensitive to water ingress. The bedding planes strike perpendicular to the tunnel axis and dip at about 50° to the north. During excavation meter-high rock wedges slid along the bedding plane in the pit, which pointed to stability problem during the subsequent tunnel driving (Figure 6).

The slickenside of the bedding planes showed striations and fluttings indicating that the movement was directed down the dip of the bedding plane, i.e. in the access pit. The drilling activities for the pre-stressed anchors were accompanied by strong water flow out of the boreholes. The anchor tests showed that the design load of the pre-stressed anchors could not be reached. Large horizontal displacements up to 14 cm were observed at the soldier piles. It was decided to double the pre-stressed anchors in order to halve the design load. Only with the reduced design load could the anchor tests be successfully performed.

The unexpected ground conditions gave rise to additional support measures and extra cost, which were covered by the clause of differing site conditions in the contract. However, the contractor had to show that the encountered ground condition differed substantially from the Geotechnical Baseline Report. For this purpose an independent geotechnical expert was called in to review the Geotechnical Baseline Report and carry out addition tests. The in-situ and laboratory tests resulted in design parameters that are substantially lower than the characteristic parameters in the Geotechnical Baseline Report. Based on these test results the design parameters were modified to take the observed support behavior during excavation of the access pit into consideration.

The changed ground conditions also made a re-design of the tunnel support necessary. To this end a two-dimensional finite element analysis with the modified design parameters was carried out. For the top heading the results obtained with the "old" (original) and "new" (modified) design parameters are shown in Figure 7. A perusal at
Figure 7 shows a strong increase in bending moment and a moderate increase in thrust.

The poor ground conditions and the small overburden of about 10-m in the vicinity of the access pit meant that the shotcrete lining should be sufficiently dimensioned to sustain the load from the ground. The stability of the shotcrete lining can be best appreciated with the bearing capacity diagram (Wu & Saraiva 1999) in Figure 8. For given section (d=30 cm) and material properties (C20/25 for shotcrete and B 500 for wire mesh) of the shotcrete lining the curve in Figure 8 was obtained by connecting all possible M-N combinations in a limit state.

The following observations can be made in Figure 8. For the old parameters the shotcrete lining has sufficient safety reserve against both bending moment and thrust. For the new parameters however the safety reserve against bending moment is almost exhausted, whereas sufficient safety reserve against thrust is available. Actually the bending moment will be reduced by creep of the green shotcrete and by non-linearity of hardened shotcrete due to cracking. Both factors had not been taken into consideration in the finite element analysis. It was therefore decided not to increase the thickness or the reinforcement of the shotcrete lining. Instead attention was focused on the stability of the tunnel face.

Investigation on the face stability showed considerable deficiency, which was caused mainly by the poor ground conditions and the unfavorable dipping of the bedding planes. Furthermore the deficient face stability was aggravated by the presence of ground water. In view of the low overburden instability at the driving face would induce collapse to the ground surface. This was a too high risk for the gas pipelines some 3.5 m behind the soldier piles. In order to minimize this risk we proposed that the top heading should be carried out under the protection of an advancing umbrella. The umbrella is composed of 15-m long steel pipes with a diameter of about 12 cm. The distance between two adjacent steel pipes was about 30 cm. The steel pipes were ar-

Figure 9. Start of top heading under the protection of umbrella

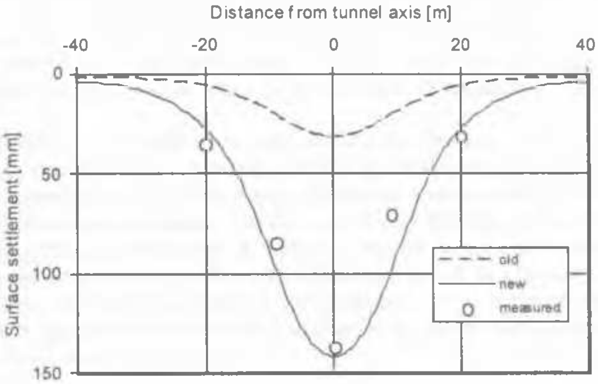

Figure 10. Settlement trough with old and new parameters

ranged around the section of the top heading (Figure 9). During drilling the steel pipe was used as casing and was then left in the ground. An injection of about 3 bar in the steel pipe helped improve the bond between the steel pipe and the surrounding rock mass. The umbrella was advanced with an overlapping of about 4 m. In this way continuous protection of the top heading could be achieved. The tunnel face under the umbrella was to be supported by 10-m long horizontal anchors of reinforcing steel rod. In addition advancing de-watering borings were prescribed to mitigate the negative impact of ground water on face stability.

The calculated settlement troughs with the old and new parameters are shown in Figure 10. The maximum settlement of about 15 cm was too large for the pipelines. We proposed to place hydraulic jacks under the pipelines to compensate the settlement from tunnel driving. The measured settlements on the ground surface are also shown in Figure 10. It can be concluded that the modified parameters provided a realistic description of the ground properties.

Retrospectively the proposed support measures turned out to be effective. The large settlements were in fairly good agreement with our prediction. Water ingress was mainly along the de-watering boreholes and was mostly under 2 l/min. For the first 50 m a driving rate of 2 m/d could be reached. The low driving rate was mainly due to the installation of the umbrella and horizontal anchors at the driving face.

After four shifts of umbrella, i.e. some 50 m behind the soldier piles the rock quality showed gradual improvement. The majority of rock in tunnel section was classified as strongly weathered, so that the tunnel was driven without the umbrella. Between 60 m and 70 m wide radial cracks were observed at the crown (Figure 11). The excavation works were halted and the reading frequency of the displacement measurement was in-

Figure 11. Radial crack in shotcrete lining at 60 m

creased. In addition the shotcrete lining between 50 m and 90 m was strengthened by additional radial anchors.

The radial cracks were apparently caused by the differential settlement between the adjacent tunnel sections with and without the umbrella. The umbrella is not only suitable for increasing the face safety but also for reducing settlement.

6 CONCLUSIONS

The ground response to excavation and support depends on a number of factors such as ground properties, excavation scheme and support measures. For shallow tunnels the support should be applied immediately after excavation in order to avoid excessive deformation and retain the inherent strength of the ground. The sedimentary rock in our project was characterized by the inhomogeneity, anisotropy and weathering. In particular the dipping of the bedding planes played an important role for the face stability. The following features can be drawn from our experience:

- In general the settlement was small, mostly under 10 mm. Large settlement was caused by adverse ground conditions in totally decomposed rock formation and partially by de-watering. The settlement was well predicted with the unloading-reloading modulus.
- The horizontal displacement in tunnel was characterized by small convergence of less than 10 mm. This indicates that despite the low overburden the ground still possesses some overconsolidation.
- Despite adverse dipping of the bedding planes most of the tunnels could be driven without any stabilizing measures at the face. This was ascribed to the high horizontal pressure and to the high rock strength.
- The tectonic action give rise to joints and fractures with close spacing, which is particularly relevant for the roof stability. Therefore advanced spiling was prescribed for most of the tunnels.
- In case stabilizing measures were necessary, e.g. in strongly weather and fully decomposed rock with adverse dipping of the bedding planes, the conventional support method by leaving the central part of excavation (supporting core) to a later excavation was not always applicable. Due to the adverse dipping the supporting core itself was not stable. An effective alternative was horizontal anchors

with length around 10 m. Anchor of glass fiber is a better alternative, since it places no hindrance for excavation.

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