Observational method ensuring the stability of a deep excavation in a clayey slide mass

Stabilité d'un excavation profonde dans des matières de glissement argileuses maintenu par la méthode d'observation

P.Kudella – *Institute of Soil Mechanics and Rock Mechanics, University of Karlsruhe, Germany*

ABSTRACT: The Ursulaberg tunnel in Pfullingen/Germany is constructed as an open cut in a sensitive weathered Middle Jurassic side-hill. The use of the observational method was not planned originally, but it proved to be very useful after unexpected deformations of the deep anchor wall were recorded. Owing to additional measurements, a satisfying mechanical model could be established, by which the deformations endangering serviceability and for the design of temporary struts were evaluated.

RÉSUMÉ: Le tunnel de Ursulaberg à Pfullingen/Allemagne est construit en fouille à ciel ouvert et situé dans une pente du Jura Moyen décomposé. L'emploi de la méthode d'observation n'était pas projeté au commencement, mais il c'est montré d'aventage après l'observation des grandes déformations unprévues du blindage profonde et de l'ancrage. Expliqué par un modèle mechanique sur la base des mesures additionelles, les déformations se manifestaient qu'un problème d' état limite d'usage et une étrésillonnage temporaire était fabriqué.

1 INTRODUCTION

The observational method is recommended in situations where the prognosis of the construction's geotechnical performance is difficult or uncertain. According to Eurocode 7, the principles of the observational method should be applied from the beginning. The present example demonstrates that the observational method is useful even for buildings conventionally planned, if problems with the assessment of serviceability limit state are encountered during construction.

About 40 kms south of Stuttgart, a two-lane road tunnel known as "Ursulaberg-Tunnel" is under construction. With this tunnel, the new alignment of the national road B 312 crosses the town centre of Pfullingen. As in many similar situations in densely populated countries, building the new road was required without restricting the existing traffic or affecting the residential areas around. Helpful in this case was the possibility to plan the tunnel along an abandoned railway line in an open cut of 1200 m length and 10 to 17 m depth. Difficult geological conditions were the price of this logistically comfortable solution.

2 GEOLOGY

2.1 *Preliminary investigation*

The tunnel cuts into the hillside of the Echaz River in the mountains of the "Schwäbische Alb". Prevailing formation is the Middle Jurassic, famous for its pronounced hill-creep and landslide potential in the region. The natural slope angle is about 7°. It had been shown by 8 core borings and additional penetration soundings, that the Jurassic claystones are overlain by Quarternary hillside loam and debris with varying depths between 2 and 12 meters. These can be described as gravels and rock fragments in a predominantly silty to clayey matrix without a stabilizing grain skeleton.

The Middle Jurassic consists of claystones, marlstones and sandstones. Generally, the tunnel base reaches unweathered rock strata (weathering class V2-V0 after Einsele et al. 1985, Fig.1) whereas the side walls are situated in a natural weathering profile, reaching from thinly stratified claystone (V3) to plastic silty clay (V5, final weathering product).

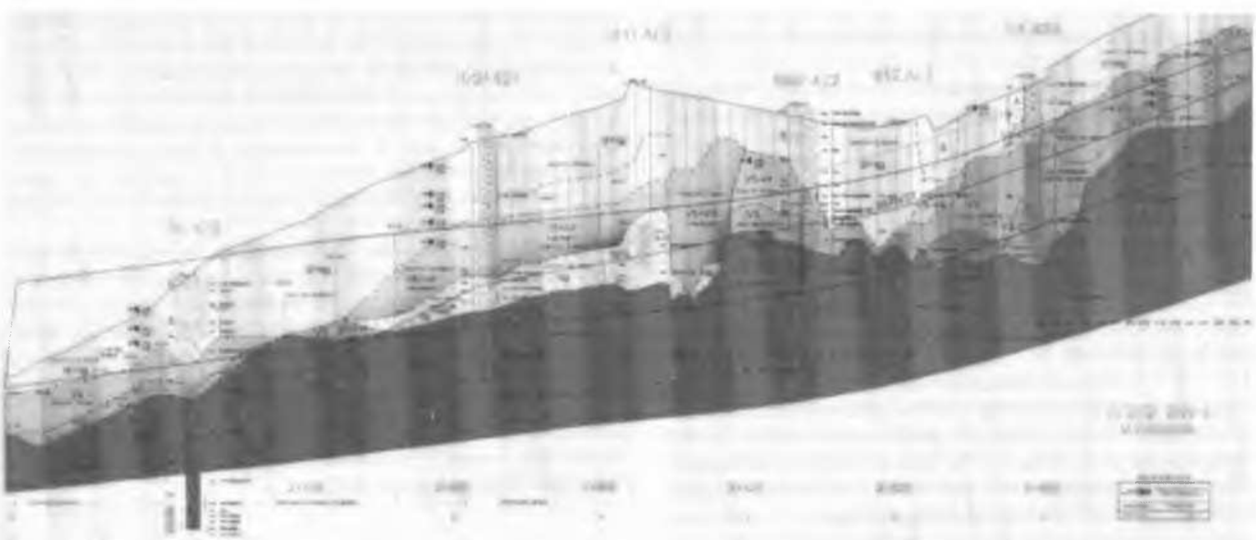

Figure1. Updated geological documentation, detail km 1+900 to km 2+750 (note redeposited weathering profile in the centre of the drawing)

Figure 2. "Undisturbed" (left) and "disturbed" (right) weathering profile

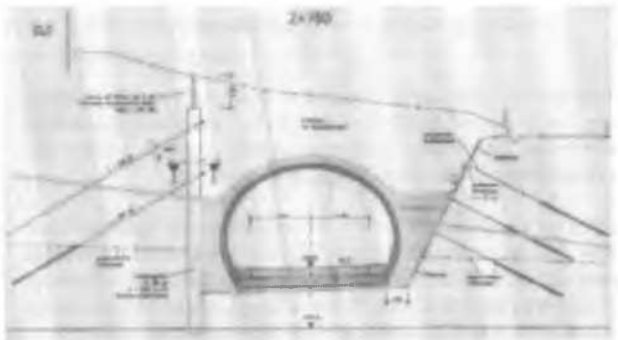

Figure 3. Standard cross section (tender) with pile (left) and anchor wall (right), design phreatic line and backfill

In some locations, the hillside debris shows clear indications for repeated redeposition. In some cases, not only the hillside loam, but the complete weathering profile has been partially eroded and replaced by alluvial gravels or turned completely upside down by a paleo-landslide (Fig. 2).

2.2 *Groundwater*

Side-hill seepage within the Middle Jurassic joints and the hillside debris has little, but irregular yield depending on rainfall. This makes dewatering ineffective, but still necessary. A design groundwater table had been given and any permanent changes of the natural draining conditions had to be avoided using special measures.

2.3 *Updated investigation*

Continuous geological monitoring during pile boring and excavation works supplemented the preliminary investigation. It concentrated on the mapping of the hillside layers and the weathering profiles, thus enabling all parties involved to adjust assumptions or parameter in the statical calculation if appropriate (Fig. 1).

Owing to considerable deformations (see below), the documentation was later complemented by extensometer and inclinemeter drillings.

3 CONSTRUCTION PRINCIPLE

3.1 *Tunnel building*

Unusual for the open-pit construction method, the tunnel profile has not a rectangular, but an oval cross-section resulting from economic optimization. It is designed to bear the overburden load of 2 to 8 meters by taking advantage of the combined backfill and subsoil lateral reaction pressure. Sophisticated FE studies had been made to determine the quality requirements of the crushed limestone backfill taking into account variable earth pressures of the natural ground and the interaction between lost retaining wall, backfill and tunnel lining stiffness.

Figure 4. Excavation pit with soldier piles (left) and anchor wall (right)

3.2 *Retaining structure*

For abt. half the tunnel length, the retaining construction for the pit excavation uses an anchor wall (in the tender planned with 60° inclination, see Fig. 3, but later steepened to 85°) with a shotcrete shell of 25 cm. Depending on the excavation depth, between 3 and 7 layers of prestressed anchors with an approximate spacing of 2 meters and with loads of 300 kN were required. Only the lowest 1-2 layers with 450 kN reach into the solid claystones. The others, with lengths of 10 to 14 m, act like a soil block nailing. For those, it had been very cumbersome to reach the high design loads requiring at least 2 phases of re-grouting. On the other hand, all individual anchors were subject to pull-out tests with an unusually high safety factor of 1.4.

Bored soldier piles with deep anchors and shotcrete filling were used in the remaining areas where existing buildings were closer than 20 meters.

For both wall types, the design was based on slip circle calculations. Characteristic shear parameters of the soil had been given by the geologist separately for each weathering class, but in the statical calculation, they had in some cases been averaged too optimistically over the slip circle height. Safety of intermediate construction stages was assumed to be sufficient with temporary berms between lowest fixed anchors and bottom of excavation. Water pressures were not accounted for (more details in Gerold et al. 2000).

4 SERVICEABILITY LIMIT STATE PROBLEMS

4.1 *Deformation measurement*

Deflections of the two wall heads using prism reflectors had been first monitored following an advice of the supervising engineer as a compensation for the impossible proof of individual anchor failure. Alarmed by unexpected movements of the anchor wall head during initial measurements, the section of the disturbed weathering profile had then been equipped with additional settlement points and 3 extensometers. These extensometers were installed parallel to the soil anchors to monitor the extension between shotcrete wall and reference points 10, 15, 20 and 35 m in the ground.

Problems started, when the excavation approached the section with "disturbed" weathering profile (see Fig. 2) which corresponded unfortunately to the maximal wall height (17 m). Within five months, both lateral and vertical deformations accumulated to 120 mm, firstly accelerating and then slowing down in relation to the excavation progress, eventually turning into a constant creep even in periods without any further construction activity. More alarming, the actual extension zone enlarged far beyond the extension of the anchorage. At the same time, cracks developed in neighbouring houses.

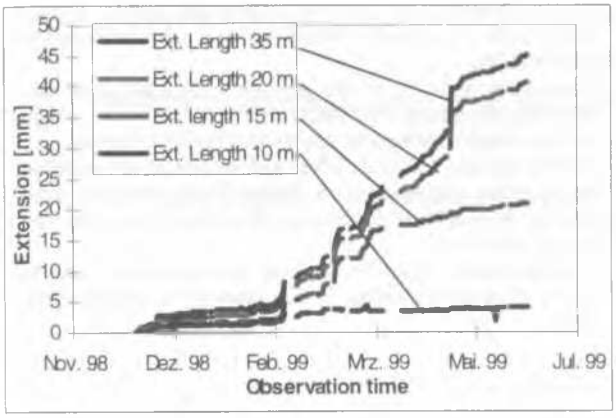

Figure 5. Recorded extensometer extensions

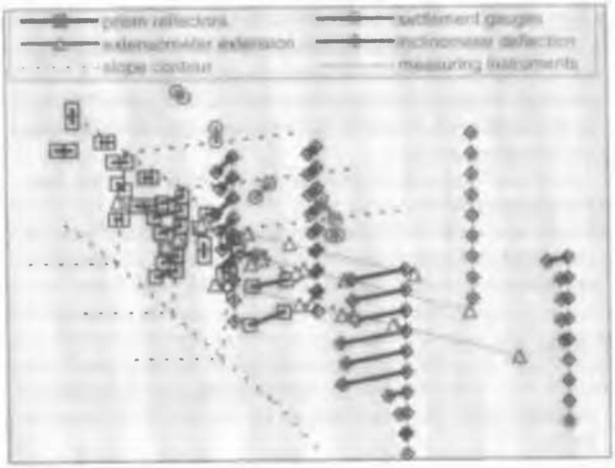

Figure 6. Plot of relative magnitude of measured deformation vectors within two weeks (between 25.5.99 and 9.6.99, shown in 3D perspective)

4.2 *Tentative interpretation and countermeasures*

First interpretations of the deformations included:

- Water or flushing air pressure build-up due to excessive anchor grouting
- Heavy rainfall and subsequent seepage
- Lacking support in some construction stages
- Constant-volume deformation after grouting plastic zones
- Underestimation of earth pressures due to lacking vertical support

and combinations thereof.

The contractor hoped to improve the vertical support of the shotcrete shell using a combination of vertical "concrete feet" in the debris and steel piles bored into the stable rock, but these measures did not reduce the deformations.

4.3 *Further observation*

A clear interpretation of the measured displacement pattern was not possible before 5 additional inclinometers in two measuring sections 10 m and 20 m behind the retaining wall had been installed. Further coring of the inclinemeter boreholes revealed an inclined (2 – 12°) thin plastic clay layer (V5) between V0-claystone surface and redeposited weathering profile.

After only a few weeks of monitoring it became obvious that over a length of 120 meters, the complete overlying mass was gliding downwards on this reactivated slip-plane like a "drawer" (Fig. 6). Corresponding settlements and surface extension reached as far as 25 m affecting the houses.

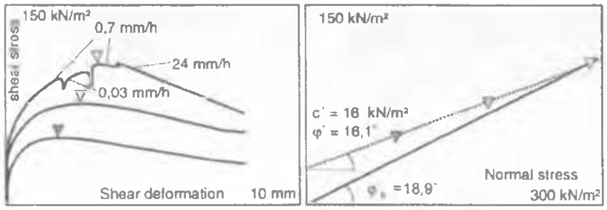

Figure 7. Shear box tests with remoulded V5 material

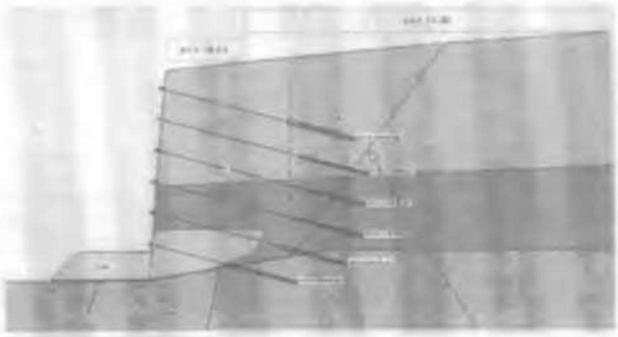

Figure 8. Mechanical model for back calculation km 2+300 (with berm)

4.4 *Mechanical explanation model*

Shear box tests have been carried out on soil samples from the critical plastic weathering product. They revealed that the former assumption of $\varphi = 17{,}5°$, $c = 7{,}5$ kN/m² was too optimistic, especially after disturbance or changes of water content. The effective cohesion vanishes after a few millimeters of shear deformation and leaves behind a material better described with a residual shear strength of $\varphi = 19°$, $c = 0$ (Fig. 7). Repeated Grouting during the construction of the anchored shotcrete wall usually required 10 days between excavation and anchor prestressing. It can be assumed that the bulk deformations inevitably occurring during this waiting time, are already sufficient for a considerable strength drop.

In a similar way, the overlying silty soil mass loses its cohesion rapidly during inundation. Due to grouting again, most drainage holes were ineffective although adjacent soil region seemed to accumulate rainfall water flowing downhill.

Based on residual shear parameters, multiple-block failure mechanisms (Goldscheider 1974) were recalculated. The model geometry was adjusted according to the monitored deformation (Fig. 8). Note, that the conventional automatic slip circle variation cannot find the most critical failure configuration if weak layers and slip surfaces involved are thin and plane !

If the slip-plane is assumed to be continuous the calculation yields safety factors close to unity. This worst-case assumption, however, is not necessary: temporary seepage forces develop after rainfall (independent of yield) leading to another minor safety reduction. Even more critical than the final state is the construction stage where the excavation reached its final depth, but the lowest anchors reaching competent rock are not installed yet.

An explanatory model for the observed creep could thus be established step by step.

4.5 *Evaluation*

Due to the above results, the occurring deformations at first questioned the serviceability. There was no observation of acceleration as for a near creep failure. On the other hand, safety against the ultimate limit state was not adequate.

Pronounced rate dependency is a typical feature of such sensitive, clayey sliding masses: partial mobilization of shear strength exceeding a certain level leads to corresponding creep rates (Gudehus 1976). The same behaviour could be established qualitatively by rate-dependent tests of the V5-clay (Fig. 7). The proof of ultimate limit state does not – like elsewhere - automati-

Figure 9. Temporary struts between retaining walls front of tunnel lining

cally include the proof of serviceability limit state.

On the other hand, for people living next to an 17 m deep excavation for almost one year, the ongoing movements are rather a question of safety than of serviceability.

4.6 *Temporary strutting*

It was therefore decided to provide countermeasures by manufactoring a set of 10 movable steel struts (Fig. 9) which could be installed step by step across the excavation above the tunnel formwork. Placed at a spacing of 4 m, they were designed to provide the missing force between required and back-calculated safety margin.

In addition, measures to accelerate adhesion of the bearing structure were taken: The placement of the concrete shell was anticipated by shifting the formwork to the endangered area and by backfilling it with lean concrete instead of limestone rockfill. The tunnel section concerned could thus be completed without further problems.

5 CONCLUSIONS

5.1 *Geotechnical design guidelines for Jurassic slide masses*

The experience gained from the construction of the Ursulaberg tunnel emphasizes the need to:

- Assume earth pressures close to at-rest values, especially if there is no vertical support for wall friction to develop
- Adopt realistic residual shear strengths in clayey slide masses, especially in combination with constructions which cannot avoid temporary soil softening
- Model all critical construction stages, especially for time-dependent soil behaviour
- Prove realistic failure mechanisms which reflect the kinematics of the retaining system as well as potential pre-construction weak soil layers and slip surfaces
- Not design a retaining system in clayey soil for anchor forces close to the physical limit or requiring excessive grouting and hence soil disturbance.

5.2 *Working with the observational method*

At the same time, this example shows the advantages of a consequent adoption of the observational method even when started later during the course of construction:

- Deformation monitoring during construction, as early as possible, serves as a tool to identify critical ground conditions or weak performance of the construction and enables adjustments in the construction process before a mechanical model is established.
- Consequent updating of the geological documentation and interpretation during the construction progress leads to more reliable design assumptions and to improved predictions.
- Further measurements, sampling and lab testing are essential as prediction and observation differs. They provide a database to develop new models which explain better the observed behaviour.
- Countermeasures must be planned and realized in time to meet any potential shortage in serviceability or ultimate limit safety.

5.3 *Requirements*

The application of observational method requires close cooperation of all parties involved and high flexibility to react to unexpected findings. Tender, contracts and site supervision must therefore:

- Conduct comprehensive geotechnical investigation and supply all data before inclusive prices are finalized
- Develop a contractual framework that encourages all parties involved to provide and to use new information rather than to hide or neglect them
- Allow qualified task-oriented decisions, even differing from the original plan
- Ensure that extra efforts are payed when necessary to minimize total costs and risks, so that every party is rewarded for cooperation.

5.4 *References*

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