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ABSTRACT: Several tunnels in the alpine region must be started in rockfall deposits before encountering solid rock. The most difficult condition that engineers must be aware of is when the inclination of the surface is close to the friction angle of the nearly cohesionless soil. In addition, tunnels often have to cross these zones at an unfavourable, acute angle.

This paper deals with two cases of major traffic tunnels under construction in central Switzerland. Both projects consist of twin tunnels for railway (Alptransit Gotthard) and motorway (main North-South highway through the Lopper Mountain near Lucerne) use, respectively.

It is shown how the results of structural and geotechnical analyses, including the effects of asymmetry, have to be integrated in a suited design and excavation method. Jet grouting and low-pressure grouting techniques, together with steel tube reinforcement in the roof, are used. Each tunnel is excavated in several phases. A minimum distance between the excavation faces of the two neighbouring tubes must be respected. Any section of the more advanced tunnel must be completely supported before the arrival of the face of the neighbouring tunnel.

The monitoring systems consist of surface geodetic measures, vertical inclinometers in the slope, horizontal inclino-extensometer measurements from inside the tunnel at each excavation step and convergence measurements.

Displacements monitored during construction give a good understanding of the real behaviour and show how they match with the displacements predicted by numerical models.

1 INTRODUCTION

The present article shows the calculation and design procedure of a tunnel in rockfall deposits, taking into account the presence of blocks and an extreme slope angle. The results will be discussed and compared in the light of two concrete examples.

2 ANALYSIS

Proof of sufficient bearing capacity for all construction phases, as well as the calculation of the determinant deformations, must be carried out. The following important points will be treated:

- Geometry
- Geological model of the site and soil parameters
- Safety definition
- Danger scenarios
- Modelling
- Results.

2.1 Geometry

Both of the tunnel tubes to be built are located close together and can influence each other. The influence is strongest when both tunnel tubes are built at the same time. The influence of one tunnel tube on the other should be checked in the case of staggered drilling over time.

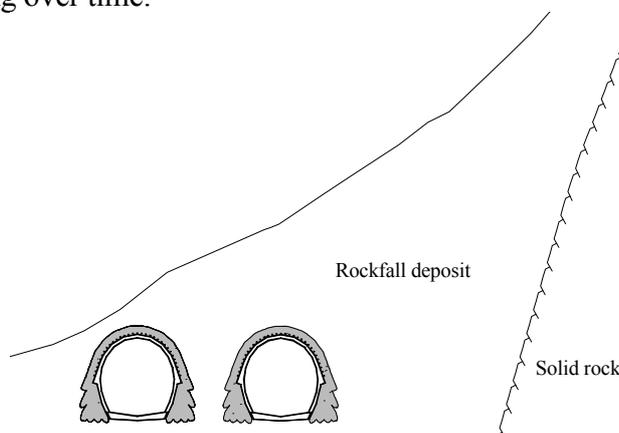


Figure 1: Tunnel cross-section (example Case 1).

Both tunnels lie in an area of steep slope (slope angle α) at the bottom of a vertical rock face. Deformation sensitive housing structures, which must be protected under every circumstance, are located downhill from the tunnels.

2.2 Geological model of the site and soil parameters

Rockfall material is normally formed of layers of sand and gravel, which have varying percentages of blocks and voids. The variation is very large due to the history of deposit. The density is also highly variable. The slope of the layers (deposition angle) roughly corresponds to their natural internal friction angles. The layers are partially indented in each other.

Boreholes were carried out in the areas of the two objects at hand. The borehole cores were investigated and soil classification was carried out.

The results of the soil classification ranged from silty sand (SP), through gravel (GM, GP and GW), to frequent hard blocks without fill material in the voids. The percentage of blocks varied between 40 and 60% of the total volume. The largest blocks reached side lengths up to approximately 8 m. Empty spaces of several cubic meters had to be dealt with and the construction ground was above the groundwater table.

No laboratory or in situ tests were carried out on the rocks in the loose soil section, in order to obtain geotechnical parameters, because the ground formed by loose blocks and rocks is generally unsuitable for laboratory tests. This means that for all calculations, empirical geotechnical parameters had to be estimated, based on grain size, angularity and density. The soil parameters furnished by the geologist required further interpretation before being used in modelling for geo-mechanical analysis.

On the basis of the history of deposit and the lack of clayey components, a zero long-term cohesion ($c' = 0$) was assumed. A short-term cohesion value should be taken into account because of the layer indentation. Evidently, the friction angle φ must be larger than the slope angle α .

Locally, the definition of the soil parameters φ and c' are only partially meaningful, since the individual blocks are sometimes as large as the tunnel structure itself.

The M_E value given by the geologist is a first loading modulus and is by definition independent of overburden. The tunnel has a variable overburden from 4 m to 50 m, therefore, an increase in M_E value as a function of depth would be more plausible. Since the in place soil is not suitable for testing to determine the M_E value, due to its grain size, a prudent value was used for the calculations. This assumption leads to an overestimation of deformation.

2.3 Safety definition

The analyses were divided into geotechnical verifications (base failure, tunnel face stability, etc.) and verifications of structural elements (verifications of girders, shotcrete, etc.). This division leads to the application of different safety definitions.

The verification concept according to Swiss Standards (SIA) is based on the following condition

$$S_d \leq \frac{R}{\gamma_R} \quad (1)$$

In addition, safety is interpreted as a working load factor. Thus, the safety definition reads as follows:

$$F = \frac{R/\gamma_R}{S_d} \quad (2)$$

2.4 Danger scenarios

The following geotechnical danger scenarios were differentiated and investigated, based on a risk analysis.

- Slope instability
- Outflow of sand and gravel lenses
- Breakdown of blocks
- Stability of the tunnel face and benches
- Settlement and base failure under foundations (crown or floor).

The danger scenarios were adapted for each construction phase.

The most crucial danger scenario is the instability of all or part of the slope.

Deformations and the resulting changes in stress fields occur due to the work in the ground. As the ground itself has no bearing reserve, every change in stress leads to a failure state in the soil. As soon as a bearing ring is completed which can take up the primary stresses in the ground, a new equilibrium state is attained.

The danger scenarios during the tunnel driving were investigated using classical methods of soil mechanics as well as single analyses involving the consideration of individual blocks.

In the following, danger to the construction works and their surroundings due to slope instability only will be explained.

2.5 Modelling

The calculations were carried out using a finite difference model.

The stratification parallel to the slope could be ignored. The presence of blocks was modelled by an adjustment of the soil parameters (increase of φ' from 2° to 3°).

The elastic modulus was estimated using a reload modulus M_E' . The soil material increases in volume in the state of failure.

The tunnel cross-section was simplified as a circle and the construction phases were modelled consecutively.

During the determination of the primary stress state the following soil parameters were used: $\varphi' = \alpha$ and $c' = 0$. With these soil parameters, a limit equilibrium state exists before the beginning of the tunnel works. This means that each, even minuscule, variation in stress in the slope leads to immediate instability (safety factor = 1.00).

In a second phase, the friction angle φ' was increased by 2° to 3° . This increase of the friction angle led to a calculated safety factor for the slope of approximately 1.1.

2.6 Results

The calculations led to the following determinant failure mechanism:

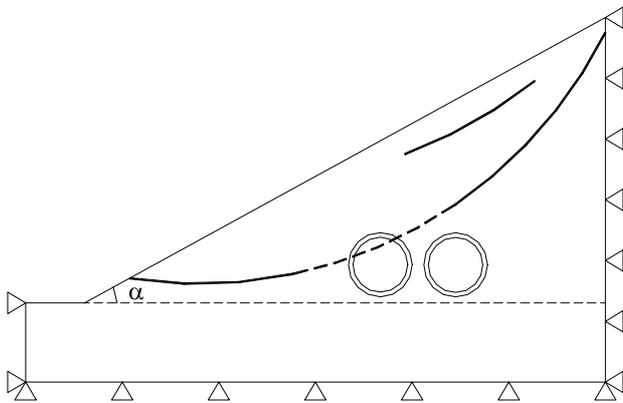


Figure 2: Static model and failure mechanism.

The transposition of the results with definition of the tunnel support is explained through two cases currently under construction:

- Case 1: Gotthard Base Tunnel South – Tunnelling through the rockfall sector.
- Case 2: Kirchenwald Tunnel National Highway A2- Hergiswil loose soil sector.

3 CASE 1: GOTTHARD BASE TUNNEL SOUTH

3.1 Project description

The Gotthard Base Tunnel is a double tube high speed railway tunnel approximately 57 km in length. The loose soil sector is located near the southern portal of the tunnel, between the open cut and the main part in solid. The underground tunnelling through the rockfall deposit represents the most challenging task of the Bodio sector.

The distance between the tube axes, along the 420 m long sector, varies between 16 m and 27 m. At a distance of approximately 195 m from the Eastern Portal, a cross passage is planned as an escape route.



Figure 3: View of the rockfall near the South Portal of the Gotthard Base Tunnel.

The minimum overburden on both tubes is approximately 4 m in the portal zone, after which it increases steadily to a value of 26 m and then decreases to about 14 m at the lot limit where solid rock begins. The total excavation area of each tube is about 90 m^2 .

The cross profile contains a double lining with complete waterproofing.

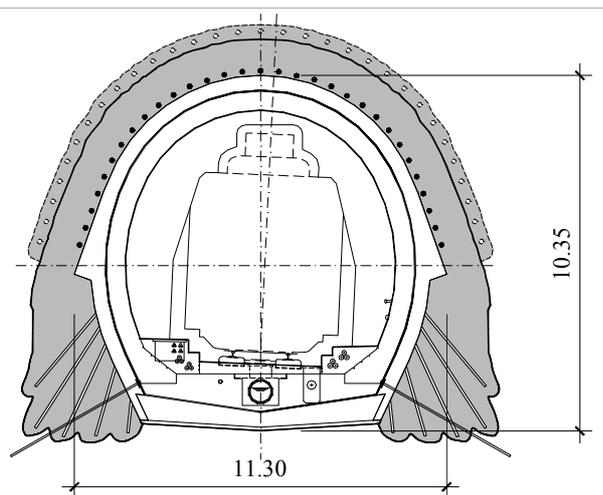


Figure 4: Final cross-section.

The total construction costs amount to approximately Euro 46'000'000.-, which corresponds to about Euro 55'500.- per meter of stabilised tunnel (not including the complete waterproofing, cast-in-place concrete inner lining and railway equipment, which will be installed after completion of the other tunnel lots).

3.2 Calculation

The slope angle over most of its area is 33° , with local values of 37° to 40° . An average friction angle value of 36° was assumed for the global model, considering the entire slope, with both tubes. Cohesion was assumed to be zero.

3.3 Excavation and safety concept

The excavation of both tubes was carried out first at the crown, followed by continuous bench excavation, under the protection of steel tube forepoling and grouting ahead. Excavating stages of 1 m at the crown and 4 m at the bench were planned. The excavation was carried out mechanically using a hydraulic excavator equipped with a drilling hammer. Large blocks of gneiss were split or blasted.

The Western tube has a minimum advance of 80 m. The excavation of the crown, followed by the bench at a maximum distance of 52 m, and the placement of complete arch support guarantees a statically sufficient ring closure at the moment of the excavation of the subsequent Eastern tube.

A 15 m long steel tube screen, with 140 mm diameter tubes, was installed for advance support. The overlap of the individual stages was 3 m. In addition, cement grouting from the tunnel face was carried out in the face section as well as up to 1 m outside of it. The resulting grouted mass distributed the load of the individual blocks to the arch.

Every meter, a girder arch was placed and sprayed with a total of 40 cm of fibre reinforced wet shotcrete for crown support.

Approximately 36 m behind the excavation face, the foundation zone of the girders was reinforced with vertical grouting. The bench was excavated and supported in a similar way at a further distance of 16 m.

Finally, an adjustment layer of lean-mixed concrete, followed by the reinforced cast-in-place concrete floor, were placed in order to form a statically sufficient ring with the girders. In the free space resulting from the inclined placement of the steel tube screen, an additional 3.5 m long reinforced arch support ring was placed.

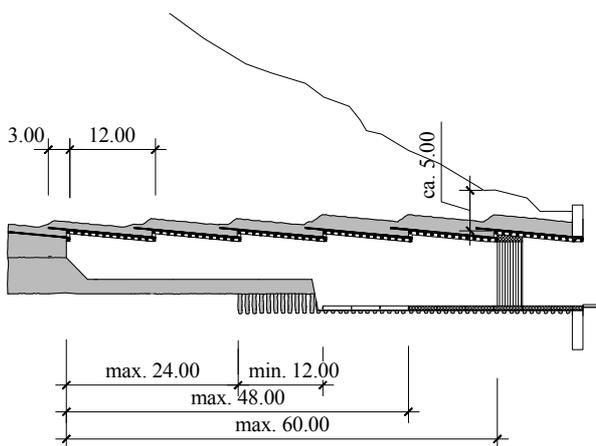


Figure 5: Excavation concept in the portal zone – Western tube.

3.4 Works supervision

During the construction works, various deformation measurement systems were placed:

- Absolute 3D surface measurements in the portal and cut-through zones.

- Vertical inclinometer measurements from the ground surface in the portal zone.
 - Absolute 3D measurements in the tunnel during driving of the crown and bench (measurement cross-section made up of 7 points every 6 m).
 - 3D measurements in a borehole above the forepoling screen, at least 6 m in front of the tunnel face.
- The actual safety state of the tunnel was evaluated and the safety investigation hypotheses were verified on the basis of the interpretation of the measurement results in conjunction with visual safety checks.

The results were used to identify possible oncoming danger and as a decision-making aid for the application of appropriate additional safety measures or modifications during construction.

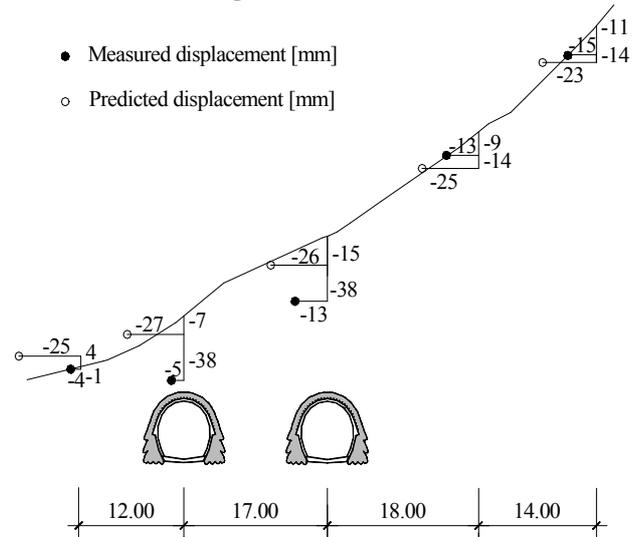


Figure 6: Surface displacements near the portal.

The measured deformations, especially in the horizontal direction, are generally less than those found through calculations. The greatest settlements, between the tubes, were mostly caused by blasting.

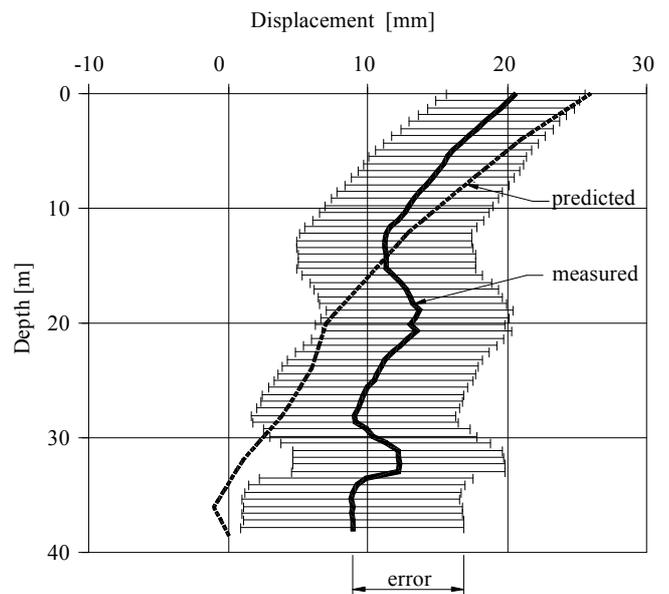


Figure 7: Diagram of inclinometer measurements near the portal.

The measured displacements correspond quite well to those obtained by calculations. The inclinometer base, located about 3 m under the floor and 4 m uphill from the Eastern tube, is locally influenced by drilling operations.

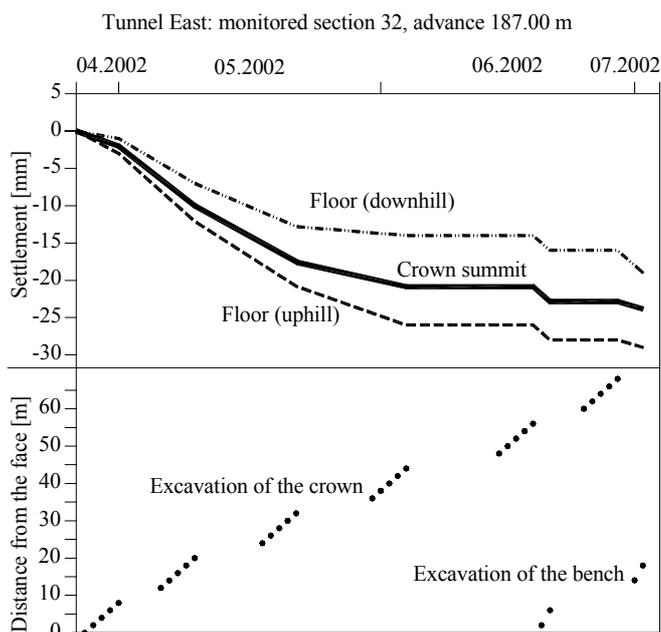


Figure 8: Diagram of measurements in the tunnel.

The measured settlements are practically in the range of the calculated values.

The extenso-inclinometer for the 3D deformation measurements in the tunnel in front of the excavation face has proven to be a very important instrument for the qualitative evaluation of tunnel face behaviour, especially at the end of each excavation phase.

3.5 Execution of the works

After a difficult initial phase, the works could be executed respecting costs and deadlines with the available installations and the proposed project optimisation. For two thirds of the total excavation volume, the average advancement cadence amounted to 0.8 m/day per tube (full cross-section) with three daily shifts. The actual advancement cadence amounts to 1 m/day per tube.

In the portal zone, large voids and caverns, some with volumes greater than 100 m³ with a corresponding increase in grout quantities, were encountered.

During the excavation of the crown, large blocks, sometimes as large as the entire excavation section and which had to be blasted, were encountered several times.

Thanks to the special software for the real time interpretation of the grouting and probing parameters (turning moment, pressure and especially advancing speed). Any immediate, crucial and useful information concerning the ground to be excavated (void space, matrix and rock contents for each boring) in front of the crown excavation was available. The results were semi-automatically graphically represented

and permitted rapid decisions concerning the material to be grouted, foreseeable problems during excavation and the justification of dimensions for the project bookkeeping.

As a result of the interpretation of the measurement results and based on the control over the geotechnical behaviour for both tubes the following optimisations were possible:

- The advance of the Western tube was reduced from the original 80 m to 66 m.
- The excavating steps for the crown excavation were increased from 1 m to 2 m.
- The bench excavation was carried out in three 4 m stages beginning at 60 m behind the tunnel face. After that, the cast-in-place concrete floor was placed in 12 m stages and the 3.5 m long arch support ring was installed systematically.

4 CASE 2: KIRCHENWALD TUNNEL

4.1 Project description

The Kirchenwald Tunnel is part of the national highway A2, the most important North-South route (Gotthard) in Switzerland. It is located in Central Switzerland, near Lucerne. The Kirchenwald Tunnel is composed of two tunnel tubes, each approximately 1.6 km long. The most important auxiliary structures are an underground branching, a ventilation station and a 125 m high ventilation shaft. The tunnel passes through a rock stretch in flinty limestone of approximately 1500 m and a loose soil stretch in rockfall material of 140 to 180 m.

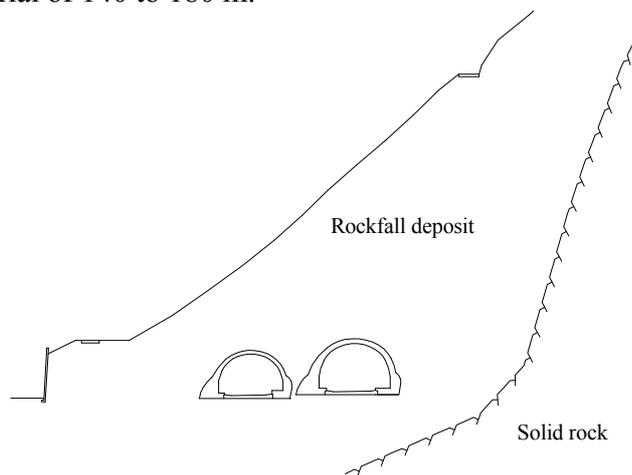


Figure 9: Tunnel cross-section.

The 187 m and 146 m long double tube stretch, is located in a transition zone between a straight line and a curve of 600 m radius. The distance between the tube axes varies between 1 m and 28 m. The minimum overburden on both tubes amounts to 4m at the portal and increases steadily to about 45 m.

The cross section is composed of a double lining with partial waterproofing. Each tube has an excavation section of 115 m² to 125 m² in the soil.

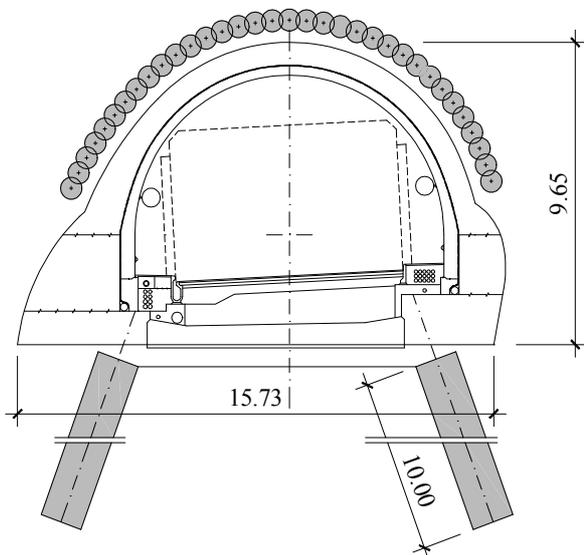


Figure 10: Final cross-section.

The costs for excavation and support amount to approximately Euro 23'000'000.-. This corresponds to about Euro 67'500.- per meter of tunnel (excluding partial waterproofing, cast-in-place inner lining and internal finishing which will be carried out after the excavation of the other construction lots).

4.2 Excavation and safety concept

Because of the lack of bedding of the arch on the downhill side, the bearing structure must be designed for the full surcharge of the soil material.

The excavation section is asymmetrically inclined. It corresponds to an ellipse inclined in the direction of the principal stresses. With this shape, a large part of the forces can be taken up as normal forces. On the uphill side, the taking up of load from the abutments to the ground is eccentric, resulting in the need for very strong reinforcing.

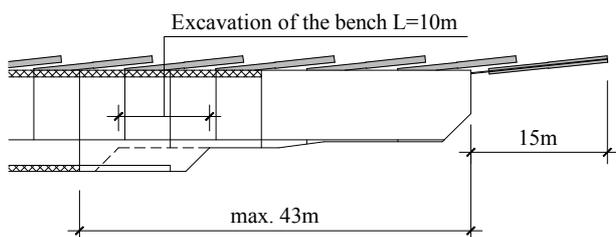


Figure 11: Crown excavation concept.

The excavation cross-section of approximately 120 m² was divided according to the German construction method. The lateral tunnels had a cross-section of about 25 m² each and were executed under the protection of a low pressure grout screen and supported with steel ribs and shotcrete. The abutment in the floor tunnel is about 3.5 m wide.

Finally, the crown was excavated under the protection of a jetted arch. 23 m behind the tunnel face, an 80 cm thick concrete arch was constructed. After a further 25 m, the floor plates were concreted in stages

of 10 m. Floor closure was therefore obtained after 48 m from the face.

The lateral adits of the uphill tunnel tube had a minimum distance of 20 m between their excavation face and the floor closure of the downhill tunnel tube. After that, the crown and the bench of the uphill tube were finished in the same way as those of the downhill tube.

Before the excavation of the crown of the second tunnel tube, the ground under the abutments of the first tube was consolidated in order to limit settlements.

4.3 Works supervision

During the construction works, various deformation measuring systems were placed:

- Absolute 3D measurements at the surface and in the tunnel.
- Inclinator measurements at the surface.
- Stress measurements in the truss of the outer lining.

The vertical displacement component of one representative point is shown in the diagram.

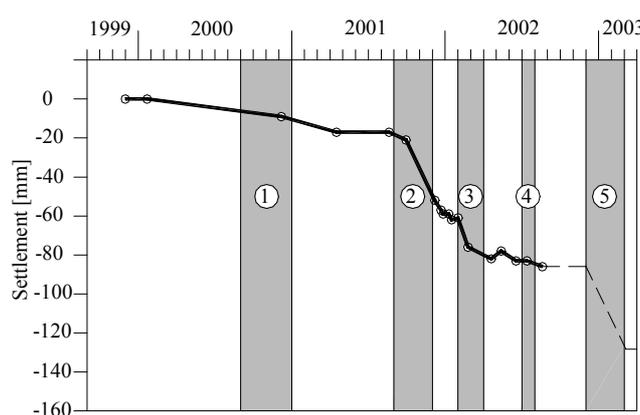


Figure 12: Surface settlements of one measurement point.

Legend:

- ① Excavation of sidewall tunnel 1
- ②+③ Excavation of crown/bench tunnel 1 + sidewall tunnel 2
- ④ Strengthening foundation tunnel 1
- ⑤ Excavation crown/bench tunnel 2

The surface displacements show good correspondence with the singular construction stages. As soon as construction was carried out in the zone where the measurement points were located, surface settlements were observed after a time-lag of approximately 2 weeks. The time-lag is especially evident for all interruptions in construction (e.g. Christmas 2001).

The second diagram shows the development of settlement over time for 3 points in a representative cross-section in the tunnel.

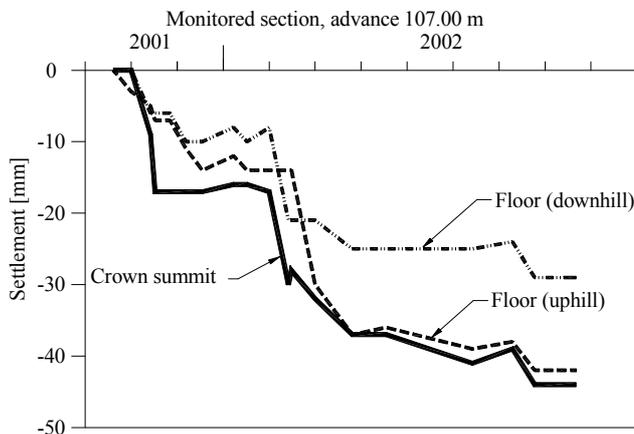


Figure 13: Diagram of measurements in the tunnel.

Before the excavation of the crown, no displacement of the abutment was observed. After the excavation of the crown, up to the placement of the stiff concrete arch, approximately 17 mm of settlement at the roof were measured. After the placement of the arch, the arch diverted the loads to the abutments, which in turn settled approximately 5 mm. The excavation section stabilised itself after the construction of the floor.

As soon as the floor adit of the second tube was excavated (distance between tunnels about 6 m), however, further settlements of 15 mm occurred, which slowly came to a stop. The excavation of the crown has not been completed yet. It is interesting to note that the cross-section tipped downhill in the first phase, and then uphill in the second phase.

Until the excavation of the crown of the second tube, up to 45 mm of roof settlement had occurred.

4.4 Execution of the works

The lateral tunnels of the first tunnel tube were excavated with a cadence of approximately 0.8 m/day with three daily shifts. During the excavation of the lateral tunnels, only about 20% of the predicted blocks were encountered. Based on this experience, the low pressure grouting pre-support was replaced by a jet grouting solution for the excavation of the crown. This change resulted in a definite decrease in over break. The construction progress at the crown was about 1.10 m/day.

The cross-section of the uphill tunnel tube had to be modified with the addition of an intermediate ceiling, creating a duct for smoke extraction. For the excavation, this meant an increase in cross-section of about 13 m². This modification also required a new design of the excavation support. Based on the settlement of the downhill tunnel tube, ground consolidation using a jet grouting procedure was carried out underneath the abutments.

5 DIFFERENCES/SIMILARITIES - COMPARISON OF THE PROJECTS

In spite of the very similar calculation models, the projects had two different construction methods and cross-sections. The main reasons for this are the following.

5.1 Construction ground

The initially very similarly interpreted construction ground behaviour turned out to be very different in reality.

The construction ground of Case 1 (Gotthard Base Tunnel) was highly inhomogeneous with sections of stacked blocks with voids as well as zones with fine-grained material. In some sections right under a stream, important water inflow was observed. The portal zones with low overburden were the most critical.

The construction ground of Case 2 (Kirchenwald Tunnel) was layered and very homogeneous with blocks located in fine-grained material without large voids. Water inflow occurred especially during precipitation, was otherwise rather modest with only damp places observed. The zone with high overburden was the most critical.

5.2 Excavation and safety concept

The wide shape of the cross section in Case 2 compared to that of Case 1, and the limited distance in the portal zone, meant that the loads on the abutments were very high. For this reason, the German construction method was chosen. The smaller cross-section in Case 1 can be driven with the crown excavation method.

The longitudinal distances were chosen as a function of space constraints for the excavation equipment (drilling machine for the installation of the steel tube screen and the vertical grouting and jetting apparatus). The distance between the completed section of the first tunnel tube and the excavation of the second was chosen to be about 20 m. The distances resulted from the hardening behaviour of the concrete.

The consolidation of the construction ground was modified in the execution phase. In Case 1, pure cement was replaced by mortar grouting, due to the greater amount of void spaces. The grouting was modified in each stage, based on the geological conditions encountered. For Case 2, low pressure grouting was replaced by jet grouting due to the presence of less blocks. Ground consolidation was carried out systematically.

6 CONCLUSIONS

Both tunnels are still in construction and the main part of the work has been successfully carried out. Both objects have been carried out in accordance with planned deadlines and costs. The planned excavation methods could be carried out with adjustments to the construction ground consolidation but without other changes. It was seen that the monitoring of the construction progress and especially the interpretation of the results are prerequisites for a technically competent supervision of the work site. The transposition of the obtained results can only bring about success in an atmosphere of collaboration among the client, the contractor, the design engineer and the works supervision and with the necessary flexibility of all of the parties.