Uetliberg Tunnel, evaluation of the soft ground heading

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ABSTRACT: The Uetliberg Tunnel is being built as part of Zurich's new West Bypass project. The road tunnel has two tubes, each 4.4 km long. The tunnel geology comprises two molasse sections, three cut-and-cover stretches, and three soft ground sections. Geological investigations showed that the soft ground sections lie in heterogeneous moraines of varying characteristics. Numerical calculations with a continuum model demonstrated that auxiliary measures required to achieve cavity stability in the soft ground sections. Various auxiliary measures were then reviewed for feasibility by means of parameter analyses. Ultimately three construction methods with appropriate auxiliary measures proved to be feasible. A risk analysis then pointed to the core construction method with steel support and shotcrete as being the least risky, so it was chosen for driving the soft ground sections.

1 INTRODUCTION

As the heart of Zurich's West Bypass, the Uetliberg Tunnel connects the Birmensdorf ring road (N20.1.4) in the west with the existing Zurich-Chur motorway (A3) in the east.

The tunnel consists of two parallel tubes (Chur tube and Basel tube), each about 4.4 km long. The two are connected every 300 m with man-size cross-cuts and every 900 m with cross-passages dimensioned for vehicles. The SOS niches are spaced 150 m apart. Portal stations with machinery rooms are located at the west and east portals. Starting from the east portal, the tunnel rises with a gradient of 1.6%.

From west to east, the Uetliberg Tunnel passes under the Ettenberg and Uetliberg hills. The Reppisch Valley lies between them. Later on, an underground ventilation facility will be built in the construction pit now used for the intermediate driving point in the Reppisch Valley. In normal operation the tunnel is ventilated by natural longitudinal air movement (piston effect of the moving traffic) in both tubes. With the underground ventilation facility, however, air can also be extracted from the tubes. The air passes through a distribution network in the tunnel's intermediate ceiling to a separate extraction gallery and then to the Eichholz exhaust air shaft, where it is blown out-of-doors.

The Uetliberg Tunnel traverses two molasse sections, three cut-and-cover sections, and the three soft ground sections of Gjuch, Diebis and Juchegg (see Fig. 1).

All of the soft ground sections and the Eichholz molasse section exhibit a standard horseshoe cross-section about 14.70 m wide, about 12.70 m high, and with a heading area between 143 and 148 m^2 (see Fig. 2). The standard cross-section for the Uetliberg molasse section is about 14.40 m wide, about 14.20 m high, and has a heading area of about 160 m^2.

2 GEOLOGY AND HAZARDS

2.1 *Geology and hydrology in the soft ground sections*

The Gjuch soft ground section, which is about 210 m long, passes through a very heterogeneous end moraine complex consisting of loamy, sandy gravel. The water table rises eastward from the middle of the tunnel cross-section to a level above the tunnel invert.

The Diebis section is about 240 m long. It lies in a ground moraine covered by washed off slope debris. The ground moraine is expected to contain the full range of particle sizes, all the way to stones and even boulders. At the beginning of the soft ground section about half of the tunnel cross-section lies in the slope debris, which then rises to the east. After about 50 m, the entire cross-section is engulfed by the moraine. The whole tunnel profile lies under the water table in the Diebis section.

The Juchegg soft ground section lies in a ground moraine that starts as gravelly sand and then turns into sandy clay. The ground moraine is topped by Uetliberg clay, which reaches about halfway up the tunnel cross-section at the Gänziloo (east) portal. After about 70 m the entire cross-section lies in the moraine. At the beginning the water table lies below the tunnel cross-section, but it starts rising at the transition from gravelly sand to sandy clay. At the transition point from soft ground to molasse, the entire tunnel cross-section lies in the ground water.

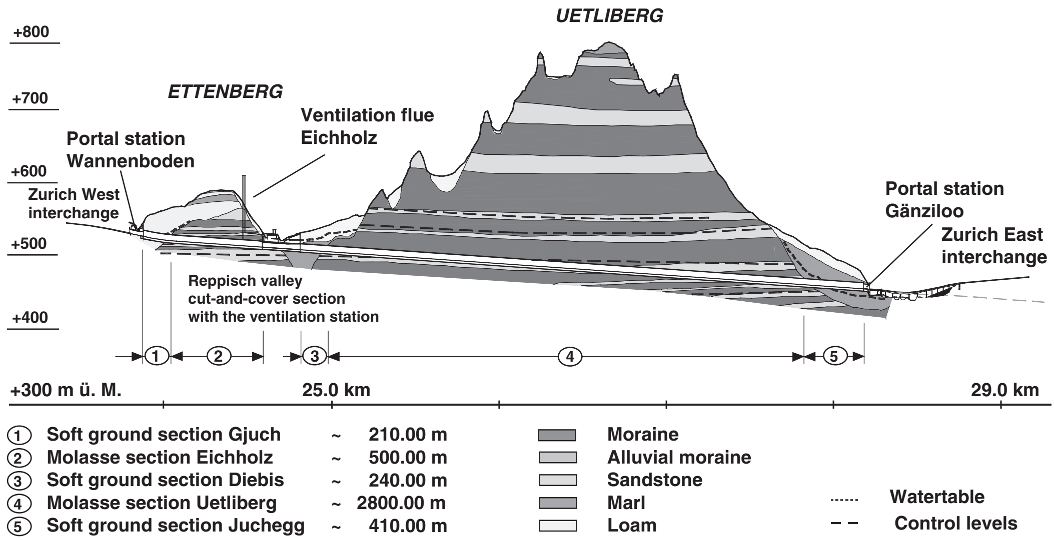

Figure 1. Longitudinal profile.

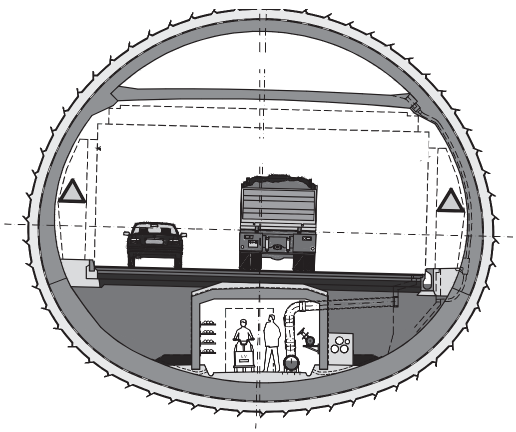

Figure 2. Standard horseshoe cross-section.

2.2 *Dangers in the soft ground sections*

The main sources of danger in the soft ground sections are:

– Extreme deformations resulting from widespread propagation of loads exceeding strength limits (plastification) in the subsoil during construction of the tunnel.
– Loosening of the rock in the roof zone as a result of roof settling. This puts heavy loads on the excavation supports and the crown lining.
– Soil settling that reaches all the way to the surface, thus affecting roadways and open areas.
– Damage to piping systems (water pipes, sewers, gas pipes, etc.) as a result of subsoil settling.
– Heavy water penetration into the tunnel with resultant washing out of the soil around the cavity.
– Collapse of the cavity and cave-ins resulting from insufficient stabilization of the soil around the cavity and in front of the face, or from insufficient strength of the excavation support.

3 MODEL CALCULATIONS WITHOUT AUXILIARY MEASURES

3.1 *Summary of the calculations*

To start with, calculations were undertaken for all of the soft ground sections without the use of auxiliary measures. Neither stabilizing nor dewatering measures were assumed. A continuum model was used for the calculations. The program used is FLAC (Fast Langrangian Analysis for Continua), which computes with the method of finite differences.

The investigation perimeter of the Diebis soft ground section was as follows:

1. Cross-section in the moraine with 20 m of overburden and a water table lying 8 m above the roof.
2. Special investigation for one tunnel tube with a span of 19 m (Basel tube near the portal).
3. Mutual effects of the tunnel tubes on each other.

3.2 *Results of the calculations*

Figure 3 shows that, in the absence of auxiliary measures, very serious deformations would occur around the crown legs and benches. Calculations indicated

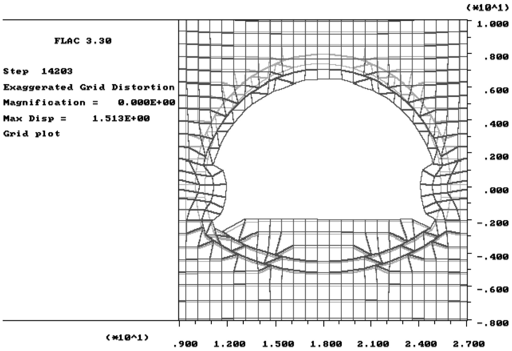

Figure 3. Large Deformations of crown base.

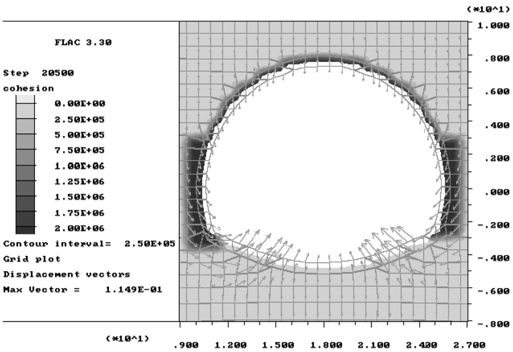

Figure 4. Invert heaves.

that deformations as great as 1.5 m would occur in the Diebis soft ground section, for example.

As indicated by Figure 4, stabilizing measures also have to be taken around the base, or else immediate closure of the invert arch is required even if the benches are stabilized. Bench stabilization merely shifts the deformation problem downward.

Under the assumptions made, the cavity would be unstable in all cases if auxiliary measures were not employed.

4 INVESTIGATION OF POSSIBLE AUXILIARY MEASURES

A preselection of possible auxiliary measures was undertaken. These are listed in Table 1 with an assessment of the fundamental feasibility of each measure.

4.1 *Lowering of the water table*

For the Diebis soft ground section, a very thorough study was carried out to determine whether and how the water table could be lowered.

Basically, drainage measures can be employed either from the surface or from the heading face. The possibility of using drainage measures starting from the surface was excluded for a number of reasons: higher cost, poorer flexibility, and impairment of the already cramped space conditions at the Landikon construction pit (excavation material, piping, construction offices, roadways, etc.). So the investigation of possible drainage measures was restricted to those starting from the heading face.

Essentially, two possibilities exist. The first involves the use of partial cross-sections (such as a side wall gallery) for drilling the drainage holes. One alternative is to use one or more side wall galleries, or a separate drainage gallery within the tunnel profile, for drainage. The second alternative is to drive the drainage gallery outside the tunnel profile. This gallery is then used exclusively for drainage.

The investigations led to the conclusion that the most economical solution is to use one side wall gallery for drainage in each tube.

It was found that systematic lowering of the water table (e.g. by means of vacuum lances) would be relatively inefficient because of the low permeabilities ($k = 10^{-6}$ to $k = 10^{-7}$ m/s). For this reason, the idea of using a systematic drainage arrangement was dropped. The use of local drilling around the heading operation was deemed to be more efficient, more economical and more flexible.

4.2 *Injections*

The feasibility of injections into the moraines of the three soft ground sections was evaluated based on the screening curves and the percentage of USCS (Unified Soil Classification System; Classification-system in Switzerland) fractions in the different moraines. For the sake of example, let us review the injectability of the Diebis section's moraine with various injection agents: If plastics were used for injection, 9.3% of the entire moraine could be injected. The figure for water glass would be 17.5%, for fine cement 7.2% and for cement 27.8%. Theoretically all of these injection agents would have to be used to cover 61% of the moraine. Even then, 39% would remain uninjectable.

Injections are basically unsuitable as a general support measure for broad, large-area soil consolidation.

4.3 *Jet grouting, pipe screen and freezing*

On the face of it, jet grouting was regarded as a possibility for use in the moraines.

A pipe screen, too, would be applicable in the moraines.

The "freezing" auxiliary measure can be used in the Diebis soft ground section, both in the slope debris and the moraine.

Table 1. Feasibility of auxiliary measures.

Relevant Ground Conditions			Feasibility of Auxiliary Measures				
Moraine	Tunnel under water table	Compactness	Lowering water table	Injection	Jet-Grouting	Freezing	Pipe screen
Juchegg							
Uetliberg clay	no	low	impossible	locally possible	possible		possible
Moraine, gravelly	no	compact	impossible		possible		possible
Moraine, loamy	possible	compact	locally possible		possible		possible
Diebis							
Slope debris	no	low	impossible	locally possible	possible	possible	possible
Moraine	yes	compact	locally possible		possible	possible	possible
Gjuch							
Moraine, washed off	yes	heterogeneous	locally possible	locally possible	possible		possible
Moraine	yes	heterogeneous	locally possible	locally possible	possible		possible

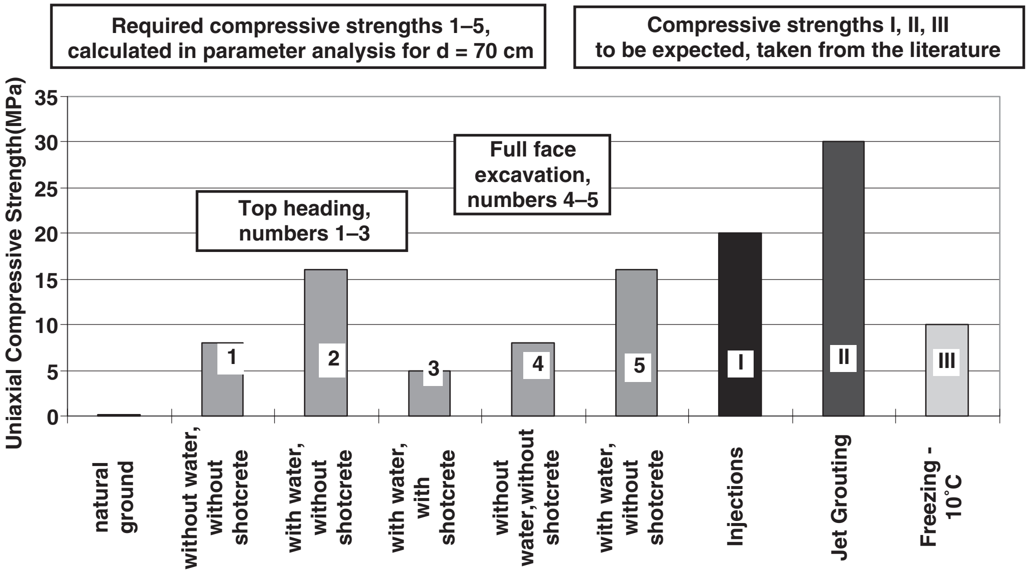

Figure 5. Results of the parameter analysis.

5 MODEL CALCULATIONS WITH AUXILIARY MEASURES

The different auxiliary measures were incorporated in the calculations as follows: For "jet grouting", it was assumed that a 70 cm thick layer of soil would be consolidated around the cavity periphery. For the model calculation with "freezing" in the Diebis soft ground section, a 2 m thick frozen ring was assumed. Most of the investigations concentrated on the condition obtaining in "crown excavation". They were based on a large-span cross-sectional geometry that represented the worst-case scenario with regard to deformations and the stressing of the auxiliary measures used.

A spectrum of the in-depth analysis is reproduced in Figure 5. The diagram summarizes the results of the parameter analyses carried out for a 70 cm thick consolidated ring (e.g. a jet-grouted vault).

The figure shows the minimum single-axis compressive strength required in the consolidated ring around the cavity according to the calculation results for five possible cases (numbers 1 to 5). Also shown are the single-axis compressive strengths to be expected (i.e., taken from the literature) for the application of (I) "injections", (II) "jetting" and (III) "freezing". The compressive strengths were determined with consolidated samples whose properties are similar to those of the natural soil of the Diebis moraine.

The following conclusions were drawn on the basis of the model calculations:

- The zones with exceeded strength limits as a result of reduced strength in the water-saturated soil propagate very widely and can even reach the surface.
- Special attention should be paid to the possibility of a cave-in during the construction phase.
- The relatively low strength of frozen soil necessitates the use of exceptionally strong excavation supports in order to reduce the extent of settling.
- After the frozen soil has thawed, settling of the surface soil is likely because of changing soil strength in the area around the body of frozen soil.
- Following thawing, the inside of the vault has to be reinforced to withstand the resultant heavy loading. If "jetting" or another support method is used instead of "freezing", it must afford load-bearing properties similar to those of a frozen body of soil.

6 POSSIBLE CONSTRUCTION METHODS, CALCULATION RESULTS

6.1 *Possible construction methods*

Now let us summarize the construction methods and the related auxiliary measures are regarded as realistic following this detailed study:

- Crown excavation with jet grouting and shotcrete: Regardless of the choice of auxiliary entire crown is excavated first, followed by excavation of the benches. The bench excavation can follow immediately after the crown excavation, or it can be done only after the crown has been cut through. A further division into bench and base is possible.
- Top heading by means of cutting shield and shotcrete: The driving of the side wall galleries is done the same as with the core method, including installation of the necessary supports. To withstand the heavy loads resulting from the crown excavation, jetting piles have to be driven under the side wall galleries for abutment reinforcement. Under the cutting shield's protection, the crown can be driven across the full width.
- Core method with steel supporting arches and shotcrete (Fig. 6): Excavation by means of the core method determines the order of driving, namely the left and right side wall galleries (top and bottom), crown, core and base. The spacing of the individual stages in relation to one another can be varied substantially depending mainly on the stability of the soil.

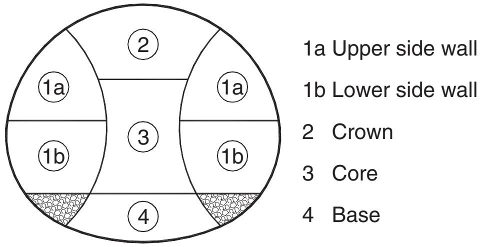

Figure 6. Core construction method.

One aspect that is common to all of these methods is the assumption of a condition with low pore water pressure. The local drainage measures employed consist of drainage and pressure relief drilling.

6.2 *Results of the calculations*

The following lists some of the calculation results for the different construction conditions.

6.2.1 *Results for "top heading with jet grouting and shotcrete"*

The jet-grouted vault with an average thickness of 70 cm creates very high stress concentrations in the crown legs. Without the formation of a suitable abutment, this can result in foundation failure. For bench excavation, a side wall with an average thickness of 1.2 m and a base support 2.2 m wide are required, as is a base invert with a thickness of 25 cm. With supports of these dimensions, calculated deformations on the order of 1 cm to 2 cm can be expected.

6.2.2 *Results for "crown excavation with cutting shield and shotcrete"*

The reinforced in-situ-concrete vault required for excavating the crown is 30 cm thick in the roof and 55 cm thick at the supports.

6.2.3 *Results for "core method with steel supporting arches and shotcrete"*

The construction state that governs the dimensioning of the inner shotcrete walls is that following the driving of the side wall galleries; these inner walls are 25 cm thick. The outer walls, which have to withstand the heaviest loads following excavation of the crown and demolition of the inner walls, require a thickness of 30 cm. Regardless of these dimensions, heavy steel arches have to be installed for immediate support.

Table 2. Results of the risk evaluation.

	Top heading with jet-grouting	Core method with shotcrete	Top Heading with cutting shield and shotcrete
Stress redistribution	1	3	2
Ingress of water	3	1	1
Face stability	3	1	1
Ground loosening	3	1	1
Strength of shotcrete	1	3	1
Deformation behaviour	1	2	2
Support	1	3	2
Durability	1	3	1
Complexity	2	1	3
Disturbance sensitivity	2	1	3
Flexibility	2	1	3
Sum	**24**	**22**	**24**
Costs per day	**174%**	**100%**	**141%**

7 SELECTION OF THE CONSTRUCTION METHOD

The construction method was chosen following a risk assessment that included the following points.

– Stress redistributions in the soil
– Water ingress
– Stability of the face
– Ground loosening
– Feasibility of the auxiliary measures
– Soil pressure at the supports (arch footing)
– Shotcrete strength development
– Deformation behavior of individual elements
– Stressing of the excavation support
– Durability of the construction
– Complexity of the construction method
– Sensitivity to disturbances.

The evaluation was carried out by assigning relative points, whereby one point stands for a low risk, two points for a medium risk, and three points for a grave risk. The alternative with the lowest point score involves the least risk. Table 2 shows the results of the risk evaluation of the three construction methods considered.

As the table shows, the alternative "core method with shotcrete" possesses the lowest risk potential and is also the least costly method. This alternative was then bid on and executed.

8 EXPERIENCE WITH THE CORE METHOD

The core method was used successfully in the three soft ground sections. It was found that this excavation method's flexibility made it possible to drive even critical stretches with unstable soil conditions without any negative incidents.

The original core method concept was ultimately modified in a number of ways for the actual heading operations:

– Pipe screen for starting the tunnel heading out of the construction pit
– Roof support with lances where the rock proved to be crumbly
– Face anchoring for stabilization during heading of the side walls
– Drainage pipes in the heading face for preventive dewatering
– Partial excavation used for driving the upper side wall galleries in difficult soil.

9 SUMMARY

The numerical model simulations constituted an important basis for decision-making in selecting the excavation method to be employed in the Uetliberg Tunnel's soft ground sections. The simulations were combined with other considerations to arrive at the optimal construction method.