Numerical Simulation of Excavating the Overburden Above a Masonry Tunnel

By Bruno Mattle, Christof Weigl and Johannes Benedikt

The Tauern railway line between Schwarzach-St. Veit and Spittal/Millstättersee is part of one of the most important railway connections through Austria, running from the German border at Salzburg through Villach and Rosenbach to the Slovenian border and via Arnoldstein to the Italian border. At its core is the 8 550-m-long Tauern tunnel between Böckstein (Salzburg) and Mallnitz (Carinthia) which was constructed as a twin-track tunnel between 1901 to 1909 and which is still operated as a twin-track tunnel by the Austrian Federal Railways (ÖBB).

The objective of the extension of the route, which began in 1988, is the continuous twin-track upgrade of the original single track Tauern line in order to increase the capacity of this significant transit route and to adapt it to the demands of modern railway traffic.

In the framework of the upgrade project a line relocation of the southern terminal of the Böckstein railway station was initiated in spring of 2001, increasing the running speed at the station and at the northern portal of the Tauern tunnel from currently 60 to 100 km/h in the future. This is why the southern terminal of the station had to be replaced 15 m to the west over a length of 400 m. It will join the existing tunnel route approximately 180 m south of the existing northern portal, which means that this tunnel section had to be removed.

The tunnel section to be removed was constructed as a mined tunnel except for the portal and the first three segments which were built as cut and cover. The tunnel intersects the alluvial cone of the Höhkarbach river.

A decisive boundary condition for the construction works was keeping the railway line operating during the entire construction works except for the removal of the masonry tunnel lining for which the route would be closed down twice for approximately 4.5 days during two consecutive weeks.

During these route closures the tunnel including the invert arch had to be removed and a provisional single-track route was established along the old tunnel alignment, with various slope protection works and stabilisation of the lining at the future northern portal, as well as new installations in the area of the track. This meant that the overburden of the tunnel had to be removed as far as possible while keeping the line open to rail traffic. Furthermore the cut had to be prepared to the greatest possible extent to ensure that the works to be carried out during the closures could be completed.

Boundary conditions

Initial situation

In the course of the tender design for the removal work it had to be ascertained how much of the overburden could be removed without endangering the stability of the tunnel and without putting the operation of the railway at risk.

First structural investigations showed that the unsupported masonry lining was not stable without the surrounding ground. Previous similar projects provided different results. During mechanical removal in the Arlberg tunnel, the tunnel masonry proved to be very stiff and compact and therefore difficult to loosen. When assuming a similar behaviour of the masonry for the Tauern tunnel is seemed

Abtrag der Überlagerung über einem gemauerten Tunnel – numerische Simulation

Für den Ausbau der Tauern Strecke der Österreichischen Bundesbahnen (ÖBB) mußte ein etwa 180 m langer Abschnitt des Tauerntunnels abgebrochen werden. Eine der wesentlichsten Randbedingungen des Projekts war die Minimierung der Bauzeit, in der die Strecke gesperrt werden mußte. Es war daher notwendig, den größten Teil der Erdabtragsarbeiten unter Aufrechterhaltung des Eisenbahnbetriebs durchzuführen. Zur Bestimmung der zulässigen Abtragsgrenzen der Überlagerung, für die eine ausreichende Sicherheit für den Bahnbetrieb garantiert werden konnte, wurden numerische Berechnungen durchgeführt. Während der Baudurchführung wurden die Ergebnisse der Berechnungen laufend mit den Meßwerten verglichen, so daß eine Optimierung der Abtragsgrenzen durchgeführt werden konnte.

In order to upgrade the Tauern Line of the Austrian Federal Railways (ÖBB) a tunnel section of approximately 180 m of the one hundred year old Tauern Tunnel had to be removed. One of the most important boundary conditions for the project was the reduction of construction time during which the route had to be closed for rail traffic. Therefore it was necessary to carry out most of the excavation works of the overburden while operating the railway line. For this reason numerical calculations were carried out to determine the allowable depth of the open cut for which sufficient safety for railway operation could be guaranteed. In the course of the construction works, the results of the calculations were continuously compared with measurements so that the extension of the open cut could be optimized.

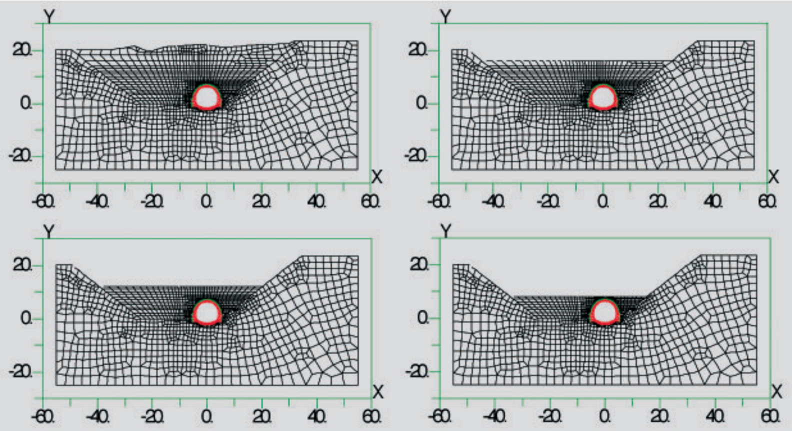

Fig. 1 Selected steps of the calculation sequence (dimensions in m).

Bild 1 *Ausgewählte Berechnungsschritte (Maße in m).*

possible to completely uncover the tunnel tube without endangering railway traffic.

However, experience from other old railway tunnels has shown that the tunnel masonry is subject to considerable damage in the course of time, e.g. due to the effect of seepage water. That was also the reason why several segments of the Tauern tunnel had to be replaced or reconstructed in the course of time. In the section to be removed grouting works had to be carried out repeatedly to seal the tunnel against seepage water.

In the course of the enlargement works at the "Patscher Tunnel" along the ÖBB's Brenner line the masonry segments proved to be mainly stable when stripping the lining. However, the unpredictable failure of a segment led to a collapse which required the route to be closed down for approximately one week.

Against the background of this experience, requirements and restrictions had to be developed for the removal works, based on calculational investigations, eliminating any endangerment of the operation of the railway and at the same time allowing the completion of all necessary works during the available route closures.

Geological conditions

During the excavation of the tunnel the geological conditions were described fairly detailed in a longitudinal profile. According to this description, the tunnel section following the northern portal is located in gravel, sand and angular gneiss blocks (1). In addition it was underlined that the tunnel in this section is subject to a high geostatic pressure which required the installation of a thick masonry tunnel lining.

Tunnel conditions

According to the common practice 100 years ago, timber framework had been installed immediately after excavation as initial support. Afterwards the final masonry lining consisting of granite – gneiss blocks was installed. The gap between the masonry and the surrounding ground was probably backfilled with excavated material. During the past decades several rehabilitation measures were carried out, as water ingress was observed at different locations (2, 3). These measures stopped the water ingress into the tunnel, but it had to be considered that the quality of the original lining worsened. Especially the mortar between the granite – gneiss blocks deteriorated.

Calculations

The open cut above the tunnel results in a change of stresses in the lining and the surrounding ground. Such a change of stresses may lead to a decrease of stability of the tunnel. Therefore calculations were carried out in order to determine the influence of the open cut on the behaviour of the tunnel. The aim of the calculations was to define certain limits within which the excavation of the ground above the tunnel could be allowed while operating the railway.

Table Parameter variations.

Tabelle *Parametervariationen.*

Calc.No.	Surrounding Ground						Lining		Backfilling	
	E MN/m²	ν –	c kN/m²	φ °	α –	k_0 -	E MN/m²	ν –	E MN/m²	ν –
1	75	0.3	0	40	0.4	0.45	6 500	0.25	50	0.35
2	50	0.3	0	40	0.4	0.45	6 500	0.25	50	0.35
3	100	0.3	0	40	0.4	0.45	6 500	0.25	50	0.35
4	75	0.25	0	40	0.4	0.45	6 500	0.25	50	0.35
5	75	0.35	0	40	0.4	0.45	6 500	0.25	50	0.35
6	75	0.3	0	35	0.4	0.45	6 500	0.25	50	0.35
7	75	0.3	0	42	0.4	0.45	6 500	0.25	50	0.35
8	75	0.3	0	40	0.3	0.45	6 500	0.25	50	0.35
9	75	0.3	0	40	0.5	0.45	6 500	0.25	50	0.35
10	75	0.3	0	40	0.4	0.35	6 500	0.25	50	0.35
11	75	0.3	0	40	0.4	0.55	6 500	0.25	50	0.35
12	75	0.3	0	40	0.4	0.45	5 000	0.25	50	0.35
13	75	0.3	0	40	0.4	0.45	8 000	0.25	50	0.35
14	75	0.3	0	40	0.4	0.45	6 500	0.2	50	0.35
15	75	0.3	0	40	0.4	0.45	6 500	0.3	50	0.35
16	75	0.3	0	40	0.4	0.45	6 500	0.25	25	0.35
17	75	0.3	0	40	0.4	0.45	6 500	0.25	75	0.35
18	75	0.3	0	40	0.4	0.45	6 500	0.25	50	0.3
19	75	0.3	0	40	0.4	0.45	6 500	0.25	50	0.4

As there were many uncertainties in the assumptions such as the current stress state in the lining and the surrounding ground, the properties of the ground, the lining and the backfilling, parametric studies were carried out in order to find the main influencing factors (4).

Calculation Model

The analyses were carried out using the finite element method, assuming plane strain conditions. Two different cross sections with overburdens of 8 and 14 m respectively were considered. In order to estimate the stress changes due to the open cut, the state of stress before the reconstruction had to be evaluated. Therefore the tunnel excavation had also to be taken into consideration in the calculations.

The excavation of the tunnel is a three-dimensional problem, deformations already occur in front of the tunnel face, before the lining is installed (stress release). This was modelled by reducing the stiffness of the elements inside the tunnel before removing them from the model. The amount of reduction depends on the stiffness of ground and support, the size of excavation and the unsupported length (6). As these parameters were difficult to estimate in this special case, the reduction of the stiffness was varied in a typical range.

After the tunnel excavation, the open cut was modelled step by step by removing individual layers of ground with a thickness of 2 m.

The calculation steps are summarised below:

- Primary state of stress,
- Stress release (reduction of stiffness of the elements within the tunnel),
- Excavation of the tunnel and installation of the lining and the backfilling,
- Open cut (remove layers of ground in several steps).

Figure 1 shows the state after excavating the tunnel and typical steps of the open cut. The calculations were carried out with the Abaqus finite element code.

Input parameters

As neither laboratory tests nor in situ tests were available, the parameters had to be estimated using comparable situations, descriptions in the literature as well as observations on site.

Ground parameters

The chosen parameters for the surrounding ground were based on data that had been derived for the design of structures nearby and on observations of natural slopes in the adjacent area. The mean values are shown below, the variations are included in the Table:

Young's modulus: E = 75 MPa,
Poisson's ratio: ν = 0,3,
Friction angle: φ = 40°,
Cohesion: c = 0 kPa,
Specific weight: γ = 22 kN/m³,
Coefficient of earth pressure at rest: k_0 = 0,45.

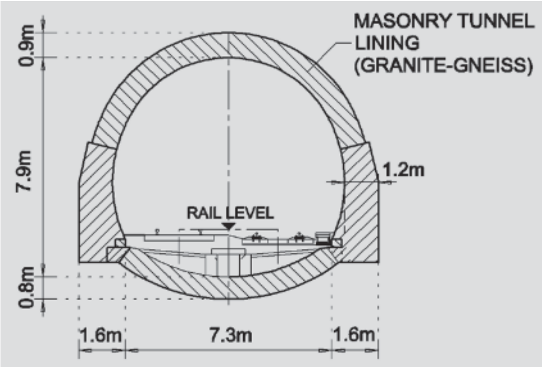

Fig. 2 Tunnel lining.
Bild 2 *Tunnelauskleidung.*

The surrounding ground was modelled using Mohr-Coulomb material behaviour.

Backfilling

It was expected that the surrounding ground rested on the tunnel lining and applied geostatic pressure. Therefore it was assumed that hardly any voids existed between the lining and the ground. This was confirmed by the records (1) that describe high geostatic pressure acting on the tunnel lining. Nevertheless there might have been some voids or soft material that had been used as backfilling. In the calculations a layer of backfill material with a thickness of 50 cm was therefore considered around the tunnel. The following mean properties were chosen:

Young's modulus: E = 50 MPa,
Poisson's ratio: ν = 0,35,
Specific weight: γ = 22 kN/m³.

The backfilling was modelled as elastic material.

Tunnel lining

The behaviour of the tunnel lining is governed by its components, the granite-gneiss blocks and the mortar between these blocks. In the circumferential direction the blocks and the mortar act together like serial springs. The total stiffness of the lining is therefore considerably lower than the stiffness of the granite-gneiss blocks. Published tests on old porphyry masonry structures have shown that the total stiffness is in the range of 50 % of the stiffness of the blocks.

In (5), the stiffness of granite-gneiss is given with 19 500 MPa as a result of laboratory tests. In order to consider the scale effect, this value was reduced to 70 %, so that the total stiffness amounted to:

$$E = 19\,000 \text{ MPa} \cdot 0{,}7 \cdot 0{,}5 \approx 6\,500 \text{ MPa}$$

This value was used as mean value. Since there may be large deviations, the stiffness was varied in a wide range.

The calculations were carried out assuming linear elastic behaviour for the lining which is a conservative assumption for analysing the stresses in the lining. These calculations were used for proving the stability of the structure. In addition the tunnel lining was modelled using Drucker-Prager material behaviour with limited tensile stresses.

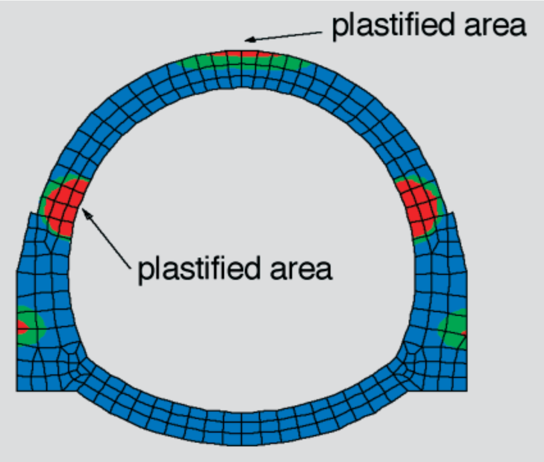

Fig. 3 Plastified areas with a remaining overburden of 1 m.

Bild 3 *Plastische Zonen bei einer verbleibenden Überlagerung von 1 m.*

The thickness of the lining in the investigated section amounts to 90 cm in the crown, 120 cm in the benches and 80 cm in the invert (Figure 2).

Variations

The Table shows the parameter variations for the analyses. The following abbreviations are used:

E Young's modulus in MPa,
ν Poisson's ratio,
φ Friction angle in degrees,
c Cohesion in kPa,
k_0 Coefficient of earth pressure at rest,
α Stress release (reduction of E inside the excavation section).

The parameters in line 1 of the Table resulted in the best agreement with the field measurements.

Design concept

The calculated stresses in the tunnel lining were integrated over the cross section, determining the axial force and the bending moment.

$$N = \int_A \sigma_x \cdot dA$$

$$M = \int_A \sigma_x \cdot z \cdot dA$$

with
N axial force,
σ_x circumferential stress,
M Bending moment related to the system axis.

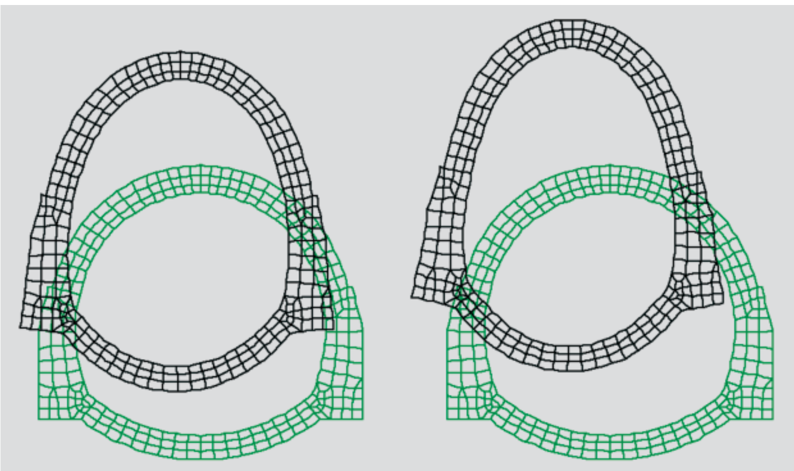

Fig. 4 Deformations of the tunnel lining for 5 and 1 m of remaining overburden.

Bild 4 *Verformungen der Tunnelröhre bei 5 und 1 m verbleibender Überlagerung.*

As long as the eccentricity e = M/N is less than d/6 (d = thickness of the lining), compression stresses act in the whole section. In this case no failure is to be expected since the compressive strength is relatively high. In case the eccentricity is more than d/6, tensile stresses act in the lining which cannot be carried by the mortar. The joints between the blocks will open. The tolerable limit according to common practice is an eccentricity which results in an open joint less than d/2.

Since the tunnel lining was modelled using elastic material behaviour, the stresses in the lining were overestimated resulting in additional safety. This was considered as reasonable, since in case of failure high personal and economic damage had to be expected.

Results

Despite the considerable variations, the results of all the calculations showed similar tendencies. The descriptions below are valid for both analysed sections, the numbers are related to the section with an overburden of 14 m. Removing several layers of ground resulted in a decrease of vertical stresses above the tunnel. The horizontal stresses also decreased, but not as much as the vertical stresses. For this reason the distribution of the geostatic pressure on the tunnel lining changed resulting in an ovalization of the tunnel (see Figure 4).

Tunnel lining

After excavating the tunnel, the lining was mainly under compression, only at the crown low tensile stresses on the inside of the lining were calculated. During the excavation of the overburden, the stress state changed continuously. The reduction of vertical geostatic pressure lead to a reversal of the stress distribution in the critical sections of crown and bench. In the tunnel crown tensile stresses were calculated on the outside of the lining, while at the benches tensile stresses acted on the inside (ovalization). The amount of tension increased with decreasing height of overburden.

For an overburden of 1 m, the stability of the tunnel lining could not be proved, as the eccentricity at the crown and bench became to big.

The analysis using a non-linear material law for the tunnel lining showed the same behaviour. After tunnel excavation, small plastified areas were found at the tunnel crown on the inner side of the lining. With decreasing height of overburden, plastified areas developed on the outside of the lining in the crown and on the inside of the lining at the benches. This calculation showed a considerable increase of plastic deformations as the height of overburden reached 1 m (Figure 3).

Deformations

The reduction of vertical stress due to the open cut lead to a heave of the tunnel. Depending on the depth of the cut, the calculated heave of the tunnel crown reached up to 15 mm. The differential deformation between the crown and the invert was considerably smaller. The elevation of the ground surface was higher on the right side of the investigated sections, and the tunnel was located in the right half of the cut. Therefore the whole tunnel moved to the left during the opening of the cut (Figure 4). This deformation represented mainly a rigid body motion and was not relevant for inducing stresses in the lining. The horizontal differential deformations between the left and the right bench, which in fact caused stresses, ranged from 5 to 10 mm.

Figure 5 shows the increase in vertical lining deformations with decreasing overburden for the chosen section with an original overburden of 14 m. During the excavation of the tunnel, the crown moved 2 mm downwards and during the opening of the cut the crown moved approximately 14 mm upwards. As the heave in the crown was greater than in the bench and the invert, an ovalization of the tunnel lining was caused.

Sensitive parameters

The calculations showed that the parameter variations do not lead to different behaviour of the structure and that the main results vary only slightly. Figure 6 shows the differential settlements between tunnel crown and invert for all calculations. The most sensitive parameters are the stiffness of the ground and the lining, while the stiffness of the backfilling hardly influences the results.

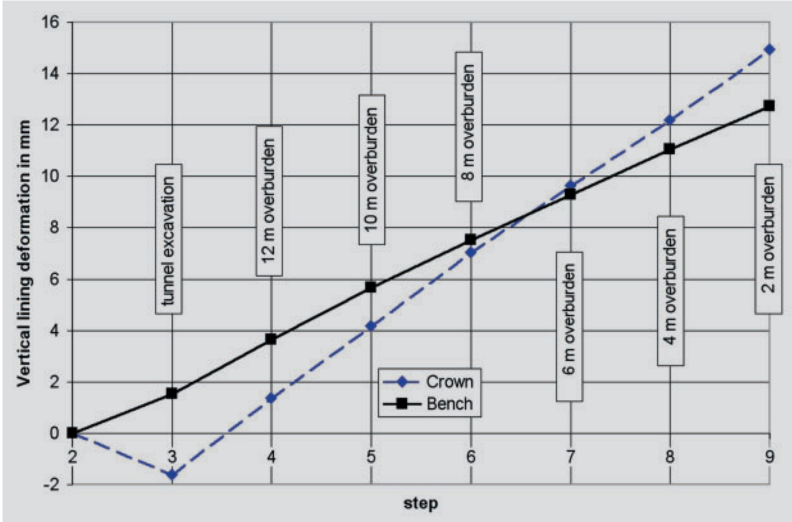

Fig. 5 Lining deformations ("+" = heave, "-" = settlement).

***Bild 5** Verformungen der Tunnelschale ("+" = Hebung, "-" = Setzung).*

Conclusions from numerical analysis

The results of the calculations led to the following conclusions. As the eccentricity in the tunnel lining increases significantly as the open cut reaches the tunnel crown, the open cut had been limited to a minimum overburden of 2 m. In addition it was not permitted to excavate unsymmetrical to the tunnel axis, since this would have lead to higher bending moments in the lining.

In order to control the behaviour during the construction a monitoring programme consisting of geodetic measurements and visual observa-

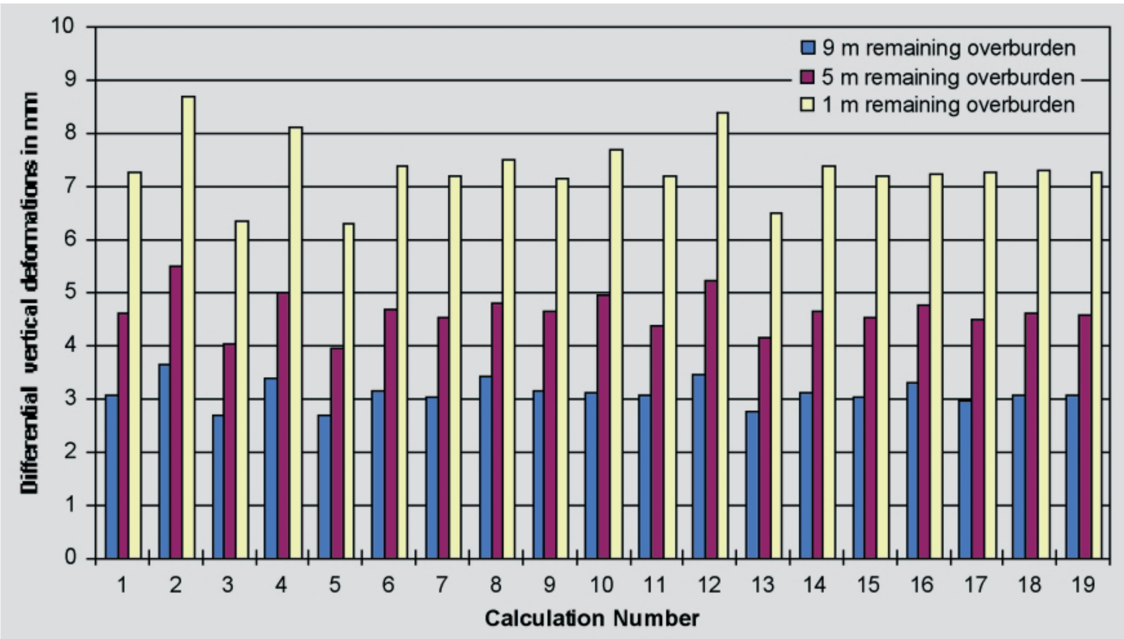

Fig. 6 Differential vertical deformations between tunnel crown and invert.

***Bild 6** Differentielle vertikale Verformungen zwischen Firste und Sohle.*

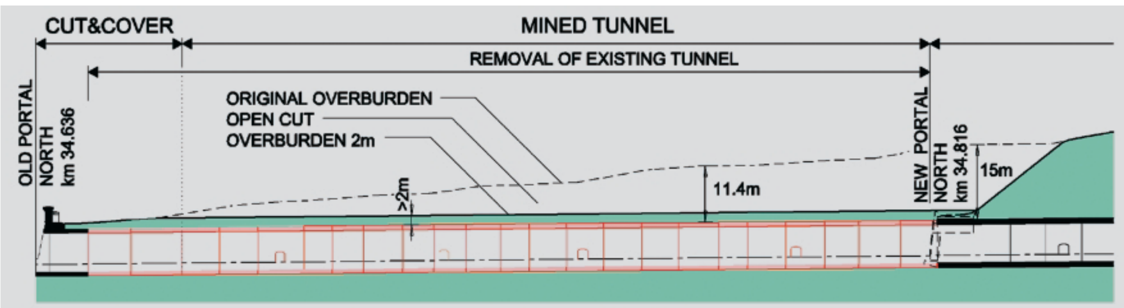

Fig. 7 Original overburden.

***Bild 7** Ursprüngliche Überlagerung.*

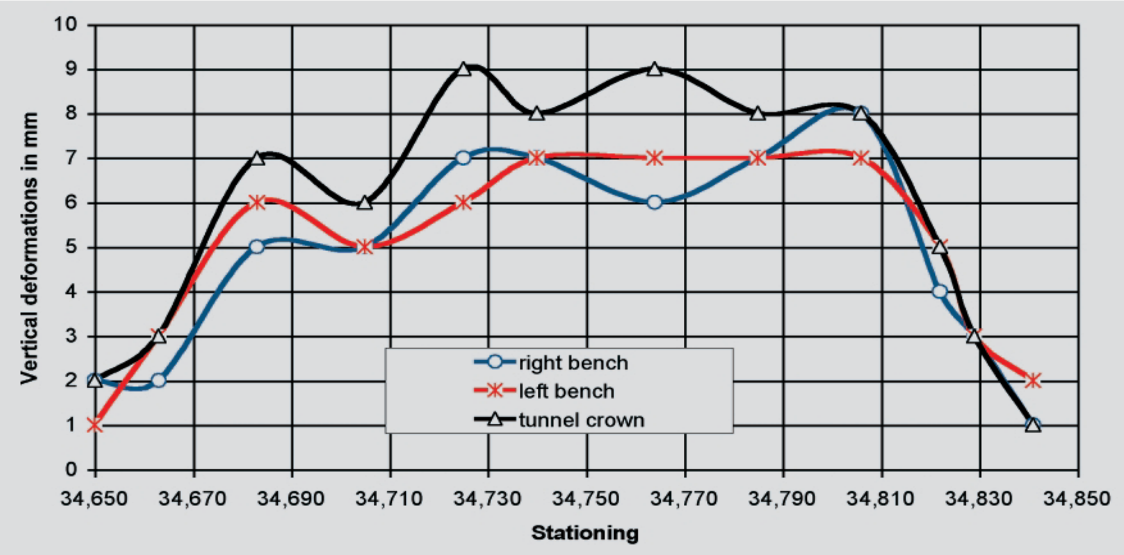

Fig. 8 Vertical deformations related to the open cut ("+" = heave).

Bild 8 *Vertikalverformungen als Folge des offenen Abtrags ("+" = Hebung).*

tions (gypsum spies and strain gauges) had been developed.

In situ measurements

The deformations of the tunnel lining were observed during the construction works. The deformations were directly related to the depth of the open cut as expected, e.g. while the deformations close to the existing tunnel portal at km 34,650 were less than 3 mm, they increased depending on the original height of overburden (Figure 7). Close to the new portal at km 34,840 where the geostatic pressure hardly changed, only small deformations were observed. Figure 7 shows the original overburden above the tunnel crown and the overburden right before reaching the defined limit of 2 m. In Figure 8 the related vertical deformations of the tunnel crown and the benches are shown.

The geodetic control measurements in the narrow tunnel tube had to be performed during short operational breaks and thus had an accuracy of ± 2 mm. As the measurements were taken up to twice a day in accordance with construction progress, measurement errors were quickly detected and corrected. Considering the necessary simplifications of the analysis model, the calculated results were in good agreement with the measurements with regard to the ground behaviour.

With respect to the behaviour of the masonry the measurements showed a stiffer behaviour than was expected from the calculations. The absolute displacements and differential deformations between the left and the right bench remained below the calculated values.

That was why new calculations were done parallel to the construction works with the aim of permitting removal works beside the tunnel in order to minimise the remaining removal works during the route closures. The overburden of 2 m in the crown was maintained in order to be able to rule out any local failure of the lining.

In a first step it could be permitted to excavate beside the tunnel in a distance of at least 8 m to the tunnel axis. The measurements were continued during these works and confirmed the results of the previous measurements. The differential deformations between the left and the right bench reached a maximum of 3 mm and were within the range of the prognosis. The differential deformations between bench and crown amounted to maximum 5 mm and thus met the calculated value. As the calculated deformations were not exceeded at this stage and because the tunnel lining was modelled with an elastic material law with which the stresses in the lining were overestimated, it could be permitted to excavate beside the tunnel as close as 6 m from the tunnel axis. Measurements were taken even after the earthworks were completed until the actual removal of the tunnel lining with the remaining overburden. They showed that hardly any additional deformations occurred during these works.

Fig. 9 Removing the remaining overburden and Stripping the lining (Foto: Alpine/ Tschaudi).

Bild 9 *Entfernen der verbleibenden Überlagerung und Abbruch der Schale.*

Removal of tunnel lining

Due to the experience gathered in other projects with delayed stripping of the masonry on account of unforeseen difficulties it was planned to cause the tunnel to collapse by blasting. However, this would have required extensive preparations in the tunnel while keeping the tunnel open to railway traffic.

That was why a concept was elaborated upon a proposal from the constructing companies in coordination with the client and designers. This concept encompassed conventional stripping with the corresponding use of machinery and was elaborated and implemented successfully. In the course of the removal works the masonry proved to be very compact. It had to be opened up after removing the remaining overburden (Figure 9) by means of breaking teeth in the crown and had to be demolished using large machinery.

It also became apparent that the lining was masoned very irregularly and interlocked tightly with the ground. The thickness of the lining varied up to several decimetres along its circumference.

Conclusion

For carrying out the earth removal works above the railway tunnel boundary conditions and limits had to be defined based on experience, available information from relevant literature and numerical investigations with variation of input parameters. In order to be able to check the results of the calculations and thus proving the stability, control measurements were taken and continuously compared with the calculations. With regard to the deformation behaviour a high degree of correspondence of the overall system with the calculations was found.

With regard to the behaviour of the masonry during the excavation above the tunnel, the lining proved to be more compact and more capable of load-bearing than was assumed in the calculations without having any masonry explorations or laboratory tests at hand.

Nevertheless, even if tests were available, non-homogeneities of the masonry as well as strength losses and damages due to ageing would have to be considered in the calculations. This would also require assumptions with regard to the numerical approaches. The maximum load-bearing capacities which was found during the stripping of the masonry should not have been applied in the calculations to the entire lining in order to ensure the continued safe railway operation. This is the reason why the chosen course of action proved to be useful since the removal of the ground could be done very close to the lining based on engineering analyses.

The remaining works performed during the route closures for a timely completion of the construction works were minimised to a sufficient extent.

Together with the control measurements it was possible to guarantee the supervision of the removal works necessary for a safe operation of the railway.

References

1. K.K. Eisenbahnbaudirektion: *Fortschrittslängenprofil Nordseite des Tauerntunnels, Planunterlagen,* 1903.
2. Österreichische Bundesbahnen: *Erneuerungsarbeiten auf der Nordseite des Tauerntunnels 1947–1949*. Kurzbericht ÖBB PE TASU.
3. Österreichische Bundesbahnen: *Kurze Zusammenfassung der Baugeschichte des Tauerntunnels*. Kurzbericht ÖBB PE TASU.
4. Thurner, R. ; Schweiger, H.F. ; Pöttler, R. ; Mattle, B.: *A practical approach for risk assessment employing reliability analysis and finite elements.* Proceedings of the 10th international Conference of IACMAG, January 2001, Tuscon Arizona, USA.
5. Czech, J. ; Huber, H.: Gesteinskennwerte aus Laborversuchen. *Felsbau* 8, 1990.
6. Kielbassa, S. ; Duddeck, H: Stress-Strain Fields at the Tunnelling Face, Three-Dimensional Analysis for Two-dimensional Technical Approach. *Rock Mechanics and Rock Engineering* 24 (1991), pp.115-132.

Authors

Dipl.-Ing. Bruno Mattle, Dipl.-Ing. Christof Weigl, Dipl.-Ing. Johannes Benedikt, ILF Consulting Engineers, Framsweg 16, A-6020 Innsbruck, Austria, E-Mail bruno.mattle@ibk.ilf.com, christof.weigl@ibk.ilf.com, johannes.benedikt@ibk.ilf.com