

# Design of the primary support of the NATM excavation of the metro station Taborstraße in Vienna based on a nonlinear 3-D-FE-model

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## Abstract

In the course of the extension of a metro line in Vienna the station Taborstraße is under construction. Two single track platform tunnels, a connection tunnel and an elevator tunnel are driven through sediments with an overburden of 12 m following an NATM excavation scheme. Time dependent three-dimensional finite element analyses have been carried out, in order to predict the settlements due to the excavation of the tunnels and to perform an ultimate limit state design. For both, the soil and the shotcrete, constitutive models with strength limits allowing an implicit design have been applied.

The actions taken in order to provide safety margins comparable to those of modern standards like the Eurocodes (which are focused on conventional models which are based on linear elastic material models for structures) and to avoid pitfalls of the numerical models are presented. The applicability of the design approaches of Eurocode 7 in connection with an implicit ultimate limit state design using time dependent strength parameters is discussed. The resulting settlements are compared with measurement results. In addition, the effects of different excavation techniques, including jet grouting, and the speed of excavation have been studied. A simple way of quantifying the effect of jet grouting columns is presented.

## Zusammenfassung

Im Zuge der Verlängerung der Wiener U-Bahn-Linie U2 wird die Station Taborstraße errichtet. Zwei eingleisige Stationsröhren, ein Verbindungstunnel sowie ein Fahrtreppentunnel werden bei einer Überlagerung von 12 m durch Sedimente zyklisch vorgetrieben. Um die Setzungen prognostizieren zu können und zur Dimensionierung der Spritzbetonschale wurden zeitabhängige 3-D Finite-Elemente-Berechnungen durchgeführt. Sowohl für den Boden als auch für den Spritzbeton wurden nichtlineare Materialgesetze eingesetzt, die die zulässigen Spannungen begrenzen und somit eine implizite Bemessung erlauben.

In diesem Beitrag wird erläutert, welche Maßnahmen getroffen wurden um Sicherheitsreserven gemäß aktueller Normen wie den Eurocodes (die vorwiegend auf lineare Materialmodelle zugeschnitten sind) zu erhalten und Schwächen der numerischen Modelle zu umgehen. Die Anwendbarkeit der drei im Eurocode 7 vorgesehenen Nachweisverfahren im Zusammenhang mit impliziter Bemessung und zeitabhängigen Festigkeitskennwerten wird erörtert. Die errechneten Setzungen werden mit geodätischen Messungen verglichen. Die Auswirkungen verschiedener Vortriebskonzepte einschließlich Düsenstrahlverfahren (DSV) wurden untersucht. Ein einfaches rechnerisches Verfahren zur Quantifizierung der Auswirkungen von DSV-Säulen wird vorgestellt.

## 1 Introduction

In the course of the extension of the metro line U2 in Vienna the station Taborstraße is under construction. Client is the “Wiener Linien”, the Vienna Transport Authority. The line will go into operation in 2008.

The station consists of two parallel single track platform tunnels with a cross section of 74 m<sup>2</sup> each. The platform tunnels are connected by a connection tunnel (cross section 59 m<sup>2</sup>) from which an escalator tunnel (cross section 63 m<sup>2</sup>) provides access to the street level. Two shafts, a large one at the western end of the station and a small one at the eastern end between the platform tunnels, have been constructed as open cut prior to the platform tunnels. The tracks lie about 18 m below street level. The tunnels run partly below 5-7-storey-buildings, partly below streets. Figure 1 shows a plan view of the station.

The tunnels are driven through sediments of the Quarternary and Tertiary following an NATM excavation scheme. Each tunnel is subdivided into top heading, bench and invert, see Figure 2. Primary support of the tunnels is a shotcrete lining with a thickness of 30 cm.. The linings will be reinforced with two layers of wire mesh. According to the tender documents (1), in order to minimise settlements the cross section of the platform tunnels should be divided into two main parts: A side drift, subdivided into top heading, bench and invert, see Figure 2, would be excavated first; then the

remaining cross section would be excavated with the same subdivision. This subdivision was the basis for a first set of analyses.

Three-dimensional finite element (FE) analyses have been performed in order to predict the settlements due to the excavation of the tunnels and to design the shotcrete lining. Of special interest are the stress concentrations in the lining which develop at the intersections in the course of the excavation process.

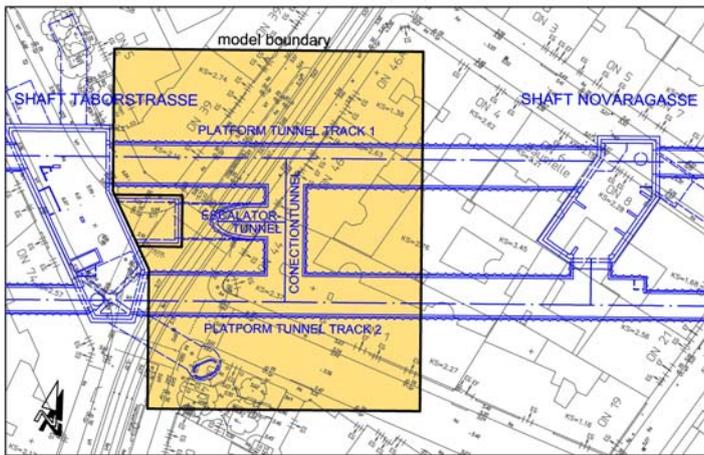


Figure 1. Plan view of the station and the model boundaries.  
Bild 1. Stationslageplan mit Modellgrenzen.

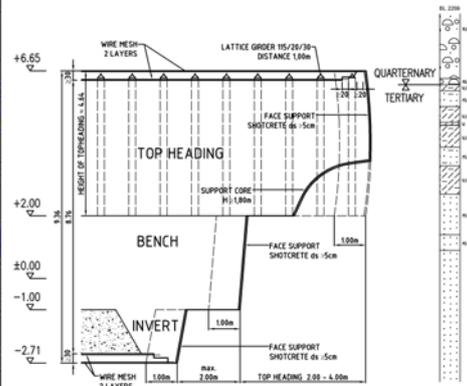


Figure 2. Excavation scheme: vertical section.  
Bild 2. Ausbruchsschema: Vertikalschnitt

The finite element model extends over the whole station area from the shaft “Taborstraße” with a length of 60 to 68 m in the direction of the line, a width of 80 m and a vertical span of 35 m. The chosen coordinate system has its origin at the height of “Wiener Null”, the local reference system, above the longitudinal axis of the connection tunnel in the middle between the two platform tunnels. Top of rail (TOR) is 15.0 m below the level of the basements or 12.5 m below “Wiener Null”. The x-direction of the coordinate system is the direction of the line (eastwards positive), the z-direction is vertically upwards.

In the following the most important properties of the numerical model are described and some characteristic results are presented. Section 5 deals with the implications of nonlinear material and structural behaviour on the interpretations of the design rules specified in the standards.

In the course of the project development, the excavation scheme has been modified and jet grouting has been introduced as an additional support measure. The numerical investigations performed in order to quantify the effects of these changes are described as well. A simple way of quantifying the effect of jet grouting columns, based on experiments, is presented.

Finally the calculated settlements are compared with measurements during the construction process.

## 2 Constitutive relations, material properties

### 2.1 Soil

The soil has been modelled by linear volume elements using the Mohr-Coulomb-material model. Three different soil layers were distinguished, with parameters according to table 1.

Table 1. Material properties of the soil layers.

Tabelle 1: Materialkennwerte für die Bodenschichten

Soil layer		Quarternar y	Tertiary (Silt)	Tertiary (Fine sand)
Description		gravel, sandy	silt, clay	sand, silty
Lower boundary	m above TOR	~ 6	~ 3	-
Specific weight (wet)	kN/m <sup>3</sup>	20	20	20
Specific weight (under water)	kN/m <sup>3</sup>	11	11	11
Modulus of elasticity	MN/m <sup>2</sup>	150	50	150

Poisson's ratio	1	0.28	0.35	0.30
Friction angle	°	35	25	27.5
Cohesion	kN/m <sup>2</sup>	0	50	0

The parameters given in the table are used in the analyses except for the following modifications: At a level of about 4 m below TOR the groundwater table has been assumed. (The groundwater table will be lowered to the level of the invert of the platform tunnels during construction of the station. The calculated settlements therefore do not contain the effects of the changes of the groundwater table.) Below this level, not only the specific weight under water is used, but also the modulus of elasticity is increased artificially by a factor of 5 in order to simulate the stiffer soil behaviour during unloading below the tunnel.

The parameters given in the table were specified as characteristic values by experienced geotechnical engineers based on site investigations and former experience with constructions at similar geological conditions in Vienna. Based on the measurement results of the excavation of the running tunnels (which were excavated earlier), lower stiffness properties of the soil than in the tender documents (1) were applied. The parameters have been chosen as cautious estimates of the mean value in the sense of Eurocode 7 (2). The strength parameters do not explicitly contain any safety factors, neither do the stiffness parameters.

The soil above the level of the basements of the buildings has not been modelled. Instead, the dead load of the soil (70 kN/m<sup>2</sup>) and the buildings (130 kN/m<sup>2</sup>), respectively, have been specified as distributed loads at the level of the basements.

## 2.2 Shotcrete

The shotcrete lining consists of layered shell elements. A constitutive model developed by Meschke and Mang (3, 4) and extended by additional creep and shrinkage terms, has been applied: It contains a strain-hardening Drucker-Prager loading surface with a time dependent hardening parameter in the compressive regime. Cracking of maturing shotcrete is accounted for in the framework of the smeared crack concept by means of three Rankine failure surfaces, perpendicular to the axes of principal stresses. The increase of elastic stiffness during hydration of shotcrete as well as the time-dependent increase of compressive strength, tensile strength, and yield surface are all considered.

The original Meschke-Mang-model describes creep effects with one single parameter, the viscosity. In the course of a research project the model has been adapted to experimental results by additional creep terms. Details of the model can be found in (5). The material parameters for the shotcrete have been chosen in a way to match the time-dependent properties of the shotcrete type SpC 25(56)/J2 (6). Different amounts of creep have been specified in the course of a parameter study (7). The values chosen for the analysis yield a relatively small amount of creep strains and little stress reduction due to creep. Thus, a conservative stress level is achieved.

Table 2. Material parameters of shotcrete.

Tabelle 2. Materialkennwerte für Spritzbeton

Parameter	Symbol	Unit	Value
Specific weight	$\gamma$	kN/m <sup>3</sup>	25
Poisson's ratio	$\nu$		0.20
Cylinder compressive strength at 28 days	$f_{cu}^{(28)}$	kN/m <sup>2</sup>	17 000
Cylinder compressive strength at 1 day	$f_{cu}^{(1)}$	kN/m <sup>2</sup>	4500
Yield stress at 28 days	$f_y^{(28)}$	kN/m <sup>2</sup>	3000
Viscosity parameter	$\eta$	h	15
Modulus of elasticity at 28 days	$E^{(28)}$	MN/m <sup>2</sup>	25 000
Factor describing the amount of irreversible creep	A	m <sup>2</sup> /kN·h <sup>-1/3</sup>	1.0·10 <sup>-8</sup>
Ultimate value for the reversible creep compliance	$C_{d\infty}$	m <sup>2</sup> /kN	1.5·10 <sup>-7</sup>
Factor describing how fast the delayed elastic strains develop	Q	m <sup>2</sup> /kN	4.0·10 <sup>-8</sup>
Factor to introduce a nonlinearity for high stress levels	k		0

### 2.3 Jet-grouting

Due to time constraints – the application of jet-grouting was introduced rather shortly before the start of excavation – the use of advanced coupled models like in (8) was not feasible. A 3-D-model was out of scope as well. A plane section through the platform tunnel was investigated, instead. Each column was simulated by only two rectangular finite elements. Temperature gradients within the columns were considered in an integrative manner, only.

As constitutive law, the extended Meschke-Mang-model was applied, again. The model parameters were derived from experimental investigations conducted by Brandstätter (9):

Laboratory tests were conducted using specimens, some cast from a mortar mix of cement PZ 275, sand and water, some taken at construction sites. The tests were carried out at three different temperature levels.

In order to adjust the parameters for the stiffness and strength development with time in the Meschke-Mang-model (4), all experimental results from laboratory specimens containing at least 50 % sand have been displayed in a single diagram for the development of strength with time (Figure 3), and a second diagram for the development of stiffness with time (Figure 4). The effect of the temperature has been taken care of by scaling the time axis as a function of the temperature (scale factors:  $T = 23^{\circ}\text{C}$ : 1.0,  $T = 40^{\circ}\text{C}$ : 1.7;  $T = 7.5^{\circ}\text{C}$ : 1/3.0).

The corresponding curves from an optimised set of parameters for the extended Meschke-Mang-model have been added in the figures. Considering the scatter of the experimental data a good agreement has been achieved. In Table 3 the chosen parameters – which are explained in more detail, together with the underlying equations, e.g. in (4) – are summarised.

Table 3. Material parameters matching the experimental results with mortar.

Tabelle 3. Materialkennwerte für DSV-Mörtel (von Versuchen abgeleitet)

Parameter	Symbol	Unit	Value
Cylinder compressive strength at 28 days	$f_{cu}^{(28)}$	$\text{kN/m}^2$	7500
Cylinder compressive strength at 1 day	$f_{cu}^{(1)}$	$\text{kN/m}^2$	1000
Modulus of elasticity at 28 days	$E^{(28)}$	$\text{MN/m}^2$	9000
Ratio between modulus of elasticity after 1 day and after 28 days	$E_1/E_{28}$		0.30
Time constant	$\Delta t_E$	h	6
Time threshold	$t_E$	h	8

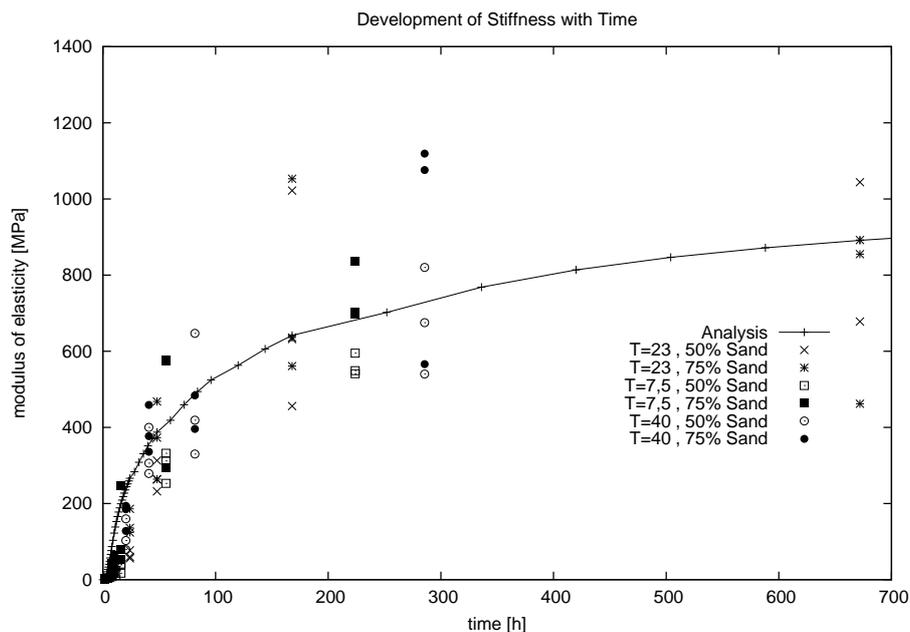


Figure 3 – Comparison of the development of stiffness

Bild 3 – Vergleich der Steifigkeitsentwicklung

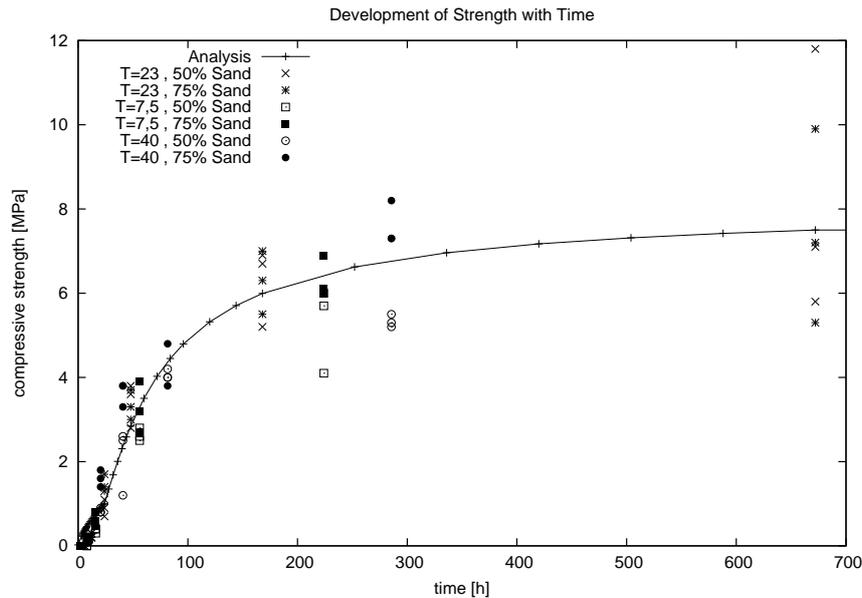


Figure 4 – Comparison of the development of strength

Bild 4 – Vergleich der Festigkeitsentwicklung

According to (9), the strength of specimens of plain cement suspension was lower than the strength of mortar specimens containing at least 50 % sand and reached about 5 to 6 MPa. Specimens from a construction site with dominantly silty-sandy soil properties yielded uniaxial strength values in the range of 0.4 to 9.8 MPa, with a mean value of 4.6 MPa. Compression tests at the construction site for the cement suspension used for the running tunnels yielded strength values in the range of 6 to 8 MPa. Due to the obviously better strength properties of the cement suspension and considering the fact that small specimens are more sensitive to local inhomogeneities, the uniaxial cylinder strength of the mortar specimens of 7.5 MPa has been used in the analyses for the material of the jet grouting columns.

The time dependent strength and stiffness properties implemented in the Meschke-Mang model could as well be fed into any concrete model which allows changing strength and stiffness in the course of the analysis. The additional specification of a high Poisson's ratio of 0.49 for very young columns (which drops to 0.30 within 20 fictive hours) has been used in the calculations described here in order to reduce the deformation of the columns of young age. (In a 2-D-model, always columns of infinite length are produced within one time interval. Deformations of freshly cut columns are overestimated because stiffness differences along the length of the column are ignored.)

Analyses with a thermo-chemo-mechanical coupled constitutive law (8) have shown that jet grouted soil columns with a diameter of 60 cm reach temperatures in the range of 10°C to 25°C. The maximum temperature develops in the centre of a column after about 15 hours. (Observations on site indicate that the temperature increase might be more pronounced.) Averaging the temperatures over the thickness of a column and the time interval of interest, a conservative mean temperature of 12° C can be assumed. This temperature is simulated in the analyses by scaling the time axis by a factor of 2.0 (compared with the figures above).

### 3 Excavation sequence, discretisation in time

The excavation of the tunnels is modelled by removal of the finite elements representing the excavated soil. In order to keep the size of the model at a reasonable level two rounds, with a length of 1 m each, of the real excavation scheme have been combined to one fictitious round. In general, only one layer of finite elements has been used to simulate such a fictitious round. The size of the time steps in the analyses has been chosen as one time step per round, the size of one time step being 24 hours.

The excavation schemes have been simplified for the analyses: The excavation of one (fictitious) round of bench and invert has been combined to one step; the distance between the excavation of the top heading and bench + invert is 4 m or two (fictitious) rounds. The excavation of a round of the remaining cross section of the platform tunnels is simulated as one single step for top heading, bench and invert and removal of the temporary lining together. The minimum distance between excavation of the side drift and excavation of the remaining cross section is 4 months according to the construction schedule.

Both platform tunnels have been driven in the same direction. The distance between the face of the top heading of the side drift and the face of the remaining cross section has been condensed to 36 m (18 days) for both platform tunnels.

The distance between the drives of the two platform tunnels has been chosen as 16 m (8 days). Thus, the number of time steps required in the analyses could be kept small. Care has been taken that the distance between the drives is still large enough to prevent an artificial mutual influence.

In the course of the introduction of jet grouting as additional support, the excavation scheme was changed to a scheme without side drift; the other basic parameters remained unchanged.

Shotcrete support is applied by stress-free activation of the shell elements simulating the newly added lining. It has been assumed that every newly excavated fictitious round is without shotcrete support, and that the time of 24 hours is used just for excavation and mucking. The second round behind the face already contains shotcrete support. The shotcrete age at the end of this second round has been set to 18 hours, i.e. the hardening of the shotcrete starts 6 hours after the end of excavation and mucking. These assumptions result in a very conservative estimate of the deformations and the strength of the shotcrete support.

Perfect bond between soil and lining as well as between older and newer parts of the lining has been assumed.

Other means of support, like forepoling rods, lattice girders or temporary support of the face, have been neglected in the analyses. The effect of jet grouting will be discussed later.

#### 4 Results using characteristic parameters, serviceability limit state design

According to the design concept of the Eurocodes, the safety, serviceability and durability of a structure have to be investigated (10). The safety will be discussed in the next section “Ultimate Limit State Design”. The serviceability limit states of concrete structures are mainly: stress limitation, crack control and deflection control. In the case of the primary lining of a tunnel, stress limitation and crack control are of minor importance, but the limitation of the displacements is a major issue. This is especially true for the settlements at the surface – which actually threaten much more the serviceability of existing buildings at the surface, and much less the serviceability of the newly excavated tunnels. (The limitation of the displacements as part of the observational method is a different issue.) In the vicinity of the station Taborstraße a maximum inclination of 1 : 500 has been specified as tolerable by the authorities. This inclination is measured in the directions parallel and perpendicular to the building walls.

##### Settlements

Using the soil and shotcrete properties according to Tables 1 and 2 analyses have been performed in order to quantify the expected settlements. A value of 1500 kN/m<sup>2</sup> has been used for the shotcrete tensile strength at an age of 28 days. In contrary to the analyses for the ultimate limit state design below, the tensile strength of shotcrete has been considered in a moderate way. Figures 5a, 5b and 5c show the calculated settlements at the end of the excavation of the two platform tunnels, after excavation of the connection tunnel, and after finishing the escalator tunnel, respectively. The displayed settlements are based on the excavation scheme with a side drift for the platform tunnels; the effects of other excavation schemes are dealt with in section 6.

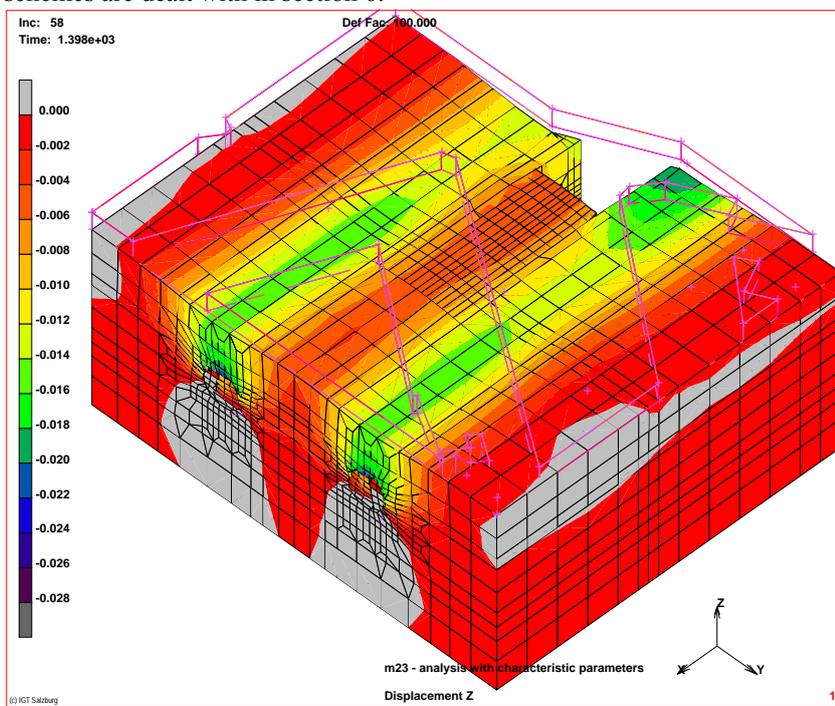


Figure 5a. Vertical displacements after finishing of the station tunnels.

Bild 5a. Vertikalverschiebungen am Ende des Vortriebs der Stationstunnel

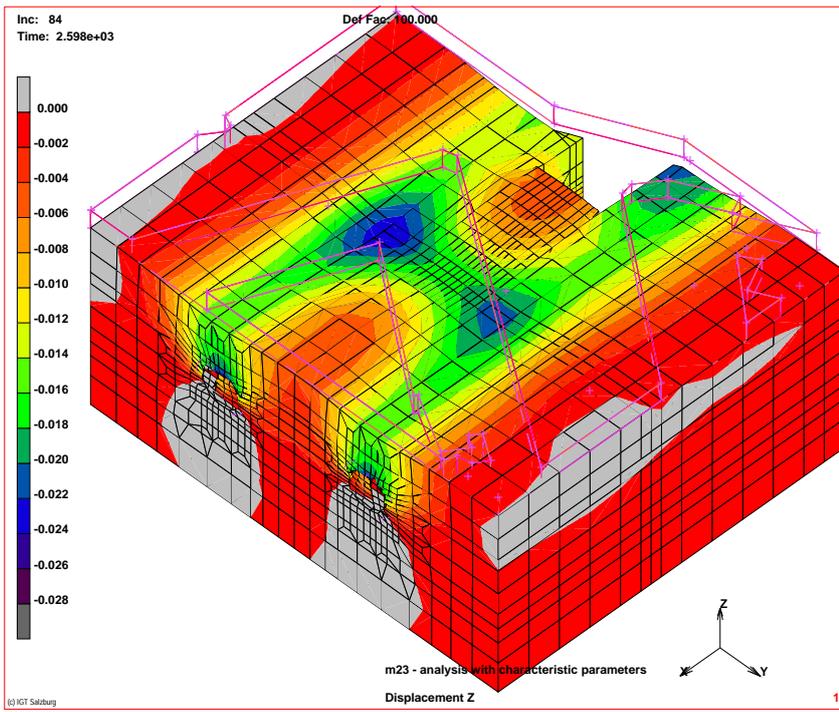


Figure 5b. Vertical displacements after excavation of the connection tunnel.  
 Bild 5b. Vertikalverschiebungen nach Ausbruch des Verbindungstunnels

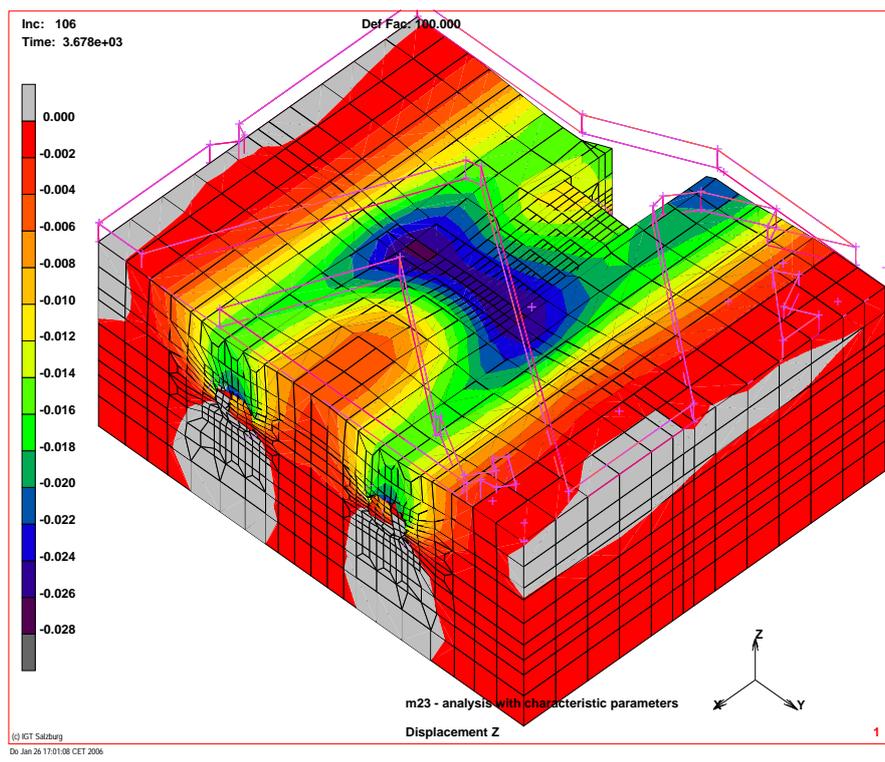


Figure 5c. Vertical displacements at the end of the analysis.  
 Bild 5c. Vertikalverschiebungen am Ende der Berechnung.

During the excavation of the side drifts of the platform tunnels the settlements amount to about 5 to 7 mm. After the full cross section has been excavated, the settlements reach 14 to 17 mm, depending on the loads due to buildings above. As expected, there is little mutual influence of the two tubes. The boundary conditions at the model boundaries (front and back) are only a crude estimate of the real behaviour at the boundaries. Therefore the calculated settlements at the front and back end of the model are inaccurate, as well as the calculated stresses in the lining. The excavation of the connection tunnel increases the settlements to 25 mm (track 2) and 22 mm (track 1), respectively. The difference is caused by the weight of the building above track 2. Without differing building loads, the settlements at

the end of the excavation (here: at track 1) would be about 1 mm larger due to the locally smaller soil stiffness, as comparative analyses have confirmed.

The excavation of the escalator tunnel affects mainly the settlements beyond the connection tunnel. The maxima remain located at the intersection between platform tunnels and connection tunnel, with 29 mm above track 2 and 28 mm above track 1. The inclination of the building walls above track 2 reach about 1 : 600.

The settlements and inclinations do not contain the effects of the ground water lowering. This has been assumed to have been taken place at the very beginning of the analyses, together with the application of the dead load of the soil.

The calculated settlements contain effects of creep of the shotcrete lining. However, creep has only a minor influence on the settlements. Comparative analyses without creep showed just 1 mm less vertical displacements at the end of the excavation process.

### Stresses and plastic strains in the soil

Figures 6 and 7 provide a perspective view of the intersection of the platform tunnel of track 2 with the connection tunnel the excavation of which has just begun. Figure 6 shows the vertical stresses in the soil. Stress concentrations at the corners of the intersections (increase of 300 to 400 kN/m<sup>2</sup>) and stress relief at the free surfaces at the faces are clearly visible. The equivalent plastic strains of Figure 7 show slight plastifications throughout the side walls of the platform tunnel. They augment to about 1 % at the unsupported faces whereas there is practically no increase at the corners of the intersection. Displacements are multiplied by a factor of 50, resulting in just visible deformations of the cross section and comparatively huge displacements of the face.

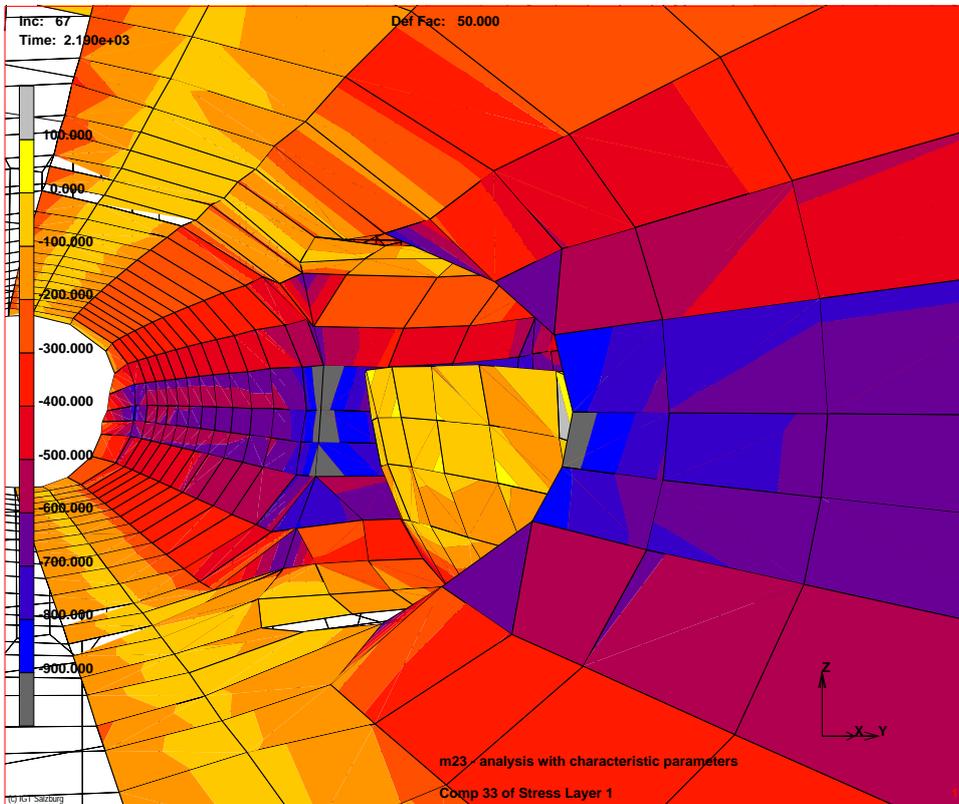


Figure 6. Vertical stresses in the soil at the beginning of the excavation of the connection tunnel. (Displacements magnified 50 times.)

Bild 6. Vertikalspannungen im Boden am Beginn des Ausbruchs des Verbindungstunnels. (Verschiebungen 50fach überhöht.)

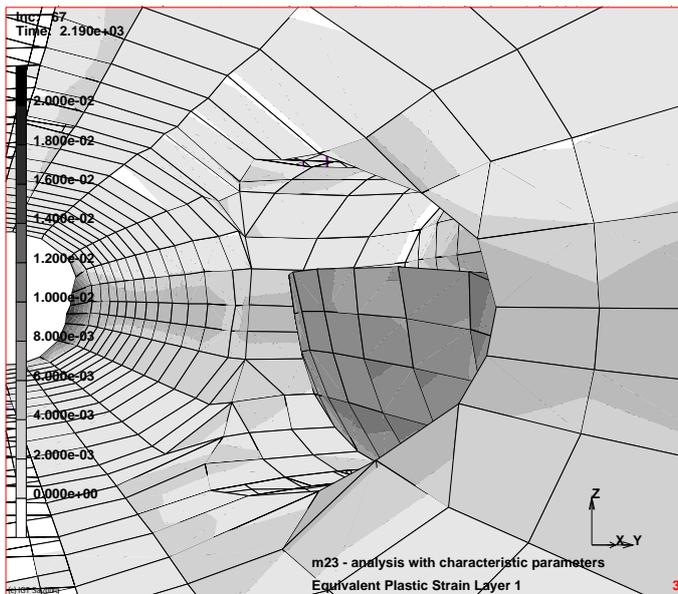


Figure 7. Equivalent plastic strains in the soil at the beginning of the excavation of the connection tunnel (analysis step 67). (Displacements magnified 50 times.)

Bild 7. Äquivalente plastische Verzerrungen am Beginn des Ausbruchs des Verbindungstunnels. (Verschiebungen 50fach überhöht.)

The load steps with the maximum plastic strains coincide with the load steps with the maximum displacements of the face. During excavation of the connection tunnel maximum displacements of 76 mm are calculated for the face of the bench and invert; the maximum face displacements for the escalator tunnel are 56 mm, again at the bench.

The subdivision of the cross section of the platform tunnels into side drift and remaining cross section has a favourable effect on the displacements of the face, as expected: they are reduced to less than 3 cm. (The numbers given here are total displacements, and do not belong only to the current excavation step.)

### Stresses in the shotcrete lining

Figure 8 contains the normal stresses in circumferential direction in the middle layer of the lining. The maximum compressive stresses occur at the side walls at the corners of the intersections. The largest stress increase due to the excavation occurs at the start of the driving of the connection tunnel in the lining of the platform tunnel of track 2.

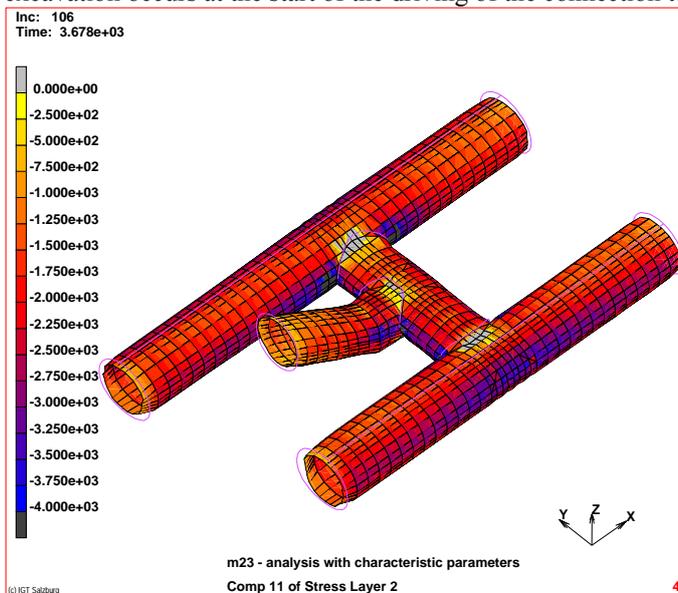


Figure 8. Normal stresses in the middle layer of the shotcrete lining in circumferential direction at the end of the analysis.

Bild 8. Normalspannungen in der Mittelfläche der Spritzbetonschale in Umfangsrichtung im letzten Zeitschritt.

The moments in the regions of maximum normal stresses in the middle layer are relatively small. Obviously the stress level is so close to the compressive strength there that the moments are reduced by creep effects and that stress redistribution within the cross section occurs.

The normal stresses in longitudinal direction increase in the vicinity of the intersections as well. However, the stress level is considerably lower than in circumferential direction. High tensile stresses at the roofs of the intersections match the corresponding tensile stresses in circumferential direction of the second part of the intersection.

Creep causes a stress reduction of about 10 % during the observed time period in general. At the locations with stress levels close to the compressive strength the creep effects are more pronounced, as had to be expected.

Figures 9a and 9b show the normal stresses in the lining in circumferential direction, again. Two steps during the excavation of the connection tunnel are displayed. The stress increase in the existing platform tunnel during the approach of the connection tunnel are clearly visible, as well as the stress changes due to relaxation.

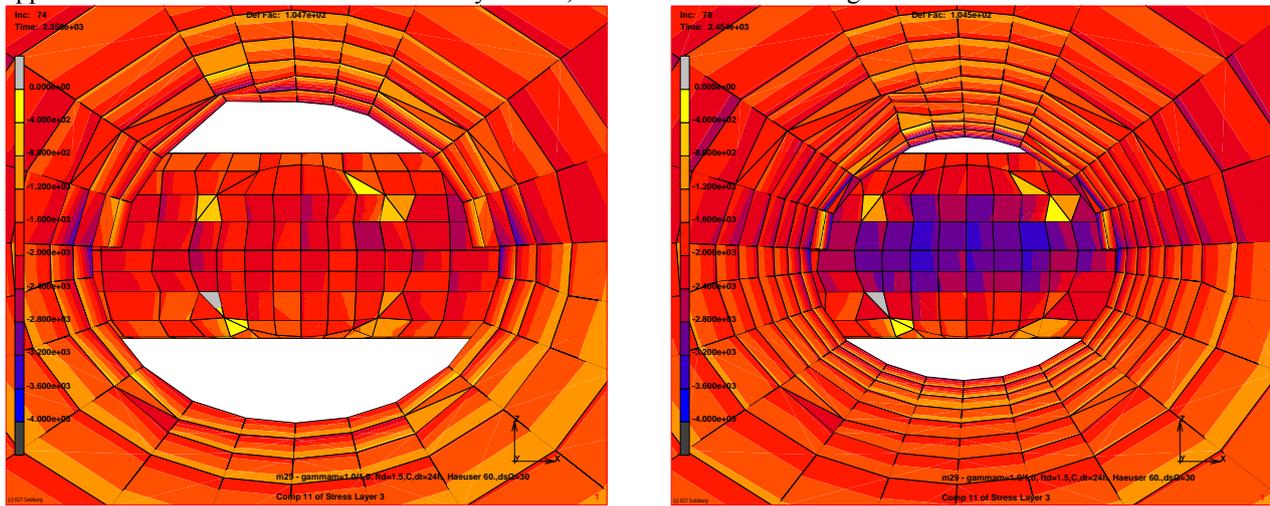


Figure 9. Stresses in the shotcrete lining in circumferential direction of the middle layer during excavation of the connection tunnel.

a – Analysis step 74

b – Analysis step 78

Bild 9. Spannungen in der Mittelfläche der Spritzbetonschale in Umfangsrichtung beim Ausbruch des Verbindungstunnels.

a – Berechnungsschritt 74

b – Berechnungsschritt 78

## 5 Ultimate limit state design

In order to perform an ultimate limit state (ULS) design safety factors have to be introduced. An approach has been chosen which follows the concept of Eurocode 7 (2): “When considering a state of rupture or excessive deformation of a structural element or section of the ground, it shall be verified that the design value of the effects of all the actions is smaller or equal to the design value of the corresponding resistance.” The design values are derived from representative values applying partial safety factors. Eurocode 7 requires following one of three different “design approaches”. They differ in the size of the partial safety factors for actions (or effects of actions), for material properties (strength parameters) of the soil and for resistances (structural properties). Design approach 1 appeared impractical for the current problem, and design approaches 2 and 3 were investigated in more detail.

The design strength properties of structural materials and the design resistances of structural elements shall be calculated in accordance with other Eurocodes (EN 1992 to EN 1996 and EN 1999) for all three design approaches. (Eurocode 2 for concrete structures (10) requires a partial safety factor of 1.5 for the compressive strength of concrete and a partial safety factor of 1.15 for the yield strength of the reinforcing steel.)

In design approach 2 characteristic values of the soil strength parameters are used. Either the actions or the effects of actions have to be augmented by a partial safety factor (a value of 1.35 has been recommended for permanent actions). Additionally a partial safety factor on ground resistances like bearing capacity, sliding or earth resistance (which do not explicitly appear in a continuum mechanics approach) is used.

In design approach 3 the partial safety factor for permanent geotechnical actions is 1.0, whereas the soil strength parameters  $\tan \phi$  and  $c$ , with  $\phi$  being the friction angle and  $c$  being the cohesion, are reduced by a (recommended) factor of 1.25. The partial safety factor on ground resistances is 1.0.

The concepts of Eurocode 7 cannot be applied directly in FE-analyses with nonlinear material models. A common approach for determining the overall stability of slopes or excavation pits is following design approach 3 and performing analyses with partial safety factors on the soil strength parameters. The resistances of the structural elements (e. g. sheet pile walls, struts, anchors) have to be reduced by partial safety factors specific to the type of structural element (11).

This approach can be adapted to the current example easily: Partial safety factors on the soil parameters are used in combination with a partial safety factor on the compressive strength of the shotcrete. Live loads should be increased by the quotient of partial safety factor on variable actions and partial safety factor on permanent actions.

Design approach 2 cannot be implemented perfectly because ground resistances do not appear directly in FE-analyses. The Austrian guidelines RVS 9.32 (12) follow this approach in general (applying the partial safety factor on effects of

actions), and additionally request a parameter study for geotechnical parameters. So uncertainties in the ground resistances are covered indirectly. (If one ignores the safety factor on ground resistances, augmenting effects of actions together with reducing the strength parameters of the structural elements matches the concept of using global safety factors.) In the current example things are still more complex: There is no easy way to increase the sectional forces (i. e. the effects of actions) in the shotcrete lining: Increased sectional forces imply increased stress states which might be outside the current failure surface. The following remedy has been taken: Instead of augmenting the sectional forces (or stresses) in the shotcrete the strength parameters of the shotcrete have been reduced a second time.

Considering design approaches 2 and 3, for the ULS-design of the metro station the following two sets of partial safety factors have been applied:

partial safety factor for	analysis 1 (design approach 3)	analysis 2 (design approach 2)
permanent actions	1.0	1.0
soil strength parameters	1.25	1.0
shotcrete strength parameters	1.5	$1.5 \cdot 1.35 \approx 2.0$

The tensile strength of shotcrete has been neglected in both analyses. However, in order to incorporate the effects of the reinforcement (two layers of quadratic wire mesh with an area of 2.83 cm<sup>2</sup>/m each) in a simple way an artificial tensile strength of 500 kN/m<sup>2</sup> has been assumed. In uniaxial tension, application of this value results in a lower load carrying capacity of 30 cm of shotcrete than two layers of wire mesh. The model parameters chosen allow some tension softening; so the ductility of the steel is taken care of.

Both analyses showed convergence for all time steps investigated. Analysis 1 shows more pronounced displacements of the free surfaces at the face and larger plastic strains of the soil, whereas in analysis 2 the sectional forces in the shotcrete lining are limited to smaller values.

It has to be emphasised that a separate design of the reinforcement of the shotcrete lining, by feeding the most unfavourable combination of normal forces and moments into a design chart, can be omitted. The constitutive law of the shotcrete implicitly limits the sectional forces to values bearable by the lining. As soon as the sectional forces get close to the load bearing capacity of the lining, the material behaves softer, the stresses are redistributed to areas with lower stress levels, such allowing a more economic design. With the viscoplastic model, during short periods of time even stresses beyond the current failure surface are permitted. Local failure is detected, but results in divergence and termination of the calculation only if the ability of the structure to redistribute the stresses has been exhausted.

Although it has been tried to make conservative assumptions wherever possible, the numerical model contains assumptions on the unsafe side, as well. The most important ones are summarized below, together with countermeasures taken:

- The discretisation in space is too coarse: Using just one row of linear elements per round prevents deformation of the unsupported walls of the round at the face. At the intersection with the connection tunnel, the discretisation of the platform tunnels is finer. There the unsupported wall deforms slightly in inward direction, but without visible effect on the overall behaviour.
- At each analysis step, convergence is tested against a threshold of unbalanced forces. In order to be sure that the threshold is of reasonable size, the unbalanced force vectors – which have a maximum at the faces – have been compared with the resistance the support at the faces can provide and be shown as being harmless. An additional investigation of the face stability has been considered as dispensable.
- Viscoplastic models permit an excess of the prescribed ultimate stresses during short time intervals, and shear failure is not covered by the numerical model applied. The tensile strength has been set lower than the reinforcement chosen would have permitted. The stresses in the lining at selected excavation stages have been fed into a tool for ultimate limit state design; it has been confirmed that the reinforcement calculated by the tool (both in bending and shear) were lower than the chosen reinforcement.

Generally speaking, the effect of many support measures (face support, forepoling etc.) has been neglected, and helps in counteracting deficiencies of the numerical model.

## 6 Comparison of excavation techniques

### Jet grouting

In the course of the project development, and based on experience gained from the excavation of the running tunnels, the excavation scheme has been adapted. An umbrella of jet grouting columns serves as an additional primary support. The side drift of the platform tunnels has been omitted, and the full width of the cross section is excavated instead. With the help of additional analyses the effects of these changes were investigated. The omission of the side drift – without consideration of the positive effects of the jet grouting – yielded slightly higher settlements with a maximum of 16 to 19 mm during the excavation of the platform tunnels. The effects of jet grouting have been studied using a plane FE-model and applying the material law of section 2.3. A maximum of 51 columns – each modelled by two rectangular elements – with a diameter of 60 cm have been considered. The effects of different sequences of producing the columns have been

investigated, considering the removal of the soil as well as the different age of the columns. According to previous experience a time interval of 2 hours has been assumed for the production of one column. Applying the time scale due to the conservative temperature estimate of 12° C a fictive time interval of one hour had to be used. Another 5 fictive hours (10 real hours) have been added before the start of excavation (pre-relaxation step) and 7 more fictive hours until the full excavation. Each time step in the analysis lasts one (fictive) hour. After removal of the soil of one column the material is substituted by the (stress-free) jet-grouting material. In each time interval one new column is added, the other columns gain strength and stiffness.

Two different sizes of the umbrella – one with 37 columns and one with 51 columns were compared. Further, two different sequences of the production of the columns were investigated, see Figure 10. The results show that (with the expected soil properties at the construction site) a larger number of columns, i. e. columns also in the bench, has only a small effect on the settlements. A production sequence from one toe of the crown over the top to the other toe yielded smaller displacements at the ceiling and larger settlements than a sequence starting from one toe to the crown and then continuing from the second toe to the crown. With the described conservative assumptions about the heat generation and the speed of hydration, applying a jet grouting umbrella produces about 10 % higher surface settlements than an excavation scheme with a side drift. In the analyses, the production of the columns produces settlements which are not realistic (see the note on 2-D-models in section 2.3). For the comparison with the excavation scheme with side drift, this part of the settlements has been neglected. Whereas a later start of excavation after finishing the umbrella has little influence on the settlements and inclinations on the surface, a two times faster hydration reduces the settlements to the same level as the excavation scheme with a side drift.

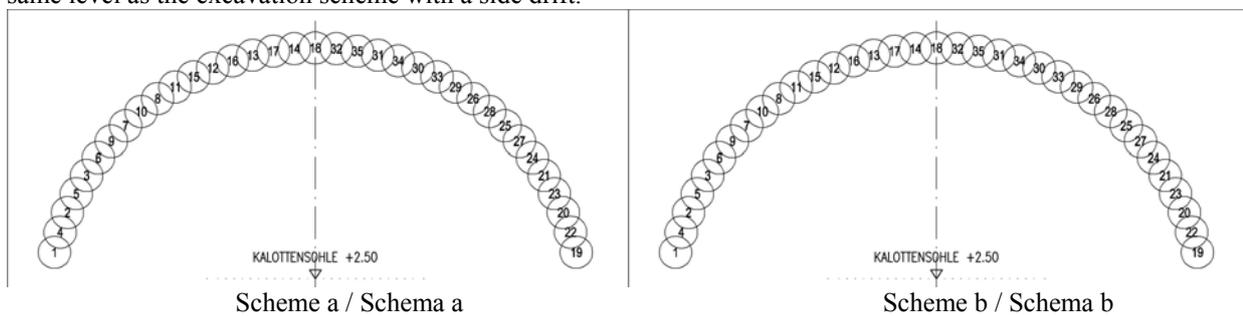


Figure 10. Construction sequence of the jet grouting umbrella with 37 columns

Bild 10. Herstellungsreihenfolge beim DSV-Schirm mit 35 Säulen

Jet grouting columns have also been applied to stabilise the face; analyses with improved soil properties at the locations of these columns have been used to quantify their effect and to verify the stability of the face.

### Speed of excavation

The jet grouting umbrellas allowed a faster progress of excavation. Additional analyses were performed in order to find out whether a speed of 7 m per day (in contrary to 2m per day according to the tender documents) could be permitted. The analyses showed that – under consideration that the shotcrete quality J3 according to (6) was used instead of J2 – the higher speed resulted in similar stresses in the shotcrete lining as with shotcrete quality J2 and the original excavation speed.

## 7 Measurement data

Figures 11 show contour bands of the surface settlements (generated from surveying data (13) at measurement points (indicated by white-filled circles and point numbers in green) by interpolation (14) at three characteristic instants of time which match the excavation stages of the calculated settlements at basement level in Figures 11. The first plot (Figure 11a), shortly after the end of the excavation of the two platform tunnels and before commencing the excavation of the connection tunnel, shows displacements mostly in the range of 20 to 30 mm above the platform tunnels. About 10 mm of the displacements are caused by lowering of the ground water table. The settlements have an increasing tendency from east to west, and have two maxima above track 1, one with 36 mm at the building Taborstraße 46 where a refilled old well had caused some trouble, and one with 38 mm in the vicinity of shaft Taborstraße.

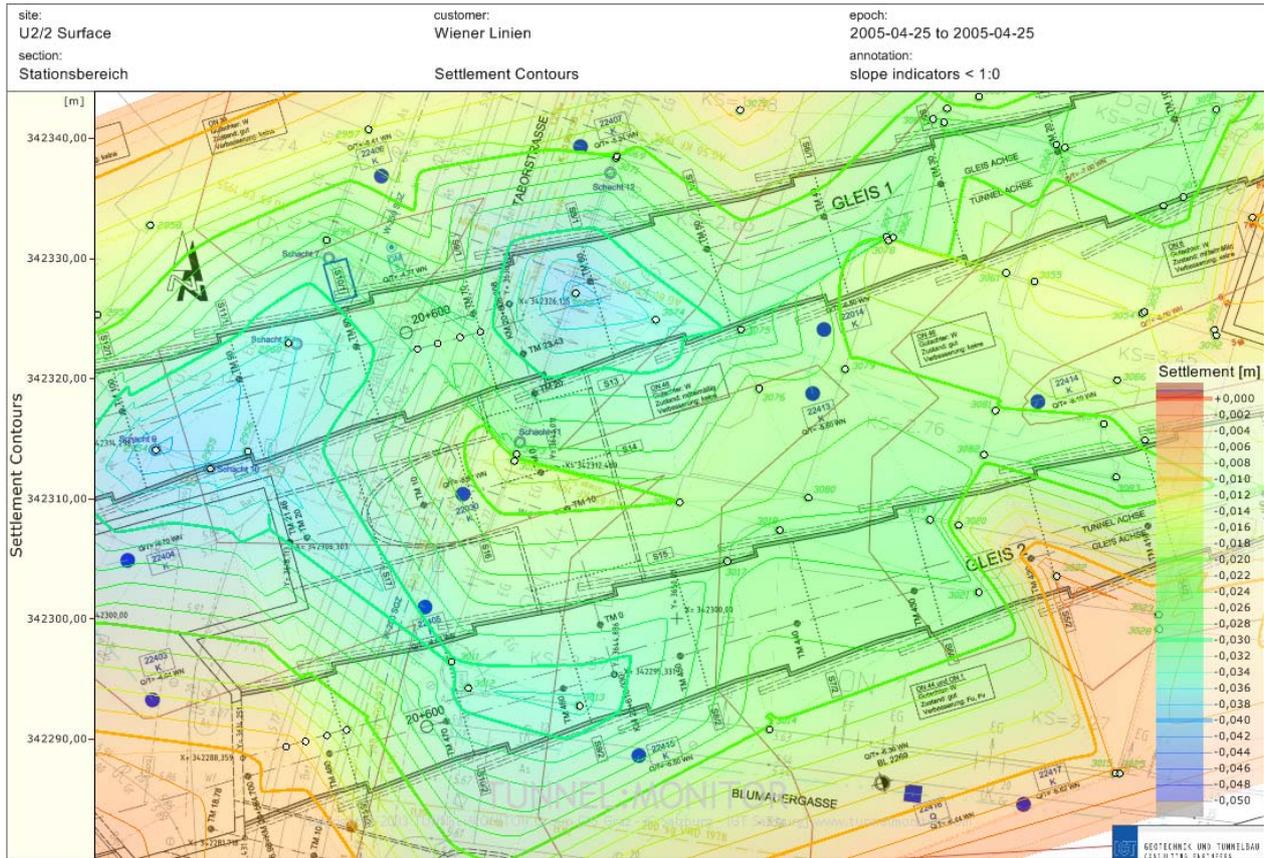


Figure 11a – Measured settlements after the end of excavation of the platform tunnels

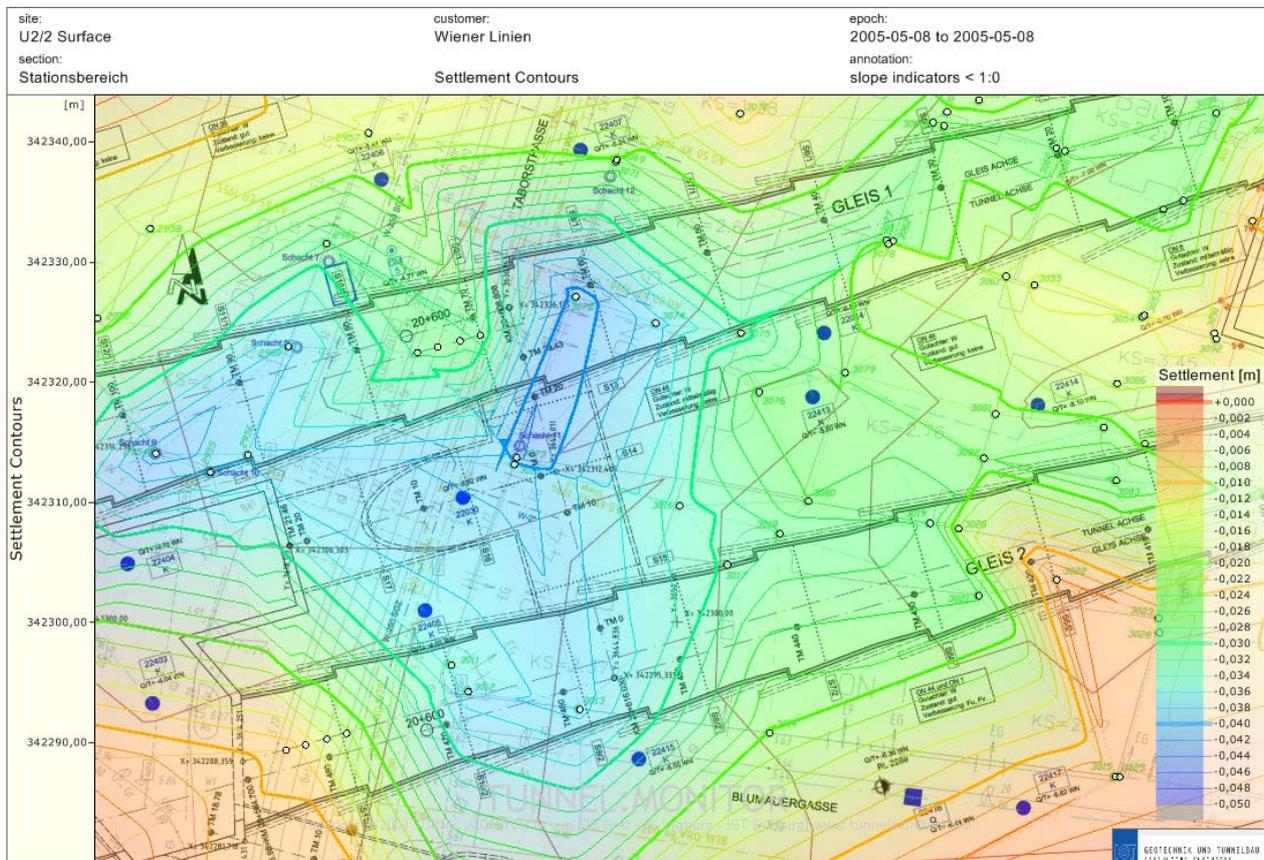


Figure 11b – Measured settlements after the end of excavation of the connection tunnel

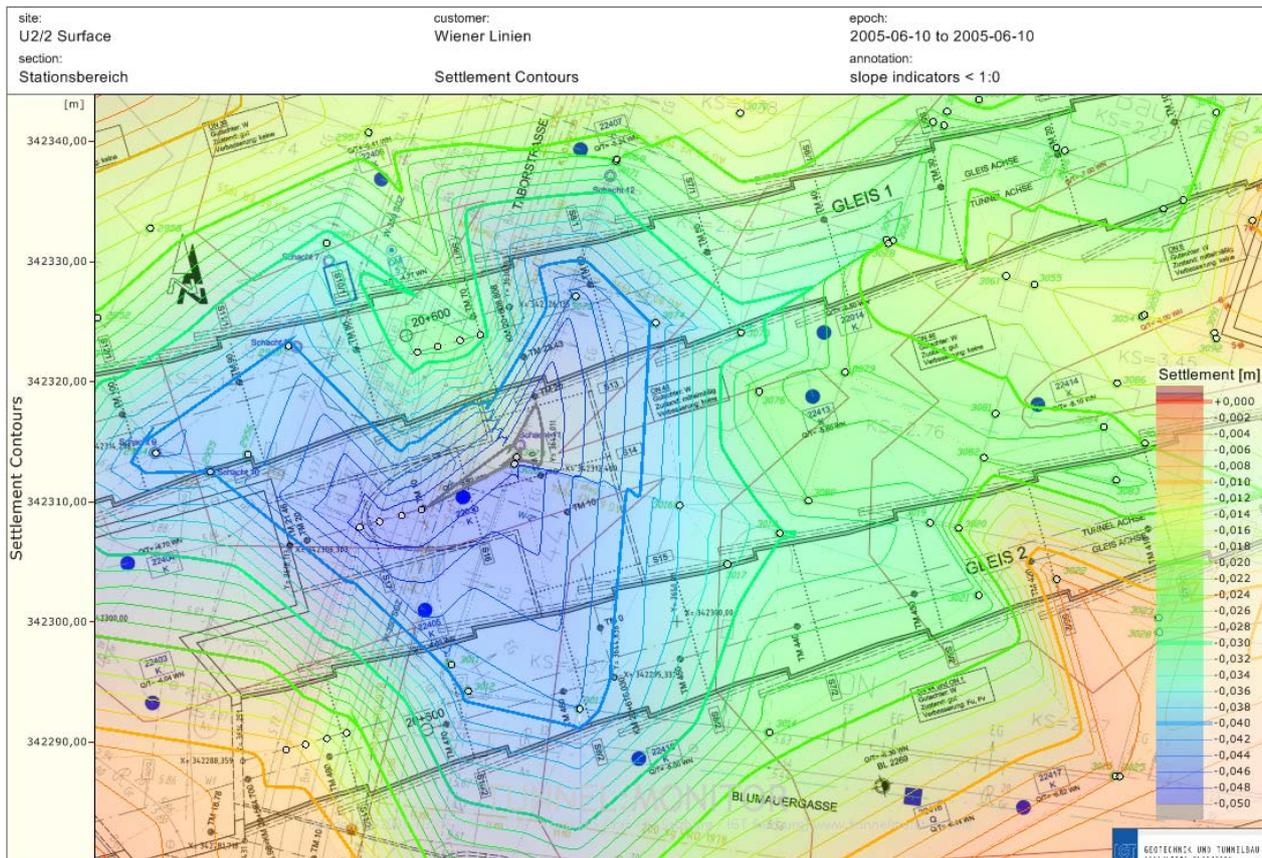


Figure 11c – Measured settlements after the end of excavation of the escalator tunnel

According to the analyses, displacements of 16 to 19 mm (for the analyses without side drift and 7 m /day excavation progress) had to be expected, excluding the effects of the ground water lowering. (The analysis results for the excavation with side drift of Figures 5 which have been published already in 2003 (15) show displacements between 14 and 17 mm for this stage of excavation.) In the vicinity of the shaft, the analyses yielded 21 mm settlements – 4 mm more than elsewhere.

After the excavation of the connection tunnel, the maximum measured settlements increased to 40 mm, again near the building Taborstraße 46. According to the analyses with side drift the excavation of the connection tunnel augments the maximum displacements by 8 mm.

The third plot, containing the measurements about two weeks after finishing the excavation of the escalator tunnel, shows a considerable increase of the surface settlements, not only directly above the escalator tunnel, also above the platform tunnels in the vicinity of the escalator tunnel a remarkable increase of displacements, up to about 7 mm, can be observed. This type of behaviour was not predicted by the analyses: There the settlement trough caused by the excavation of the escalator tunnel is narrow enough not to influence the maximum settlements above the axes of the platform tunnels.

The geologic conditions seem to favour wide settlement troughs; this is also confirmed by the fact that at some locations the settlement troughs of the two tracks almost combine to one wide settlement trough.

As a result of the wider settlement troughs, despite locally higher settlements than predicted by the analyses, the admissible inclinations parallel and normal to the building walls were met.

The trigonometric measurements in the tunnels showed displacements which were considerably lower than the calculated values. However, a comparison of measurements and analyses is difficult, because only the part of the displacements can be measured which occur after installation of the measurement equipment on the shotcrete lining and the first reading of the coordinates.

## 8 Summary and conclusions

The use of three-dimensional numerical models considering the nonlinearities of soil and primary support allows an economic design of the linings at tunnel intersections. A material law has been applied which allows an implicit design of the shotcrete lining, i.e. the material law itself guarantees that the strength of the shotcrete is not exceeded at any of the excavation stages. Following the concepts of Eurocode 7, a method has been described with which sufficient safety margins against failure, as required in an ultimate limit state design, either of the soil or the shotcrete, can be achieved. The effects of different excavation schemes, the speed of excavation, and of jet grouting columns on the stresses in the shotcrete lining and on the settlements have been investigated.

The analyses results using characteristic values of the soil and shotcrete parameters have been compared with measurements at the construction site. Reasonable agreement has been achieved, with the limitation that the width of the settlement troughs has not been predicted correctly. More advanced soil models might be required to overcome this shortcoming.

For jet grouting columns, a simple model has been presented. It is based on experimental data and can be implemented into existing computer codes in an easy way.

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