Design of the shotcrete tunnel lining of a metro station – safety considerations

H. Walter
IGT – Geotechnik und Tunnelbau, Consulting Engineers, Salzburg, Austria

ABSTRACT: In the course of the extension of a metro line in Vienna the station Taborstraße will be built. Two single track platform tunnels, a connection tunnel and an elevator tunnel will be driven through sediments following an NATM excavation scheme with part-excavations of the cross sections.

Time dependent three-dimensional finite element analyses using nonlinear material models have been performed, in order to predict the settlements due to the excavation of the tunnels and to design the shotcrete lining. Features of the viscoplastic shotcrete model are: a rapid development of strength and stiffness, cracking, and a pronounced creep at high stress levels.

At the tunnel intersections stress concentrations in the lining occur which would not be permitted according to the design rules of common standards. Variations of the material parameters of the shotcrete and the soil are used to show that safety margins comparable to those of the standards are present.

1 INTRODUCTION

In the course of the extension of the metro line U2 in Vienna the station Taborstraße will be built. The project is in the tender stage at the end of 2002 (Wiener Linien 2002), the line will go into operation in 2008. Client is the "Wiener Linien", the Vienna Transport Authority.

The station consists of two parallel single track platform tunnels with a cross section of 74 m² each. The platform tunnels are connected by a connection tunnel (cross section 59 m²) from which an escalator tunnel (cross section 63 m²) provides access to the street level. Two shafts, a large one at the western end of the station, the shaft "Taborstraße", and a small one at the eastern end between the platform tunnels, the shaft "Novaragasse", will be 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 of the station.

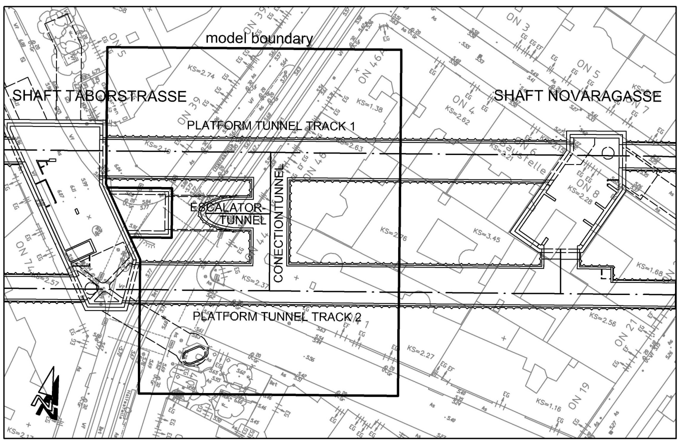

Figure 1. Plan view of the station and the model boundaries.

The tunnels will be driven through sediments of the Quarternary and Tertiary following an NATM excavation scheme. In order to minimise settlements the cross section of the platform tunnels will be divided into two main parts: A side drift, subdivided into top heading, bench and invert, see figure 2, will be excavated first; then the remaining cross section will be excavated with the same subdivision.

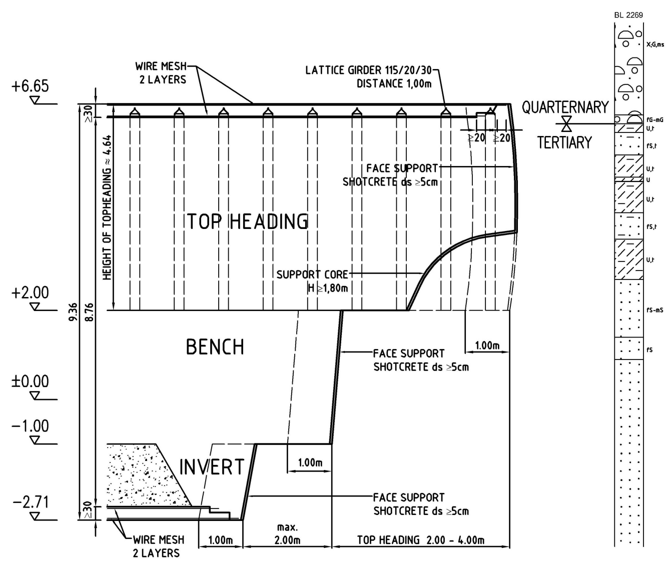

Figure 2. Excavation scheme: side drift of the platform tunnels.

Primary support will be a shotcrete lining with a thickness of 30 cm, and 25 cm at the temporary walls, respectively. The connection tunnel and the escalator tunnel will be subdivided into top heading, bench and invert, as well; the lining thickness is 30 cm, again. The linings will be reinforced with two layers of wire mesh.

Three-dimensional finite element 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.

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". T. 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. A discussion of these results leads to the observation that a design of the shotcrete lining following the procedures in standards like the Eurocode (CEN 2001, CEN 2002) can hardly be performed using the internal forces resulting from the analysis.

Therefore, the last chapter deals with the implications of nonlinear material and structural behaviour on the interpretations of the design rules specified in the standards. With the help of additional analyses it will be shown that sufficient safety margins against failure of the structure are present if the shotcrete lining with the dimensions described above is used as support.

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.

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

Table 1. Material properties of the soil layers.

Soil layer		Quarternary	Tertiary (Silt)	Tertiary (Fine sand)
Description		gravel, sandy	silt, clay	sand, silty
Lower boundary	m above TOR	~ 6	~ 3	-
Specific weight (wet)	kN/m³	20	20	20
Specific weight (under water)	kN/m³	11	11	11
Modulus of elasticity	MN/m²	150	50	150
Poisson's ratio	1	0.28	0.35	0.30
Friction angle	°	35	25	27.5
Cohesion	kN/m²	0	50	0

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. The parameters have been chosen as cautious estimates of the mean value in the sense of Eurocode 7 (CEN, 2001). 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/m²) and the buildings (130 kN/m²), 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, and extended by additional creep and shrinkage terms, has been applied.

2.2.1 *Meschke-Mang-model*

Detailed descriptions of the Meschke-Mang-shotcrete model can be found in the literature (Meschke 1996; Meschke et al. 1996). It has already been applied for 3-D-tunnel analyses (Mang et al. 1994). The main features of the model are:

A strain-hardening Drucker-Prager loading surface with a time dependent hardening parameter to account for 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. Two independent hardening and softening mechanisms control the constitutive behaviour of shotcrete subjected to compressive and tensile stresses, respectively.

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 extension of the inviscid elastoplastic model for aging shotcrete to viscoplasticity is based upon the model by Duvant and Lions.

A numerically efficient algorithmic formulation of multisurface viscoplasticity in principal axes results in a robust implementation for engineering applications.

2.2.2 *Refined model*

The Meschke-Mang-model describes creep effects with one single parameter, the viscosity. At stress levels within the current yield surface, no creep or relaxation occurs. Therefore, the adaptability to experimental creep or relaxation data is limited.

In the course of a research project (Walter et al. 1996), experiments with shotcrete specimens were conducted at the Mining University Leoben, Austria. In the tests, shotcrete prisms were loaded at an age of only 6 hours. The specimens were subjected to a large variety of load paths, some with load control and some with displacement control.

In order to match the test results, it was necessary to extend the Meschke-Mang-model. A simple engineering approach was chosen:

A model already used for the description of the long-term shotcrete behaviour (Schubert, 1988) is used to define additional creep terms: It is based on the rate of flow-method (England & Illston, 1965).

For each of the principal stresses creep strain increments for the time interval (t_i, t_{i+1}) are calculated as

$$\Delta e_{CR,u} = s_{t_{i+1}} \cdot \Delta C(t) \cdot e^{k \cdot s_{t_{i+1}}} + \Delta e_d \tag{1a}$$

with

$$\Delta e_d = \left(s_{t_{i+1}} \cdot C_{d\infty} - e_{d,t_i} \right) \cdot \left(1 - e^{\frac{-\Delta C(t)}{Q}} \right), \tag{1b}$$

$$\Delta C(t) = C\left(t_{i+1}\right) - C\left(t_i\right) \tag{1c}$$

and

$$C(t) = A \cdot t^{\frac{1}{3}} . \tag{1d}$$

In equations (1) denote

$\Delta e_{CR,u}$	the increment of creep strains in the time interval from $t_1$to t_{i+1}
$s_{t_{i+1}}$	the principal stress at time t_{i+1}

and

Δe_d	the increment of 'delayed elastic strain', e_d, i.e. of the reversible creep component, in the interval from t_i to t_{i+1}

The other variables are model parameters:

A	describes the amount of (irreversible) creep
$C_{d\infty}$	is the ultimate value for the reversible creep compliance
Q	describes how fast the delayed elastic strains develop
k	is used to introduce a nonlinearity for high stress levels.

With the help of a Poisson's ratio for creep, n_{CR}, the creep strains in the principal directions are related to each other:

$$e_{1,CR} = e_{1,CR,u} - n_{CR}\left(e_{2,CR,u} + e_{3,CR,u}\right) \tag{2a}$$

$$e_{2,CR} = e_{2,CR,u} - n_{CR}\left(e_{1,CR,u} + e_{3,CR,u}\right) \tag{2b}$$

$$e_{3,CR} = e_{3,CR,u} - n_{CR}\left(e_{1,CR,u} + e_{2,CR,u}\right) \tag{2c}$$

In equations (2) the first index of the strain values denotes the index of the principal direction.

For the back transformation of the principal values of the creep components into global directions, the same transformation matrix as for the principal stresses is used.

The three-dimensional extension of the creep law is somewhat arbitrary. Due to the lack of experimental data for creep under multiaxial stress states this simple generalisation seems to be justified. As a first guess, the value of the creep Poisson's ratio has been set equal to the Poisson's ratio for shotcrete.

Details of the implementation can be found in (Walter et al. 1996, Walter 1997). The refined model contains shrinkage terms according to the relation

$$e_{sh} = e_{sh,\infty} \cdot \frac{t}{B + t} \tag{3}$$

with e_{sh} being the volumetric shrinkage strains at time t, $e_{sh,\infty}$ the ultimate value of the shrinkage strains and B a parameter describing the development of the shrinkage strains with time.

2.2.3 *Material parameters for shotcrete*

Table 2 shows the material parameters chosen for the shotcrete. They match the properties of the shotcrete type SpB 25(56)/J2 (Betonverein 1998) which has been specified as a minimum requirement in the tender documents, very well, see figure 3. Different amounts of creep have been specified in the course of a parameter study. The values given here yield a relatively small amount of creep strains and little stress reduction due to creep. Thus, a conservative stress level is achieved. Figure 4 shows the effects of varying the parameter A. (A = 0 results in using the original Meschke-Mang-model.)

Table 2. Material parameters of shotcrete.

Parameter	Symbol	Unit	Value
Specific weight	γ	kN/m³	25
Poisson's ratio	ν		0.20
Cylinder compressive strength at 28 days	$f_{cu}^{(28)}$	kN/m^2	17 000
Cylinder compressive strength at 1 day	$f_{cu}^{(1)}$	kN/m^2	4500
Yield stress at 28 days	$f_y^{(28)}$	kN/m^2	3000
Viscosity parameter	η	15 h	
Modulus of elasticity at 28 days	$E^{(28)}$	MN/m^2	25 000
Factor describing the amount of irreversible creep	A	m²/kN·h$^{-1/3}$	$1.0 \cdot 10^{-8}$
Ultimate value for the reversible creep compliance	$C_{d\infty}$	m²/kN	$1.5 \cdot 10^{-7}$
Factor describing how fast the delayed elastic strains develop	Q	m²/kN	$4.0 \cdot 10^{-8}$
Factor to introduce a nonlinearity for high stress levels	k		0

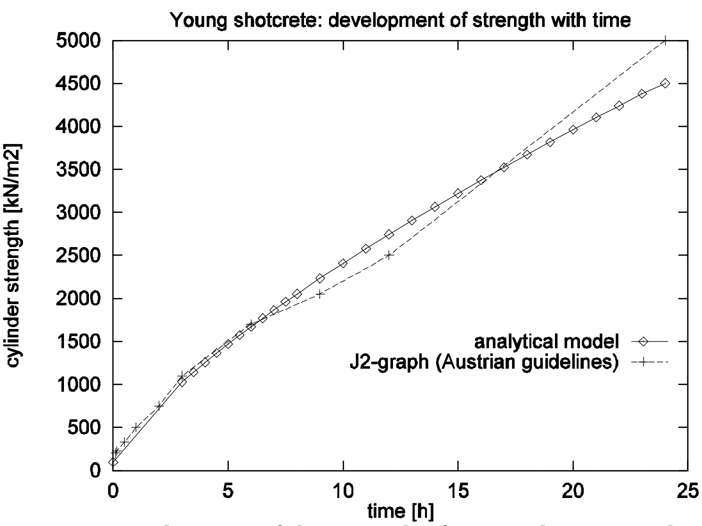

Figure 3. Development of the strength of young shotcrete with time.

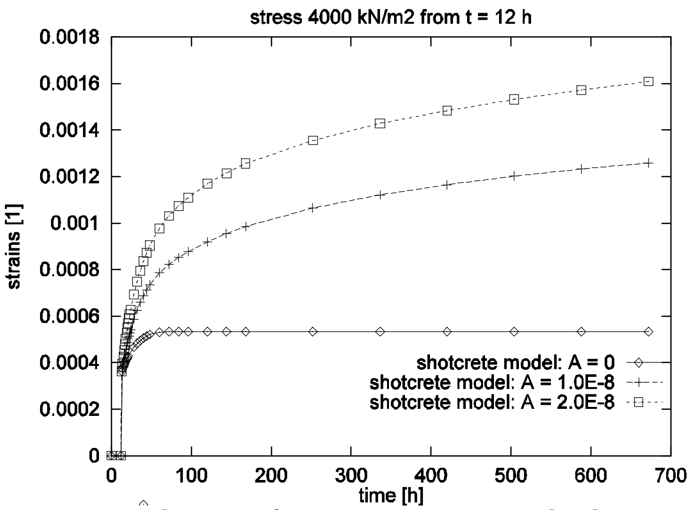

Figure 4. Development of strains at constant stress level.

3 EXCAVATION SEQUENCE, DISCRETIZATION 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. According to the construction schedule a progress of two rounds per day, i.e. one fictitious round with a length of 2 m in 24 hours, has to be expected. 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 will be driven in the same direction in the analysis. 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.

The excavation of the connection tunnel and of the escalator tunnel is simulated in two phases, again: first the top heading is removed; two rounds later bench and invert are excavated together. The direction of the driving of the connection tunnel has been chosen in the direction from track 2 to track 1 because preliminary analyses have shown that this direction minimizes the settlements and inclinations of the buildings above the station. The excavation of the escalator tunnel starts at the connection tunnel. Before the excavation of the connection tunnel and after the end of the excavation process extra time steps with increased length have been added. They cover an additional period of one month each, with the goal to observe the amount of stress redistributions caused by creep of the shotcrete.

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.

4 RESULTS USING CHARACTERISTIC PARAMETERS

With the parameters of tables 1 and 2 the station has been analysed. At most of the time steps convergence was achieved after 3 or 4 iteration steps using a tolerance of 1 % for the residual forces

4.1 *Settlements*

The figures 5 show the calculated settlements at the level of the basements of the buildings. The basement walls are indicated on the plots.

Figure 5a shows one characteristic step, analysis step 29, during the excavation of the two platform tunnels: The top heading of the side drift of track 1 has proceeded 54 m into the model, the remaining cross section 18 m. The top heading of the side drift of track 2 has advanced 34 m into the model, and the first layer of the remaining cross section has been excavated.

Figure 5b shows an instant at the end of the excavation of the connection tunnel, figure 5c depicts the settlements at the end of the analysis.

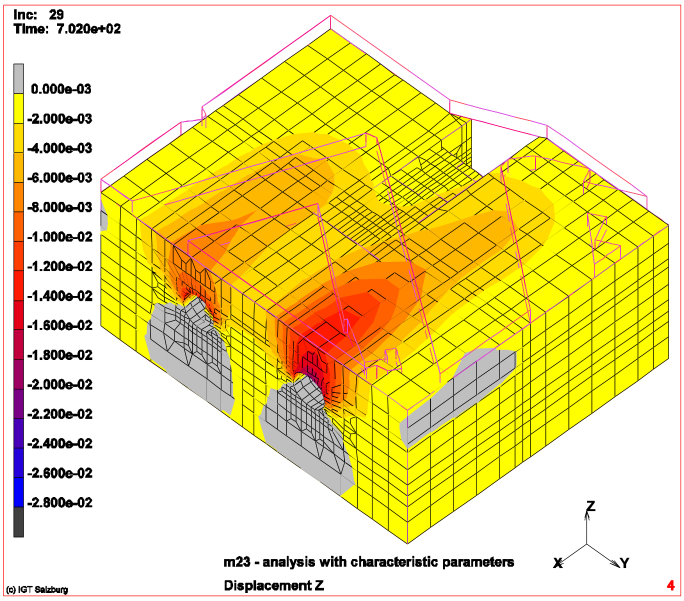

Figure 5a. Vertical displacements at analysis step 29.

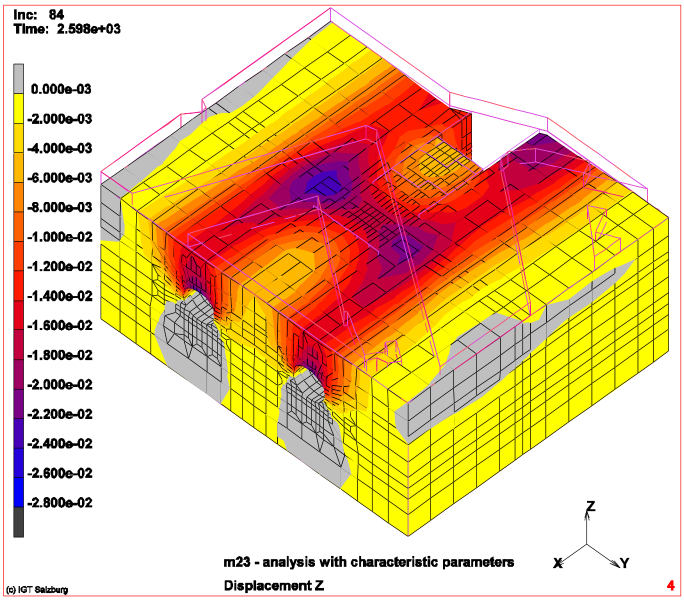

Figure 5b. Vertical displacements after excavation of the connection tunnel.

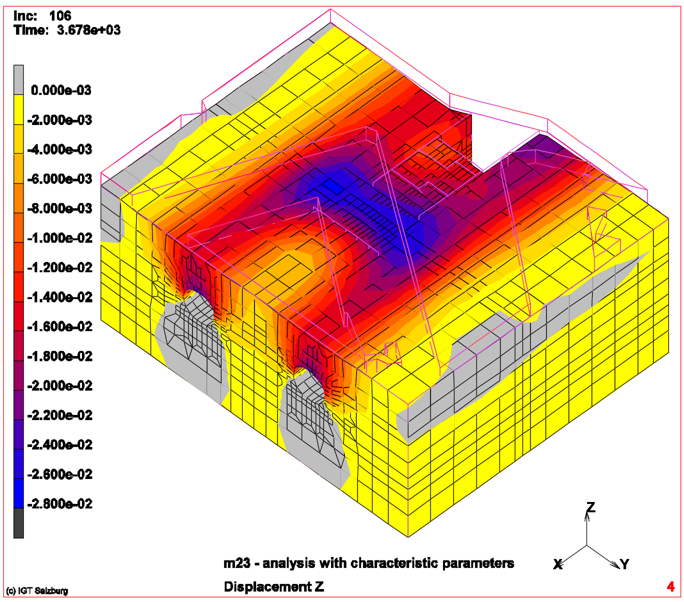

Figure 5c. Vertical displacements at the end of the analysis.

Directly above the face of the top heading of the first side drift the settlements are about 6 mm, they increase to 13 to 16 mm after the excavation of the remaining cross section. The maximum inclinations are about 1:700. The settlements increase to 21 mm above track 1, and to 24 mm above track 2 during the excavation of the connection tunnel. The excavation of the escalator tunnel creates additional settlements westerly of the connection tunnel. The maximum settlement above track 2 increases to 27 mm.

Of special interest are the maximum inclinations within the plan area of the buildings. They reach a maximum of 1:500 which is the maximum value allowed by the authorities.

The long term effects of creep of the shotcrete are small: The maximum settlements increase by less than 1 mm during the periods without excavation before commencing the connection tunnel and at the end of the analysis.

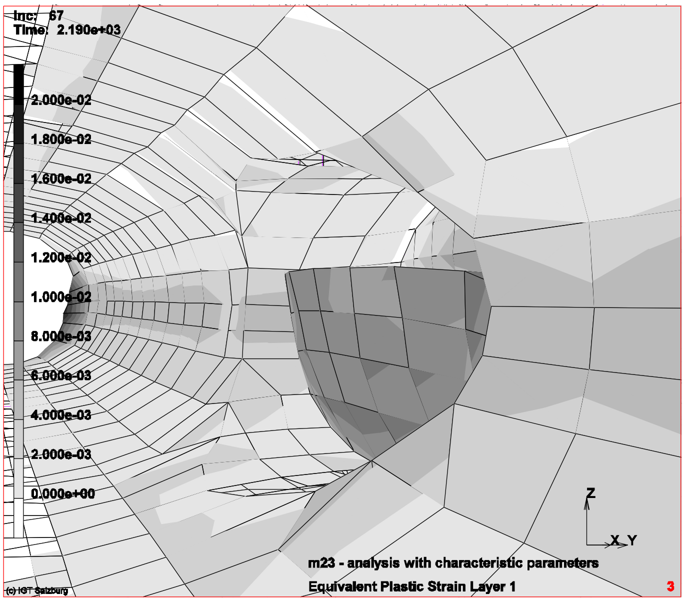

Figure 7. Equivalent plastic strains in the soil at the beginning of the excavation of the connection tunnel (analysis step 67). (Displacements magnified 50 times.)

4.2 *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/m²) 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.

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 in analysis step 77; the maximum face displacements for the escalator tunnel are 56 mm, again at the bench.

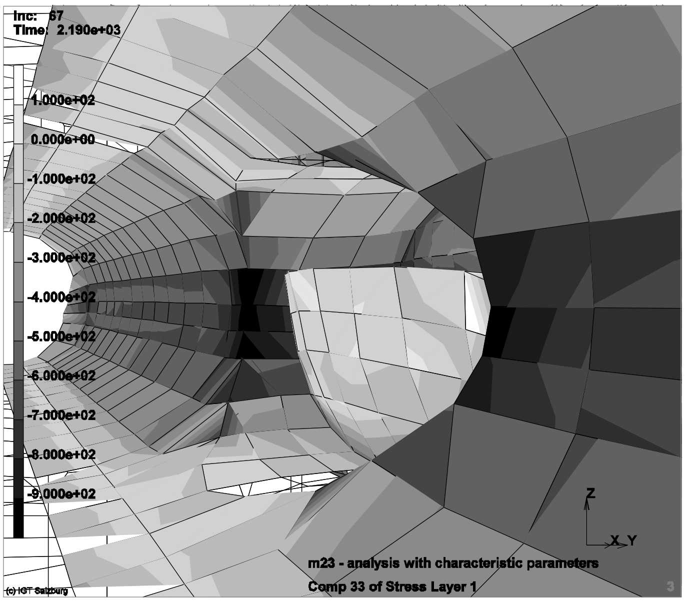

Figure 6. Vertical stresses in the soil at the beginning of the excavation of the connection tunnel. (Displacements magnified 50 times.)

4.3 *Stresses in the shotcrete lining*

The following figures contain 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. As can be observed in figures 9a to 9c the application of the shotcrete lining in the connection tunnel together with creep effects diminishes the stresses. The stresses in the young shotcrete of the lining of the connection tunnel are considerably lower than those in the platform tunnel.

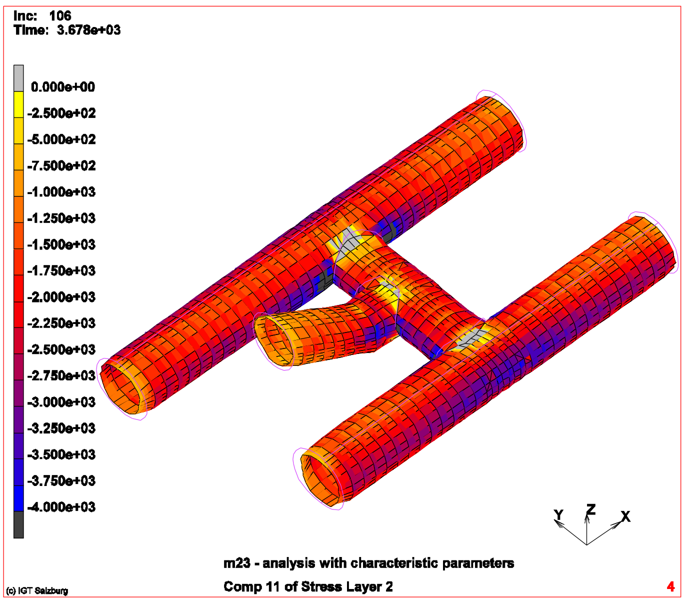

Figure 8. Normal stresses in the middle layer of the shotcrete lining in circumferential direction at the end of the analysis

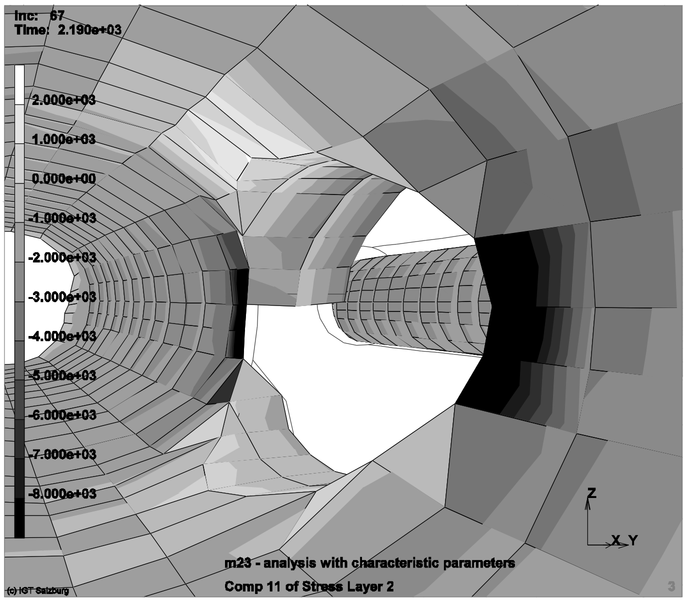

Figure 9a. Normal stresses in the middle layer of the shotcrete lining in circumferential direction at analysis step 67. (Displacements magnified 50 times.)

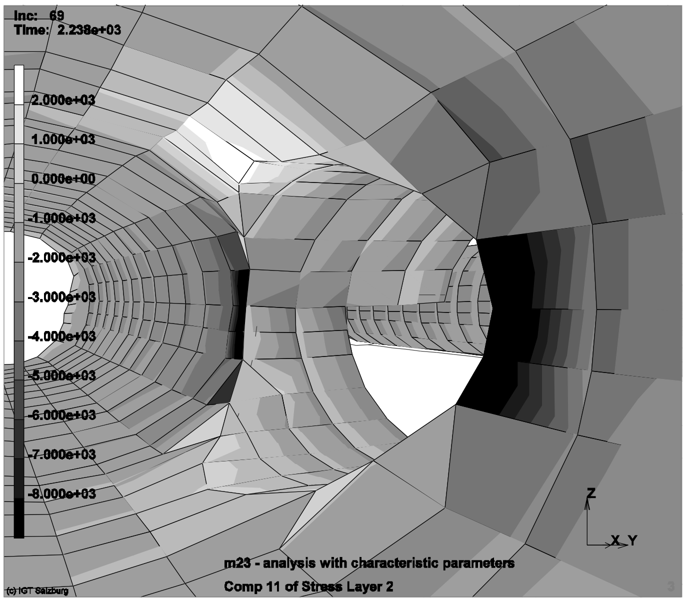

Figure 9b. Normal stresses in the middle layer of the shotcrete lining in circumferential direction at analysis step 69.

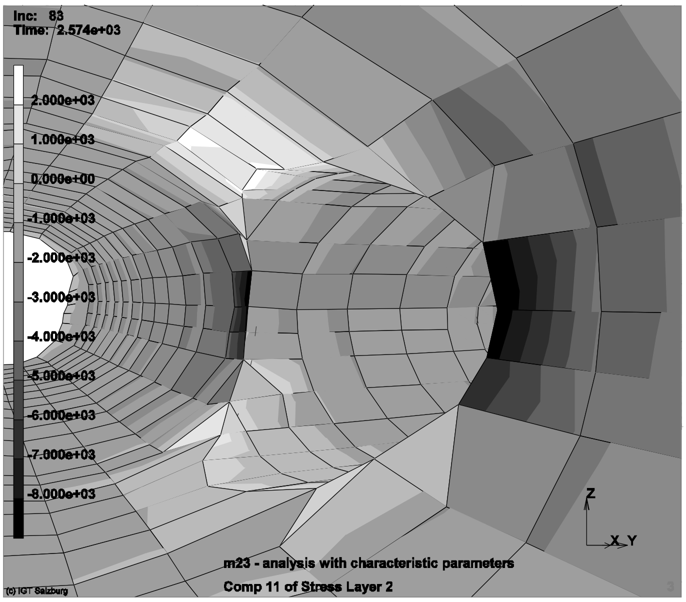

Figure 9c. Normal stresses in the middle layer of the shotcrete lining in circumferential direction at the analysis step 83.

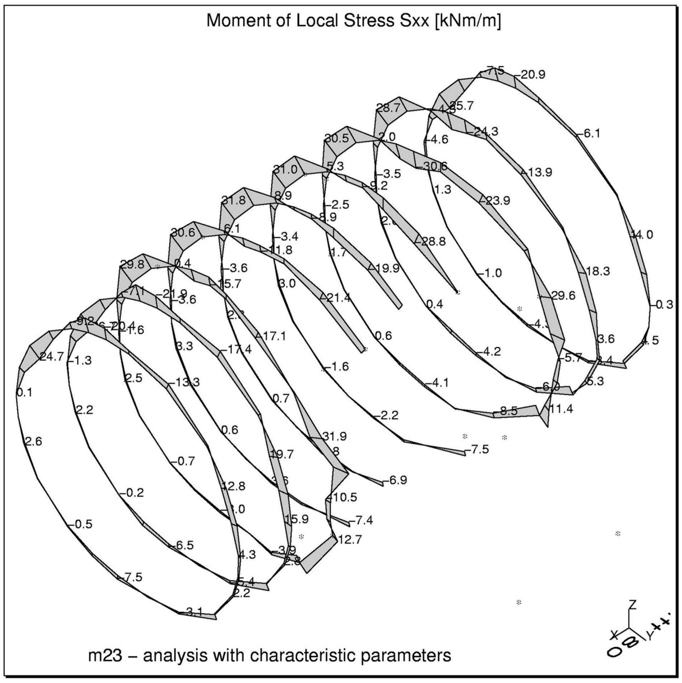

Figure 10. Bending moments in the lining resulting from stresses in circumferential direction at the end of the excavation of the connection tunnel.

The moments in the regions of maximum normal stresses in the middle layer are relatively small (Fig.10). 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.

5 DESIGN OF THE SHOTCRETE LINING, ADDITIONAL ANALYSES

The results of the previous chapter show that both, the soil and the shotcrete receive loads up to their load carrying capacity in the course of the excavation. It is clearly visible that the regions with high stress levels are locally confined. The plasticity-based mechanisms for dealing with stresses at yield built into the constitutive laws for soil and shotcrete enforce stress redistributions and prevent overloading of the soil or the shotcrete. Creep effects reduce the maximum stresses in the lining further. The good natured convergence behaviour at all analysis steps indicates that the ‘structure’ – consisting of soil and shotcrete lining – is able to carry the applied loads and that the capacity for stress redistribution is sufficient.

According to the design rules of common standards, e.g. the Eurocodes (CEN 2001, CEN 2002), however, stress concentrations in the shotcrete lining with stress levels close to the compressive strength would not be permitted and the design be considered as unsafe. There are also cross sections where a standard design would require more reinforcement than the amount which can be placed on site. Additionally, Eurocode 7 (CEN 2001) would require a check on the safety of the soil in addition to the safety of the lining.

The question now arises whether the ‘structure’ is safe in the sense of the standards, whether there is a sufficient safety margin against failure, i.e. whether the probability of failure is low enough.

In order to check the available safety margins a number of different approaches can be considered, among them:

– Analyses with characteristic parameters for shotcrete and soil, followed by a standard design of the reinforcement as described above.
– Analyses with increased loads, e.g. by increasing the specific weight and the building loads, and reducing the strength properties of the shotcrete.
– Decreasing the strength of the soil and reducing the strength properties of the shotcrete.
– Reducing only the strength of the shotcrete, but by an increased amount compared with the approach above.

The first approach is not applicable, at least not for all parts of the structure, for the reasons described above.

The second and the third approaches fit into the concept of applying partial safety factors to actions and resistances, respectively (CEN 2001, CEN 2002). The fourth approach resembles the concept of a global safety factor used in many older standards, and is similar to the first approach. In both approaches the safety of the soil is not investigated. The first approach has the advantage of yielding realistic displacements and deformations, whereas the fourth approach contains a thorough check on the load-redistribution capacity of the structure including also the soil.

In the opinion of the author the second approach is far off the physical reality. It has not been further investigated.

The third and fourth approaches are investigated by additional analyses. For the analyses the following safety factors have been chosen in accordance with (CEN 2001, CEN 2002) and applied to the strength parameters of the soil, cohesion and tangent of the angle of friction, and of the shotcrete, yield and ultimate stress in tension and compression:

Table 3: Safety factors used in different approaches

(Partial) safety factor	First approach	Second approach	Third approach	Fourth approach
Soil – cohesion	1.0	1.0	1.25	1.0
Soil – shearing resistance	1.0	1.0	1.25	1.0
Soil – dead weight	1.0	1.35	1.0	1.0
External loads	1.0	1.35	1.0	1.0
Analysis: shotcrete tensile strength	1.0	1.5	1.5	1.7
Analysis: shotcrete compressive strength	1.0	1.5	1.5	1.7
Internal forces and moments	1.35	1.0	1.0	1.0
Cross section design: Yield stress of reinforcement	1.15	1.0	1.0	1.0
Design: Compressive strength of the shotcrete	1.5	1.0	1.0	1.0

A reinforcement has not been specified in the analyses in order to avoid an overestimation of the load carrying capacity in tension. (The softening in the model is only a coarse approximation of the brittleness of the shotcrete in tension). Therefore, the safety factors for the resistance of the lining chosen for the analysis and those chosen for the design of the cross section are not directly comparable.

The differences between the approaches can be visualized by comparing the figures 12 (for the third approach) and 13 (for the fourth approach) showing results for step 67, with the figures 7 and 9a (for the first approach).

The displacements, especially the inward movement of the face of the bench, are higher than in the first approach. One reason is that higher plastic strains develop in the third approach because of the smaller yield surface; another reason is the higher stress level in the soil in the fourth approach because of the weaker lining. Both in terms of displacements and plastic strains, the third approach yields more unfavourable results than the fourth approach.

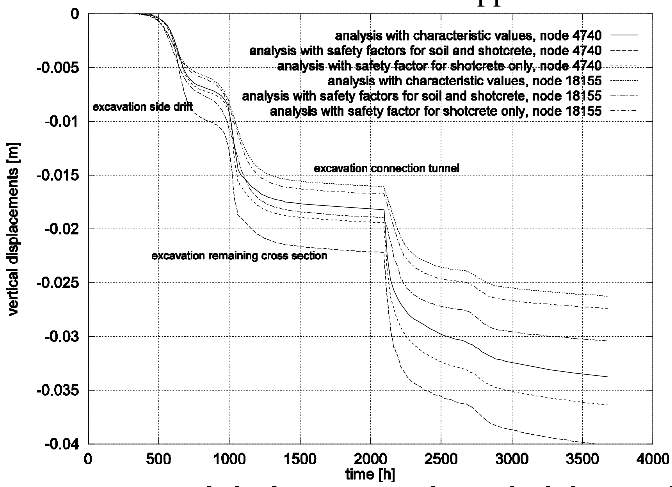

Figure 11. Vertical displacements at the roof of the tunnel (node 4740) and at basement level (node 18155) at the intersection of track 2 and the connection tunnel.

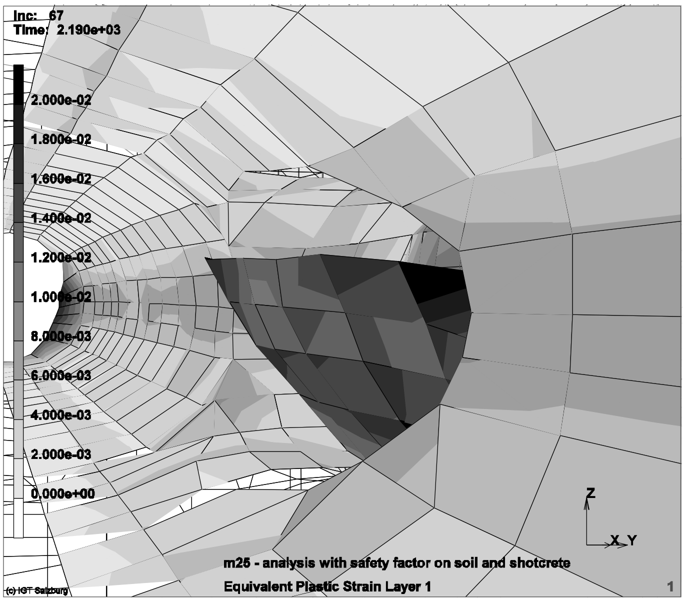

Figure 12a. Third approach: Equivalent plastic strains in the soil at step 67.

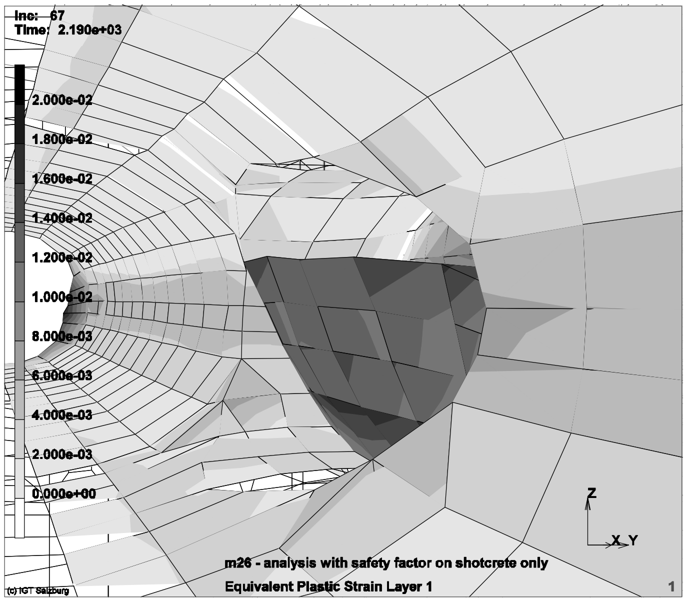

Figure 13a. Fourth approach: Equivalent plastic strains in the soil at step 67.

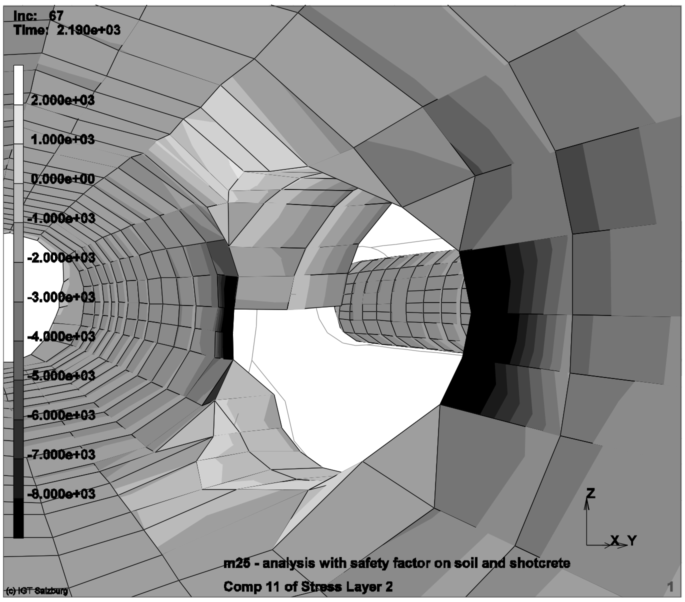

Figure 12b Third approach: Normal stresses in circumferential direction in the middle layer of the lining at step 67.

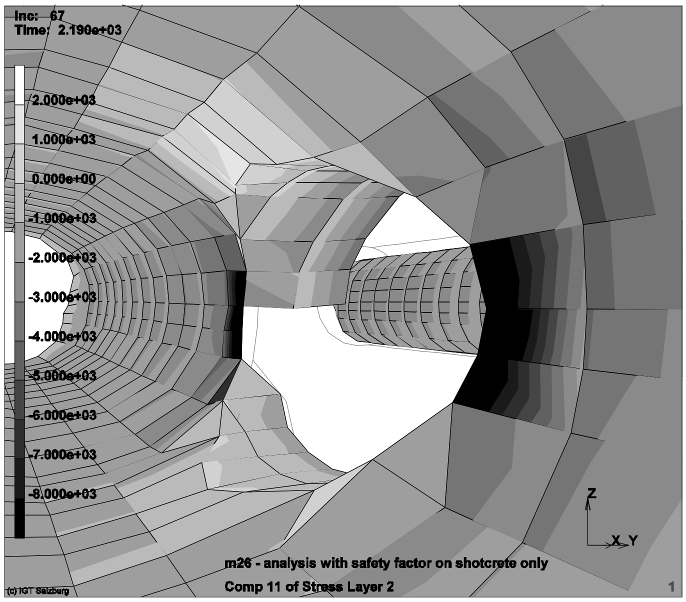

Figure 13b. Fourth approach: Normal stresses in circumferential direction in the middle layer of the lining at step 67.

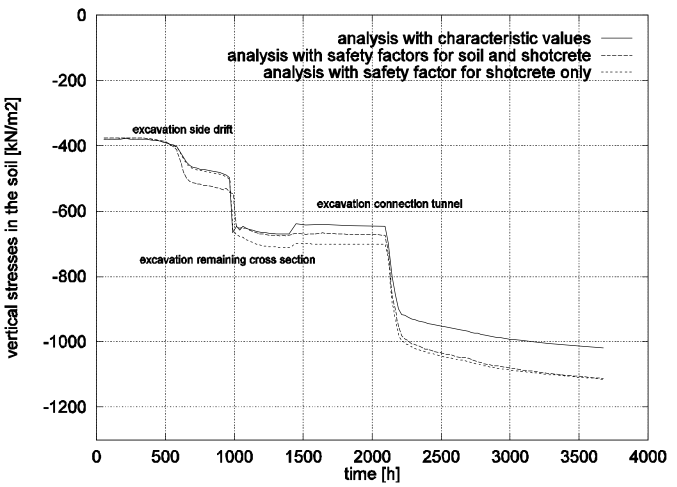

Figure 14. Vertical normal stresses at the bench in the corner with positive x-coordinate of the intersection of track 2 and the connection tunnel.

The vertical stresses in the soil are very similar: the lowest stresses yields the first approach, there is almost no difference between the third and fourth approach (Fig. 14).

The smaller the compressive strength of the shotcrete, the lower are the maximum normal stresses and the wider is the area of increased stress levels in the shotcrete lining. Due to the local confinement of the stress maxima the differences are not very obvious on the plots. (More pronounced are the differences in the regular parts of the platform tunnels visible in the back of figures 12b and 13b. The differences there originate from the different load carrying capacity of the soil and the resulting stress transfer to the lining.)

The moment diagrams in figures 15 confirm the observations made at figure 10: At cross sections with high normal stresses the moments are the smaller the smaller the compressive strength of the shotcrete because the whole cross section has reached its load carrying capacity. At cross sections with smaller normal forces the moments of the first and third approaches are very similar, only the fourth approach shows visibly smaller moments.

The number of iteration steps required for convergence is slightly increased in the third approach which is obviously due to the larger amount of plastification at the free surfaces of the soil. None of the analyses indicates a divergent behaviour.

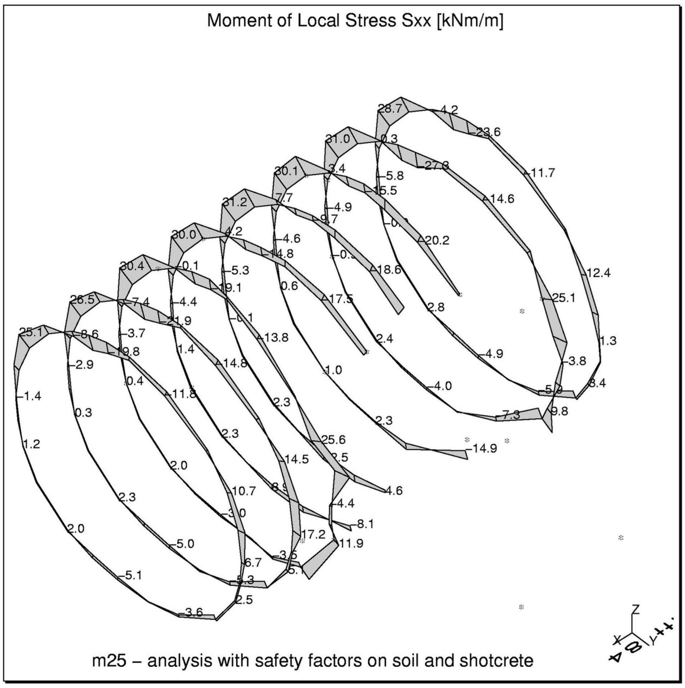

Figure 15a. Third approach (safety factor on soil and shotcrete): Moments in circumferential direction in the lining of track 2 in the vicinity of the intersection with the connection tunnel at the end of the excavation of the connection tunnel.

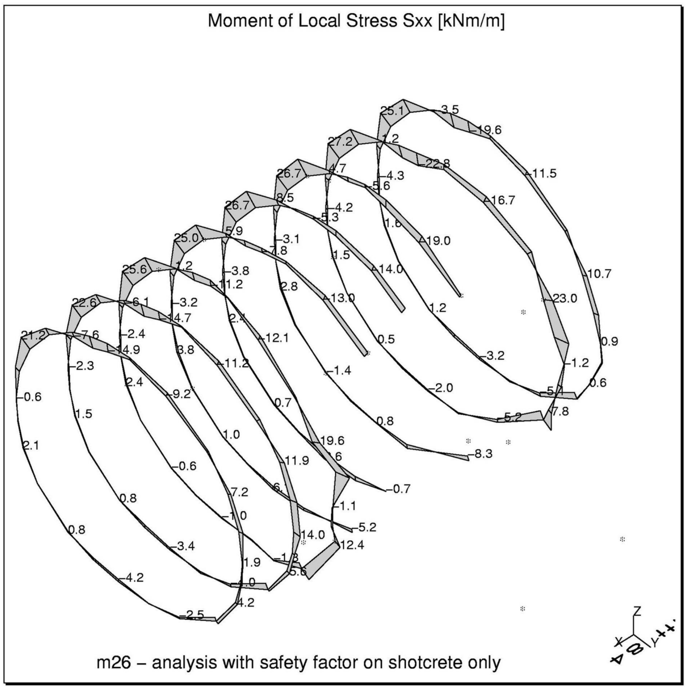

Figure 15b. Fourth approach (safety factor on shotcrete only): Moments in circumferential direction in the lining of track 2 in the vicinity of the intersection with the connection tunnel at the end of the excavation of the connection tunnel.

6 SUMMARY AND CONCLUSIONS

A first analysis of a metro station with characteristic material parameters showed small regions at the intersections of the tunnels, where the safety margins were not sufficient in terms of a standard design. Additional analyses with reduced strength parameters of soil and/or shotcrete confirmed that the regions with high stress levels are locally confined and do not endanger the convergence of the analyses. The shotcrete lining has been designed safely and safety margins comparable to those of the standards are present.
Some problems with the additional analyses still remain: Using safety margins on actions or resistances results in models which deviate from reality. There are many possibilities of applying safety margins, with results which are not directly comparable. Applying statistical methods (Thurner 2001) might be a remedy, but would currently be much too expensive for three-dimensional analyses.

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