4. Crown heading with closed invert

4.1 Österfeld Tunnel in Stuttgart, Germany

4.1.1 Introduction

The Österfeld Tunnel undercrosses residential areas as well as the railway tracks of the Gäubahn and the S-Bahn (urban railway) partly with low overburden. During the heading of the tunnel underneath the railway lines the railway traffic had to be maintained. In this area, therefore, special measures for the support of the tunnel were required. Special attention was required, because the road tunnel is located in mudstone layers of the Lias α, which are subjected to high horizontal stresses.

4.1.2 Structure

The Österfeld Tunnel, which has been excavated by means of the NATM is approx. 400 m long and part of the eastern by-pass around Vaihingen, a suburb of the city of Stuttgart. The new road connection with a total length of 1.9 km is joining the highway B 14 at the Vaihinger triangle (Fig. 4.1). The Österfeld Tunnel undercrosses the Paradiesstraße and the railway tracks of the Gäubahn and the S-Bahn (Fig. 4.2). Furthermore the new road crosses the Nesenbachtal by means of a 170 m long bridge. Following the southern end of this bridge, the road is running through the 780 m long Hengstäcker Tunnel. The subsequent road section is joining the Nord-Süd-Straße (Fig. 4.1). The new road connection was opened for traffic in September 1999.

The northern portal of the Österfeld Tunnel is located at the end of the roadway Unterer Grund. Up to the Don-Carlos-Brücke the tunnel runs parallel to the tracks of the Gäubahn and the S-Bahn (Fig. 4.2). Along the first 100 m of this section at the base of the adjacent railway trench a 9 m high angular retaining wall is located. Moreover in the area of the tunnel some residential buildings are located. One of these houses is directly undercrossed by the tunnel. Behind the Don-Carlos-Brücke the tunnel undercrosses the four tracks of the above mentioned railway lines at an acute-angle. The tunnel ends at the northern flank of the Nesenbachtal (Fig. 4.2).

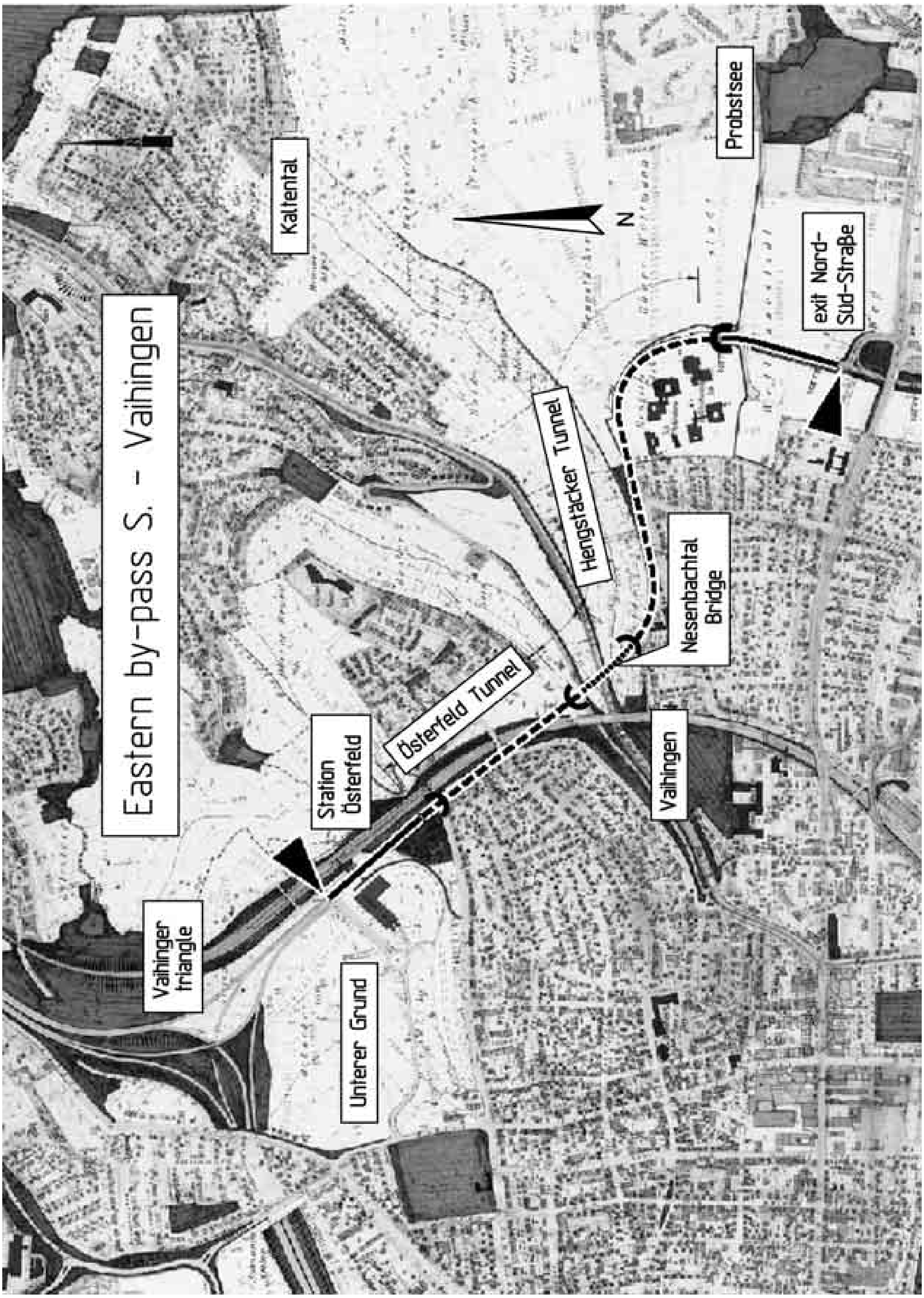


Fig. 4.1: Map of the eastern by-pass in Stuttgart-Vaihingen

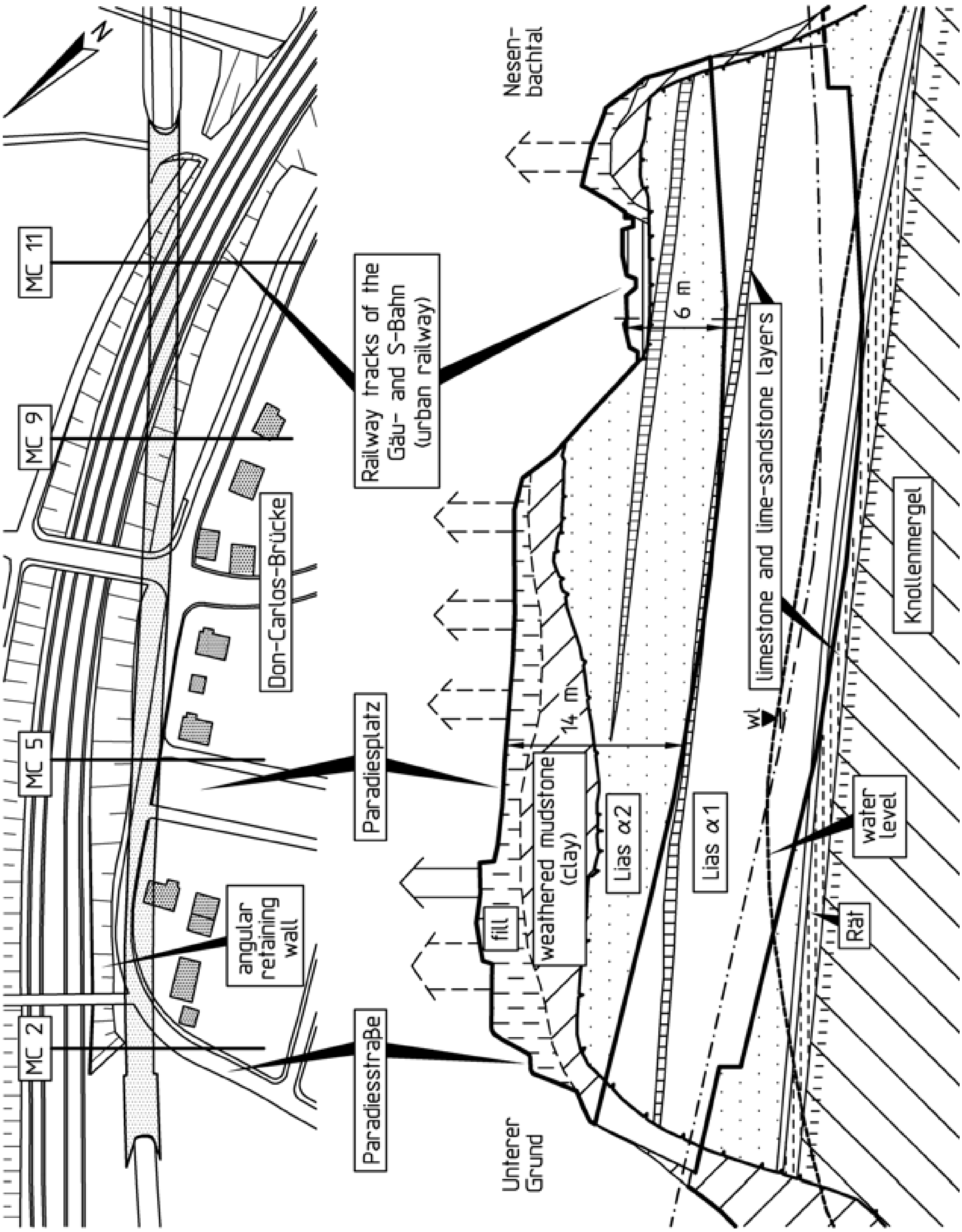


Fig. 4.2: Plan and longitudinal section of the Österfeld Tunnel

The overburden of the tunnel varies between 6 m and 14 m. The lowest overburden results at the undercrossing of the railway tracks (Fig. 4.2).

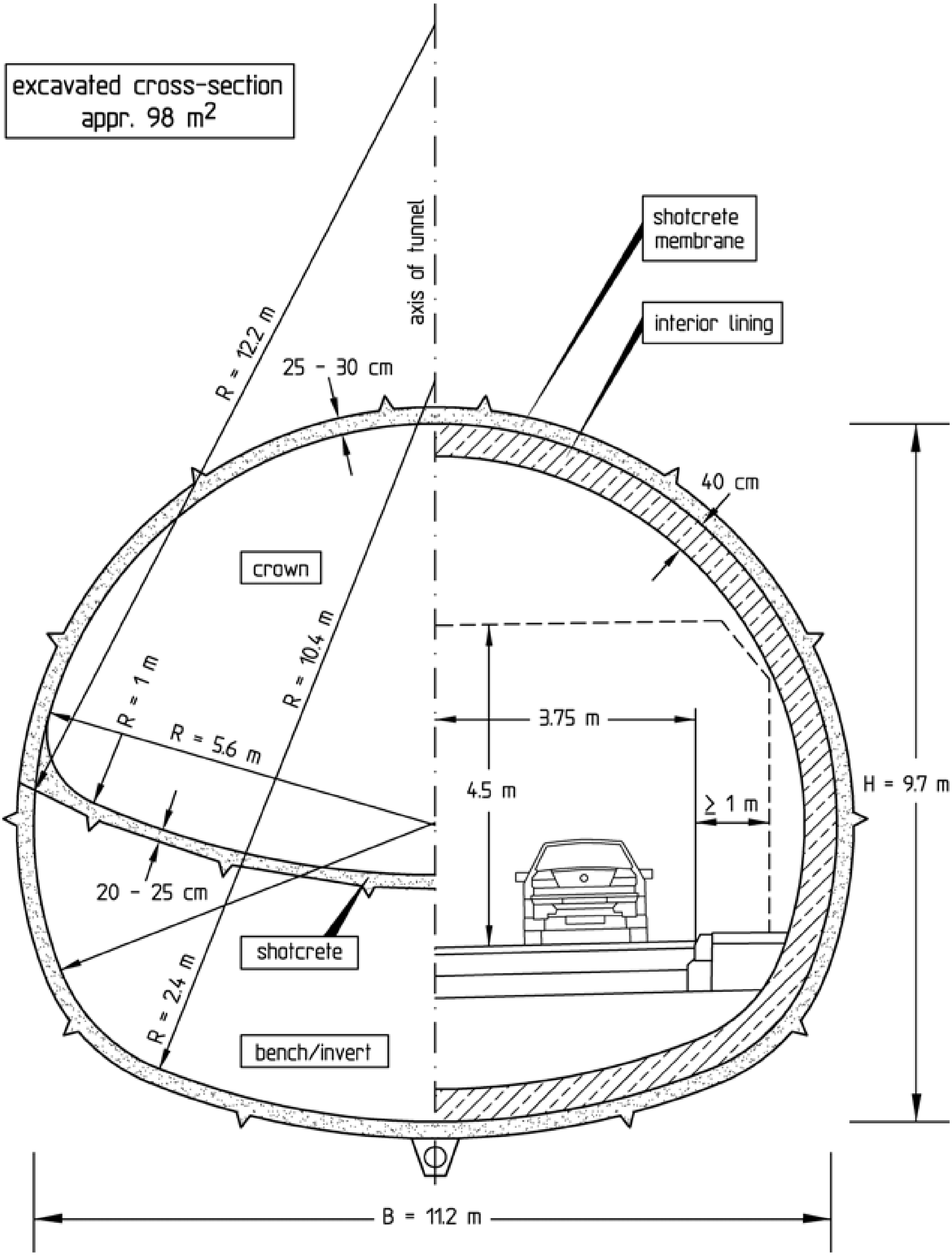


Fig. 4.3: Österfeld Tunnel, regular cross-section

For the tunnel a mouth-shaped cross-section was carried out with a total internal width of 11.2 m and a total height of 9.7 m. The excavated cross-section amounts to approx. 98 m^2. The thickness of the shotcrete membrane is 25 to 30 cm and the thickness of the interior lining consisting of watertight reinforced concrete is 40 cm. For the traffic in each direction a traffic lane of 3.75 m width is available (Fig. 4.3).

The radius of curvature at the area of the crown is R = 5.6 m. The invert (R = 10.4 m) as well as the temporary invert of the crown (R = 12.2 m) were carried out with larger radii. At the transitions from the crown to the temporary invert of the crown (R = 1 m) and from the sidewalls to the invert (R = 2.4 m) small radii were selected (Fig. 4.3).

4.1.3 Ground and groundwater conditions

During the design phase for the new by-pass road an extensive program for the exploration of the subsoil and groundwater conditions was carried out. This program includes core drillings, the investigation of soil, rock and water samples, water level observations, combined extensometer and inclinometer measurements as well as in-situ stress measurements.

According to the results of this exploration program the Österfeld Tunnel is nearly completely located in the Psilonoten- and Angulatenlayers of the Lower Jurassic (Lias α1 and Lias α2, Fig. 4.4), consisting of mudstones with single limestone and lime-sandstone interbeds. The mostly solid mudstones are transversed by bedding parallel discontinuities with small spacings and are distinctly jointed. From the portal zones up to the central part of the tunnel the degree of weathering decreases and the strength of the intact rock as well as of the rock mass, respectively, increases. The generally very hard layers of limestone and lime-sandstone are characterized by two sets of vertical joints J1 and J2, which are oriented perpendicular to the bedding planes and enable the excavation with an excavator. The almost horizontal bedding B dips parallel to the tunnel axis towards the Nesenbachtal. The sets of discontinuities B, J1 und J2 are also present in the mudstones, however, with a considerable less extent and frequency as in the limestones and lime-sandstones. Concerning the rock mechanical parameters the mudstones, the limestones and the lime-sandstones are combined to one layer (Fig. 4.5). In the present case this is permissible, because the thicknesses of the limestones and the lime-sandstones are small in comparison to the thicknesses of the mudstone layers as well as to the dimensions of the tunnel cross-section.

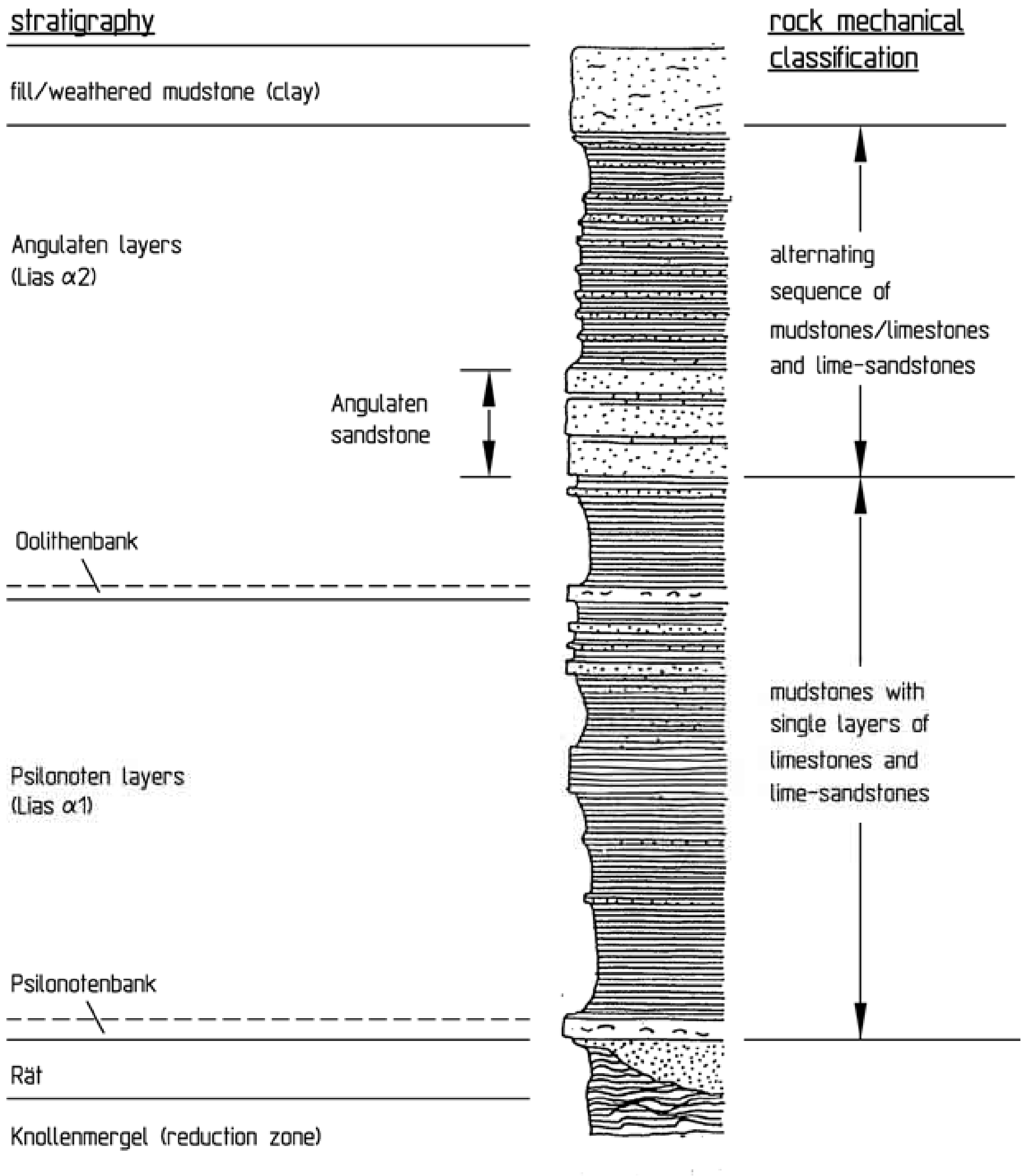


Fig. 4.4: Stratigraphical and rock mechanical classification of the ground

Locally the invert of the tunnel intersects the mudstones of the Rät, which are located below the Lias α formation (Fig. 4.4 and 4.5).

The transition to the unweathered Knollenmergel, which is located underneath the mudstones of the Rät, is formed by a disintegrated layer, the so-called reduction zone of the Knollenmergel (Fig. 4.4 and 4.5). In the reduction zone as well as in the unweathered Knollenmergel slickensides are present, which dip with 20° up to 40° and strike in all directions (Fig. 4.5).

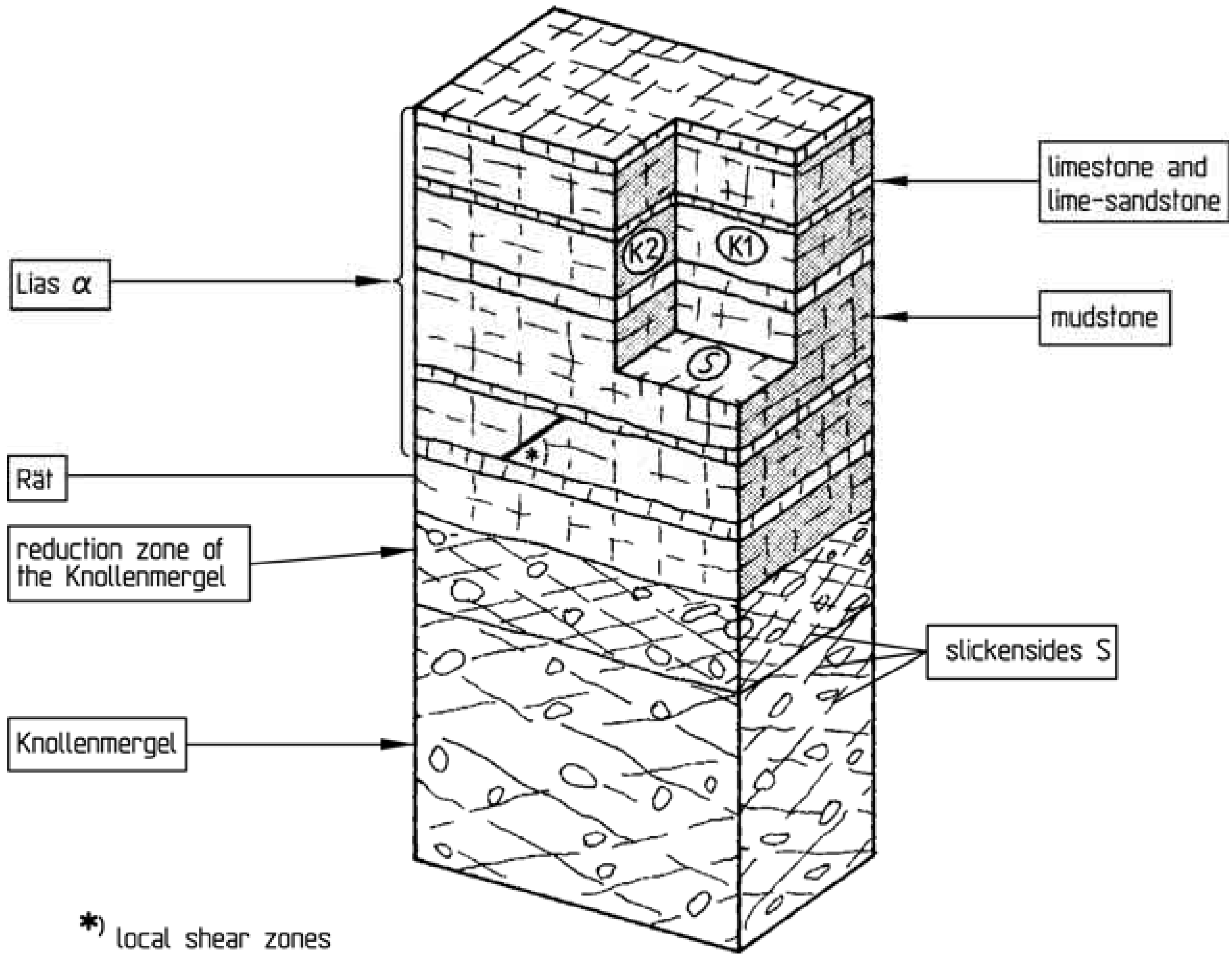


Fig. 4.5: Lias α, Rät and Knollenmergel, structural model (Wittke, 1990)

On top of the above described layers of the Lias α up to the ground overlying strata of reclaimed fill and weathered mudstone (clay) are located (Fig. 4.2). The thickness of these layers varies between 3 and 8 m. In the area of the railway trench the rock surface is situated only a few decimeters below the ground surface.

The structural model for the Lias α, the Rät and the Knollenmergel, illustrated in Fig. 4.5, was developed within the scope of other projects in the city of Stuttgart with comparable ground conditions (Wittke, 1990).

The soil mechanical parameters derived for the layers close to the surface are based on the results of laboratory investigations on samples taken from core drillings located in the area of the tunnel. Due to the grain-size distributions and moisture contents as well as data gained from experience a mean modulus of deformation of $E = 25$ MN/m^2 as well as mean effective shear parameters of $\varphi' = 20°$ and $c' = 10$ kN/m^2 are assumed (Table 4.1).

The elastic behaviour of the Lias α can be approx. described by a transversally isotropic stress-strain-law. Based on the volumetric distribution of the mudstones and the limestones and lime-sandstones the mean elastic constants were calculated according to Wittke (1990). In these calculations the mudstones were assumed to be transversally isotropic and the limestones and lime-sandstones were treated as isotropic elastic rocks. The elastic constants of the mudstones, limestones and lime-sandstones were derived from the results of dilatometer tests and laboratory tests on rock samples. Also the experiences gained from other projects in the area of Stuttgart were utilized for the estimation of these parameters. The general relationships for the elastic constants of alternating sequences consisting of transversally isotropic rocks are given in Salamon (1968).

layer	deformability	strength
Fill and clay	$E = 25$ MN/m^2	$\varphi' = 20°$, $c' = 10$ kN/m^2
Mudstones with single layers of limestones and lime-sandstones	$E_1 = 1000$ MN/m^2 $E_2 = 500$ MN/m^2	Bedding B: $\varphi_B = 20°$ $c_B = 0$ Jointing J_1, J_2: $\varphi_J = 30°$, $c_J = 40$ kN/m^2
Rät and reduction zone of the Knollenmergel	$E = 150$ MN/m^2	Discontinuities in the Rät and slickensides in the reduction zone: $\varphi_D = 17.5°$, $c_D = 10$ kN/m^2
Knollenmergel	$E = 1000$ MN/m^2	Slikensides: $\varphi_S = 17.5°$, $c_S = 10$ kN/m^2

Table 4.1: Mean values of soil and rock mechanical parameters

The shear strength of the mudstones is dominated by the shear strength of the discontinuities, which is significantly smaller than the shear strength of the intact rock. For the shear strength of the bedding planes a friction angle of $\varphi_B = 20°$ and a cohesion of $c_B = 0$ are assumed. For the joints of sets J1 and J2 an angle of friction of $\varphi_J = 30°$ and a cohesion of $c_J = 40$ kN/m^2 are estimated (Table 4.1). A tensile strength normal to the discontinuities is not taken into account.

With regards to the rock mechanical properties the Rät and the reduction zone of the Knollenmergel are combined to a uniform layer. On the basis of experience for the modulus of deformation a value of $E = 150$ MN/m^2 is assumed (Table 4.1). In laboratory tests on rock samples unconfined compressive strengths ranging from $\sigma_u = 0.3$ MN/m^2 to 5.0 MN/m^2 were determined. Though these values are quite low, here also the strength on the discontinuities is decisive. The shear parameters of the discontinuities in the Rät and the slickensides at the reduction zone of the Knollenmergel, respectively, are estimated to be $\varphi_D = 17.5°$ and $c_D = 10$ kN/m^2 (Table 4.1). A tensile strength normal to the discontinuities here also is not accounted for.

The unconfined compressive strength of the unweathered Knollenmergel is somewhat higher than that of the reduction zone. Decisive for the strength of the unweathered Knollenmergel are however the slickensides with shear parameters of $\varphi_s = 17.5°$ and $c_s = 10$ kN/m^2 (Table 4.1). The modulus of deformation of the unweathered Knollenmergel was not evaluated. From experience gained from other projects in the area of Stuttgart with comparable subsoil conditions (Wittke, 1990) for this parameter a value of $E = 1000$ MN/m^2 is estimated (Table 4.1).

According to the results of investigations, carried out in the past at different structures in the Lias α high horizontal in-situ stresses are to be expected (Grüter, 1988; Wittke, 1990; Wittke 1991). Thus in two exploratory boreholes in-situ stress measurements using the overcoring technique (Kiehl and Pahl, 1991) were carried out. These measurements resulted in horizontal in-situ stresses in an order of magnitude of $\Delta\sigma_H = 0.2$ to 1.9 MN/m^2 for the mudstone layers of the Lias α. These stresses have to be accounted for in addition to those resulting from the dead weight due to horizontally confined in-situ conditions.

In the portal zones the ground-water level is located below the invert of the tunnel. In the remaining area the water table is located somewhat above the invert of the tunnel. In the central section of the tunnel it is levelled at the middle of the height of the cross-section of the tunnel (Fig. 4.2).

4.1.4 Fundamentals of the design

Because of the small overburden, the high horizontal stresses and the low shear strength of the bedding parallel discontinuities in the Lias α, in which the tunnel is located, special problems concerning the stability of the tunnel during construction arise. After the excavation of the tunnel the horizontal stresses have to be transmitted around the tunnel's cross-section. As a consequence stress concentrations at the roof and the invert of the tunnel occur, which may be considerably higher than in the horizontal stresses present in the undisturbed state of stress (in-situ state). If the tunnel remains unsupported over a greater section the mudstones would be highly stressed in horizontal direction. Because of the above mentioned comparable low strength shear failures on the bedding parallel discontinuities above the roof and beneath the invert of the tunnel would occur. Moreover a buckling of thin mudstone layers would be possible.

To avoid failures and collapses under these difficult conditions a bolted shotcrete support has to be always installed immediately after excavation. Thus the stresses can be transmitted around the tunnel mainly through the shotcrete. To achieve a stable stage of construction the support must be adequately designed and must have an early bearing capacity. High requirements with regard to a high early strength of the shotcrete are to be fulfilled.

Because of the large cross-section of the tunnel, which amounts to 98 m^2 (Fig. 4.3), as well as for reasons of the construction process and for stability it was decided to subdivide the cross-section in crown, bench and invert. Because of the same reason and in order to minimize surface subsidence the crown was designed with a shotcrete membrane at the temporary invert. Also during crown excavation as well as bench and invert excavation, respectively, short round lengths and an early closing of the shotcrete support were foreseen.

An alkali free shotcrete with a quick-setting spray cement was used according to the dry shotcrete mixture transport technique of the Rombold and Gfröhrer company , which is described in Balbach and Ernsperger (1996) (see Chapter 2.1.2). This shotcrete is characterized by a fast development of strength as well as a high ultimate strength. Although it is a shotcrete with a concrete grade of B25 corresponding to a C20/25 according to EUROCODE 2 (EC2), it was well known from experience gained from other projects that this shotcrete develops a considerably higher strength comparable to a concrete grade of B45 corresponding to a C 35/45 (EC2). As a conservative assumption the design of the shotcrete membrane, therefore, was based on a concrete grade of B35 corresponding to a C30/37 (EC2). The strength of the shotcrete during construction was continuously checked by concrete tests.

4.1.5 Stability analysis for the stages of construction

The stability analyses for the stages of construction of the Österfeld Tunnel were carried out at vertical slices according to the finite element method (Wittke, 2000). In the mudstone layers of the Lias α in addition to the stresses due to dead weight horizontal stresses $\Delta\sigma_H$ varying from 0.5 to 1.5 MPa - depending on the location of the considered computation section - were simulated. Analyses without consideration of increased horizontal stresses were carried out too.

To account for the displacements which occur ahead of the temporary tunnel face and before the shotcrete membrane is installed in the two-dimensional analyses a stress relief was simulated by reducing the Young's modulus of the rock mass to be excavated (Wittke, 2000):

$$E_{red} = a_v \cdot E \qquad \text{with } a_v \leq 1 \tag{4.1}$$

In the analyses the so-called stress relief factor a_v is varied between 1.0 (no stress relief) and 0.5. In the next step of analysis the excavation as well as the installation of the shotcrete membrane was simulated simultaneously.

In the scope of the review of the design two- and three-dimensional analyses using the computer code FEST03 (Wittke, 2000) were carried out. By means of these analyses the displacements monitored during excavation of the tunnel were back analyzed and the assumptions and thus the parameters taken as a basis for the stability analyses were checked. In the following the steps as well as the results of these analyses are explained by means of an example.

In Fig. 4.6 the finite element mesh used for the three-dimensional analyses is illustrated (Hauck et al., 1998). The dimensions of the computation section are 100 m x 100 m x 40 m. The computation section is subdivided into 9074 isoparametric elements with 12720 nodes. The computation section is modeling the area of the Paradiesplatz, which is located approx. 120 m south of the portal "Unterer Grund" (Fig. 4.1 and 4.2). The setup of the finite element mesh enables the modeling of the stages of excavation, of the shotcrete membrane as well as of the subsoil profile and the railway trench. The analyses were carried out assuming an elastic-viscoplastic stress-strain behaviour for the ground. For the mudstones of the Lias α with single layers of limestones and limesandstones as mentioned above a transversally isotropic stress-strain behaviour in the elastic domain as well as increased horizontal in-situ stresses were simulated. The soil and rock mechanical parameters as well as the three-dimensional finite element mesh are shown in Table 4.1 and Fig. 4.6.

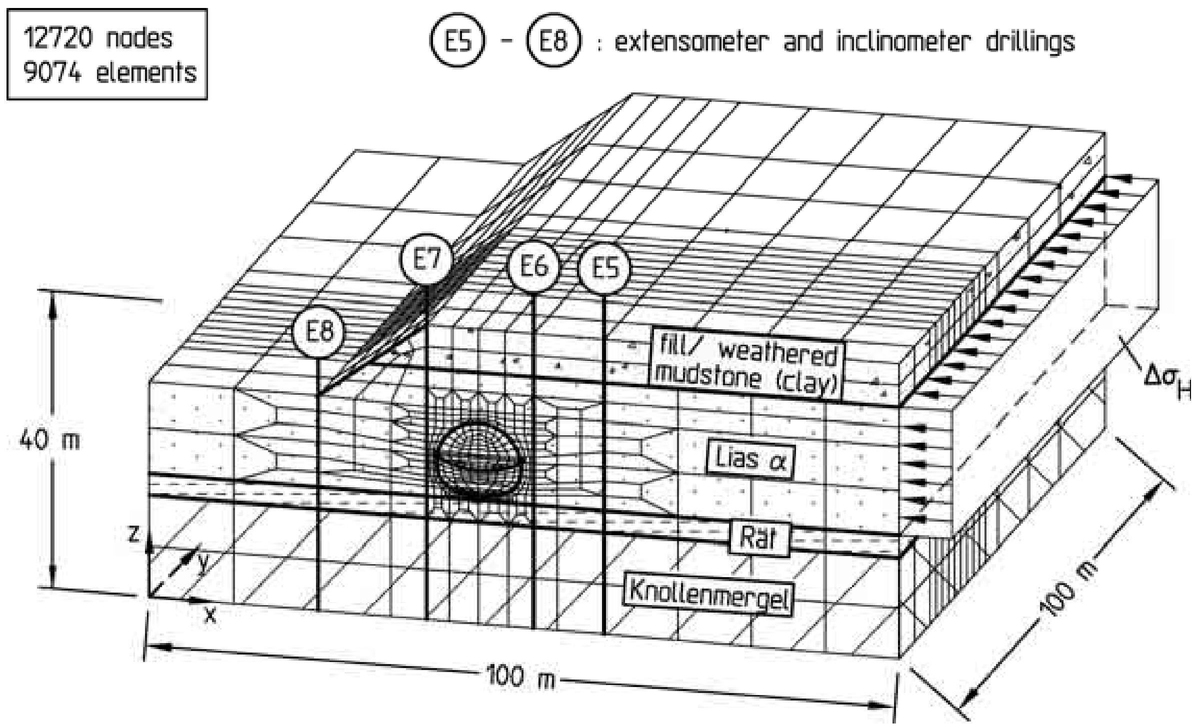


Fig. 4.6: Computation section, finite element mesh and arrangement of extensometers and inclinometers

1st and 2nd step: In-situ state of stress before construction of the railway
3rd step: Excavation of the existing trench for railway tracks

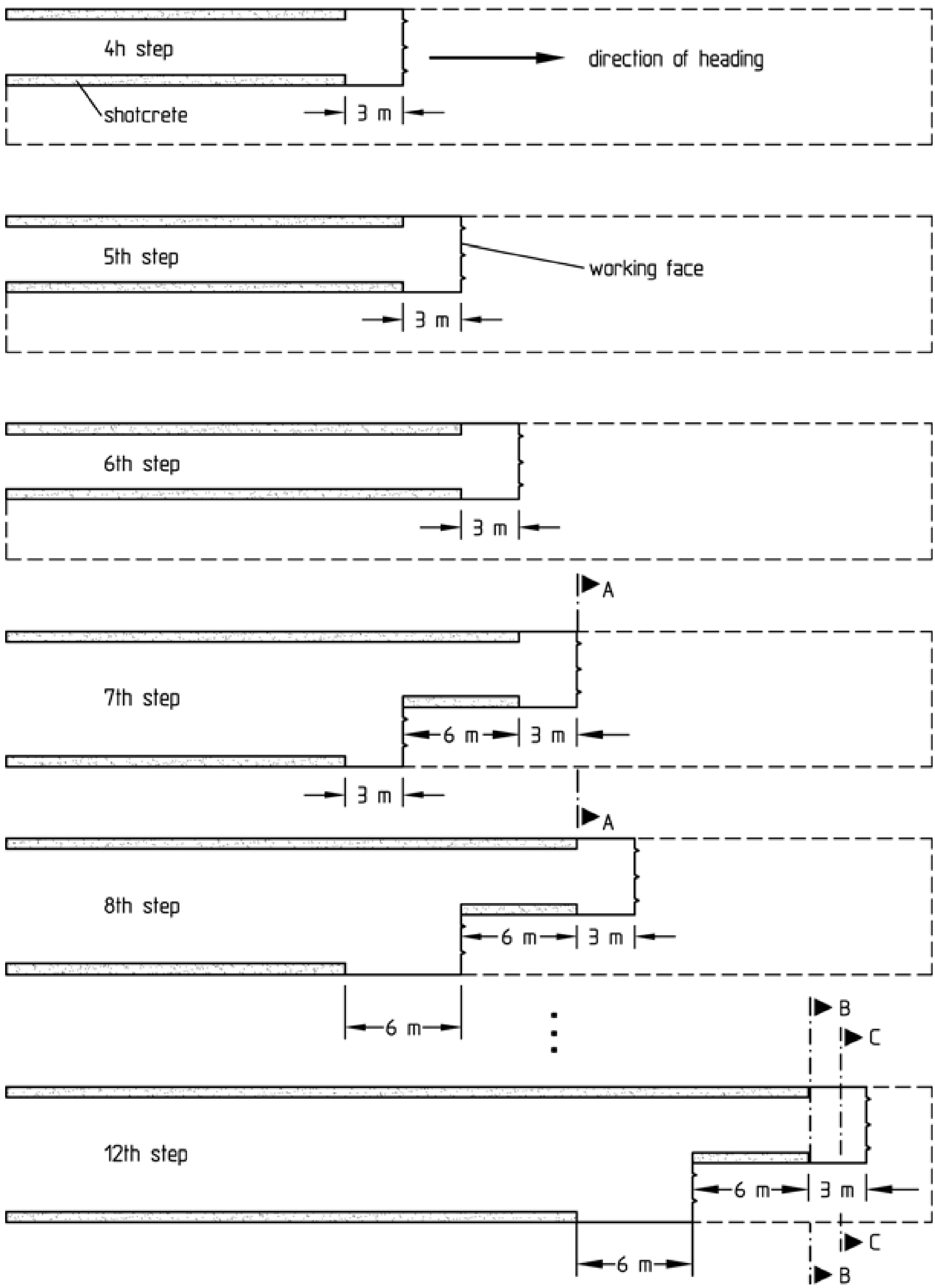


Fig. 4.7: Steps of analysis for simulation of the heading of the tunnel

In the three-dimensional analysis the stages of construction arising during the heading of the tunnel were simulated in several steps, which approximately correspond to the driving and support of the tunnel in reality (Fig. 4.7). The applied so-called "step by step" method is explained in detail in Wittke (2000).

In the two first steps of the analysis the in-situ state of stress due to dead weight and the additional horizontal stresses in the Lias α were calculated. For the simulation of the increased horizontal stresses $\Delta\sigma_H$ the nodes located at the boundary plane with the coordinate x = 100 m were displaced in x-direction. These displacements lead to horizontal stresses, which correspond to the stress $\Delta\sigma_H$ existing in the Lias α (Fig. 4.6). In the first step of analysis the whole computation section was horizontally loaded by these displacements. The unit weight γ was accounted for, however, only for the Lias α. In the second step of the analysis the soil layer underneath the surface as well as the Rät and the Knollenmergel, in which increased horizontal stresses are not existing, were substituted by materials, which have the same mechanical parameters as before, which however are not weightless any longer ($\gamma > 0$). Since the new materials were installed stress-free into the already deformed corresponding elements (Wittke, 2000) and because in the second step of the analysis the horizontal displacements of the boundary x = 100 m were not changed, the soil underneath the surface, the Rät and the Knollenmergel are loaded only by the dead weight and not subjected to increased horizontal stresses.

The excavation of the railway trench was simulated within the third step of analysis. In the steps 4 up to 12 the crown excavation as well as the excavation of the bench and the invert following at a certain distance were simulated. In the computation case, which is illustrated in Fig. 4.7, unsupported round lengths of 3 m for the crown excavation and of 6 m for the excavation of the bench and the invert were simulated. These are larger than the real round lengths applied during construction which are mentioned below (Chapter 4.1.6). Hereby the development of strength of the shotcrete, which was not considered in the analysis, was roughly simulated. By modeling greater round lengths it was taken into account that the young shotcrete develops its complete bearing capacity only after a number of days (see Chapter 2.1). Therefore the distance between the load bearing shotcrete support and the tunnel face modeled in the analysis is larger as one round length in reality. The excavation of the bench and the invert was simulated stepwise in a similar manner as the crown excavation (Fig. 4.7).

By means of three-dimensional analyses it is possible to compute the stress redistributions in the area of the temporary tunnel face, which lead to stress concentrations in the rock mass ahead of the tunnel face, which is not yet excavated, as well as in the rock mass adjacent to the excavated cross-section and in the support already installed. To demonstrate the three-dimensional carrying behaviour the development of the calculated vertical displacement of a point at the roof during the heading of the tunnel is illustrated in Fig. 4.8. Up to the 7th step of analysis, in which the tunnel face passes the considered point, already 50 % of the final displacement due to excavation are evaluated as advancing displacement.

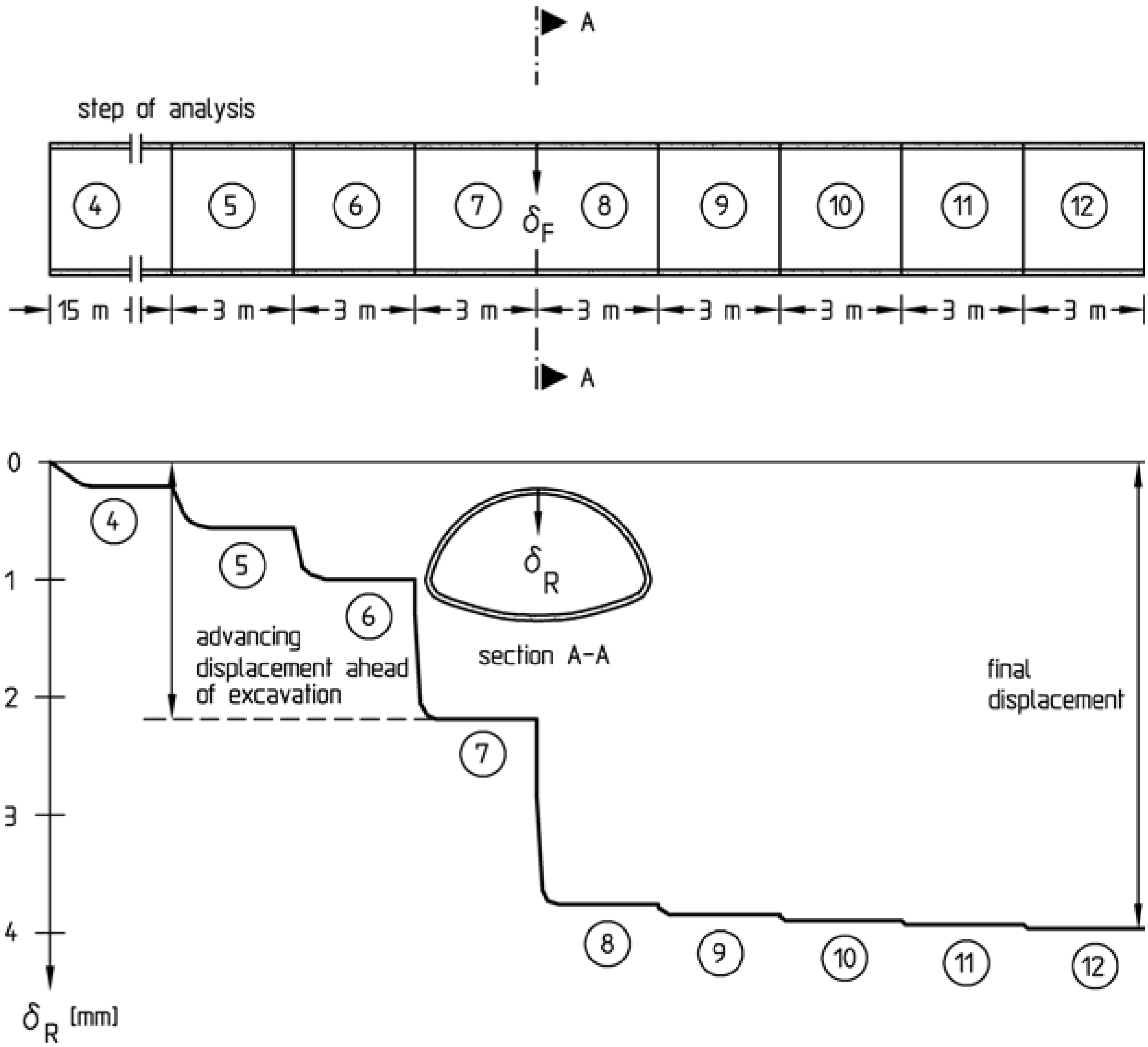


Fig. 4.8: Vertical displacements of a selected point at the roof in the course of analysis

Fig. 4.9 illustrates the calculated principal normal stresses in the rock mass resulting from excavation and support of the crown (12th step of analysis) computed for a cross-section perpendicular to the axis of the tunnel, and located in the area of the unsupported crown (cross-section C-C in Fig. 4.7). For the considered computation case an increased horizontal stress of $\Delta\sigma_H$ = 1 MN/m^2 existing in the Lias α was assumed. As a result of computation an arch is formed in the rock mass around the crown. As a consequence of the increased horizontal stresses beneath the crown's invert near the contour of the excavation stresses up to 1.6 MN/m^2 are computed. Above and underneath the unsupported cross-section of the crown as well as at the foot of the slope of the railway trench and also in the Knollenmergel the strengths on the discontinuities are exceeded (Fig. 4.9).

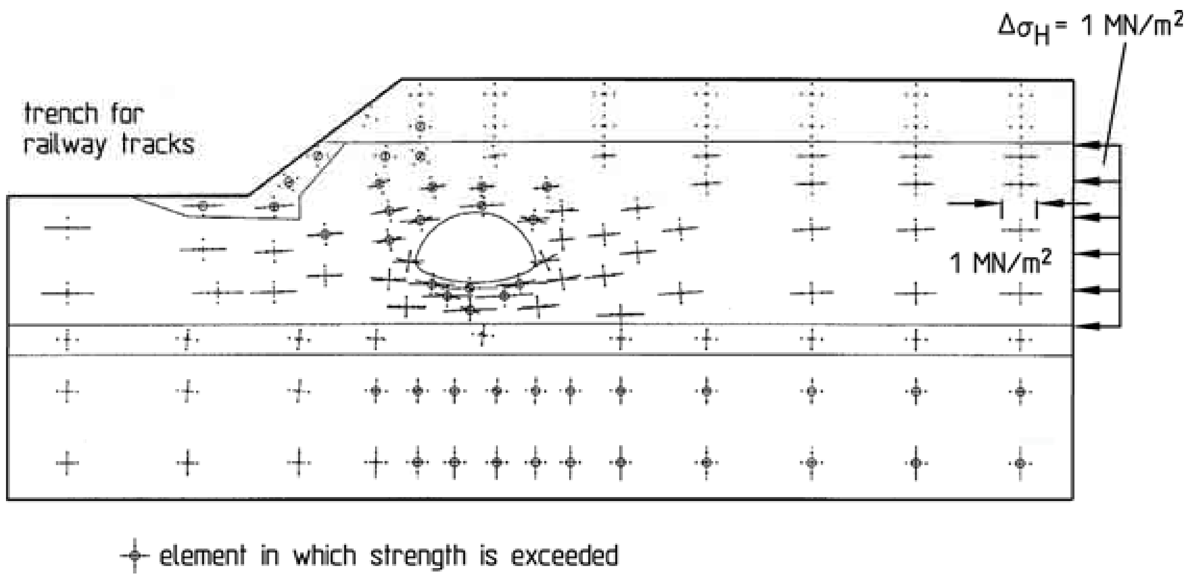


Fig. 4.9: Principal normal stresses and subsoil areas in which strength is exceeded resulting from the crown heading , 12th step of analysis (section C-C in Fig. 4.7)

In a corresponding illustration Fig. 4.10 shows the displacements due to the crown heading. Fig. 4.10 represents the differences between the nodal point displacements computed for steps 12 and 3 of the analysis for cross-section B-B (Fig. 4.7). The displacements oriented towards the excavated opening amount to approx. 6 mm. For the foot of the slope of the railway trench displacements of approx. 10 mm are computed (Fig. 4.10).

The comparison of the results of the two- and three-dimensional analyses shows that for the two-dimensional analyses a stress relief factor of a_v, which ranges from 0.5 to 0,8, is to be taken into account to achieve displacements corresponding to the results of the three-dimensional analyses.

According to the results of the comparative analyses the shotcrete membrane can be designed with a thickness of 25 cm and a reinforcement consisting of an inner and outer steel fabric met Q 295 considering a safety factor of 1.35.

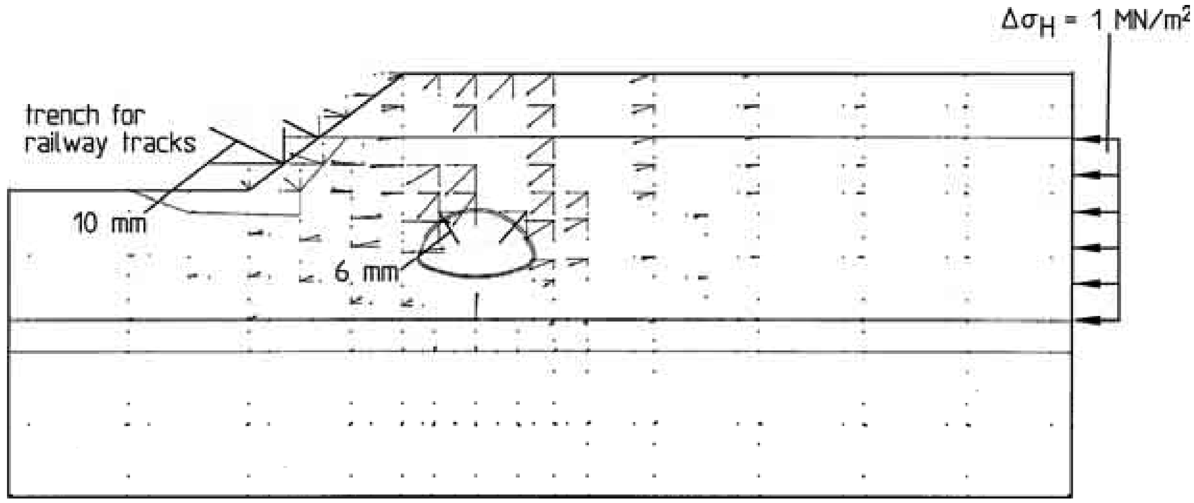


Fig. 4.10: Displacements due to the crown heading, 12th - 3rd step of analysis (section B-B in Fig. 4.7)

4.1.6 **Excavation and support**

For the excavation of the tunnel with a total cross-section of 98 m^2 (Fig. 4.3) a tunnel excavator was used. The heading was subdivided into a crown excavation and an excavation of bench and invert following the crown at some distance (Fig. 4.11).

The distance from the excavation of the bench and the invert to the crown was chosen to at least 50 m. The round lengths for the crown's excavation were chosen between 80 cm and 1.2 m. For the excavation of the bench and the invert round lengths ranges from 1.6 to 3.0 m (Fig. 4.11). Thus during the crown's excavation as well as the excavation of the bench and the invert an early closure of the shotcrete membrane was realized. This measure has proven to limit the subsidence especially during the undercrossing of the buildings and the railway to a low level.

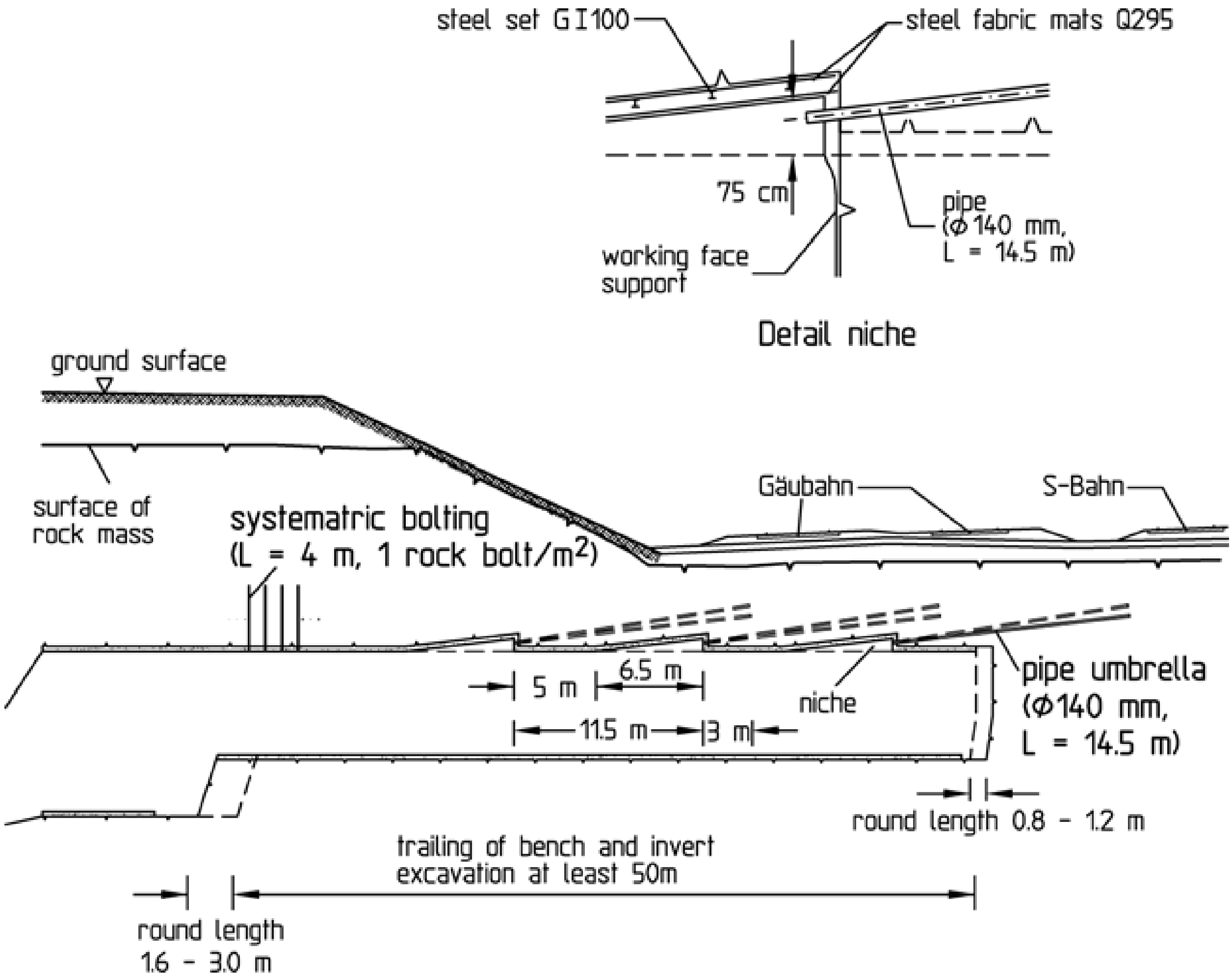


Fig. 4.11: Undercrossing of the Gäu- and S-Bahn (urban railway), longitudinal section

The regular support of the tunnel consists of a shotcrete membrane reinforced by two layers of steel fabric mats and with thicknesses varying from 25 cm to 30 cm. The temporary invert was supported by a 20 cm to 25 cm thick shotcrete membrane. Also steel sets and a systematic bolting around the crown and the bench are part of the support (Fig. 4.3 and 4.12). In Fig. 4.13 details of the support at the foot of the crown are illustrated.

Due to the low overburden and the high frequency of the discontinuities near the ground surface in the area of the two portals grouted spiles were used as advancing support. The intensively jointed mudstones located immediately above the Oolithenbank (Fig. 4.4) turned out to be caving to a major degree. In case of a unfavorable location of this rock layer at the tunnel roof, therefore, also in greater distance to the portals the installation of grouted spiles were required.

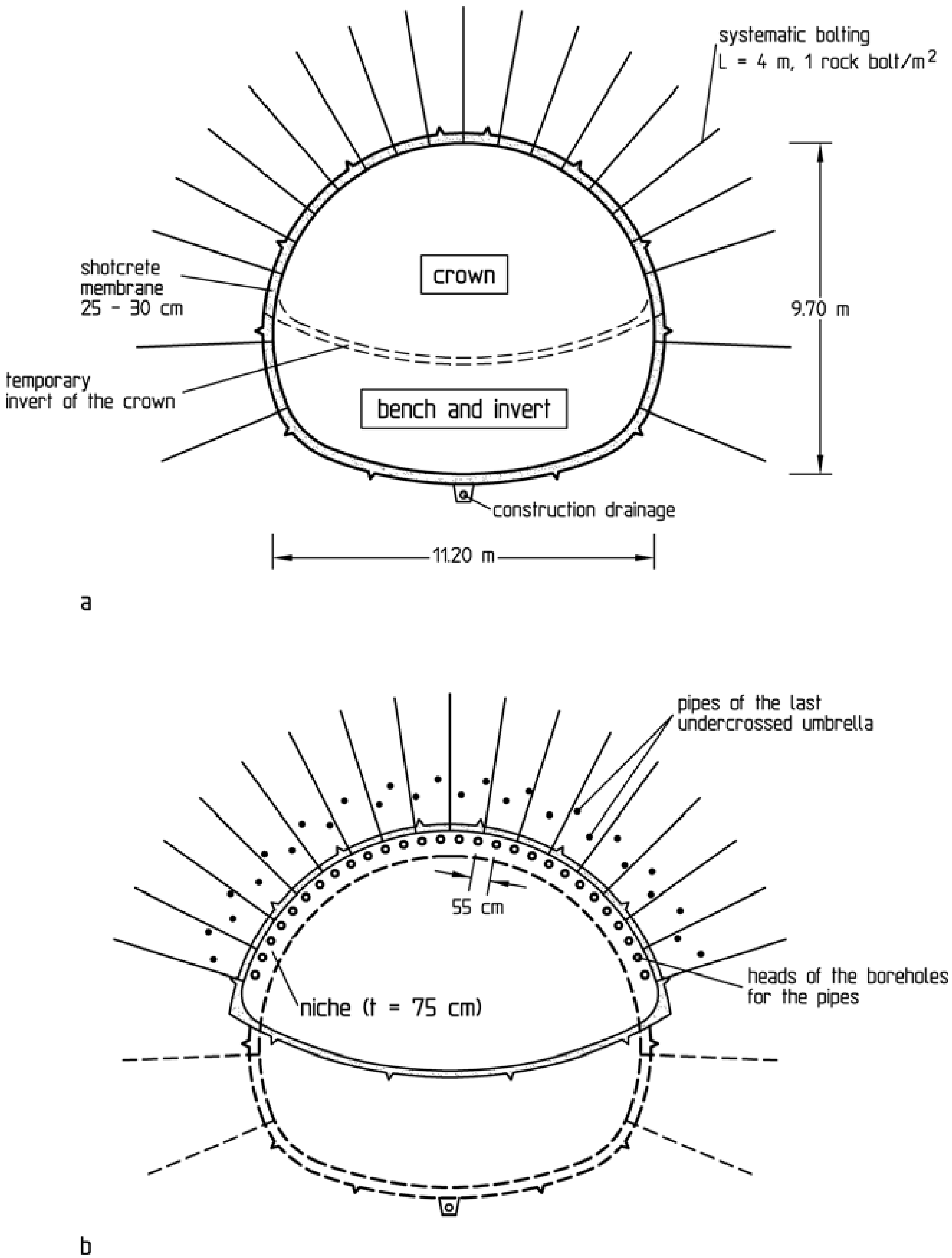


Fig. 4.12: Tunnel cross-section with support measures: a) Regular cross-section; b) undercrossing of the railway tracks

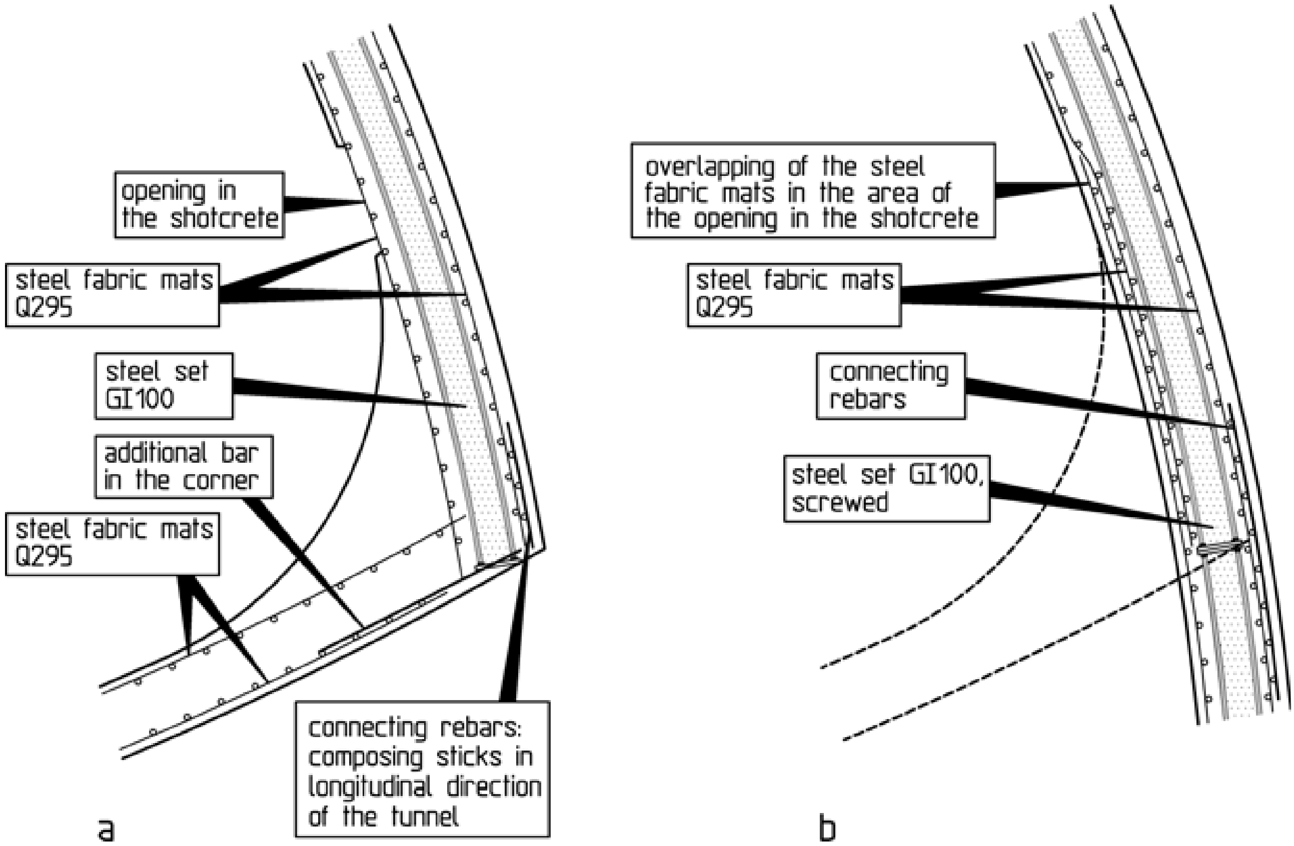


Fig. 4.13: Design of support in the area of the crown's foot: a) Crown excavation; b) excavation of the bench and the invert

Fig. 4.14: Crown excavation under a pipe umbrella

In the area of the railway trench the excavation of the crown was carried out underneath the protection of a total of ten advancing pipe umbrellas (Fig. 4.11, 4.12 and 4.14). The steel pipes in each case were installed from niches and have a diameter of 140 mm and a length of 14.50 m. The overlap between two successive pipe umbrellas was selected to 3 m. The lengths of the niches, which were expanded continuously up to a maximum depth of 75 cm, is 6.5 m (Fig. 4.11). Before the excavation of the bench and the invert in the area of the niches was carried out, the niches were filled with shotcrete.

The pipes consisting of 2 m long pieces were installed using a solid eccentric bit with an overcut of approx. 1 cm. Using prefabricated openings for injection (Fig. 4.12) as well as a double packer the annulus was grouted by means of cement based suspension. From the recorded grout volumes it could be concluded that by this procedure only the annulus between the pipe and the borehole wall was filled with grout. An appreciable grouting of the rock mass located between the pipes was not achieved. Finally, in a separate working operation, the steel pipes were filled with suspension (Hauck et al., 1998).

The average completion time for a pipe umbrella was 5 days. During the corresponding interruption of the crown heading the bench and the invert excavation was carried out which was started after the installation of the first pipe umbrella because of their higher rate of advance. In this way an optimum rate of advance could be achieved (Hauck et al., 1998).

For the time interval between the excavation and the installation of the shotcrete membrane the bearing behaviour of the steel pipe umbrella in the longitudinal direction of the tunnel is activated. In other words the space between the tunnel face and the load bearing shotcrete support in this stage is bridged by the pipes. As a consequence caving and loosening of the rock mass in this area as well as resulting subsidence are largely avoided.

4.1.7 Monitoring program and interpretation of the measuring results

During the heading of the tunnel an extensive monitoring program with special emphasis on the area of the undercrossing of the railway was carried out. Before the excavation started four main measuring cross-sections (MC2, MC5, MC9 and MC11, Fig. 4.2) with vertical combined extensometer and inclinometer measuring equipments on both sides of the tunnel (Fig. 4.6) were installed.

On the surface above and aside of the tunnel, at the buildings, at the "Don-Carlos-Brücke" as well as at the sleepers of the railway tracks points for levelling were installed. The measuring program was complemented by underground convergency and displacement measurements within various tunnel cross-sections, which were carried out parallel to the tunnel driving.

Eventual subsidence of the railway tracks was monitored by optical measurements carried out from fixed points located outside of the area of the railway tracks, by means of installation of reflectors, which were fixed at the sleepers.

The measuring results show small subsidence of the ground surface with a maximum of 2.5 cm. During the undercrossing of the house Paradiesstraße no. 69 the maximum vertical displacement of the building could be limited to 12 mm. The differential settlements of the building were so small that the admissible angular rotations of the building were not reached and thus no visible damages of the building occurred.

In the area of the railway tracks also no inadmissible subsidence or differential settlements could be observed.

The results of the monitoring were evaluated and interpreted by means of finite element analyses (Hauck et al., 1998).

Exemplarily the measuring results of the measuring cross-section 5 (MC5), which is situated in the area of the "Paradiesplatz" (Fig. 4.2) will be discussed. It reflects the situation, in which the tunnel is running immediately adjacent to the slope of the railway trench and the roof of the tunnel is approx. located at the elevation of the railway trench.

The horizontal displacements measured by inclinometers in four boreholes during the crown's excavation above the tunnel are oriented towards the railway trench and amount up to 9 mm (Fig. 4.15a). In the height of the tunnel's cross-section the measured horizontal displacements are oriented on both sides towards the excavated opening and amount 6 to 7 mm. These displacements lead to horizontal convergencies of the side walls of the tunnel.

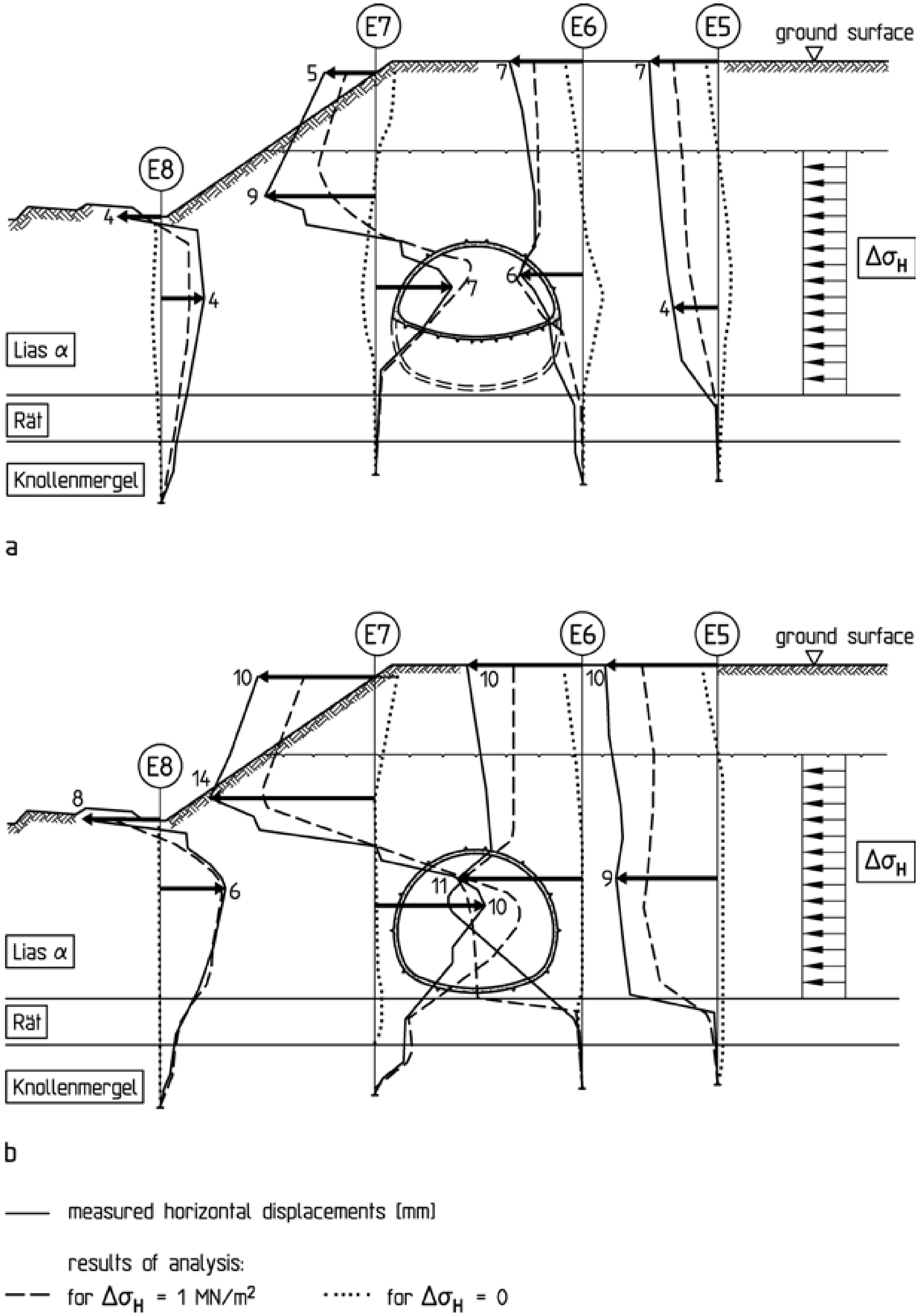


Fig. 4.15: Measured and calculated horizontal displacements (MC5): a) Crown excavation; b) full excavation of the tunnel's cross-section

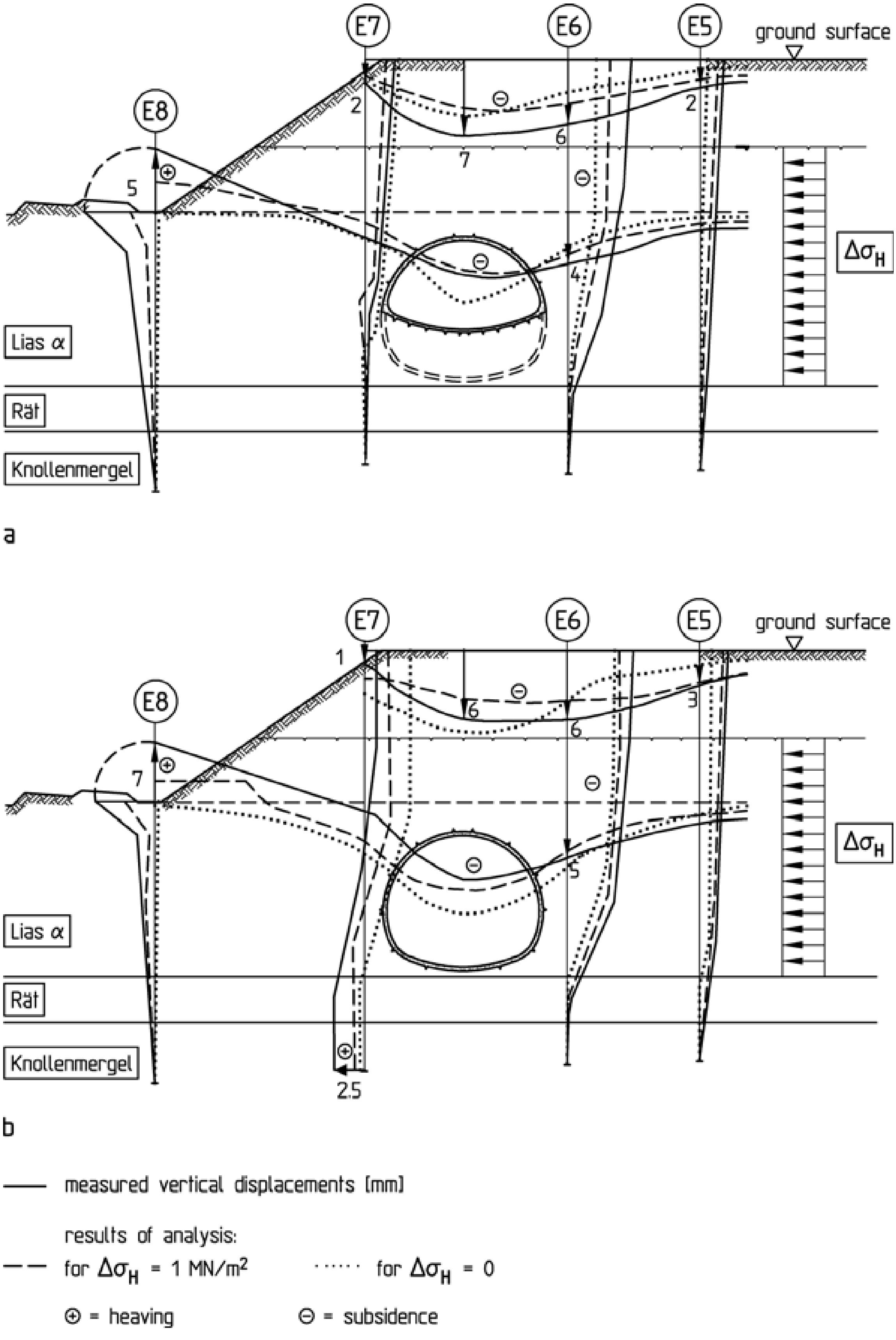


Fig. 4.16: Measured and calculated vertical displacements (MC5): a) Crown excavation; b) full excavation of the tunnel's cross-section

Above the roof of the tunnel a change of direction of the horizontal displacements occurs (drilling E7). This change can be observed also at the foot of the slope and was indicated by the tilting of the masts of the contact wire in the direction of the railway tracks.

The following excavation of the bench and the invert of the tunnel has lead to an increase of the horizontal displacements from 50 up to 100 % (Fig. 4.15b).

The measurement of the vertical displacements resulting from the crown excavation in the measuring cross-section 5 show a subsidence at the ground surface above the tunnel as well as in the rock mass above and adjacent to the tunnel. On the other hand in the area of the railway trench small heavings were measured (Fig. 4.16a).

During the following excavation of the bench and the invert no significant changes of the vertical displacements were measured (Fig. 4.16b).

The interpretation of the results of the measurements by means of finite element analyses leads to the result that the measured displacements can only be understood, if increased horizontal stresses in the rock mass are taken into account. The rock mechanical parameters on which the stability analyses are based on as well as the increased horizontal in-situ stresses in the order of magnitude of $\Delta\sigma_H \approx 1$ MN/m^2 could be veryfied by the comparison of measured and calculated displacements (Fig. 4.15 and 4.16).

4.1.8 Conclusions

The design and construction of the eastern by-pass of Stuttgart-Vaihingen can be considered as a challenge. Based on the experience gained in connection with large tunneling projects for road as well as for urban railway traffic carried out in the city of Stuttgart the complex and partly new tasks, which are related to the heading of the Österfeld tunnel, could be solved rather excellently.

With regard to the stability and the displacements resulting from tunnel driving special attention was required because of the small overburden as well as the increased horizontal stresses and low shear strengths on the bedding parallel discontinuities of the mudstones of the Lias α, in which the tunnel is located.

These problems were solved by the following measures:

- Subdivision of the excavation of the tunnel's cross-section in crown, bench and invert,

- installation of a shotcrete membrane and a systematic bolting immediately after excavation,

- short round lengths and an early closing of the shotcrete support during the crown's excavation as well as the excavation of the bench and the invert,

- reinforced shotcrete support of the temporary invert of the crown,

- use of a shotcrete with a quick-setting cement with a high early and ultimate strength,

- carrying out of steel pipe umbrellas in the area of the undercrossing of the railway tracks with low overburden.

By these measures the excavation of the tunnel could be carried out with very little subsidence at the ground surface. At no stage of construction the railway traffic was affected.

Moreover it has been found that the results of the three-dimensional analyses using the finite element method, which were carried out within the scope of this project, have made an essential contribution for prognosis as well as for the design of the measures for excavation and support.

4.2 Road tunnel "Elite" in Ramat Gan, Israel

4.2.1 Introduction

The two-lane road tunnel "Elite" was headed in Ramat Gan, a city in the Tel Aviv area. The tunnel was started from a underground parking lot located adjacent to the 260 m high Gate Tower (Fig. 4.17), the highest building in the Middle East. The tunnel undercrosses eight-lane Jabotinsky Street to Tel Aviv in the area of an intersection (Fig. 4.18).

Fig. 4.17: Gate Tower, Ramat Gan (Israel)

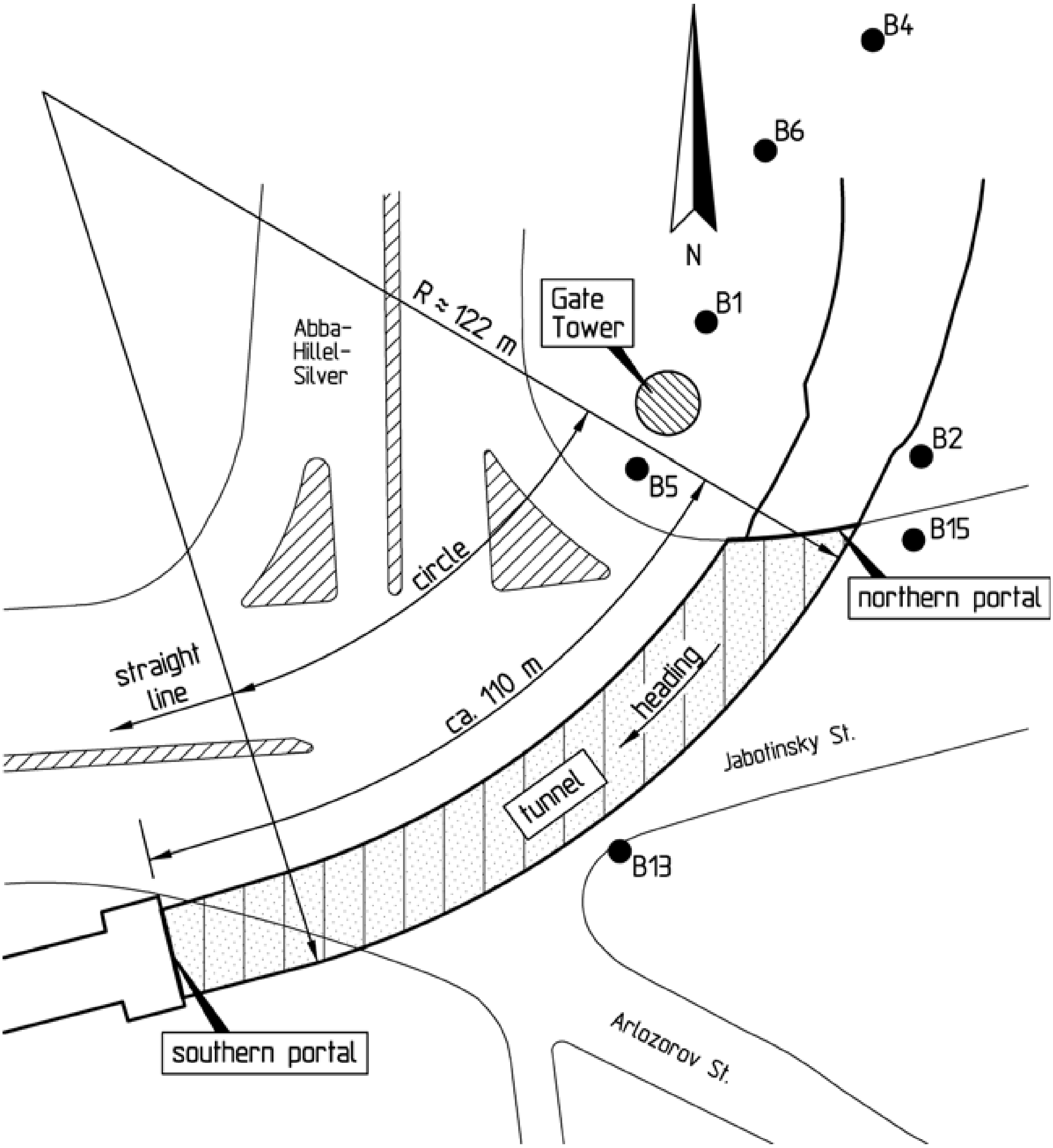


Fig. 4.18: Elite Tunnel, site plan with drill points

4.2.2 Structure

The axis of the approx. 110 m long Elite Tunnel runs firstly along a circular arc with a radius of approx. 120 m and then changes into a straight line (Fig. 4.18).

The tunnel clearance is approx. 4.9 m high and approx. 10 m wide (Fig. 4.19). The tunnel is located in a calcareous sand with low

cohesion (Kurkar), which contains local lenses of cohesionless fine sands (see Chapter 4.2.3). The overburden ranges from 3 to 4.5 m.

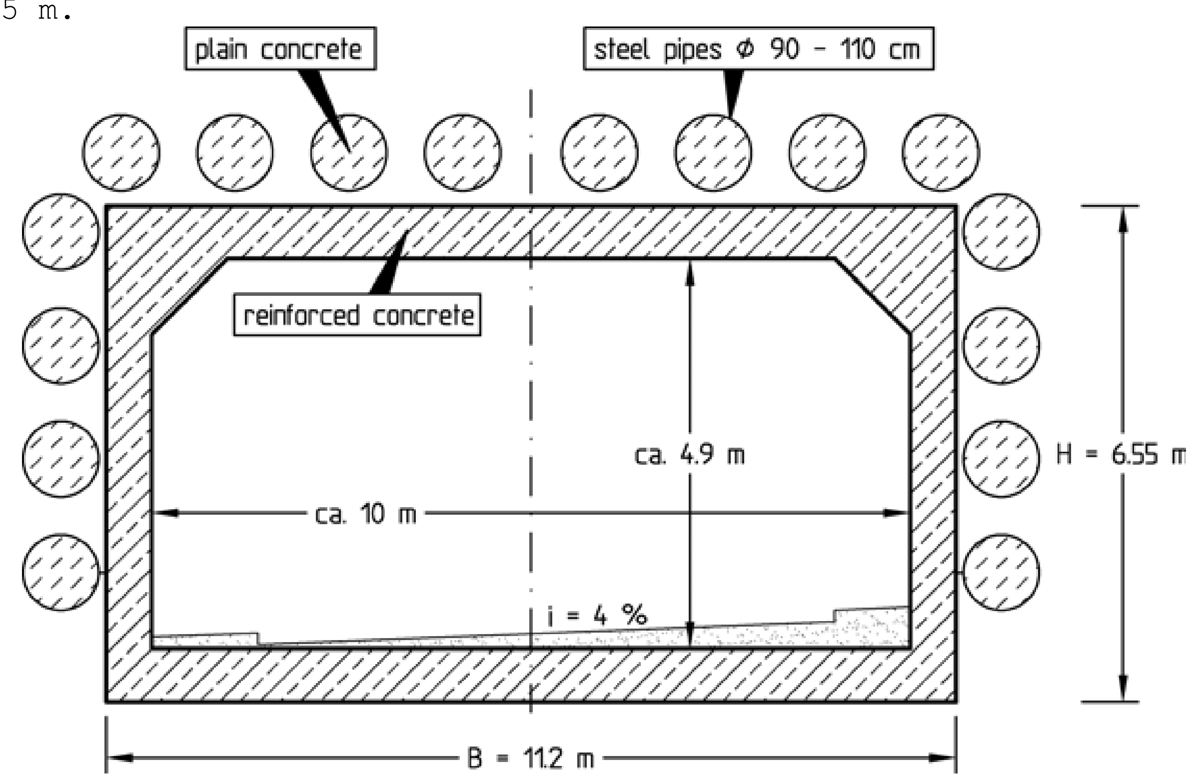


Fig. 4.19: Elite Tunnel, tender design, cross-section

The tender design included the advance installation of steel pipes 90 to 110 cm in diameter above and adjacent to the tunnel over the entire tunnel length using a microtunneling machine (Fig. 4.19 and 4.20). Under the protection of this umbrella of steel pipes filled with concrete, the tunnel was subsequently to be excavated. The approximately rectangular reinforced concrete tunnel cross-section (Fig. 4.19) was to be constructed in blocks in the process with the excavation being interrupted for each block (Fig. 4.20).

In cooperation with Walter construction company (Walter Bau AG), WBI prepared a contractor's design proposal described in Chapter 4.2.4. This contractor's design proposal is based on the NATM and was later carried out.

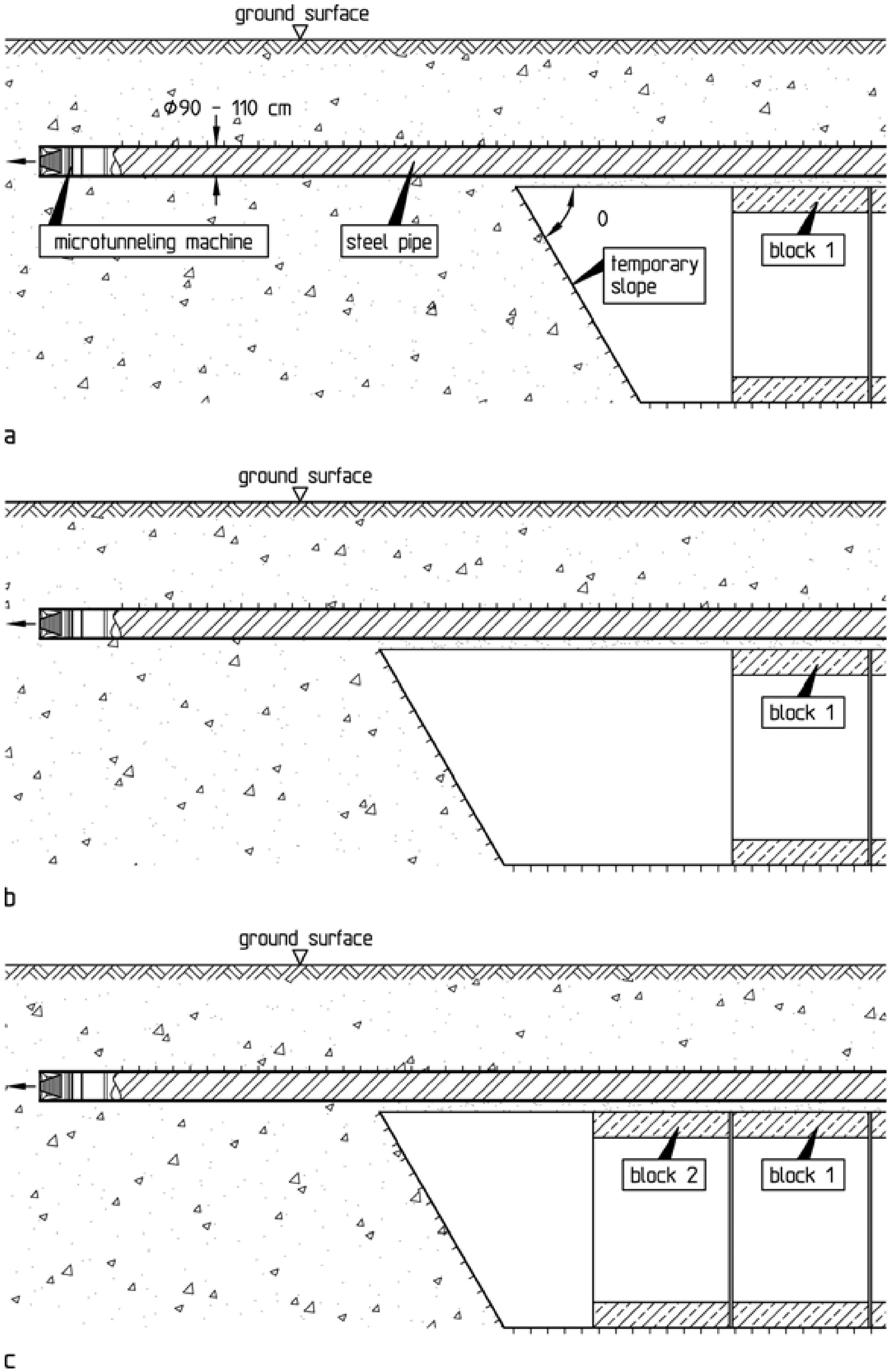


Fig. 4.20: Elite Tunnel, tender design, construction stages: a) Installation of block 1; b) excavation; c) installation of block 2

4.2.3 Ground and groundwater conditions

In the course of the exploration for other construction projects, eight boreholes were sunk in the tunnel area. Fig. 4.18 shows the drill points of seven of these boreholes.

In Fig. 4.21 the drill logs of four boreholes are projected onto a longitudinal section through the tunnel axis. According to these borehole logs, fill or clayey sands and clays exist down to a depth of 2 m. Below, medium dense to dense calcareously bonded, partially cemented sands with a fraction of gravel are encountered (Fig. 4.22). In these slightly cohesive sands, termed "Kurkar", locally cohesionless fine sands are intercalated, as mentioned above.

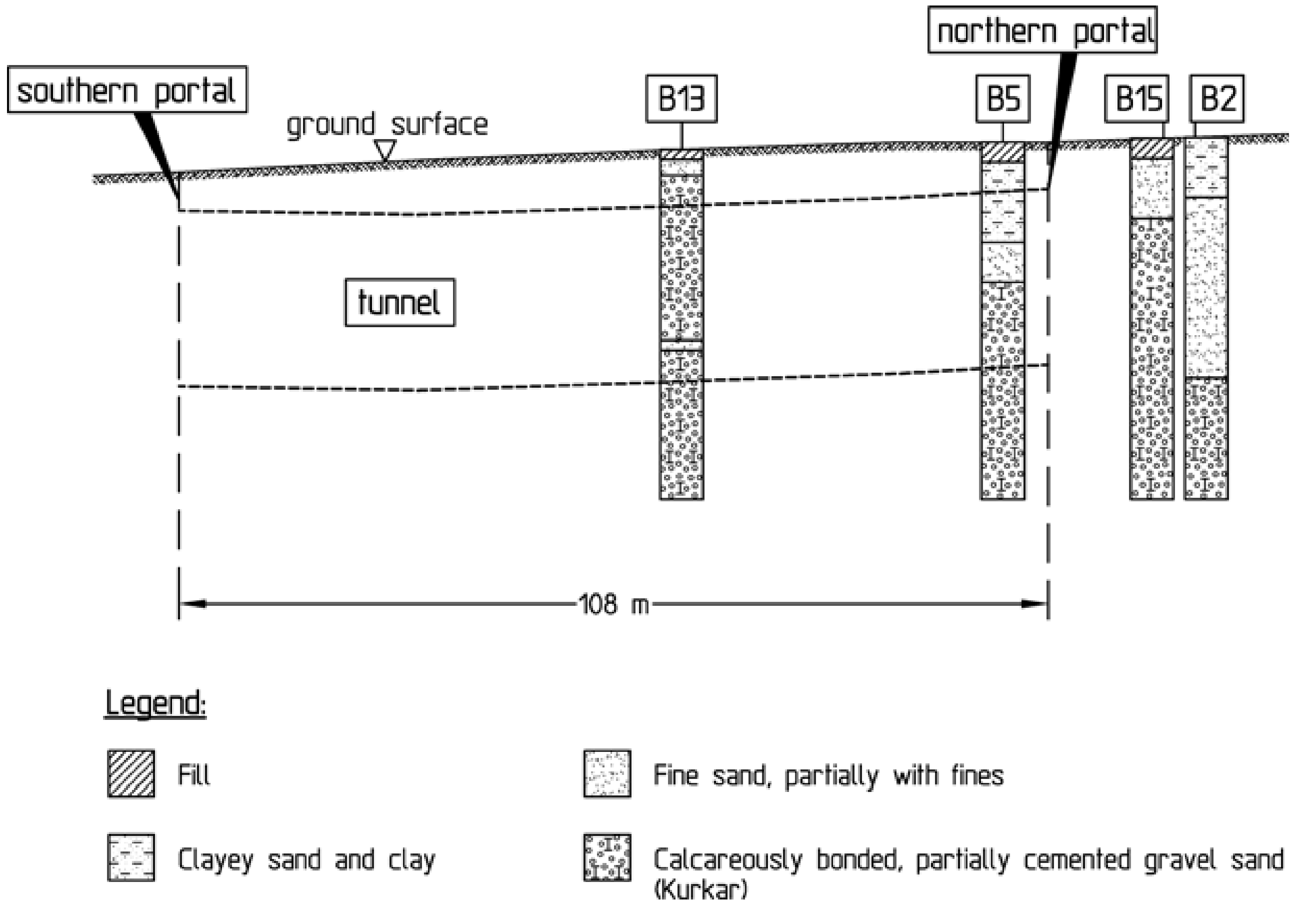


Fig. 4.21: Elite Tunnel, longitudinal section with drill logs

To assess the relative density and the bulk modulus, Standard Penetration Tests (SPT) were carried out in each borehole. Fig. 4.23 shows exemplarily the drill log and the SPT results for bore-hole B13 located closest to the tunnel alignment (see Fig. 4.18).

Fig. 4.22: View of the temporary tunnel face located in the "Kurkar" formation

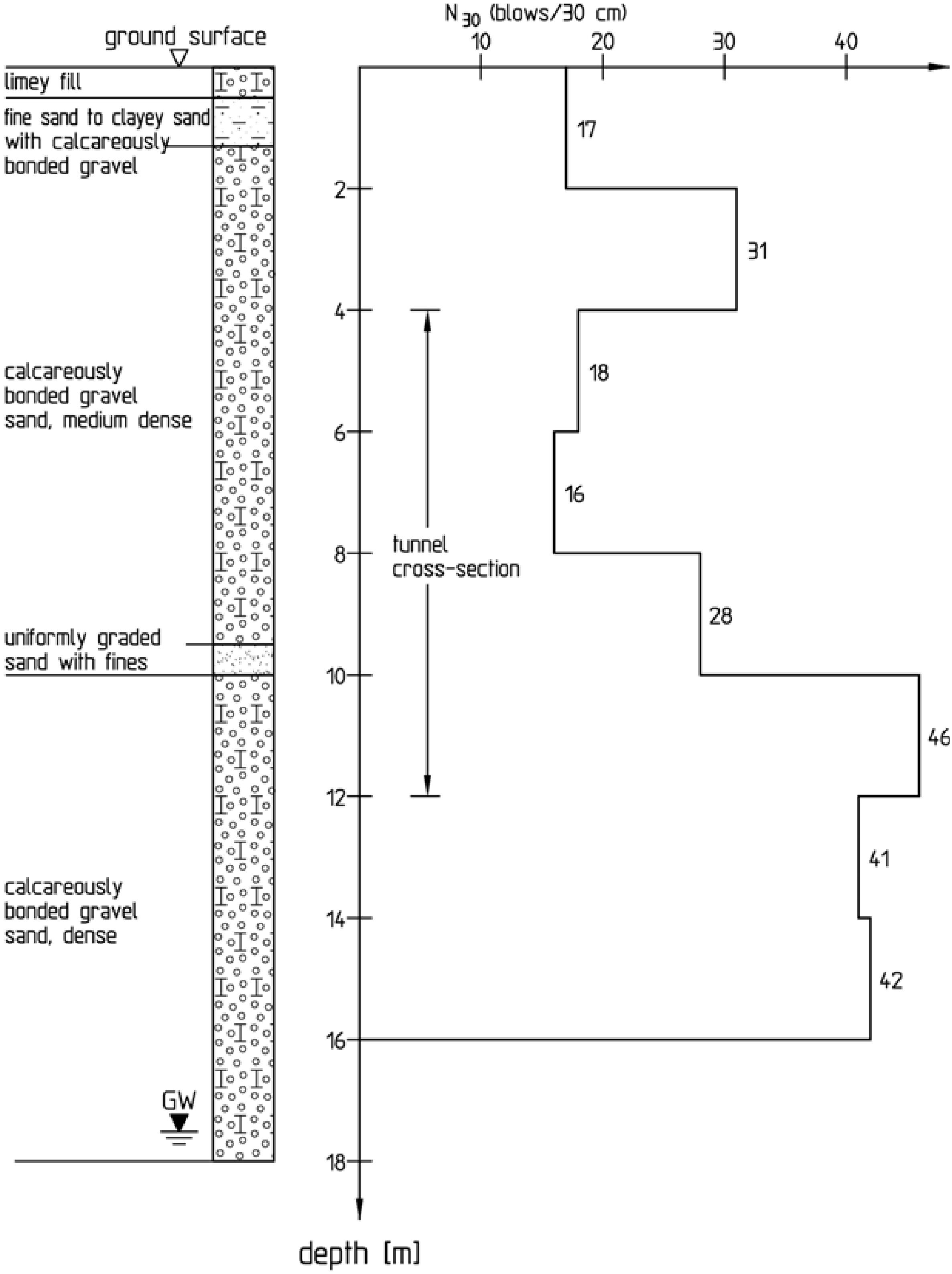


Fig. 4.23: Borehole B13, drill log and Standard Penetration Test (SPT), results

According to this, Kurkar is encountered at the level of the tunnel cross-section, with the exception of a thin soil layer of uniformly graded sand with fines. On the basis of the blow counts per 30 cm of penetration depth determined with the SPTs ranging between N_{30} = 16 and N_{30} = 46, the Kurkar and the uniformly graded sand can be classified as medium dense to very dense (D = 0.5 to 0.7) according to DIN 4094, appendix 1 (1990). An estimate of the bulk moduli according to DIN 4094, appendix 1 (1990) on the basis of the determined values for N_{30} leads to moduli ranging from approx. 40 to approx. 100 MN/m^2.

The groundwater table was found at 0 to 1 m a.s.l. in the exploration boreholes, which is equal to 6 to 7 m below the tunnel's invert (Fig. 4.23).

4.2.4 Design

Fig. 4.24 shows the tunnel cross-section according to the contractor's design proposal prepared by WBI together with Walter Bau AG. The excavation contour circumscribes the clearance with a height of approx. 8.2 m and a width of approx. 12.1 m. Because of the low overburden, the roof was designed shallow with a radius of curvature of 11.75 m. For the sidewalls and the invert large radii were selected as well with 11.55 m and 14.06 m, respectively. At the transitions from the roof to the sidewalls and from the sidewalls to the invert the selected radii are comparatively small with 2.65 m and 2.12 m. The excavated cross-section amounts to approx. 85 m^2.

For construction management reasons and for reasons of the stability of the tunnel face, the cross-section is subdivided into the crown with temporary invert support and the trailing bench and invert (Fig. 4.24 and 4.25). The height of the crown amounts to 5.6 m. The transitions from the sidewalls to the temporary invert have radii of 1.7 m. The temporary invert of the crown is rounded with a radius of 18.38 m (Fig. 4.24).

A thickness of 25 cm is selected for the shotcrete membrane in the vault. In the area of the bench, the invert and the temporary crown invert the shotcrete membrane is planned with a thickness of 20 cm. The thickness of the reinforced concrete interior lining amounts to 40 cm (Fig. 4.24).

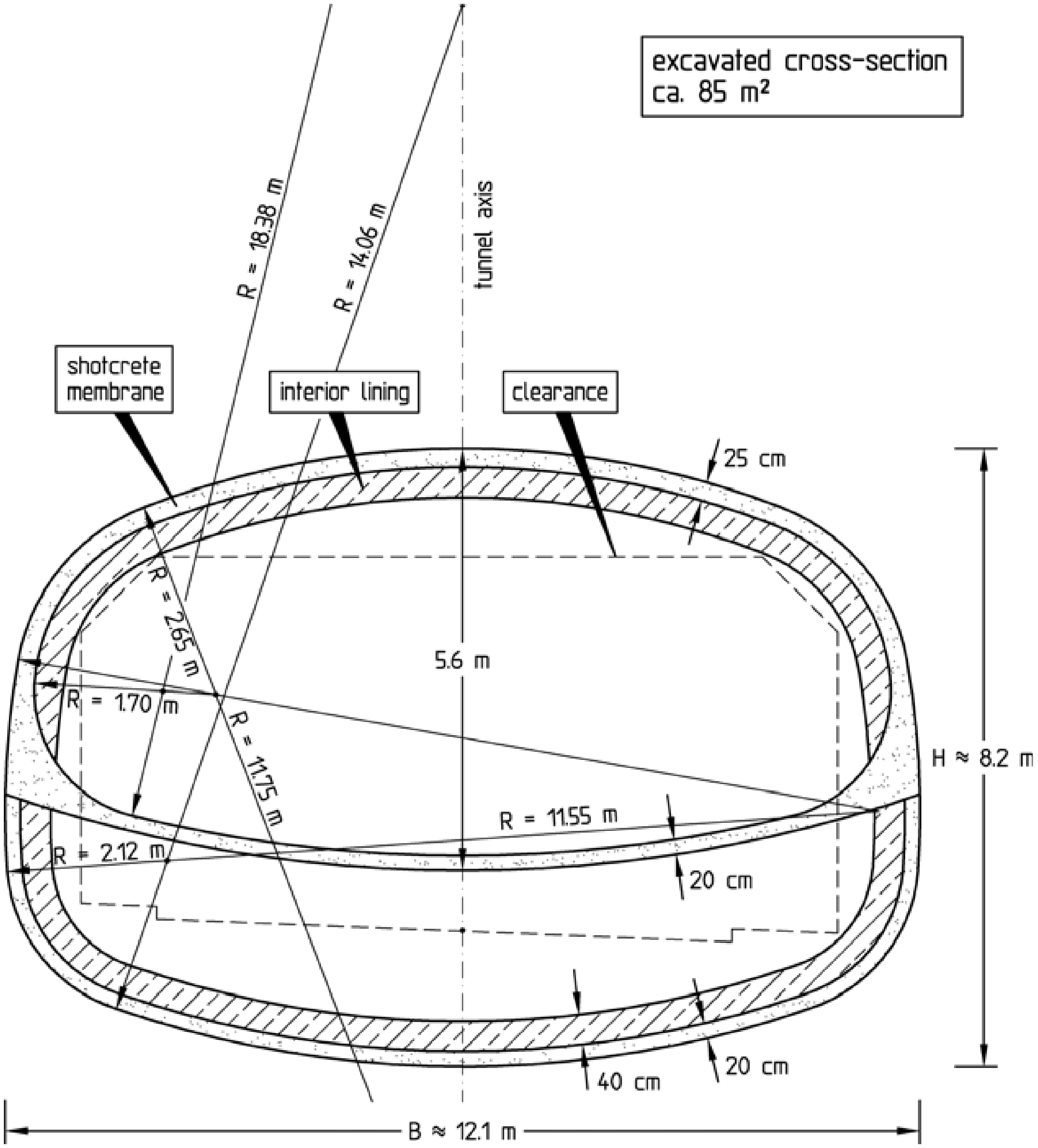


Fig. 4.24: Elite Tunnel, contractor's design proposal, cross-section

To protect the work space and to relieve the area of the temporary tunnel face, the tunnel is planned to be excavated under the protection of pipe umbrellas consisting of 12 m long steel pipes with a diameter of ca 17 cm (Fig. 4.25 and 4.26). The pipes are spaced at approx. 40 cm and have a wall thickness of 7 mm. Four rebars are entered into each pipe to increase the section modulus. Further, the pipes are filled with B25 concrete (Fig. 4.26). The steel pipes overlap by 3 m (Fig. 4.25). As mentioned above, the pipe umbrellas are designed to bridge the work space and should therefore be able to carry the entire load resulting from overburden and traffic. Their design is therefore based on these loads (see Chapter 4.2.5).

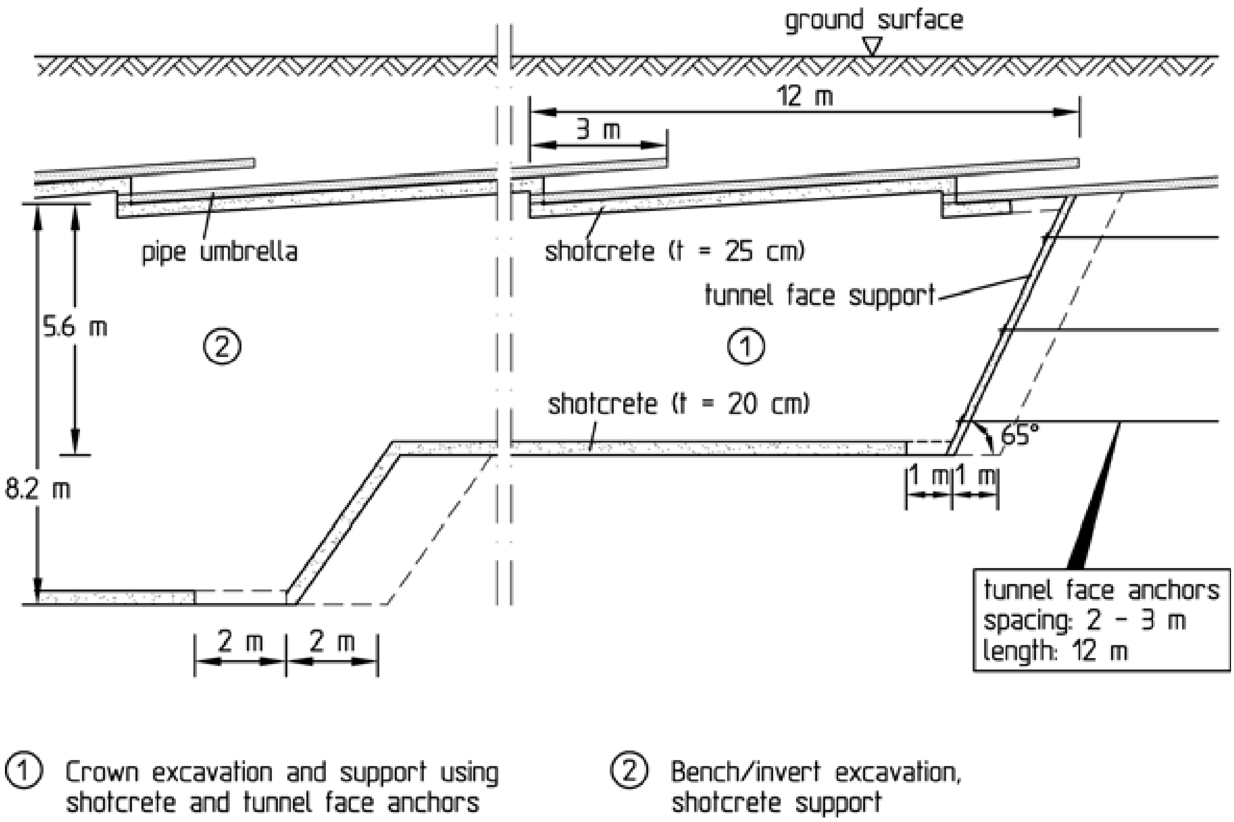


Fig. 4.25: Elite Tunnel, contractor's design proposal, excavation and support, longitudinal section

If the sand has a cohesion or an apparent cohesion, the gaps between the pipes can be bridged by the arching effect. If dry, loose sand or cohesionless fill appears in the roof area, the gaps between the pipes must be supported, e. g. by the installation of additional pipes (Fig. 4.26). Alternatively, the gaps may be bridged by steel plates welded to the pipes, or they may be supported using spiles (see Chapter 4.2.6).

The contractor's design proposal includes a steep temporary tunnel face inclined at approx. 65° and supported by reinforced shotcrete and tunnel face anchors (SN-anchors) spaced at 2 to 3 m. The tunnel face anchors shall be 12 m long, just as the pipes (Fig. 4.25). By installing the anchors in parallel with the construction of the pipe umbrellas, the interruptions of the heading necessary due to the construction of the pipe umbrellas should be minimized.

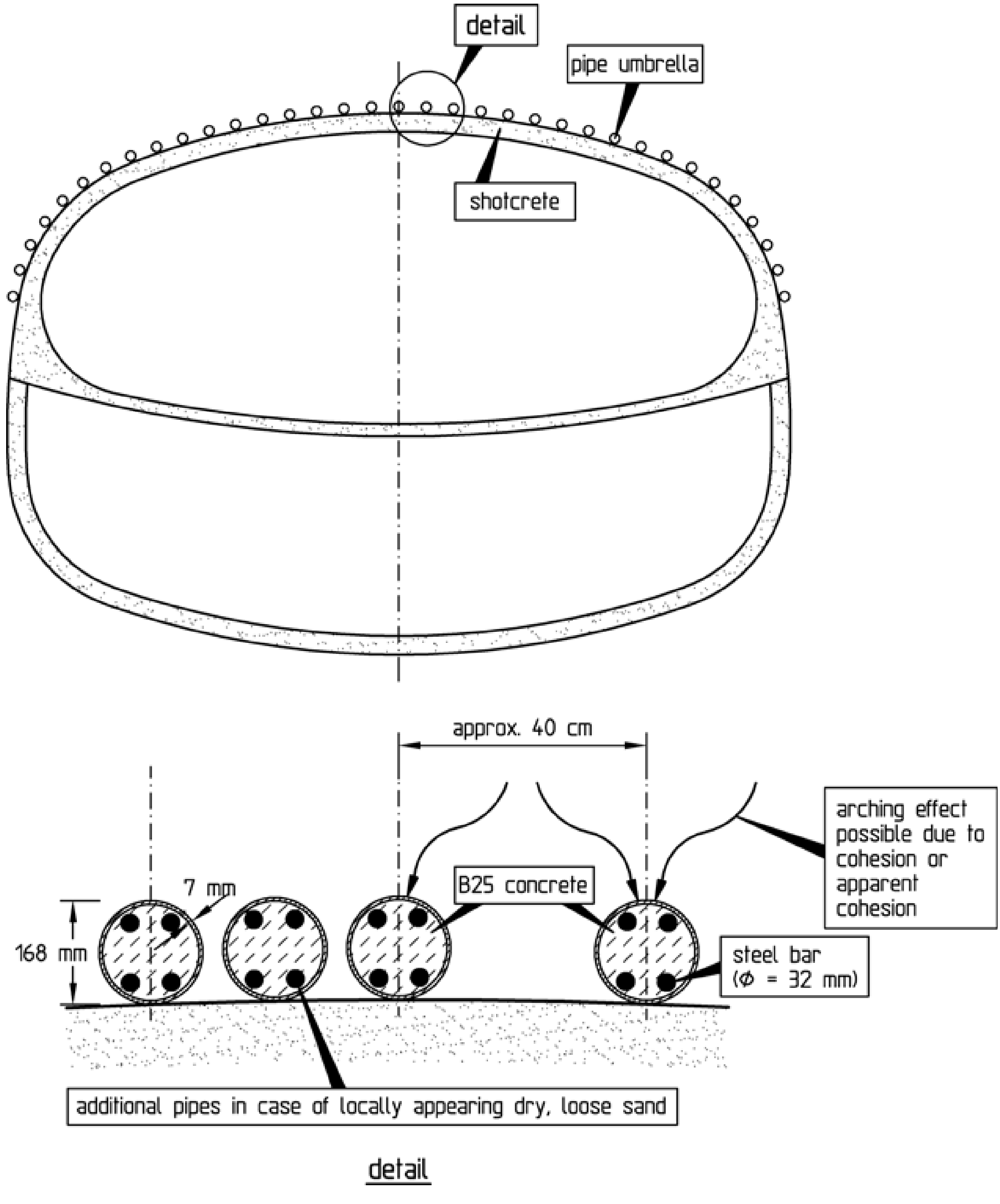


Fig. 4.26: Elite Tunnel, contractor's design proposal, cross-section with pipe umbrella and detail

Round lengths of approx. 1 m were specified for the crown heading (Fig. 4.25). After each round, the shotcrete support including the closing of the temporary invert are to be completely installed before the excavation continues. This way the unsupported span in the crown never amounts to more than 1 m. Round lengths of approx.

2 m were specified for the excavation of bench and invert (Fig. 4.25). Here as well the shotcrete support are to be completely installed after each round including its closing at the invert, so that the unsupported length amounts to 2 m at the most.

The contractor's design proposal has essentially two advantages over the tendering design (see Fig. 4.19 and 4.20). The heading and the installation of the reinforced concrete interior lining are carried out in two separate working steps, which leads to a considerable gain of time and thus to cost savings. Further, the contractor's design proposal does not require the use of a microtunneling machine.

4.2.5 Stability analyses

Design of the shotcrete support

For the dimensioning of the shotcrete support two-dimensional analyses were carried out using the program system FEST03 (Wittke, 2000).

Fig. 4.27 shows the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters the analyses were based on. The computation section consists of a 48 m wide, 45 m high and 1 m thick slice. The FE-mesh was divided into 630 isoparametric elements with a total of 3958 nodes.

For the nodes on the bottom boundary (z = 0) and on the lateral boundaries (x = 0 and x = 48 m) sliding supports were selected as boundary conditions. On the two planes perpendicular to the tunnel axis, equal displacements were prescribed for the nodes with equal x- and z-coordinates (Wittke, 2000). All nodes were assumed fixed in y-direction. The traffic load acting on the ground surface was accounted for by a surface loading (p_t = 23 kN/m^2). The overburden amounts to 4.5 m (Fig. 4.27).

The ground was subdivided into two soil layers. Down to a depth of 8.5 m a medium dense sand was assumed with a Young's modulus of E = 100 MN/m^2 and a Poisson's ratio of ν = 0.35, corresponding to a bulk modulus of E_s = 160 MN/m^2. Below that, a dense sand with E = 250 MN/m^2 and ν = 0.35 corresponding to E_s = 400 MN/m^2 was specified. To be on the safe side, no cohesion was assumed for any of the two sands (c' = 0). An angle of friction of φ' = 30°

was specified. For the shotcrete membrane, a Young's modulus of $E = 15000\ MN/m^2$ was assumed (Fig. 4.27).

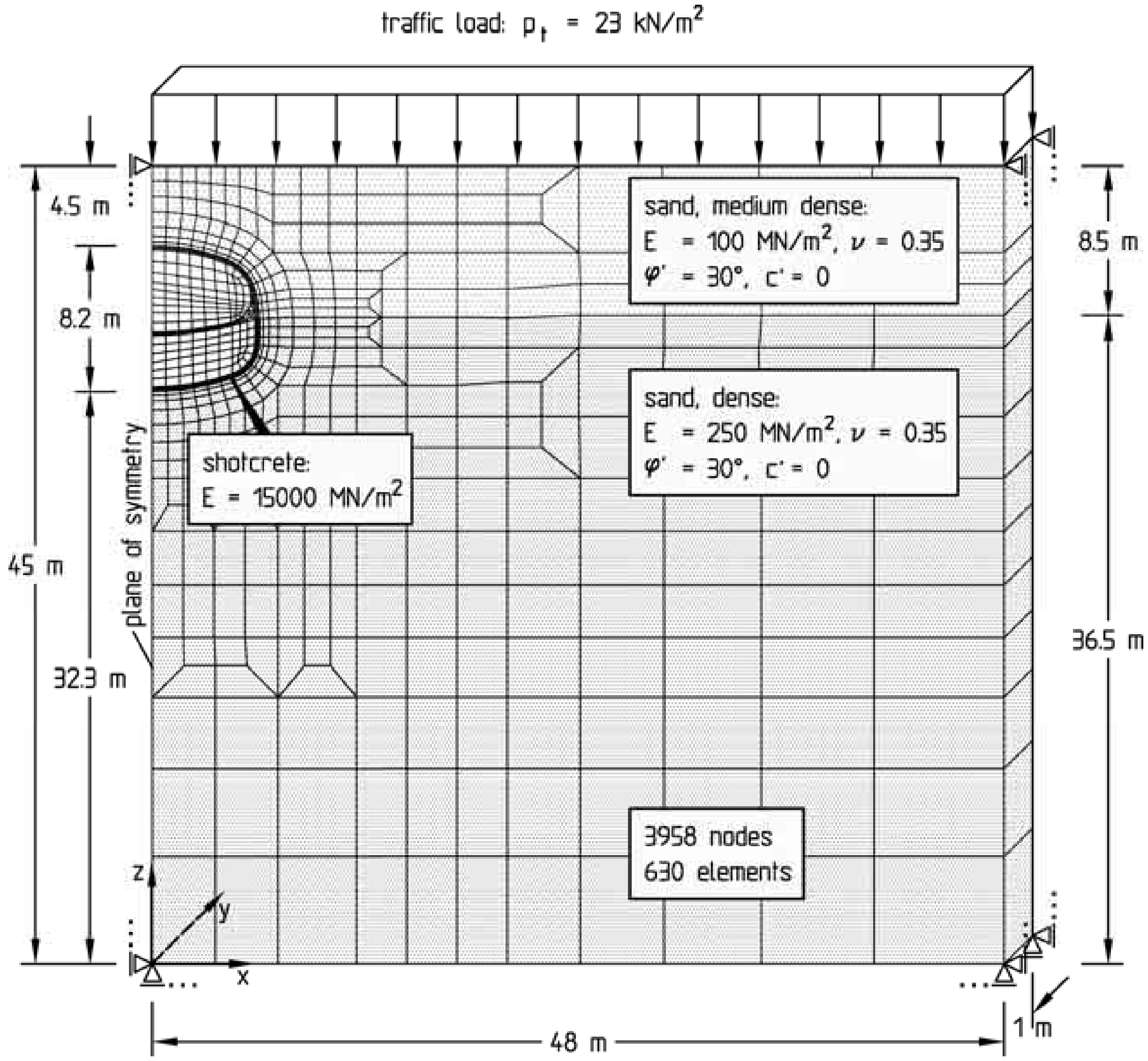


Fig. 4.27: Computation section, FE-mesh, boundary conditions, ground profile and parameters for two-dimensional analyses

Since the ground profile and the cross-section of the tunnel are symmetrical to the tunnel axis, only one half of the tunnel cross-section was modeled (Fig. 4.27).

Fig. 4.28 illustrates the computation steps. The 1st computation step comprises the determination of the state of stress and deformation resulting from the dead weight of the ground and the traffic load p_t. In the 2nd computation step the excavation and the shotcrete support of the crown are modeled. The 3rd computation step represents the excavation and shotcrete support of bench and invert.

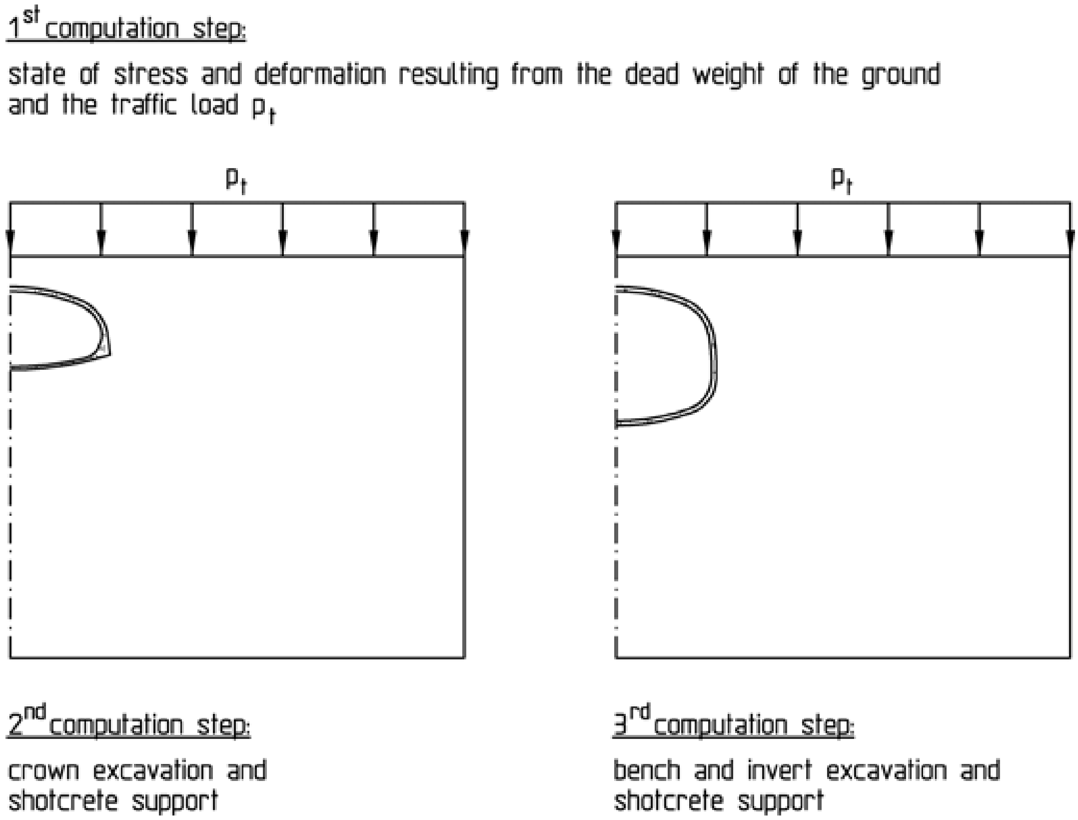


Fig. 4.28: Two-dimensional analysis, computation steps

It should be pointed out that the simulation of excavation and support in one computation step leads to a overestimation of the loading of the shotcrete membrane, since the displacements preceding the excavation, which have already occurred before the support is installed, are not taken into account in the analysis. This in turn results in the tunneling-induced ground surface subsidence being underestimated.

Fig. 4.29 shows the displacements of the ground surface and the tunnel contour (crown) computed for the 2nd computation step relative to the 1st computation step. The largest displacements result at the roof with 25 mm and at the ground surface with a maximum of 22 mm. These values change only marginally in the 3rd computation step.

Fig. 4.30 depicts the bending moments and normal thrust in the shotcrete membrane determined for the 2nd and 3rd computation step. Because of the small radius in the case of the crown heading the bending loading of the shotcrete support is larger in the 2nd computation step than in the 3rd computation step. The largest moments result on the sidewalls. In the 3rd computation step the largest bending moment occurs at the transition from the roof to the sidewalls. Compressive normal thrust are computed for the entire tunnel circumference.

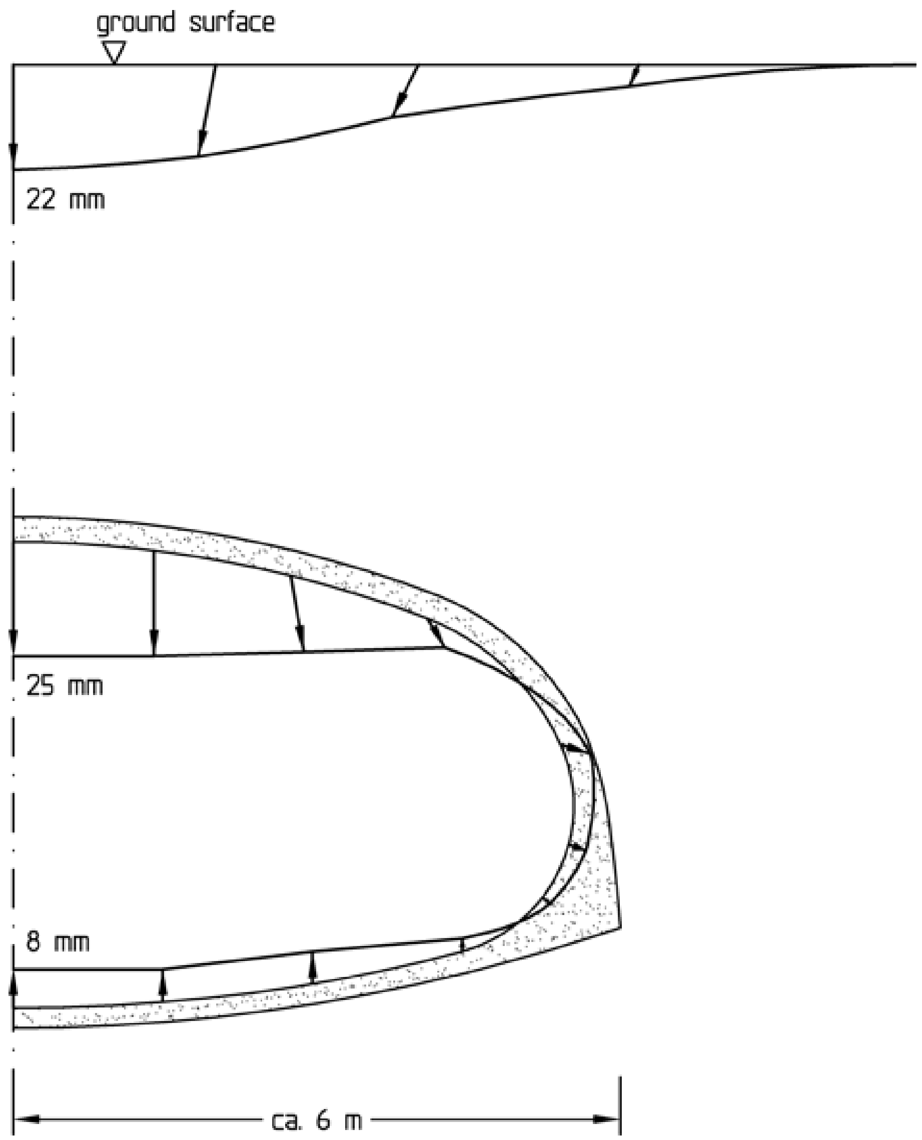
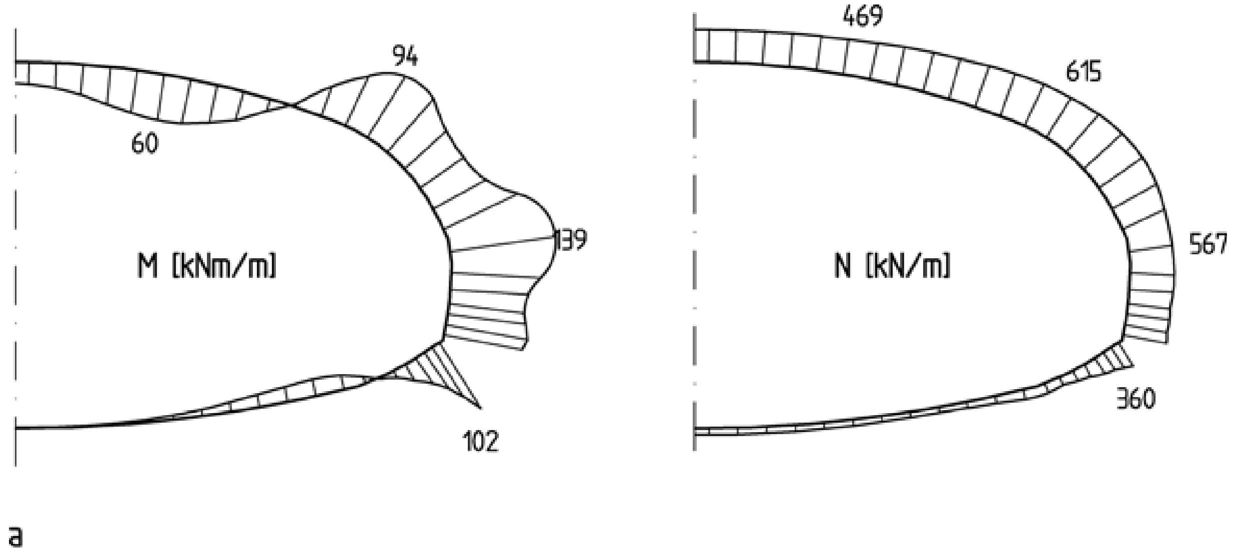


Fig. 4.29: Displacements, 2nd – 1st computation step

In Fig. 4.31 the statically required amounts of reinforcement are given for the design of the shotcrete membrane according to DIN 1045 (1988) for a safety factor of $\eta = 1.45$. This factor of safety includes the safety factors of $\eta_t = 1.6$ for the traffic load (p_t) and $\eta_\gamma = 1.4$ for the overburden weight ($\gamma \cdot H_o$) which were predetermined by the constructor:

$$\eta = \frac{\eta_t \cdot p_t + \eta_\gamma \cdot \gamma \cdot H_o}{p_t + \gamma \cdot H_o} \approx 1.45 \tag{4.2}$$



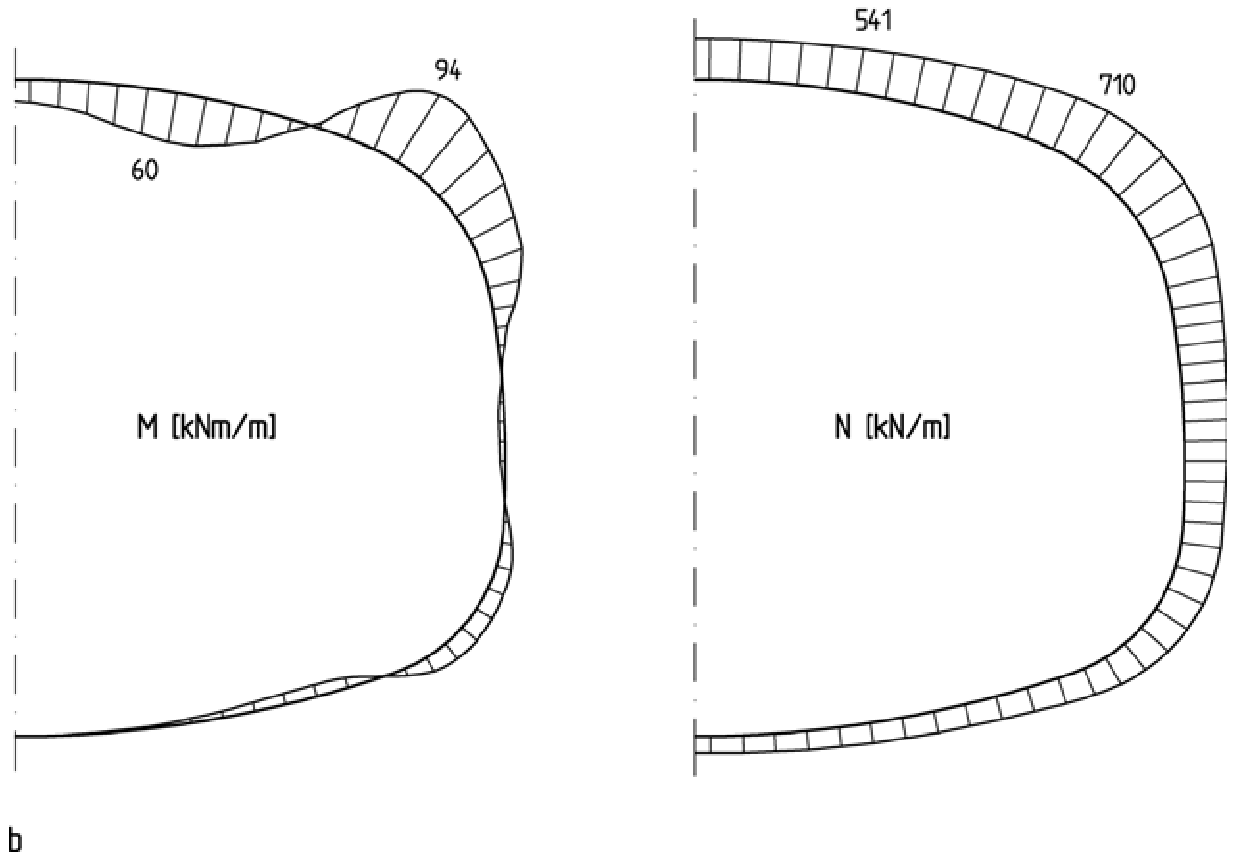


Fig. 4.30: Stress resultants in the shotcrete membrane: a) 2nd computation step; b) 3rd computation step

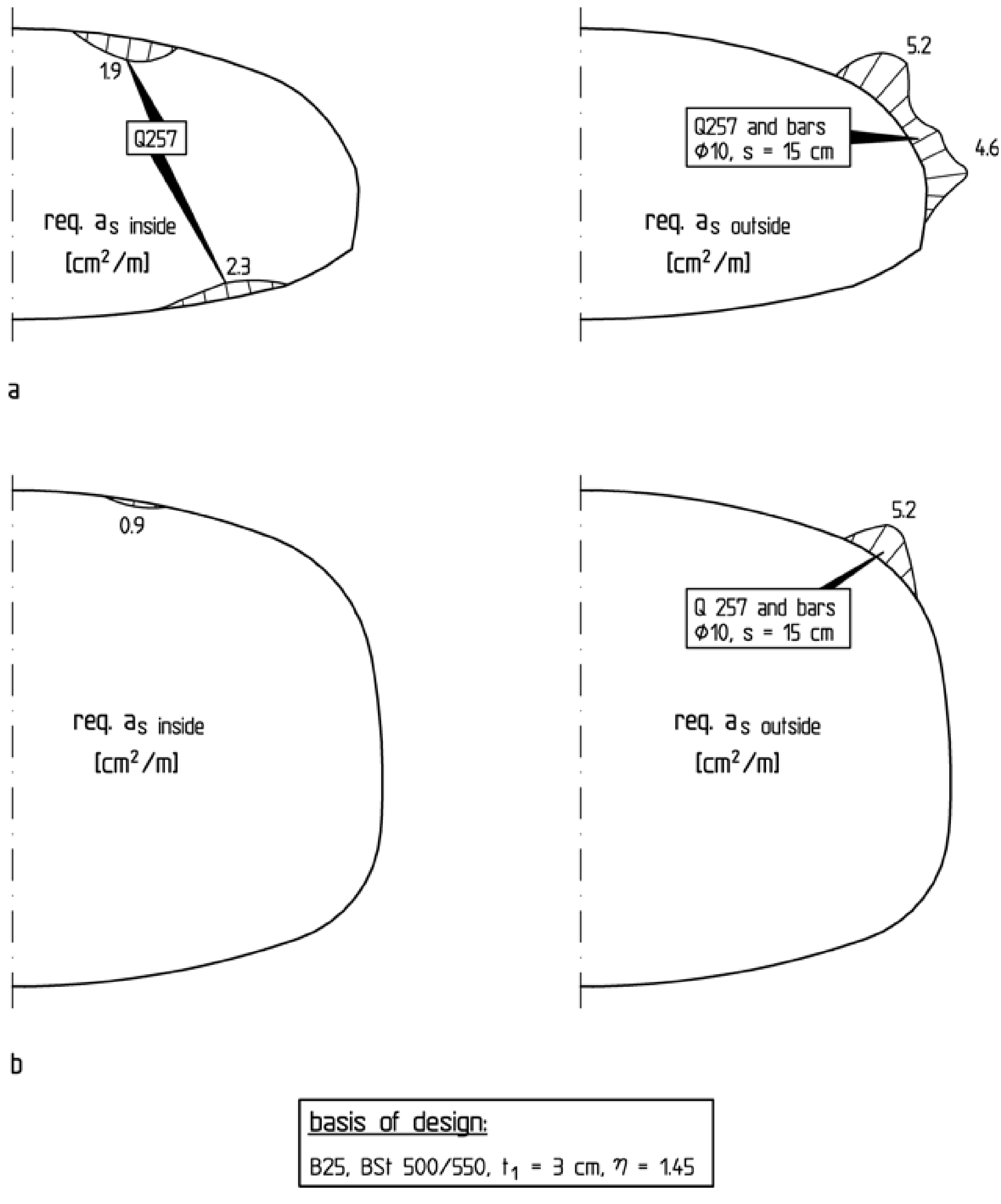


Fig. 4.31: Statically required reinforcement of the shotcrete membrane: a) 2nd computation step; b) 3rd computation step

For a concrete grade of B25, a steel grade of BSt 500/550 and a surface distance of the reinforcement of t_1 = 3 cm, steel cross-sections of ≤ 2.3 cm^2/m on the inside and ≤ 5.2 cm^2/m on the outside are evaluated as maximum statically required reinforcement.

These can be covered by steel fabric mats Q295 on the inside and on the outside and by supplementary reinforcement within the upper sidewall area (Fig. 4.31). In the bench and invert area, the inside steel fabric mat Q295 can be omitted. Fig. 4.32 shows the design and the reinforcement of the shotcrete membrane in the area of the crown's foot.

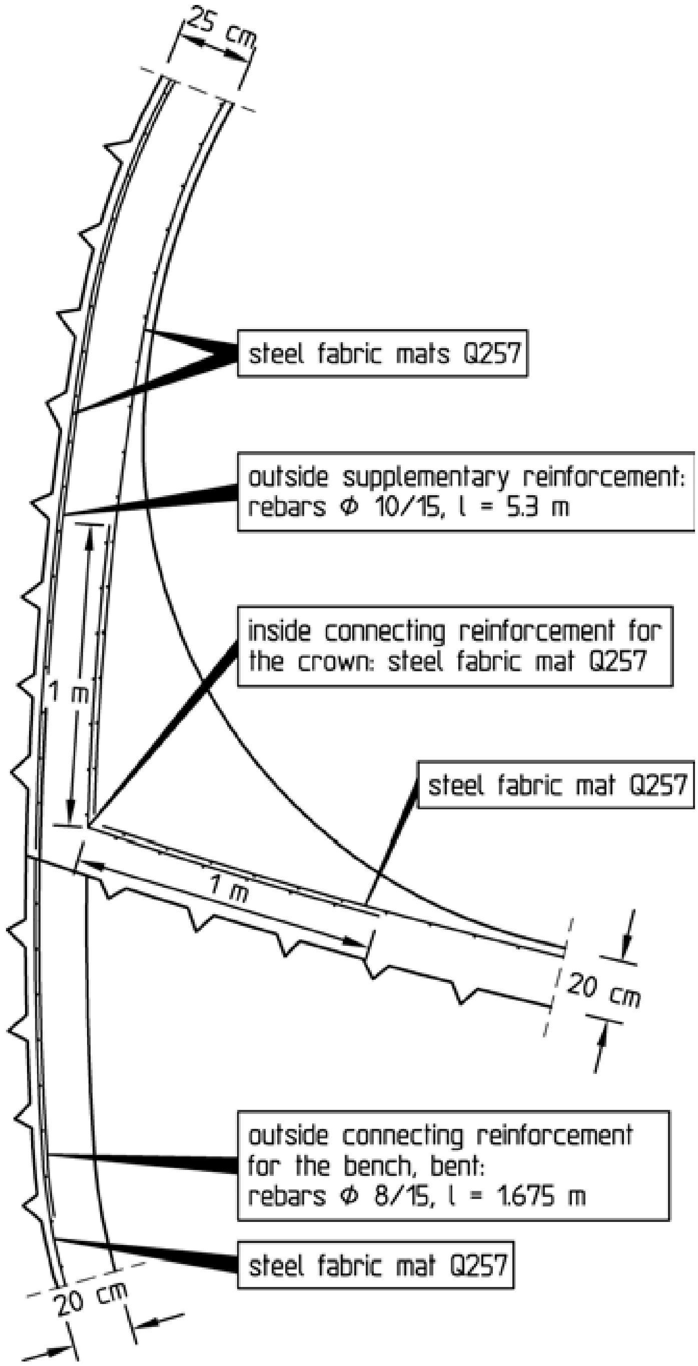


Fig. 4.32: Design and reinforcement of the shotcrete membrane in the area of the crown's foot

Stability of the temporary tunnel face

To investigate the stability of the temporary tunnel face, three-dimensional analyses were carried out using the program system FEST03 (Wittke, 2000).

Fig. 4.33 shows the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters these analyses were based upon. The computation section is 40 m wide, 40 m high and 62 m long. The FE-mesh was divided into 5848 isoparametric elements with a total of 14945 nodes.

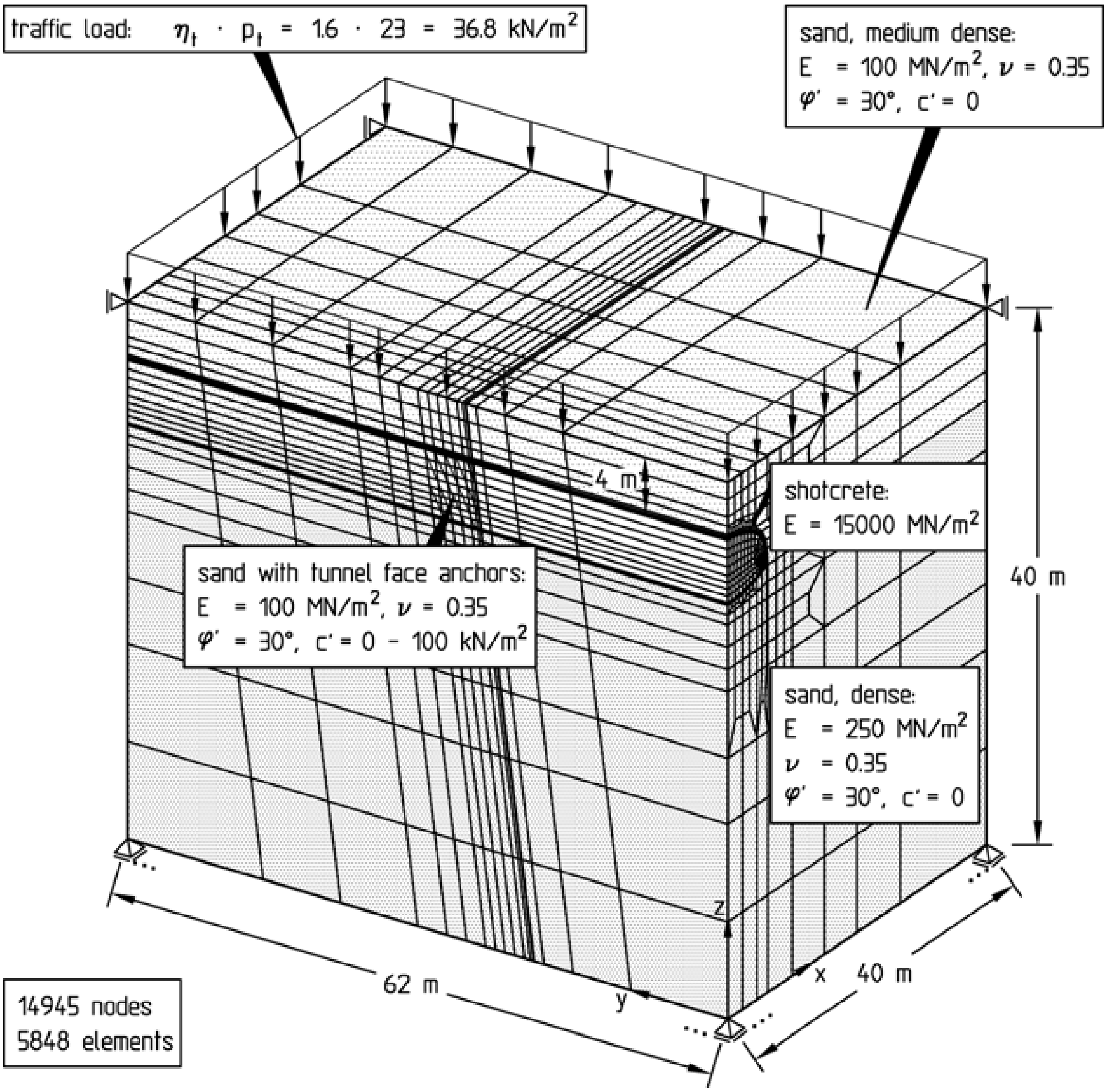


Fig. 4.33: Computation section, FE-mesh, boundary conditions, ground profile and parameters for three-dimensional analyses

Sliding supports were selected as boundary conditions for the nodes on the bottom boundary (z = 0). The nodes on the planes x = 0 and x = 40 m were assumed fixed in x-direction, and the nodes on the planes y = 0 and y = 62 m were fixed in y-direction. The traffic load acting on the ground surface was increased by the safety factor η_t = 1.6 and applied as a uniform surface load. An overburden of 4 m was specified (Fig. 4.33).

The ground profile and the parameters corresponded to the assumptions made for the two-dimensional analyses. The weight of the soil (γ = 20 kN/m^3) and thus also the overburden pressure were increased by the safety factor η_γ = 1.4.

The tunnel face was assumed inclined at 80°. The tunnel face anchors were considered by a cohesion in the respective area (Fig. 4.33). Five cases were investigated: c' = 0 (no tunnel face anchors), c' = 25 kN/m^2, c' = 50 kN/m^2, c' = 75 kN/m^2 and c' = 100 kN/m^2, corresponding to an anchor arrangement with a raster spacing between 1 m x 1 m and 2.1 m x 2.1 m.

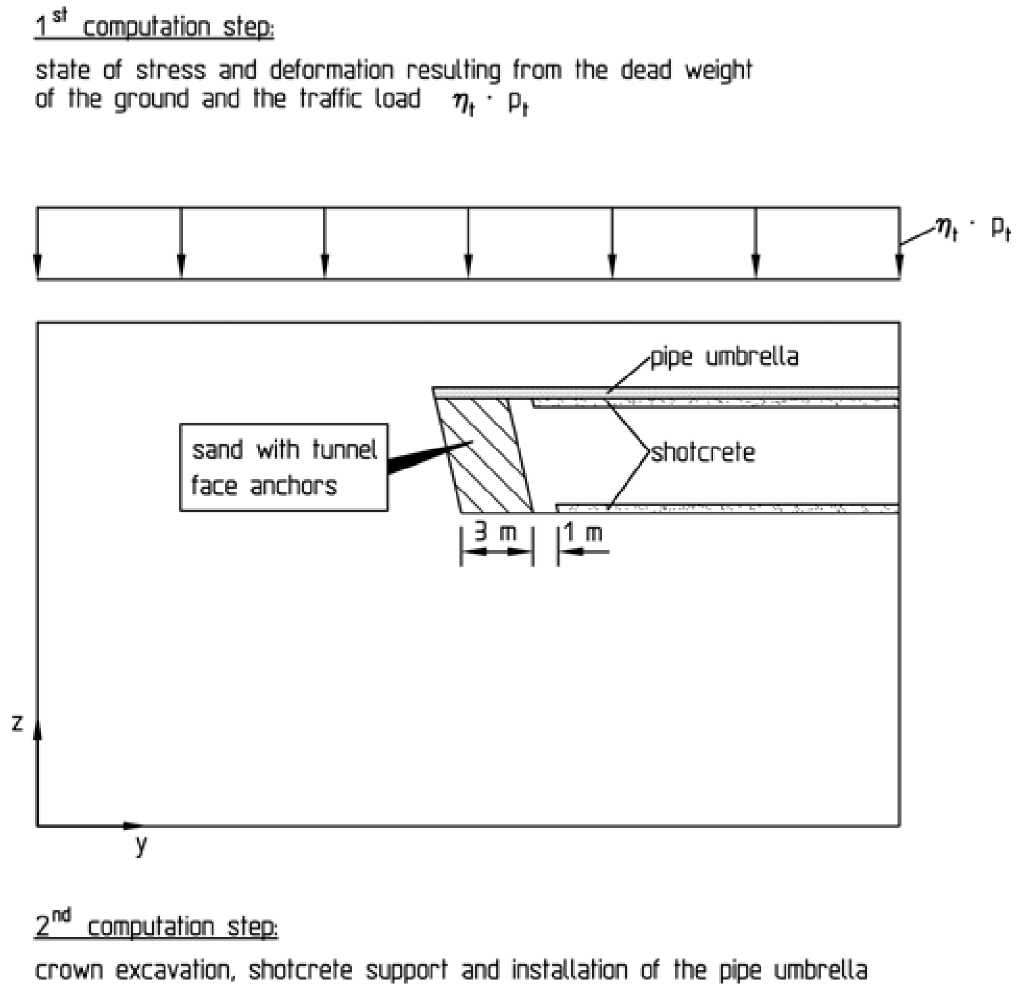


Fig. 4.34: Three-dimensional analyses, computation steps

The three-dimensional analyses were carried out in two computation steps as a simplification (Fig. 4.34). Following the analysis of the primary state in the 1st computation step, the excavation and simultaneous shotcrete support of the crown and the installation of the pipe umbrella were simulated in the 2nd computation step. The unsupported area adjacent to the tunnel face was assumed to be 1 m deep. The shotcrete support of the tunnel face was not taken into account.

It should be pointed out that with these analyses as well the tunneling-induced displacements of the soil are underestimated, since the support exerted by the shotcrete membrane is overestimated with the computation sequence outlined in Fig. 4.34. For a realistic computation of the displacements, one of the two procedures for the simulation of a three-dimensional tunnel heading described in Wittke (2000) (step-by-step method or iterative method) would be necessary. The analyses, however, had the only purpose of assessing the stability of the temporary tunnel face.

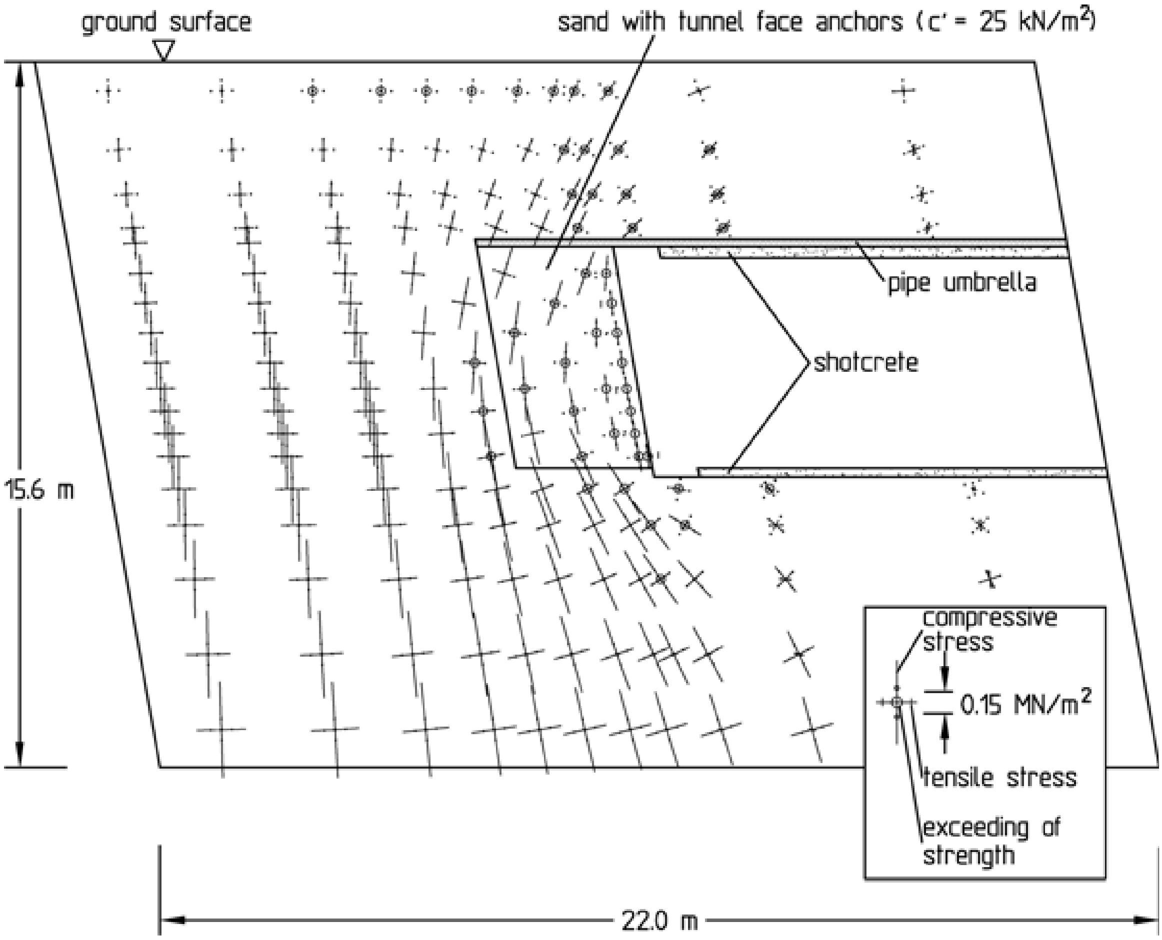


Fig. 4.35: Principal normal stresses computed after completion of the viscoplastic iterative analysis and elements with exceeded strength ($c' = 25$ kN/m^2), 2nd computation step (vertical section through the tunnel axis)

As a consequence of the excavation, the strength of the soil is locally exceeded in the area of the tunnel face. Fig. 4.35 shows the principal normal stresses computed for the 2nd computation step and the elements with exceeded strength, specifically marked, in a vertical section through the tunnel axis for the case of a cohesion of the anchored area of c' = 25 kN/m^2. A corresponding representation of the computed displacements is given in Fig. 4.36. The displacements result from elastic and inelastic deformations of the soil. The latter are computed in a viscoplastic iterative analysis (Wittke, 2000). Fig. 4.35 and 4.36 show the results of the 2nd computation step after completion of the viscoplastic iterative calculation.

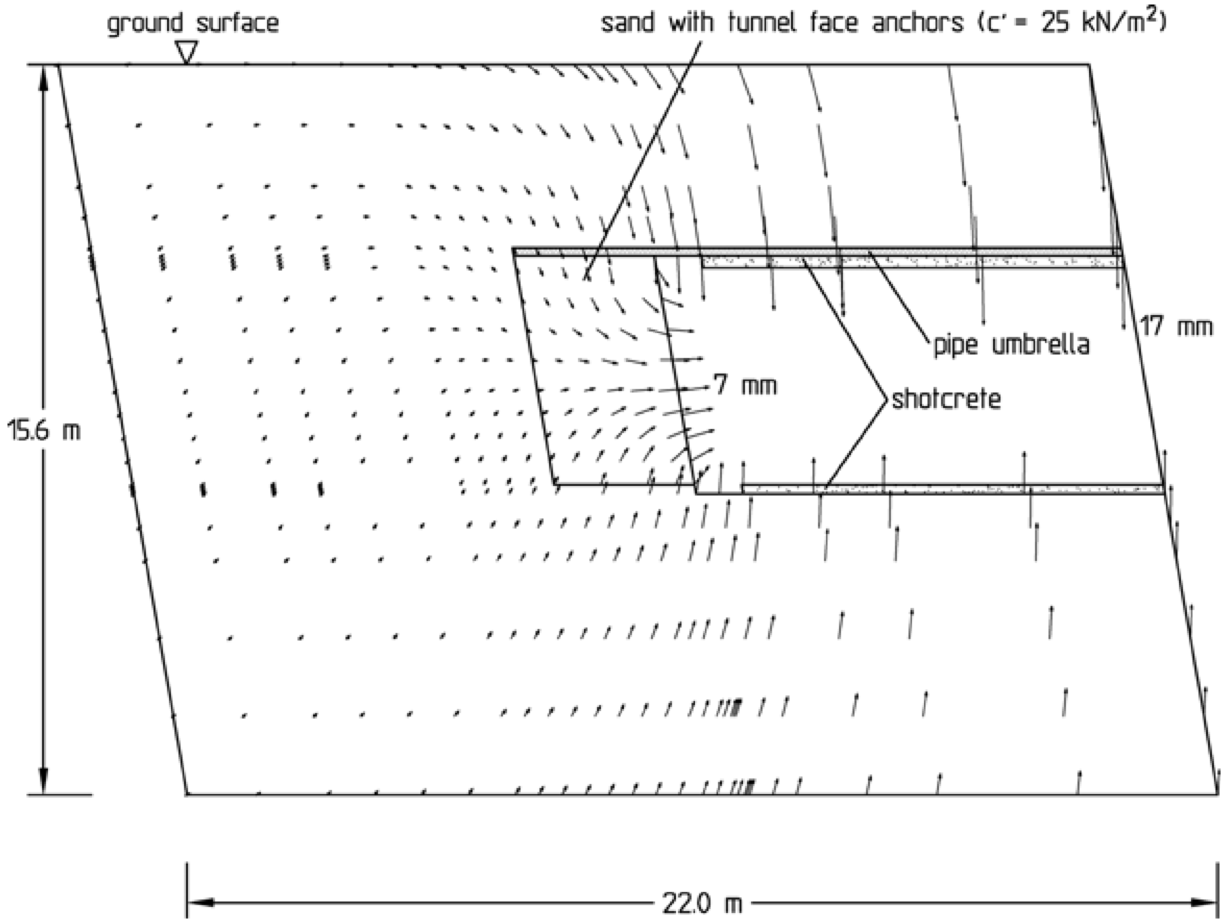


Fig. 4.36: Displacements computed after completion of the viscoplastic iterative analysis (c' = 25 kN/m^2), 2nd – 1st computation step (vertical section through the tunnel axis)

The criterion for the proof of stability of the tunnel face is the convergency of the nodal displacements in the course of the viscoplastic iterative analysis. Fig. 4.37 shows the development of the displacements computed for a node on the tunnel face in the course of the viscoplastic iterative analysis in the 2nd computation step for the five investigated cases. For the case without tunnel face anchors (c' = 0), the convergency of the displacement of this node cannot be proven by the analysis. In all other cases (c' ≥ 25 kN/m^2) the displacement converges in the computations. For c' = 25 kN/m^2 a displacement of only 7 mm results (Fig. 4.36 and 4.37). The cohesion of 25 kN/m^2 corresponds to an anchor raster of 2.1 x 2.1 m.

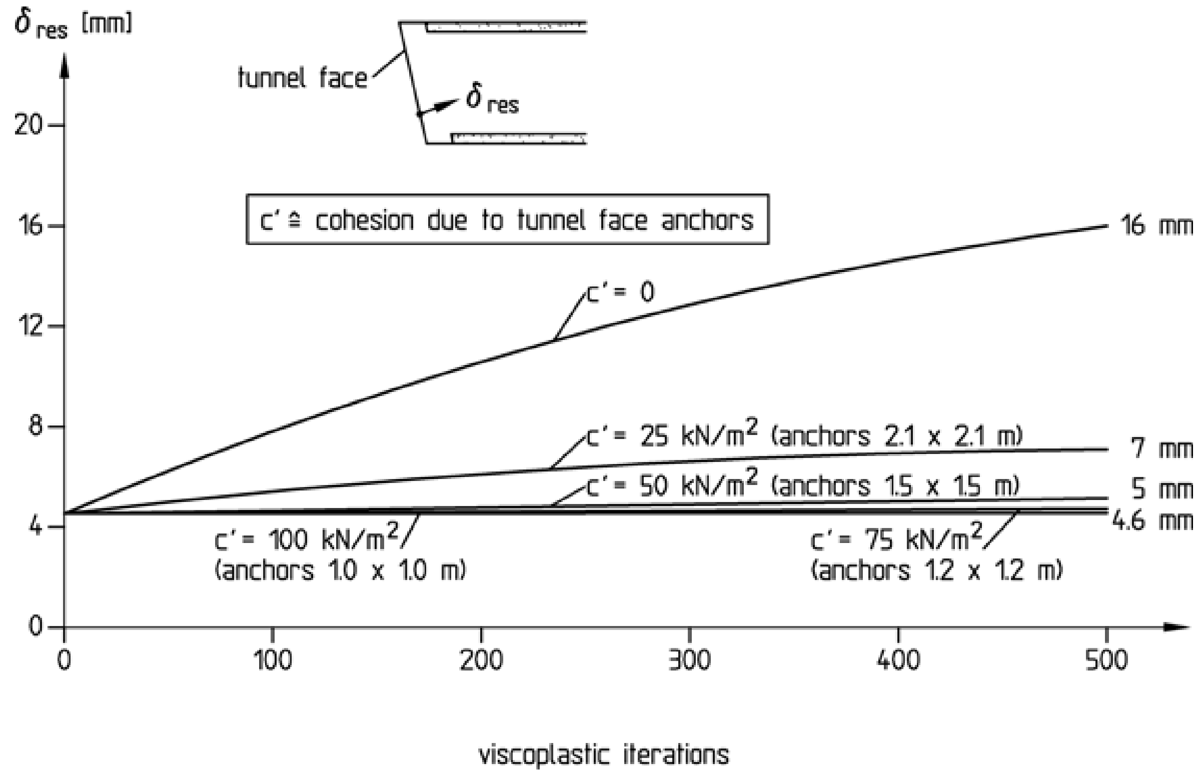


Fig. 4.37: Displacement of a node on the tunnel face in the course of the viscoplastic iterative analysis, 2nd computation step

Design of the pipe umbrella

As mentioned above, the pipe umbrella must be able to carry the loads from traffic and from the weight of the overburden. As for the proof of stability of the tunnel face, the traffic load was increased by the factor of safety η_t = 1.6 and the overburden pressure of the soil by the factor of safety η_γ = 1.4 for the design of the pipe umbrella, as requested by the client.

As explained in Chapter 4.2.4, the round length for the crown heading amounts to 1 m and the shotcrete support is to be installed and temporarily closed at the invert after each round. For the design of the pipe umbrella it is conservatively assumed that its maximum free span amounts to approx. 2 m. It is taken into account here that the green shotcrete does not have any bearing capacity yet directly after its application (see Chapter 2.1.3).

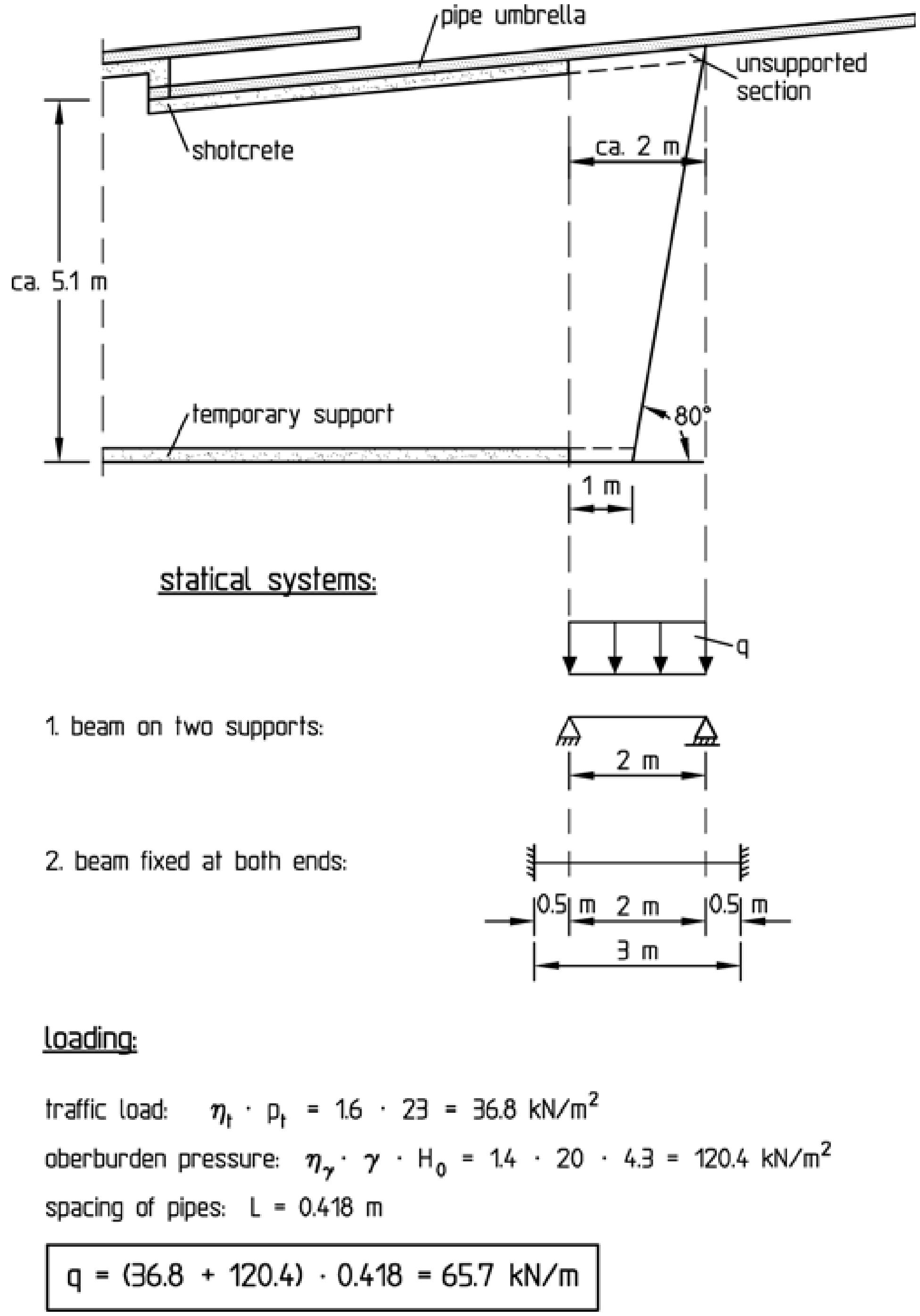
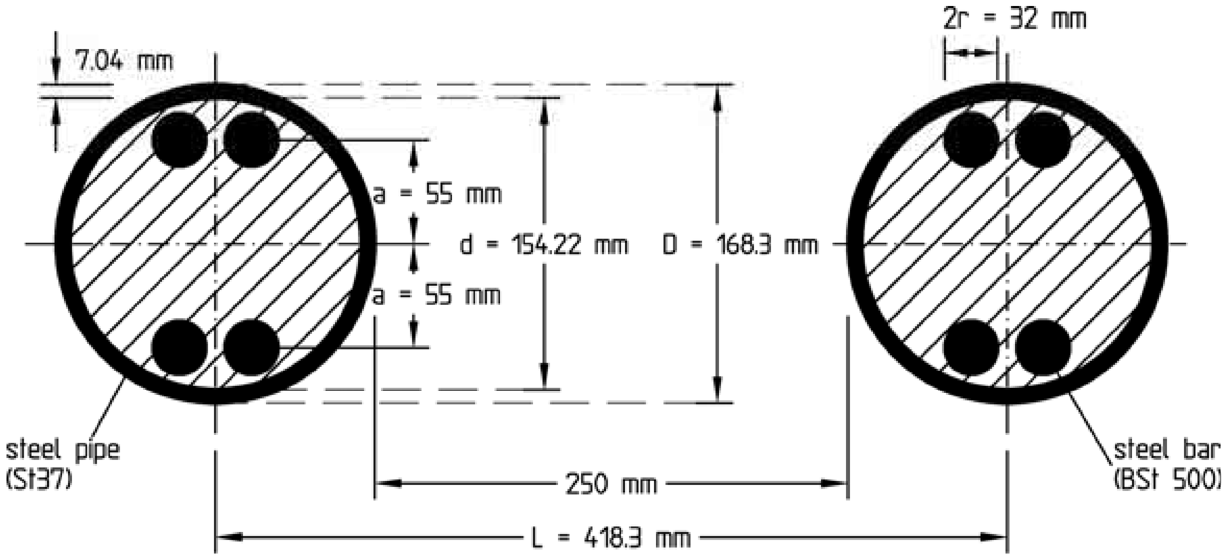


Fig. 4.38: Design of the pipe umbrella, statical systems and loading

At 80°, the inclination of the tunnel face is assumed somewhat steeper than in the design (see Fig. 4.25). This results in a distance between the tunnel face and the closed support of approx. 2 m at the roof and approx. 1 m at the temporary crown invert (Fig. 4.38).



$$I = \frac{\pi\,(D^4 - d^4)}{64} + 4 \cdot \frac{r^4 \cdot \pi}{4} + 4 \cdot a^2 \cdot \pi \cdot r^2$$

$$= \frac{\pi\,(168.3^4 - 154.22^4)}{64} + 16^4 \cdot \pi + 4 \cdot 55^2 \cdot \pi \cdot 16^2$$

$$= (11.61 + 0.206 + 9.731) \cdot 10^6\ \mathrm{mm}^4 = \underline{21.55 \cdot 10^6\ \mathrm{mm}^4}$$

$$W = \frac{I}{D\,/\,2} = \frac{21.55 \cdot 10^6\ \mathrm{mm}^4}{84.15\ \mathrm{mm}} = \underline{256 \cdot 10^3\ \mathrm{mm}^3}$$

Fig. 4.39: Design of the pipe umbrella, calculation of the section modulus of the pipes

The beam on two supports and the beam fixed at both ends are considered as statical systems for the design of the pipe umbrella. For the reasons given above, the maximum span of the beam is assumed as 2 m. For the fixed beam, 0.5 m each on both ends of the beam are added to the length and assumed to be fixed. From the superposition of the traffic load and the overburden pressure, taking into account the spacing of the pipes (L = 418 mm), the loading of the beam results to q = 65.7 kN/m (Fig. 4.38).

In Fig. 4.39 the calculation of the section modulus W of the pipes, which is required for the stress proof is specified. For the proof of safety, the computed stresses are compared to the yield stress of the pipes made from steel of the grade St37 ($\beta_y = \sigma_{adm} = 240$ N/mm^2). This is permissible, since the assumed loads were provided with factors of safety. The beam fixed at both ends is decisive for the design with a computed tension of 192.2 N/mm^2 (Fig. 4.40).

Loading of pipe umbrella (see figure 4.38):

$$q = (36.8 + 120.4) \cdot 0.418 = 65.7 \text{ kN/m}$$

Admissible stress (yield stress):

steel pipe (St 37) : $\sigma_{adm} = 240 \text{ N/mm}^2$

1. Beam on two supports:

$$M = \frac{q \cdot l^2}{8} = \frac{65.7 \cdot 2^2}{8} = 32.85 \text{ kNm}$$

$$\sigma = \frac{M}{R} = \frac{32.85 \cdot 10^6 \text{ Nmm}}{256 \cdot 10^3 \text{ mm}^3} = 128.3 \text{ N/mm}^2 < \sigma_{adm}$$

$$\delta_{max} = \frac{5}{384} \cdot \frac{q \cdot l^4}{E \cdot I} = \frac{5 \cdot 65.7 \cdot 2^4 \cdot 10^{12}}{384 \cdot 210000 \cdot 21.55 \cdot 10^6} = 3.0 \text{ mm}$$

2. Beam fixed at both ends:

$$M = \frac{q \cdot l^2}{12} = \frac{65.7 \cdot 3^2}{12} = 49.3 \text{ kNm}$$

$$\sigma = \frac{M}{R} = \frac{49.3 \cdot 10^6 \text{ Nmm}}{256 \cdot 10^3 \text{ mm}^3} = 192.2 \text{ N/mm}^2 < \sigma_{adm}$$

$$\delta_{max} = \frac{1}{384} \cdot \frac{q \cdot l^4}{E \cdot I} = \frac{1 \cdot 65.7 \cdot 3^4 \cdot 10^{12}}{384 \cdot 10^6 \cdot 210000 \cdot 21.55} = 3.1 \text{ mm}$$

Fig. 4.40: Design of the pipe umbrella, stresses and deflection

The deflection of the pipe umbrella is estimated at approx. 3 mm (Fig. 4.40).

4.2.6 Construction

The Elite Tunnel was the first tunnel in Israel to be constructed by the NATM. Particularly high demands were therefore made on the technical construction supervision provided by WBI. The demands focused mainly on the works for the excavation and the installation of the shotcrete membrane, which had to be continuously supervised.

Differing from the design the pipes only had a length of 10.4 m. Since the overlap of the pipe umbrellas amounts to 3 m, it was possible to excavate for 7.4 m under one umbrella before the pipes for the next umbrella had to be installed (Fig. 4.41).

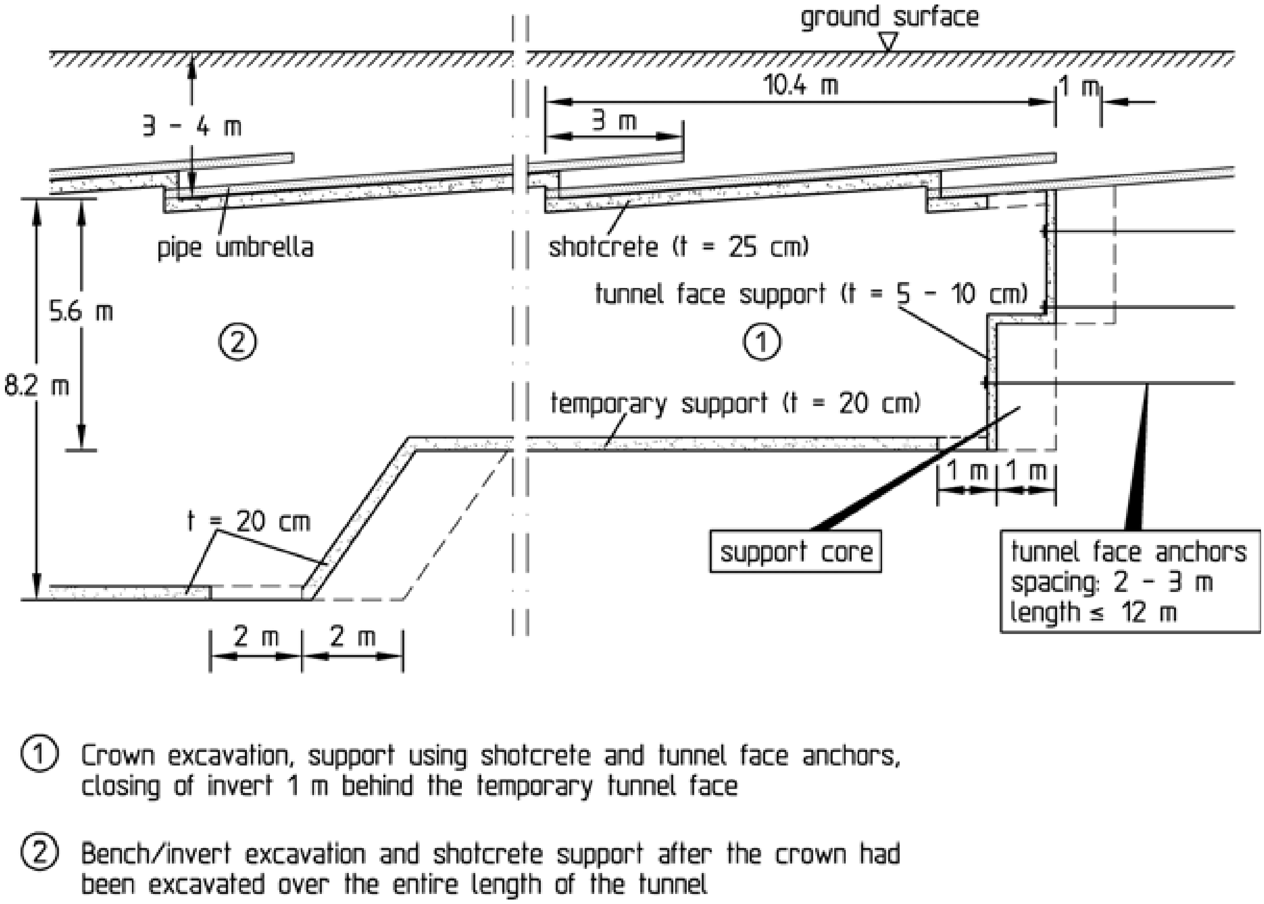


Fig. 4.41: Elite Tunnel, excavation and support, longitudinal section

At the roof and above locally fill or layers with cohesionless, fine grained, loose sands were encountered. Here, the gaps between the steel pipes had to be supported (Fig. 4.42). In the area of the first two pipe umbrellas the gaps were supported by steel plates welded to the pipes (Fig. 4.42a). In the area of pipe umbrellas 3 to 14 cemented rebars (spiles) were installed between the steel pipes (Fig. 4.42b). The drillings for the spiles served at the same time to explore the ground in advance. In the area of the last two pipe umbrellas, additional steel pipes, filled with B25 concrete but not reinforced, were installed (Fig. 4.42c).

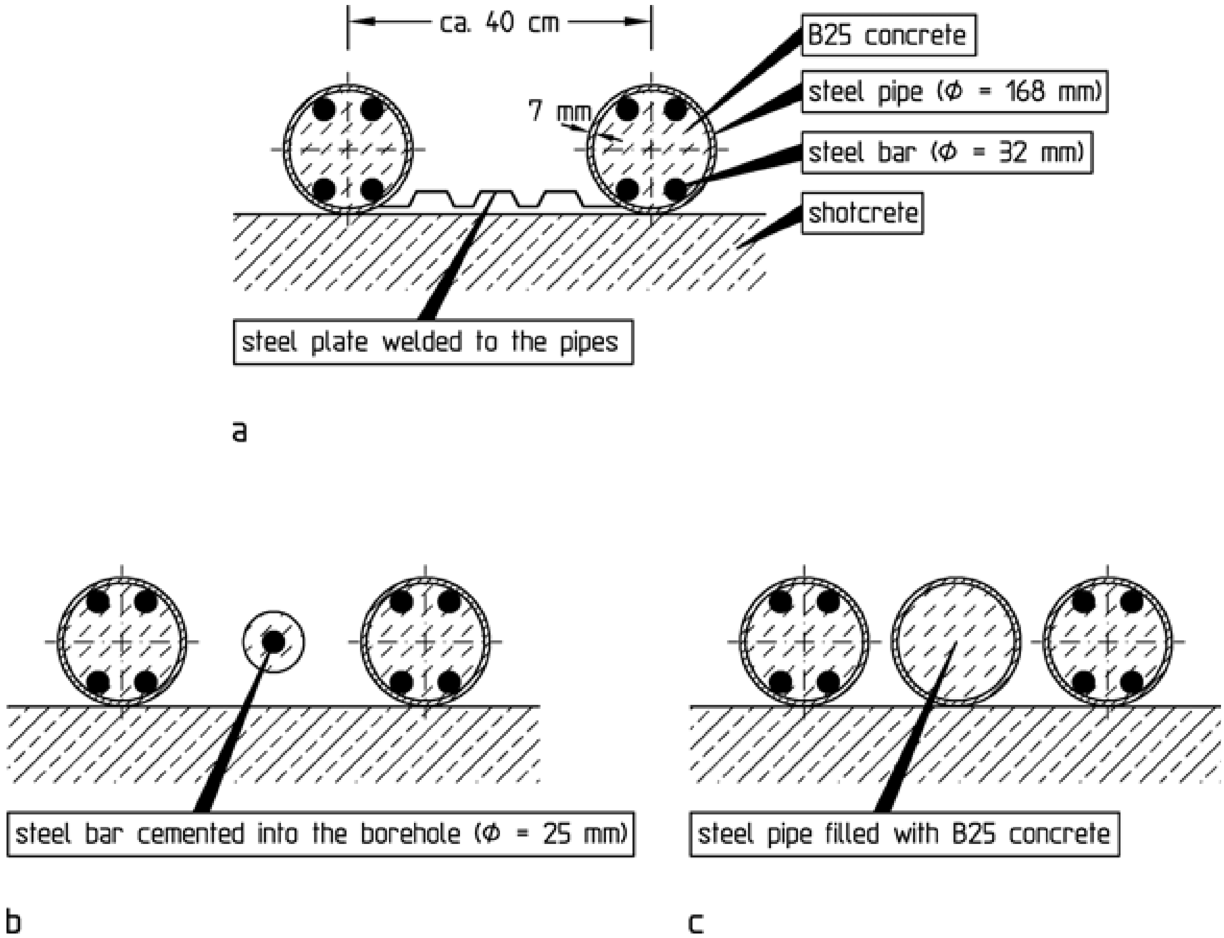


Fig. 4.42: Support of the gaps between the pipes in case of locally occurring dry, loose sand or fill: a) Pipe umbrellas 1 and 2; b) pipe umbrellas 3 to 14; c) pipe umbrellas 15 and 16

Due to reasons of construction, a vertical tunnel face was carried out. For stability reasons the crown generally had to be excavated in several steps and supported immediately with reinforced shotcrete (t = 5 to 10 cm) (Fig. 4.41).

Fig. 4.43 depicts a geotechnical mapping of the crown face at chainage 17. In Fig. 4.44, the supported crown is shown at chainage 36. It can be seen here that in the middle of the crown a support core was carried out (see Fig. 4.41). Further pictures from crown heading construction are shown in Fig. 4.45 to 4.47.

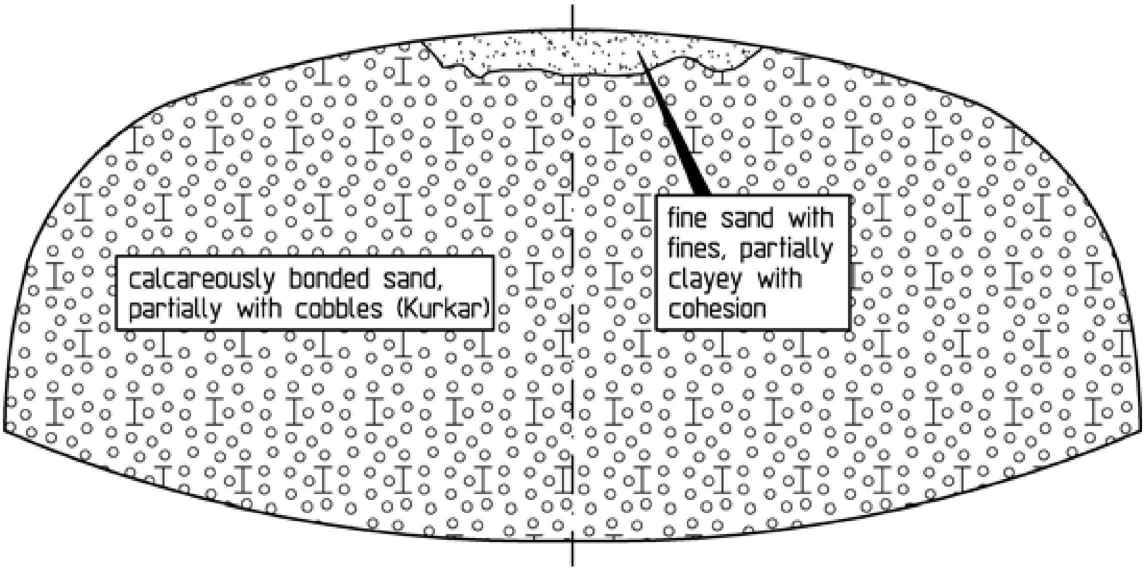


Fig. 4.43: Geotechnical mapping of the crown face, chainage 17

Fig. 4.44: View of the supported crown face, chainage 36

Fig. 4.45: Start at the northern portal

Fig. 4.46: Construction of a pipe umbrella

Fig. 4.47: Crown excavation in parts

The excavation of bench and invert was only started after the crown of the entire tunnel had been excavated (see Fig. 4.41).

4.2.7 Monitoring

To measure the subsidence due to the tunneling, 13 leveling points were installed at the ground surface above the tunnel roof, and eight measuring cross-sections with roof bolts were installed in the tunnel. The measurements served to monitor the stability of the tunnel.

Fig. 4.48 shows exemplarily the subsidence of the ground surface and the tunnel roof measured during the crown heading with the temporary face located at chainage 39. The subsidence of the ground surface amounts to approx. 20 to 50 mm, the measurable subsidence of the roof accounts for approx. 15 to 20 mm.

Fig. 4.48 further shows the ground surface subsidence measured after completion of the crown heading. It amounts to approx. 20 mm in the area of the northern portal and to approx. 40 to 60 mm in the remaining tunnel section.

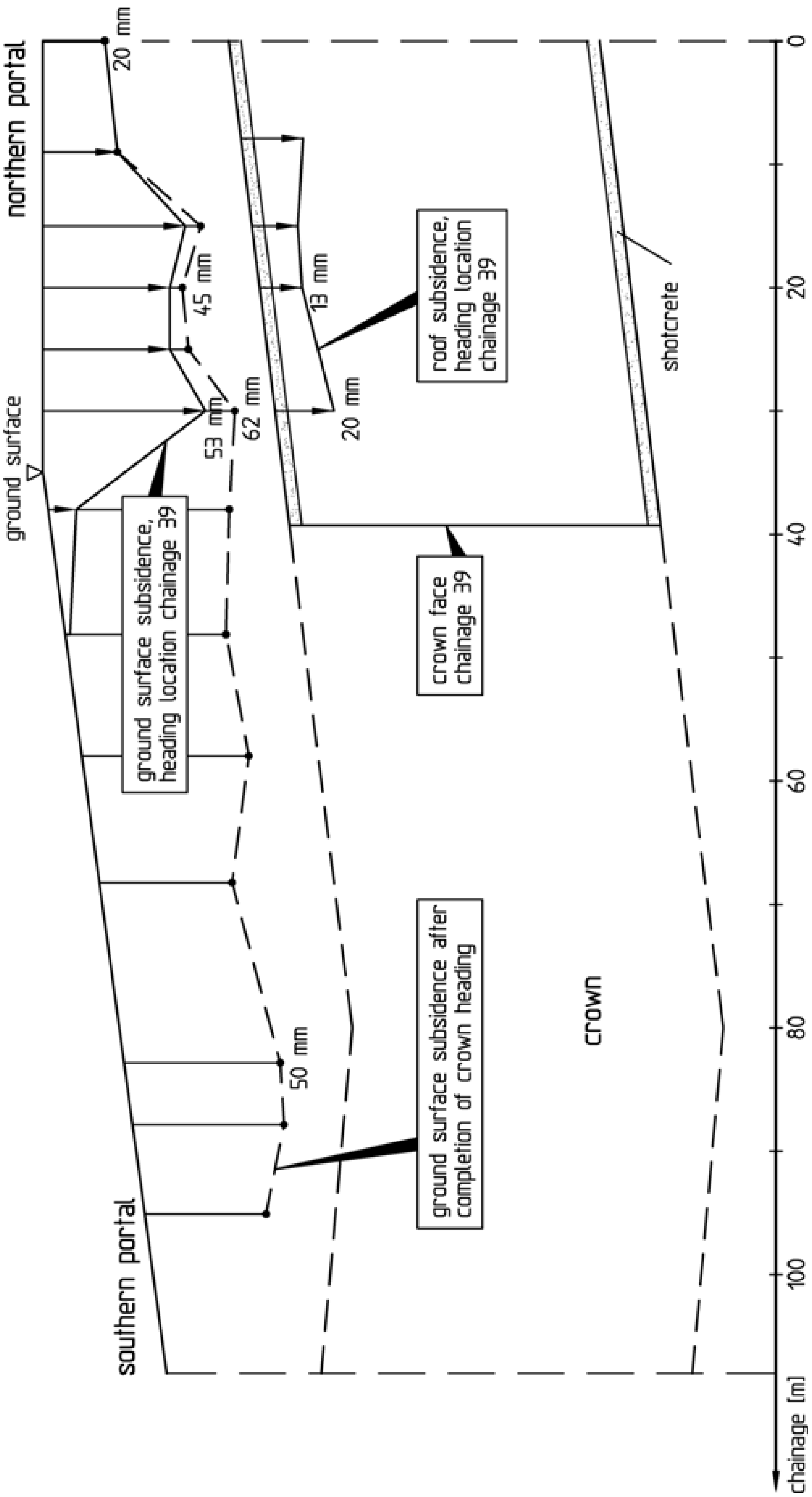


Fig. 4.48: Surface and roof subsidence measured during crown heading

For the comparison of the measured subsidence with the results of the stability analyses it must be noted that the subsidence preceding the excavation, which has already occurred before the shotcrete support is installed, was not captured in the analyses (see Chapter 4.2.5).

The subsidence that has occurred at the ground surface before the shotcrete support is installed amounts to approx. 20 mm in the example of Fig. 4.48. By adding this subsidence to the calculated subsidence of approx. 22 mm (see Fig. 4.29) and to the estimated deflection of the pipe umbrella of approx. 3 mm (see Fig. 4.40), a total subsidence of approx. 45 mm can be derived from the analysis results which agrees well with the measured subsidence (see Fig. 4.48).

4.2.8 Conclusions

The Elite Tunnel in Ramat Gan crosses through a medium dense to dense, calcareously bonded, slightly cohesive sand termed "Kurkar". Loose, cohesionless sands are locally embedded. The overburden amounts to 3 to 4 m. Since the tunnel undercrosses an eight-lane road, the subsidence had to be kept small.

The tunnel was excavated by the NATM under the protection of a pipe umbrella as a crown heading with a closed support at the temporary invert and trailing bench and invert excavation. The crown was excavated in several parts with a vertical tunnel face, a support core and short round lengths and supported by reinforced shotcrete and tunnel face anchors. The shotcrete membrane was installed and closed at the invert at a distance of approx. 1 m to the tunnel face (see Fig. 4.41).

Following this procedure the tunnel was excavated in a stable way. The ground surface subsidence amounted to approx. 4 to 6 cm.

The results of the FE-analyses contributed essentially towards the design, the statics and the specification of the excavation and support measures.

4.3 City railway tunnel to Botnang, in Stuttgart, Germany

4.3.1 Introduction

In the early 1990's the Stuttgart city railway line U9 was improved up to Botnang terminal. As a part of this project the "Herder Street" and "Lindpaintner Street" stops were connected by a 550 m long, double-tracked tunnel (Fig. 4.49a). The tunnel undercrosses the Botnang saddle in a wide turn as well as the Gäubahn and some built-up areas (Fig. 4.49).

4.3.2 Structure

Between the Botnang portal and chainage km 4+392 the two-tracked standard profile was constructed over a length of 379 m (Fig. 4.49b) with a height of approx. 8.5 m and a width of approx. 10.5 m (Fig. 4.51). From chainage km 4+392 to the Herder Street portal at chainage km 4+239.5 the height of the cross-section increases from approx. 8.5 to approx. 14 m (enlarged profile, Fig. 4.49b). The maximum cross-section at the Herder Street portal has a height of approx. 14 m and a width of approx. 12 m (Fig. 4.50). The maximum overburden amounts to some 47 m (Fig. 4.49b).

The maximum cross-section at chainage km 4 + 239.5 is shown in Fig. 4.50. The shotcrete membrane is 30 to 35 cm thick, the thickness of the reinforced concrete interior lining amounts to 60 cm. In the area of the sidewalls and the invert, with R = 13.16 m and R = 14.5 m, respectively, comparatively large radii were selected for the rounding of both linings. In the roof area a smaller radius of R = 4.6 m was designed for statical reasons. At the transitions from the sidewalls to the invert smaller radii of R = 1.1 m were selected. The excavated cross-section amounts to approx. 142 m^2.

4.3.3 Ground and groundwater conditions

To explore the ground and groundwater conditions core drillings were sunk along the tunnel alignment and equipped as observation wells (Fig. 4.49b).

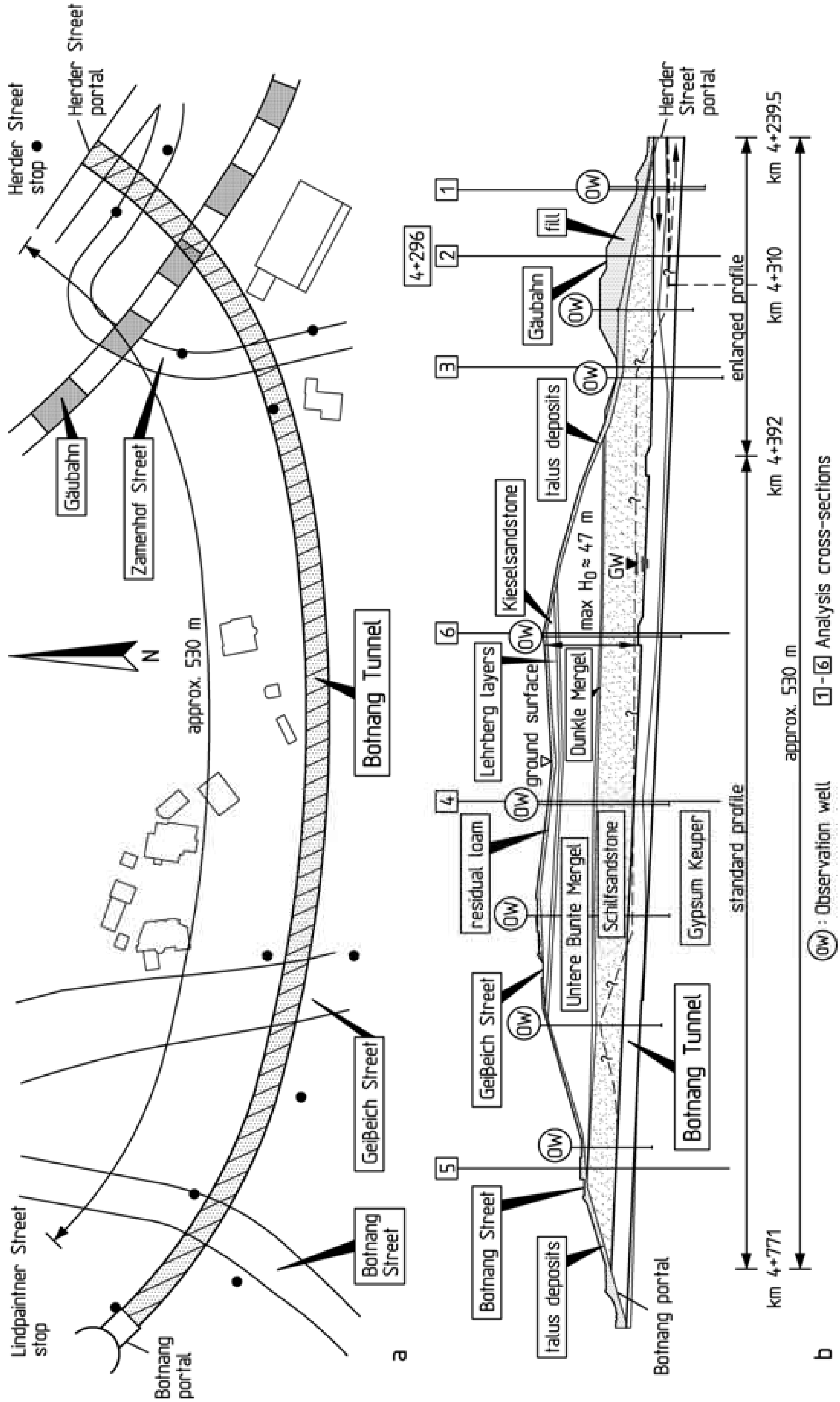


Fig. 4.49: City railway tunnel to Botnang: a) Site plan; b) longitudinal section with ground profile

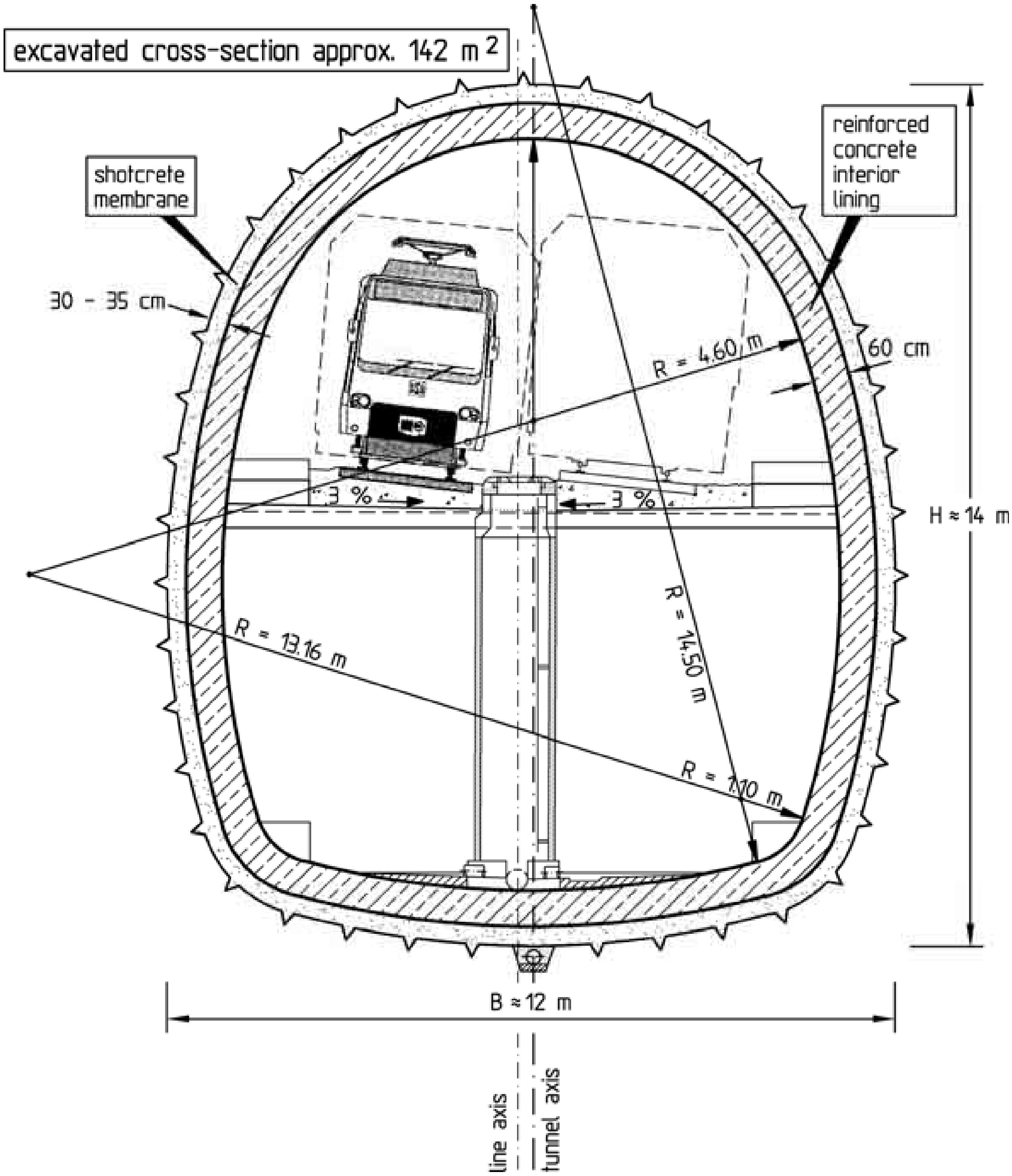


Fig. 4.50: City railway tunnel to Botnang, maximum cross-section (km 4+239.5)

The following formations exist in the area of the tunnel alignment from top to bottom (Fig. 4.49b):

- Fill,
- residual loam, talus deposits,
- Kieselsandstone layer and Lehrberg layers (claystones and marlstones),

- Untere Bunte Mergel (marlstones),
- Dunkle Mergel and Schilfsandstone layer (marlstone, sandstones and claystones),
- Gypsum Keuper.

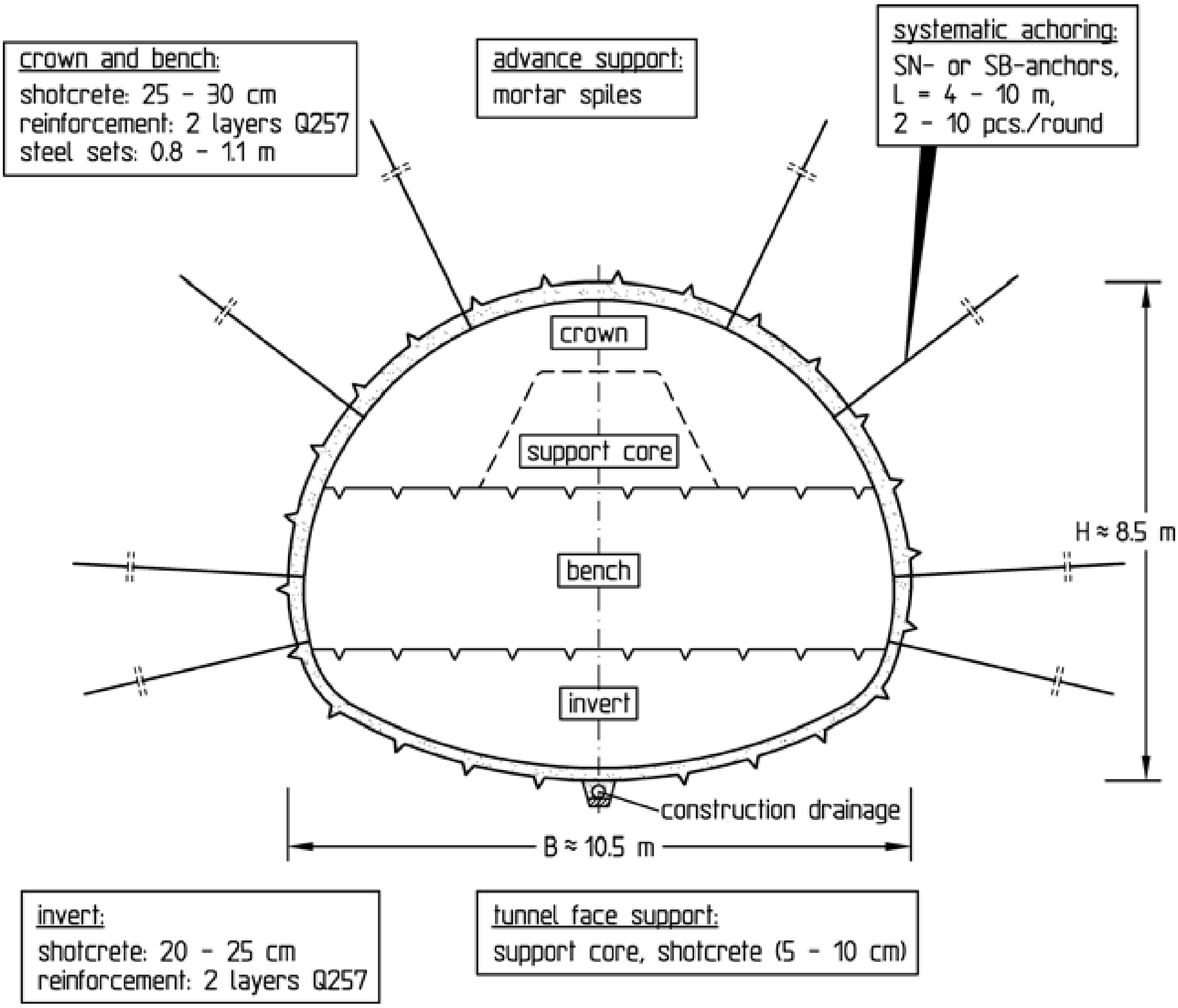


Fig. 4.51: Excavation and support, standard profile, excavation class 7A, cross-section

Fill of larger thickness occurs at the ground surface and at the portal areas. It mainly consists of firm to stiff sandy silt with embedded solid and hard rock fragments and to a minor degree also building rubble. Fill with a thickness of up to 17 m was found at the settlement-sensitive Gäubahn embankment, which the tunnel undercrosses over a tunnel length of 100 m (Fig. 4.49b).

The flanks of the Botnang saddle are covered with talus material, the thickness of which usually amounts to 1 to 2 m. Up to 4 m of thickness are reached in shallowly dipping terrain where the talus material changes into residual loam. The latter consists of soft to stiff silts containing heavily varying amounts of solid and hard rock fragments. In the area of the Botnang Tunnel portal an inclinometer was installed already before the start of construction. The measurements gave no indication of slope movements.

Layers of partially plastic leaching silts appear in the marls, which are belonging to the Keuper formation.

The Schilfsandstone layer is composed of a series of gray-green, gray and brown, mostly solid and hard marlstone and sandstone banks that are weathered close to the surface. The upper part of the profile is dominated by marlstones with a varying however high fine sand content. Gray-black, sand-free claystones are intercalated in some areas as well. The thickness of this layer amounts to approx. 25 m. The lower part of the Schilfsandstone layer, located within the tunnel's cross section, consists mainly of hard sandstone banks with clay flasers. The fine- to medium-grained sandstones are clay-bonded or cemented by immediate siliceous grain bonding. Quantitative mineralogical investigations yielded quartz contents ranging from 30 to 40 %.

The thickness of the layers varies between 10 and 100 cm. The bedding parallel discontinuities are occasionally marked by soft or firm clay layers, on which the banks tend to separation.

Sandstones of the Schilfsandstone layer have a medium to wide joint spacing, while joints in clay- and marlstones are narrow- to medium-spaced. Two vertical joint sets exist, which intersect at an angle of 60 to 70°. Their acute angle includes the N-S-direction. Thin sandstone banks are fractured into plates, while thick banks are fractured into columns rather. The joints are mostly rough, slightly undulating and closed. Open joints and clayey coatings occur mostly in the hillside area (slope dilatation) as well as close to the surface.

The Gypsum Keuper consists of an alternating sequence of soft to firm silts, stiff claystone layers and hard claystone and dolomite layers. The Gypsum Keuper layers encountered are almost completely leached. In several boreholes single, strongly leached gypsum nodules as well as thin gypsum layers were found.

The soil and rock mechanical parameters listed in Table 4.2 are based on the results of borehole expansion tests using the borehole jack model "Stuttgart", on laboratory tests on samples taken from the exploration boreholes as well as on experience gained from other projects located in comparable ground conditions. The stability analyses (see Chapter 4.3.5) were based on these parameters.

Layer	Deformability	Strength
Fill in the Gäubahn area	$E = 10\ MN/m^2$ $\nu = 0.35$	$\varphi' = 27.5°$ $c' = 5\ kN/m^2$
Talus material/ residual loam	$E = 5\ MN/m^2$ $\nu = 0.4$	$\varphi' = 27.5°$ $c' = 5\ kN/m^2$
Kieselsandstone	$E = 2000\ MN/m^2$ $\nu = 0.2$	Joints J: $\varphi_J = 40°$, $c_J = 20\ kN/m^2$
Lehrberg layers, Untere Bunte Mergel, Dunkel Mergel	$E = 300\ MN/m^2$ $\nu = 0.3$	$\varphi = 30°$ $c' = 130\ kN/m^2$
Schilfsandstone, portal areas	$E = 150\ MN/m^2$ $\nu = 0.3$	$\varphi = 35°$ $c' = 30\ kN/m^2$
Schilfsandstone	$E = 500\text{-}1000\ MN/m^2$ $\nu = 0.2$	Bedding B: $\varphi_S = 40°$, $c_S = 50\ kN/m^2$ Joints J: $\varphi_K = 40°$, $c_K = 50\ kN/m^2$
Gypsum Keuper, weathered	$E = 20\ MN/m^2$ $\nu = 0.35$	$\varphi = 27.5°$ $c' = 25\ kN/m^2$
Gypsum Keuper, unweathered	$E = 300\text{-}500\ MN/m^2$ $\nu = 0.3$	$\varphi = 27.5°$ $c' = 25\ kN/m^2$

Table 4.2: Soil and rock mechanical parameters

According to the results of the piezometer measurements, the undisturbed groundwater table is located above or at the tunnel roof almost over the entire tunnel length. In the area of the Herder Street portal it falls to about the middle of the tunnel's cross-section (Fig. 4.49b).

4.3.4 Design

From the Herder Street portal to chainage km 4 + 310 the double-tracked standard profile was planned to be excavated first as a temporary stage. After that the final profile was to be excavated from chainage km 4 + 310 to the Botnang portal. The final profile is equal to the enlarged profile from chainage km 4 + 310 to chainage km 4 + 392 and to the double-tracked standard profile starting at chainage km 4 + 392. After the cut-through the cross-section was planned to be enlarged in the invert to the final profile from chainage km 4 + 310 backwards to Herder Street (Fig. 4.49b).

Fig. 4.51 shows the standard profile divided into crown, bench and invert and the excavation and support measures planned for the standard excavation procedure. The excavation class was designated as 7A according to the recommendations of the working group "Tunneling" of the German Geotechnical Society (DGGT, 1995: Table 1).

Crown and bench were each excavated with an immediate installation of the support. The crown face may only be ahead of the bench face by a maximum of approx. 3 m (Fig. 4.52, phase I in the left portion). Shotcrete with a thickness of 25 to 30 cm and reinforced by two layers of steel fabric mats Q257, support arches spaced at 0.8 to 1.1 m and a systematic anchoring using SN-anchors were planned (Fig. 4.51). Mortar spiles should be used as advancing support.

The support should be closed at the invert no more than 2.4 m behind the invert excavation and no more than some 16 m behind the excavation at the roof (Fig. 4.2, phase I in the left portion). The reinforced shotcrete should be placed at the invert with a thickness of 20 to 25 cm (Fig. 4.51).

The tunnel face was planned to be excavated steeply and in steps, with a support core in the crown area. Immediately after excavation the tunnel face was to be supported using 5 to 10 cm thick shotcrete, possibly reinforced by a steel fabric mat Q257 (Fig.

4.52, left portion). During the excavation the shotcrete support should be opened in sections, and the exposed partial areas should immediately be sealed again by shotcrete after the excavation.

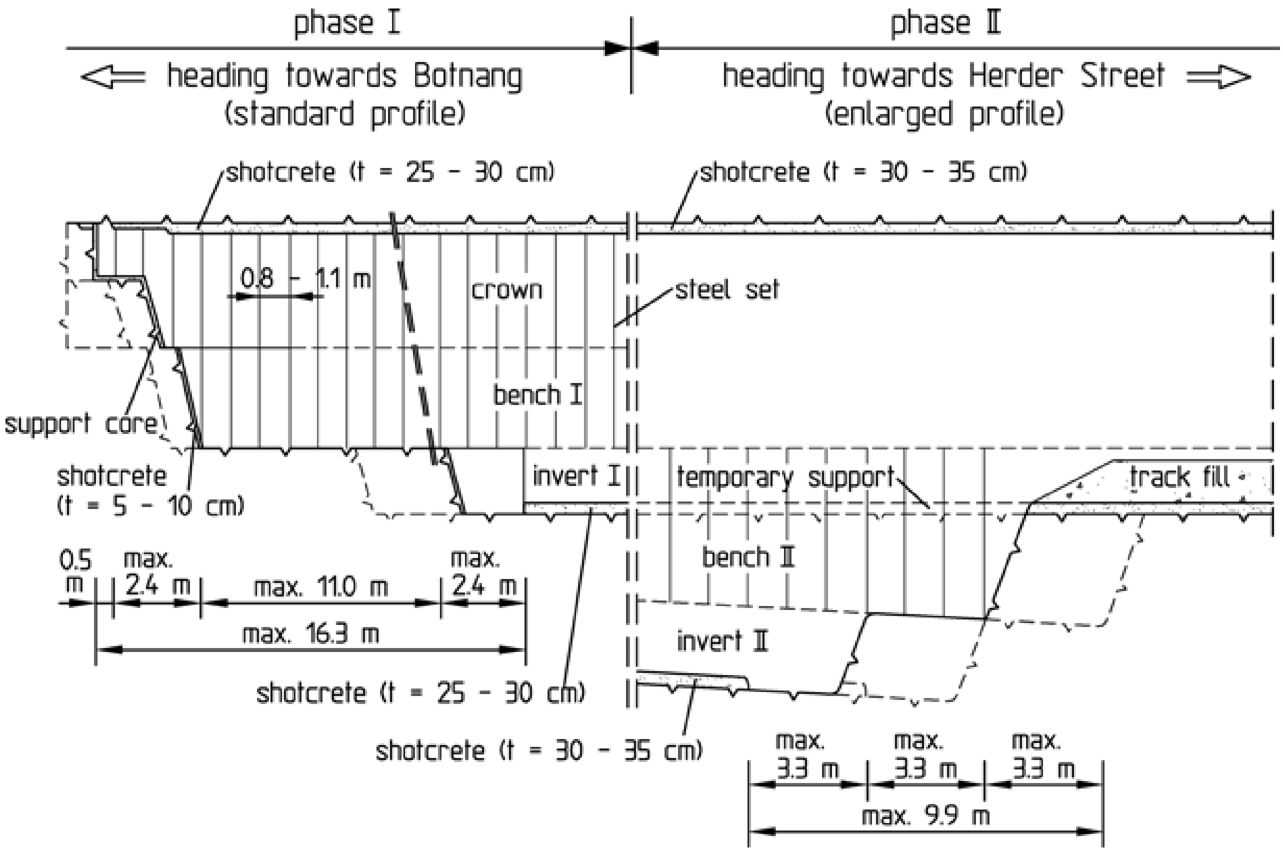


Fig. 4.52: Excavation and support, longitudinal section

Fig. 4.52 (phase II, right portion) and 4.53 show the excavation and support measures planned for the later lowering of the invert in the area with enlarged cross-section.

In the upper part of the cross-section (crown, bench I and invert I), excavation and support was to be carried out as for the standard heading, but with a thicker shotcrete membrane (t = 30 to 35 cm) and systematic anchoring (see Fig. 4.51 and 4.53). A reinforced shotcrete membrane with a thickness of 30 to 35 cm should also be installed in the lower part of the cross-section (bench II and invert II). The support should be closed at the invert no more than 3.3 m behind the excavation of invert II and no more than 9.9 m behind the excavation of bench II (Fig. 4.52, right portion).

The support of the tunnel face and the advancing support should be constructed as for the standard heading.

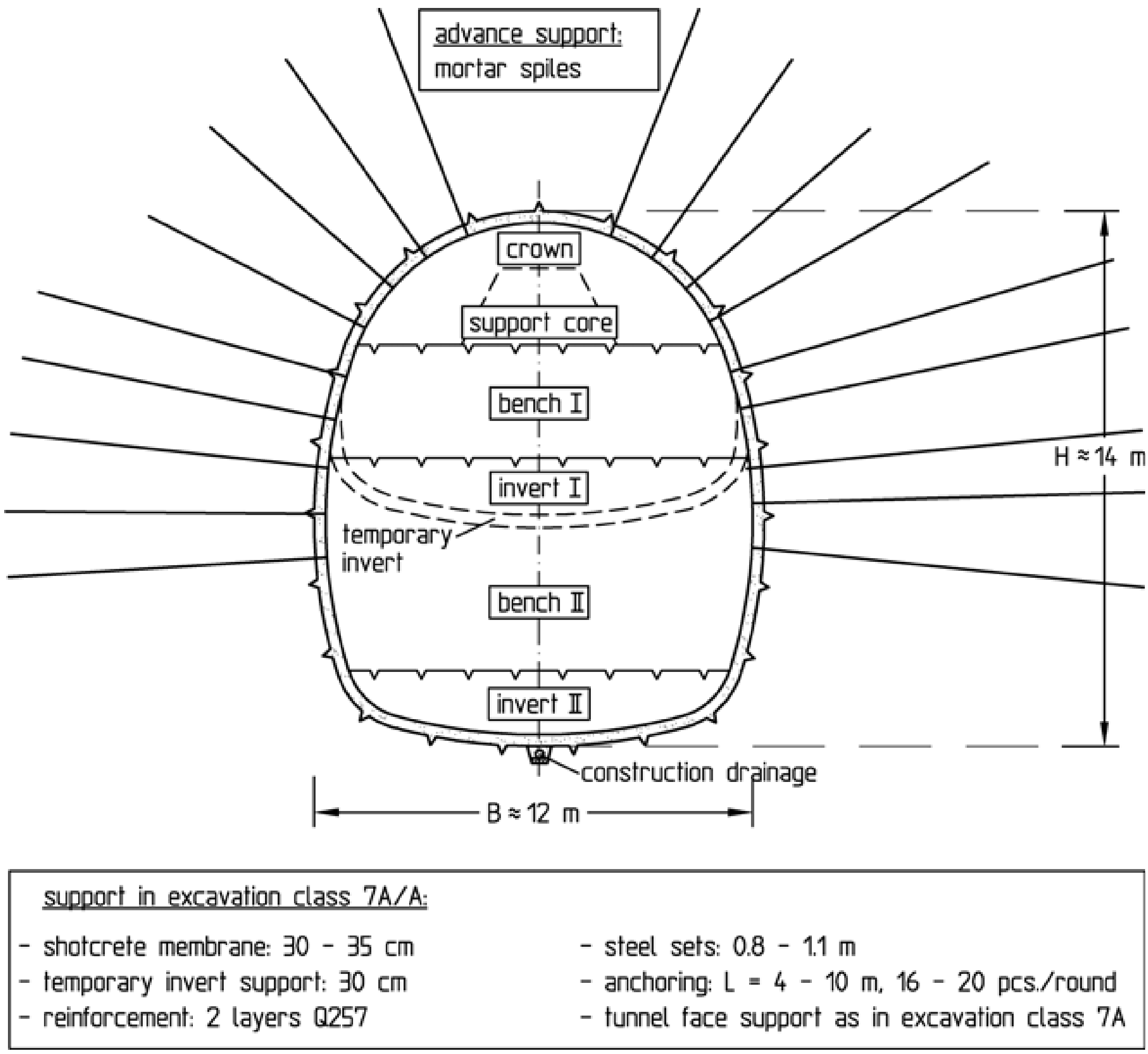


Fig. 4.53: Excavation and support, maximum cross-section, lowering of the invert, cross-section

4.3.5 Stability analyses for the design of the shotcrete support

For the design of the shotcrete support, two-dimensional FE-analyses on vertical slices were carried out with the program system FEST03 (Wittke, 2000). Fig. 4.49b shows the locations of the analysis cross-sections investigated in the final design analyses. The analyses were based on the parameters given in Table 4.2.

In the following, analysis cross-section 2 will be exemplarily treated. It is located at chainage km 4+296 in the area of the undercrossing of the Gäubahn (Fig. 4.49b). In Fig. 4.54 the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters are shown. The computation section consists of a 63.9 m high, 45.7 m wide and 1 m thick slice of rock mass. The FE-mesh was divided into 1104 isoparametric elements with a total of 1320 nodes.

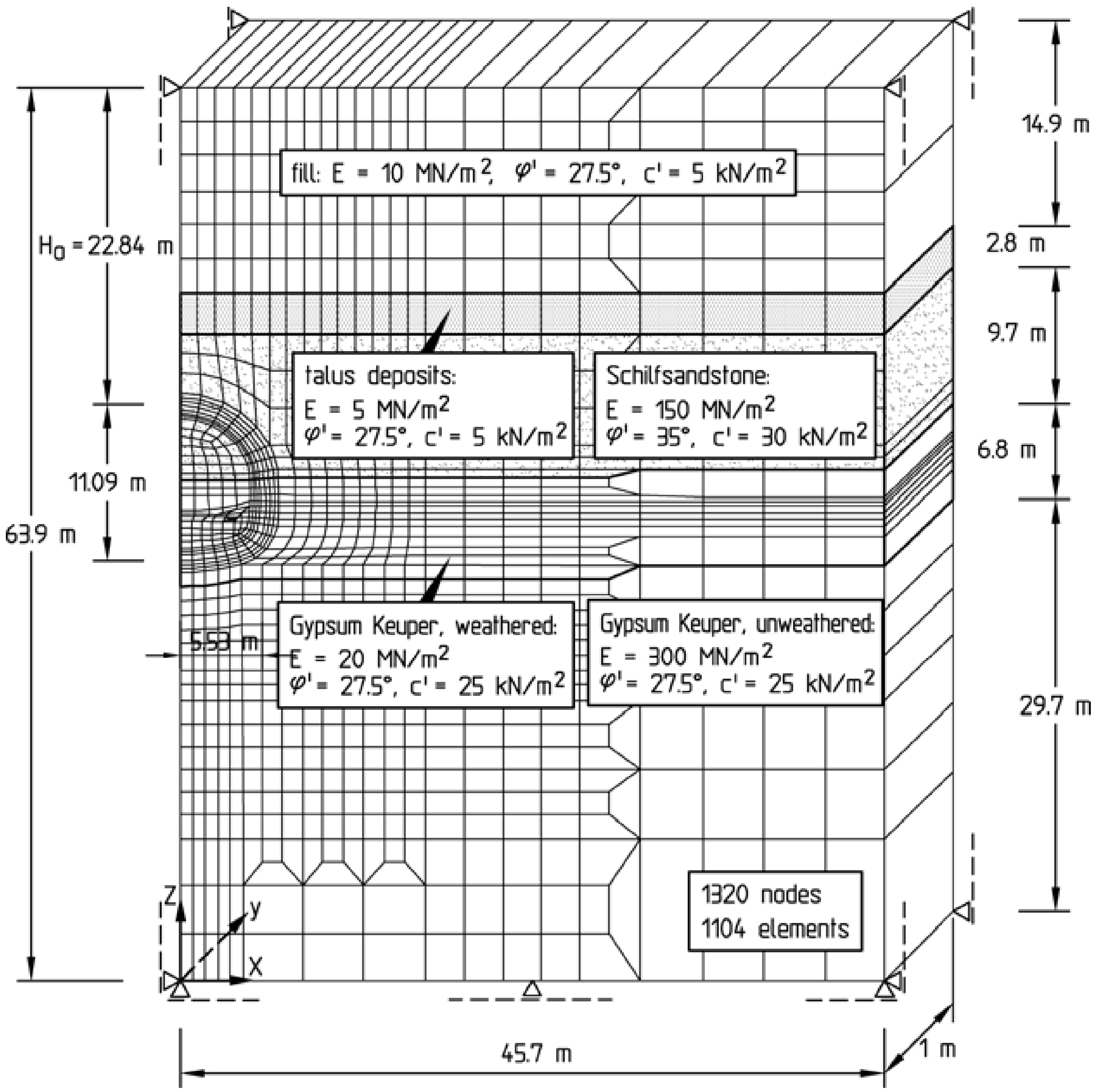


Fig. 4.54: Computation section, FE-mesh, boundary conditions, ground profile and parameters, analysis cross-section 2 (km 4+296)

For the nodes on the lower boundary plane (z = 0) and on the lateral boundary planes (x = 0 and x = 45.7 m), sliding supports were introduced as boundary conditions (Fig. 4.54). For the two planes perpendicular to the tunnel axis (y = 0, y = 1 m), equal displacements were assumed as boundary conditions for nodes with equal x-and z-coordinates (Wittke, 2000). All nodes were assumed fixed in y-direction.

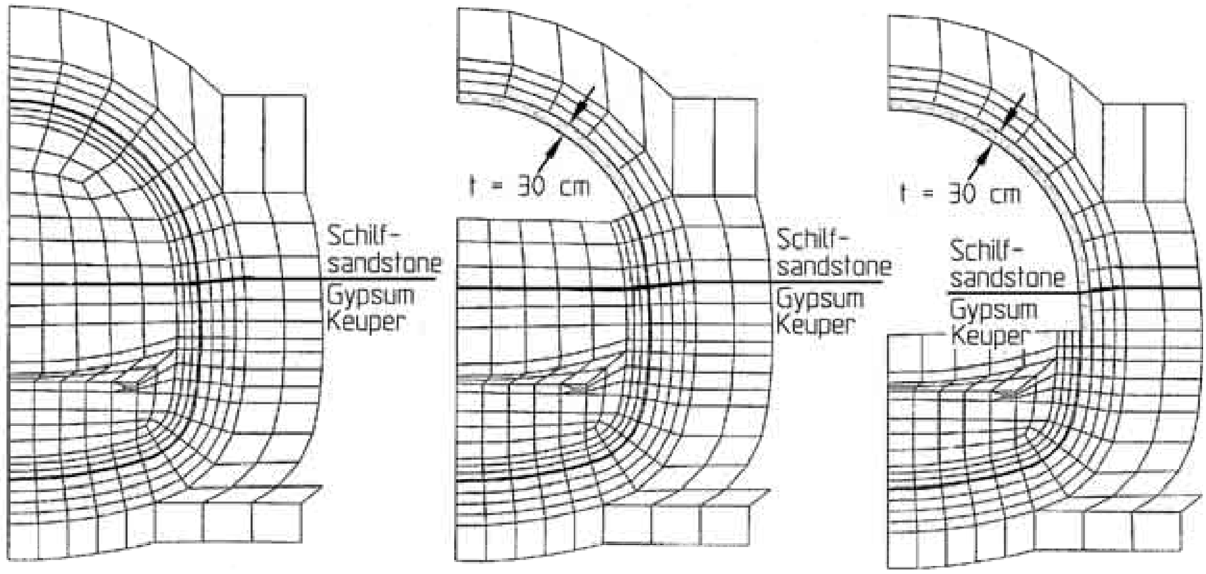
In the area of analysis cross-section 2, the tunnel has approx. 23 m of overburden. The upper part of the cross-section is located in the Schilfsandstone, the lower in the weathered Gypsum Keuper. Above the Schilfsandstone, hillside loam and the fill of the embankment constructed for the Gäubahn are modeled, with thicknesses of approx. 3 m and 15 m, respectively. The weathered Gypsum Keuper is underlain several meters below the tunnel's invert by the unweathered Gypsum Keuper (Fig. 4.54).

The vertical section through the tunnel axis constitutes a plane of symmetry with respect to the geometry of the tunnel cross-section and to the ground profile. Therefore, only one half of the tunnel was modeled in the analysis (Fig. 4.54).

Fig. 4.55 and 4.56 show the six computation steps used to simulate the excavation and the support during tunneling according to the design (reference case). In the 1st computation step, the state of stress and deformation resulting from the dead weight of the ground (in-situ state) is determined. In the 2nd computation step the excavation of the crown and its support using shotcrete are simulated. The 3rd computation step comprises the excavation and shotcrete support of bench I. In the 4th computation step excavation of the invert I and the closing of the temporary support at the invert are simulated. Since the first two construction stages (computation steps 2 and 3) were simulated with an open invert, this analysis sequence accounts for a late closing of the invert as specified in the design with a distance of approx. 16 m to the tunnel face (see Fig. 4.52). The connection of the temporary invert support to the sidewall was simulated at first without any curvature corresponding to the design (Fig. 4.55 and 4.56).

In the 5th and 6th computation step, the excavation of bench II and the excavation of invert II with simultaneous installation of the shotcrete support also at the invert are simulated.

Fig. 4.57 presents the principal normal stresses determined for the 6th computation step (complete excavation), as well as those areas in which the strength has been exceeded. It can be seen that the plastic zones are limited to the area of the tunnel contour.

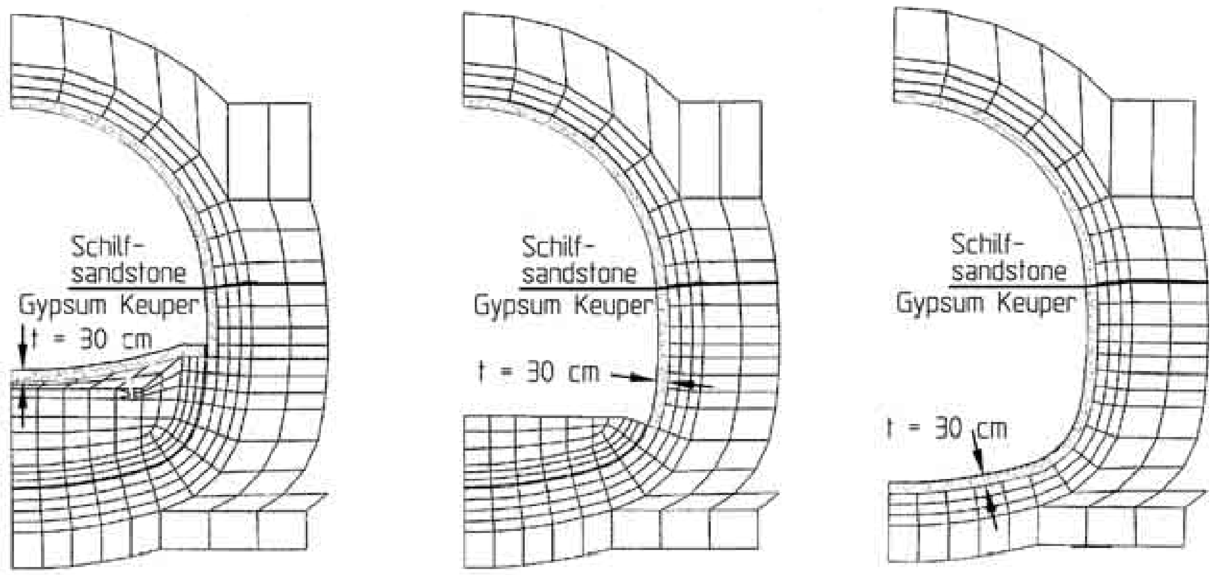


1^{st} computation step:
state of stress and deformation resulting from the dead weight of the ground

2^{nd} computation step:
crown excavation and shotcrete support

3^{rd} computation step:
excavation of bench I and shotcrete support

Fig. 4.55: Analysis cross-section 2, reference case, computation steps 1 to 3



4^{th} computation step:
excavation of invert I and shotcrete support

5^{th} computation step:
excavation of bench II and shotcrete support

6^{th} computation step:
excavation of invert II and shotcrete support

Fig. 4.56: Analysis cross-section 2, reference case, computation steps 4 to 6

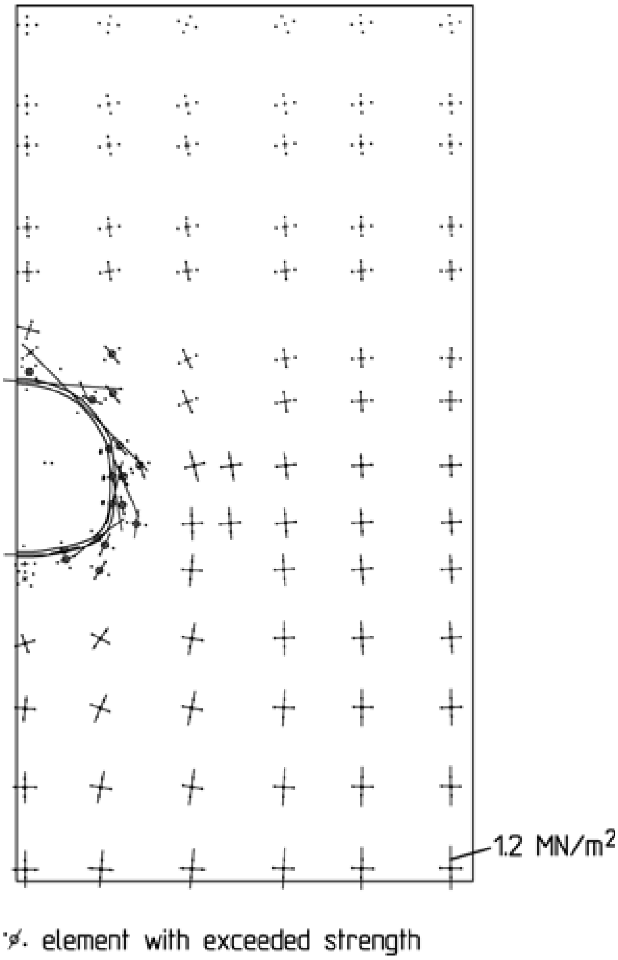


Fig. 4.57: Analysis cross-section 2, reference case, principal normal stresses and elements with exceeded strength, 6th computation step

Fig. 4.58 depicts the displacements computed for the 6th computation step and thus the displacements in the stage after the excavation of the total cross-section. The analysis results in comparatively large vertical displacements of 12 cm at the roof and 7.4 cm on the ground surface.

Fig. 4.59 shows the stress resultants computed for the stage after the excavation and support of invert I (4th computation step) and of the total cross-section (invert II, 6th computation step). In the 4th computation step (excavation of invert I) extremely high bending moments result in the area of the connections of the temporary invert support to the sidewalls (Fig. 4.59a). For this loading, the shotcrete membrane cannot be reasonably designed. In the 6th computation step as well, comparatively large bending moments occur in the area of the sidewalls (Fig. 4.59b). The design of the shotcrete membrane for a concrete grade of B25, a shotcrete thickness of t = 35 cm and a safety factor during construction of 1.35 yields that in the lower sidewall areas a higher amount of reinforcement of the shotcrete membrane compared to the original design is required (Fig. 4.60).

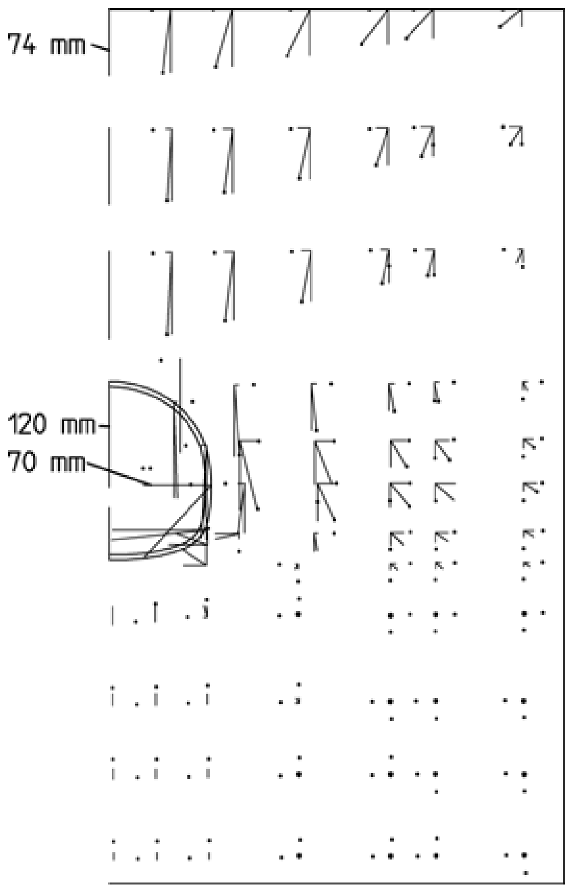
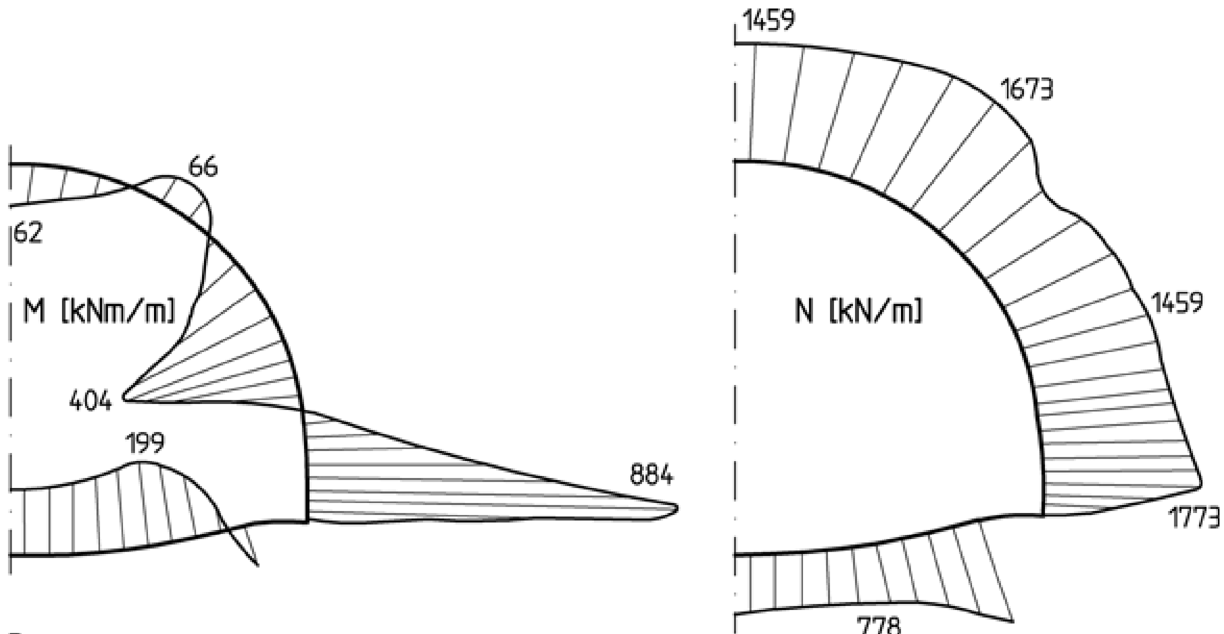
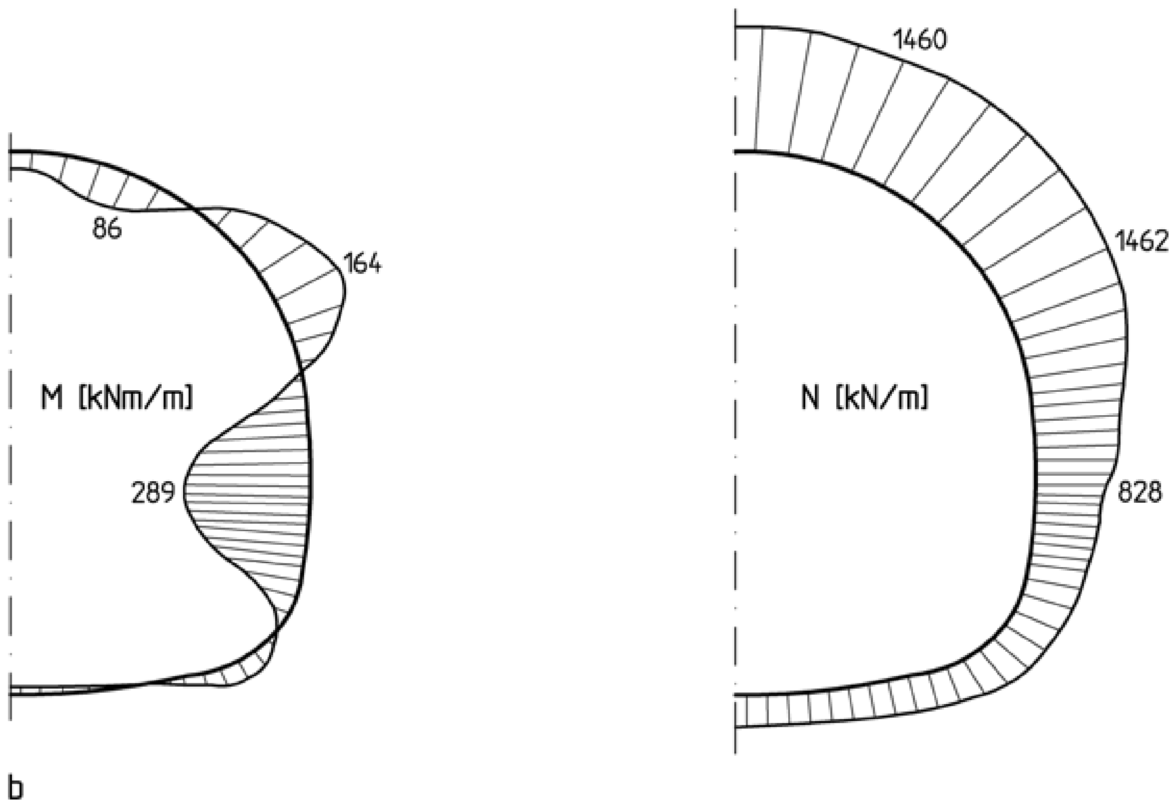


Fig. 4.58: Analysis cross-section 2, reference case, displacements, 6^{th} – 1^{st} computation step

Subsidence of the computed magnitude (Fig. 4.58) could also not be permitted for the undercrossing of the Gäubahn. On the basis of these analysis results and of the displacements measured during the heading (see Chapter 4.3.7), it was decided in agreement with all parties concerned to close the support at the invert earlier in order to keep the subsidence smaller. The computation steps for the analyses simulating an early support closing at the invert are shown in Fig. 4.61 and 4.62. The excavation of bench I and invert I as well as the installation of the temporary invert support were simulated in one computation step (3^{rd} computation step, Fig. 4.61). In order to reduce the loading of the shotcrete membrane in the area of the connection of the temporary invert with the side-walls, a rounded connection was modeled (Fig. 4.61).

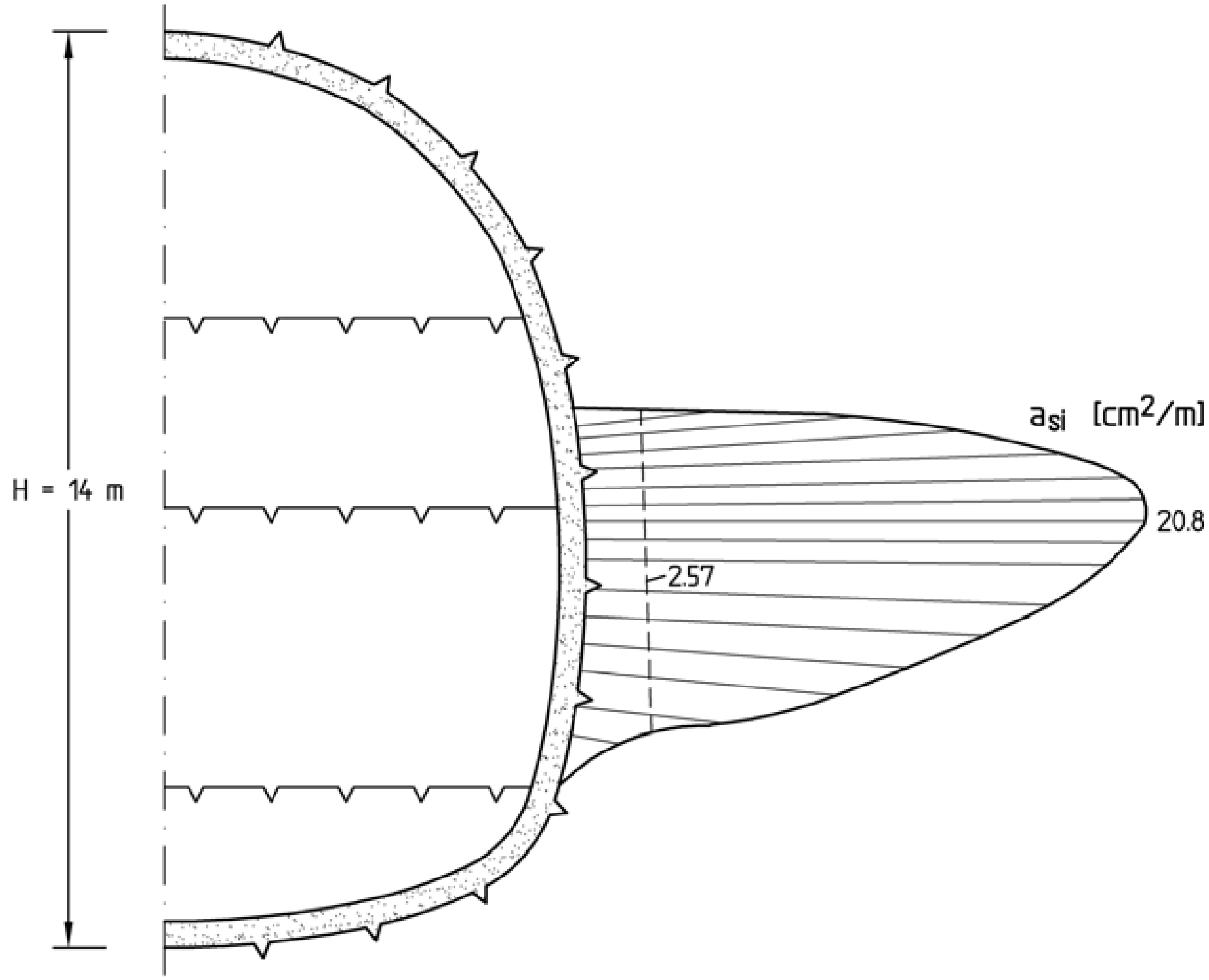


a



b

Fig. 4.59: Analysis cross-section 2, reference case, stress resultants in the shotcrete membrane: a) 4th computation step; b) 6th computation step



Basics of design: concrete grade B 25,
steel grade BSt 500/550, t = 35 cm, t_1 = 3 cm, η = 1.35

Fig. 4.60: Analysis cross-section 2, reference case, statically required inside reinforcement of the shotcrete membrane

To reduce the high bending loading of the shotcrete membrane in the sidewall areas after the excavation of the complete cross-section (H ≅ 14 m, see Fig. 4.59 and 4.60), an anchoring of the shotcrete membrane in the sidewall areas by untensioned anchors was accounted for in the 4^{th} computation step in addition to the excavation of bench II. The anchoring in the sidewall areas was modeled by truss elements. Five untensioned anchors per tunnel meter were assumed in the analysis with a cross sectional area of 4.5 cm^2 each. By the 5^{th} computation step the excavation and support of the invert was simulated (Fig. 4.62).

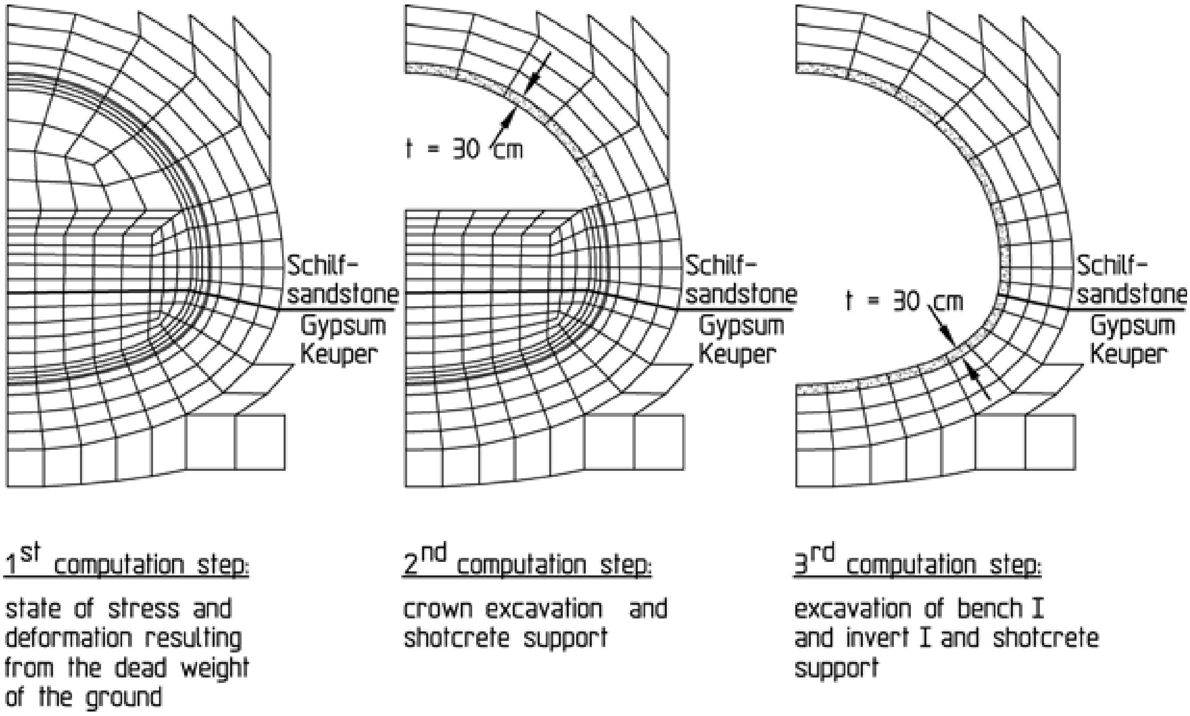


Fig. 4.61: Analysis cross-section 2, case 2, early closing of the invert and rounding of the temporary crown invert, computation steps 1 to 3

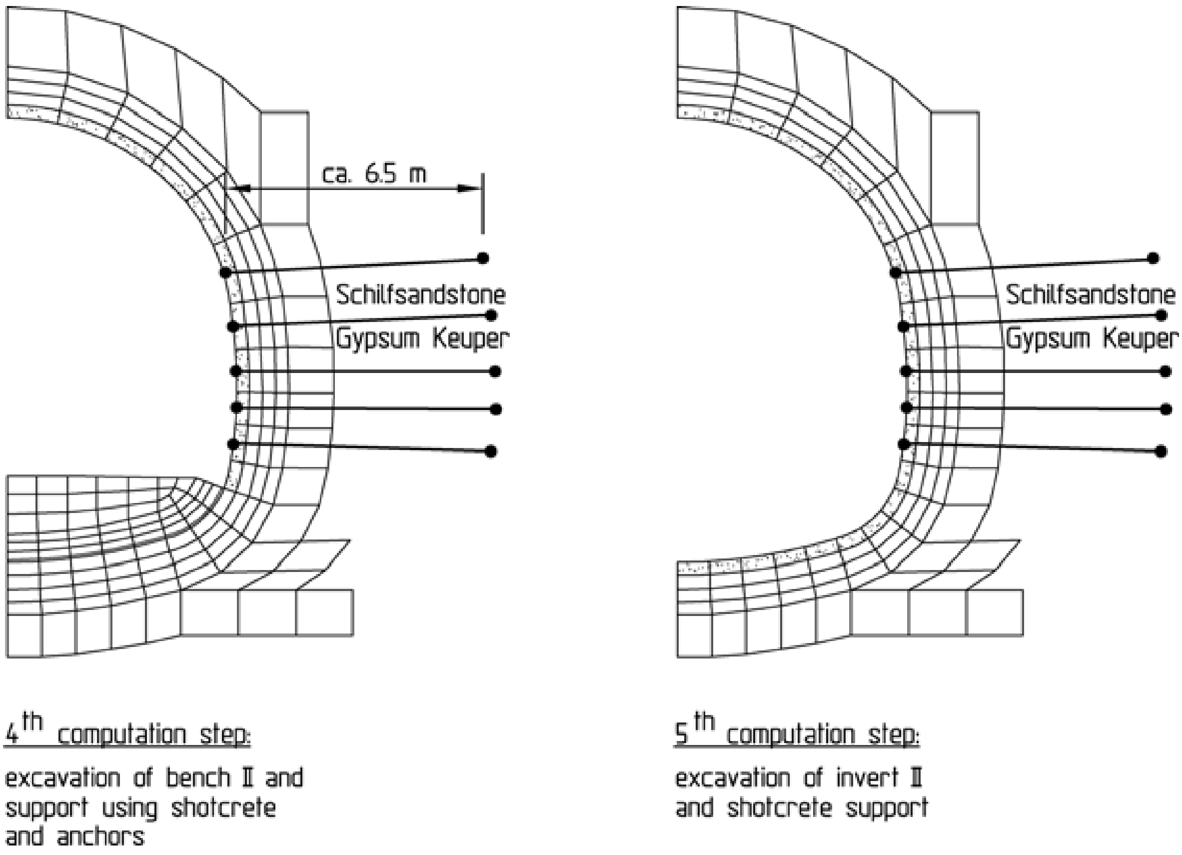


Fig. 4.62: Analysis cross-section 2, case 3, early closing of the invert and support of the sidewalls using anchors, computation steps 4 and 5

The results of this analysis show first, that the computed displacements decrease considerably due to the simulation of an early closing of the invert and of the anchoring (see Fig. 4.58 and 4.63). The subsidence of the ground surface amounts to only 27 mm as opposed to 74 mm in the reference case. The horizontal displacements of the sidewalls decrease from 70 mm in the reference case to 18 mm.

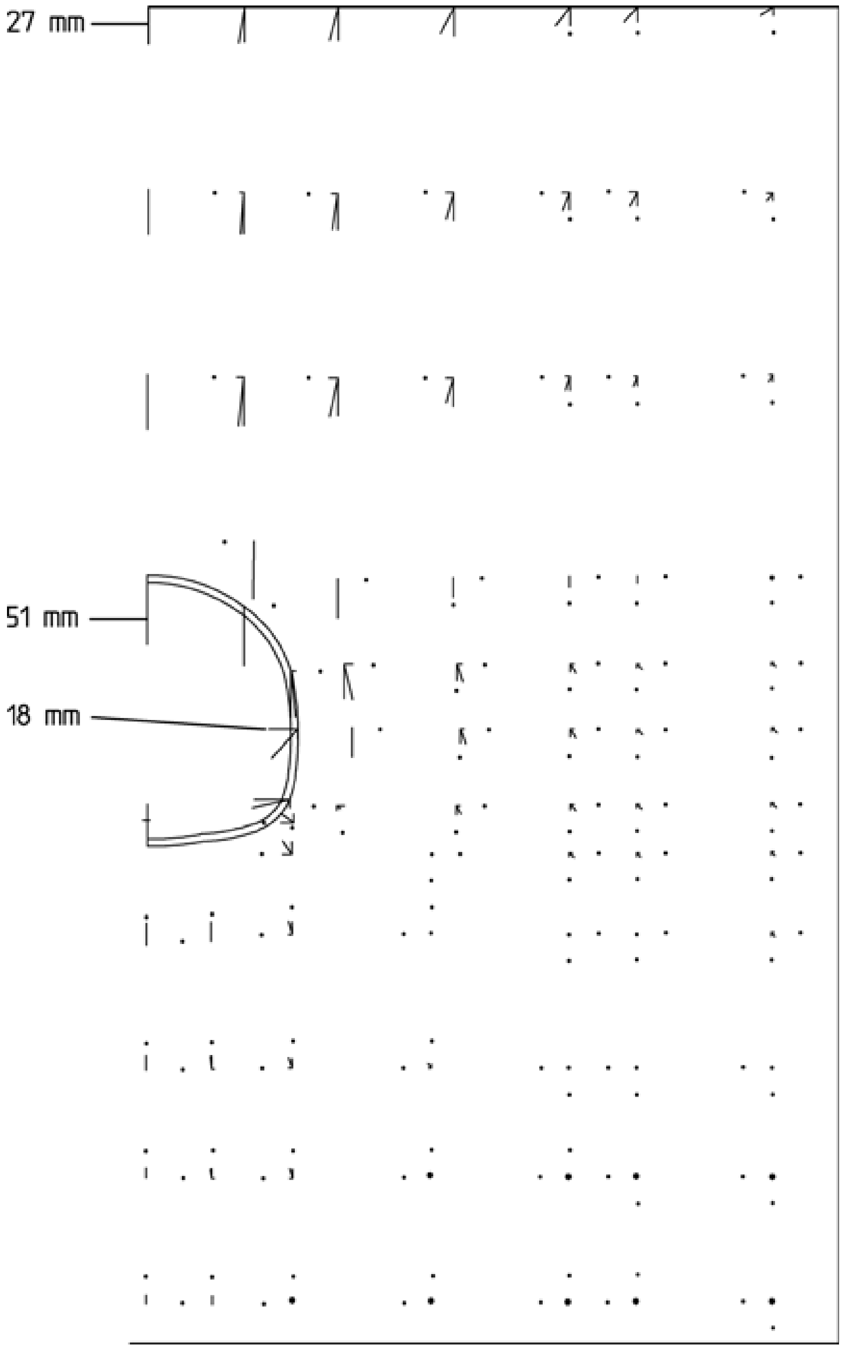


Fig. 4.63: Analysis cross-section 2, case 3, displacements, 5^{th} – 1^{st} computation step

The rounding of the cross-section in the area of the invert leads to a marked reduction of the bending moment (see Fig. 4.59a and 4.64). For a concrete grade of B25, a shotcrete support thickness of t = 35 cm and a factor of safety of 1.35 the analysis yields that supplementary reinforcement is required in the lower sidewall area in addition to the planned steel fabric mats Q257. Further, in the area of the greatest change in bending moment corresponding to the greatest shear force a shear reinforcement is necessary. It can be covered by diagonally bent-up reinforcement.

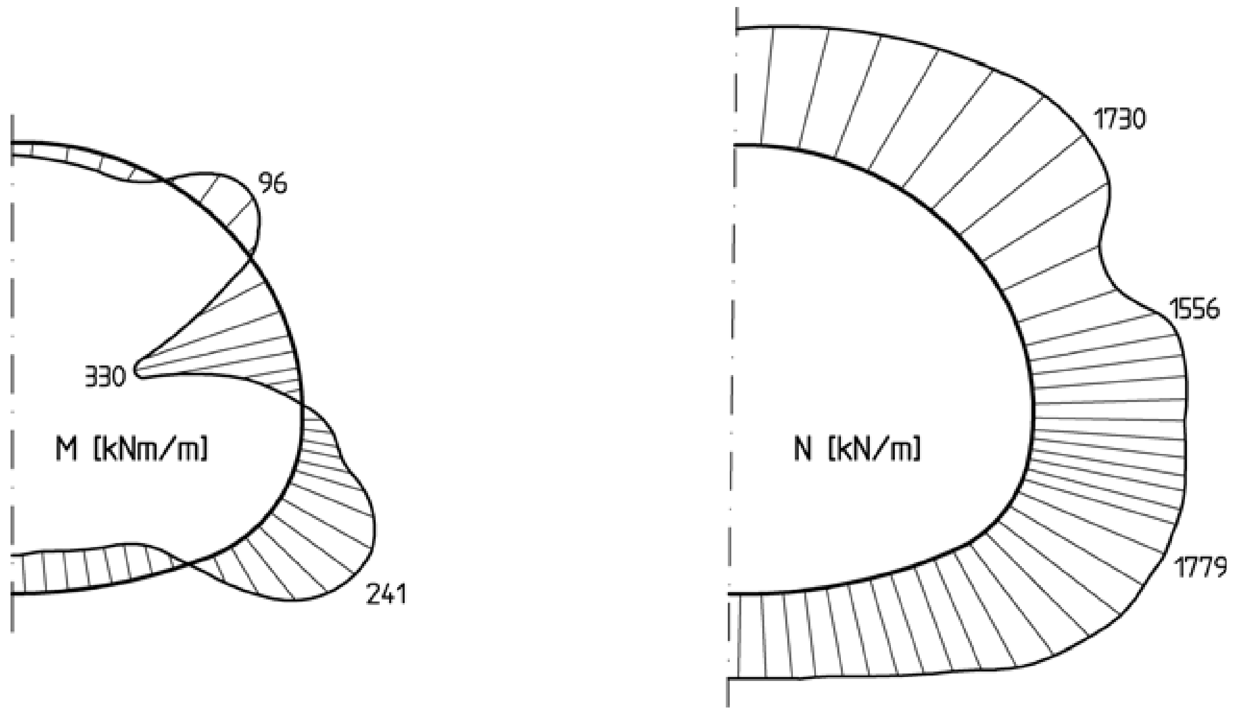


Fig. 4.64: Analysis cross-section 2, case 2, stress resultants in the shotcrete support, 3rd computation step

In Fig. 4.65a the tensile anchor forces determined in the 5th computation step (see Fig. 4.62) are given. Values of up to 100 kN (10 t) per anchor are computed.

The bending moments in the shotcrete membrane computed for the stage after the excavation of the complete cross-section (5th computation step) with the anchoring taken into account are given in Fig. 4.65b. Compared to the reference case (without anchoring) a strong reduction of the bending loading becomes evident (see Fig. 4.59b and 4.65b). The dimensioning yields that no supplementary reinforcement is required in the sidewalls if steel fabric mats Q257 are installed. At the transition from the sidewalls to the temporary invert, however, supplementary reinforcement in addition to the mats is required.

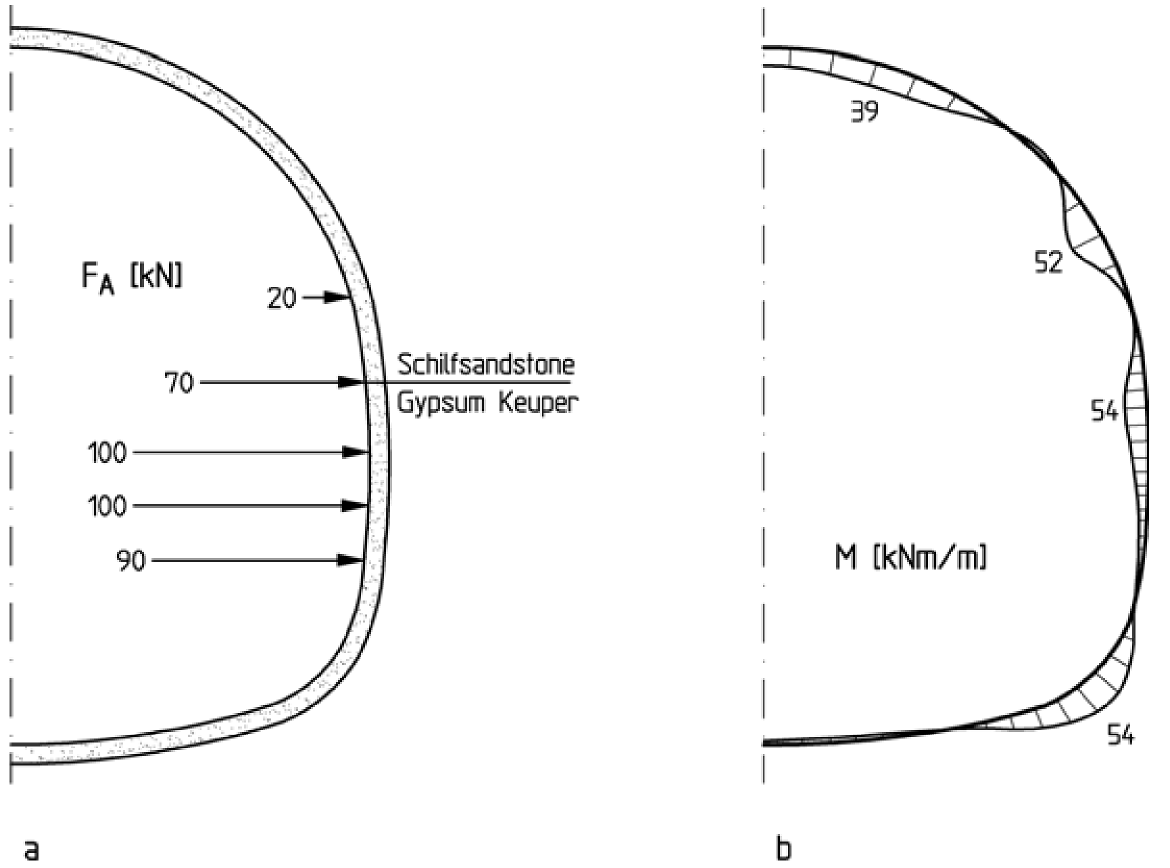


Fig. 4.65: Analysis cross-section 2, case 3, 5th computation step: a) Tensile anchor forces; b) bending moments

4.3.6 Construction

Fig. 4.66 shows the excavation and support measures carried out within the area of the enlarged cross-section during the heading (Beiche and Kagerer, 1993).

The shotcrete membrane was constructed with a thickness of 35 cm and rounded in the transition zones from the sidewalls to the temporary crown invert. The reinforcement included inside and outside reinforcement mats Q257 and supplementary reinforcement in the transition zones. Further, according to the design a systematic anchoring of the vault and the sidewalls was carried out (Fig. 4.66). In the central sidewall areas (invert I, bench II) the anchoring was intensified.

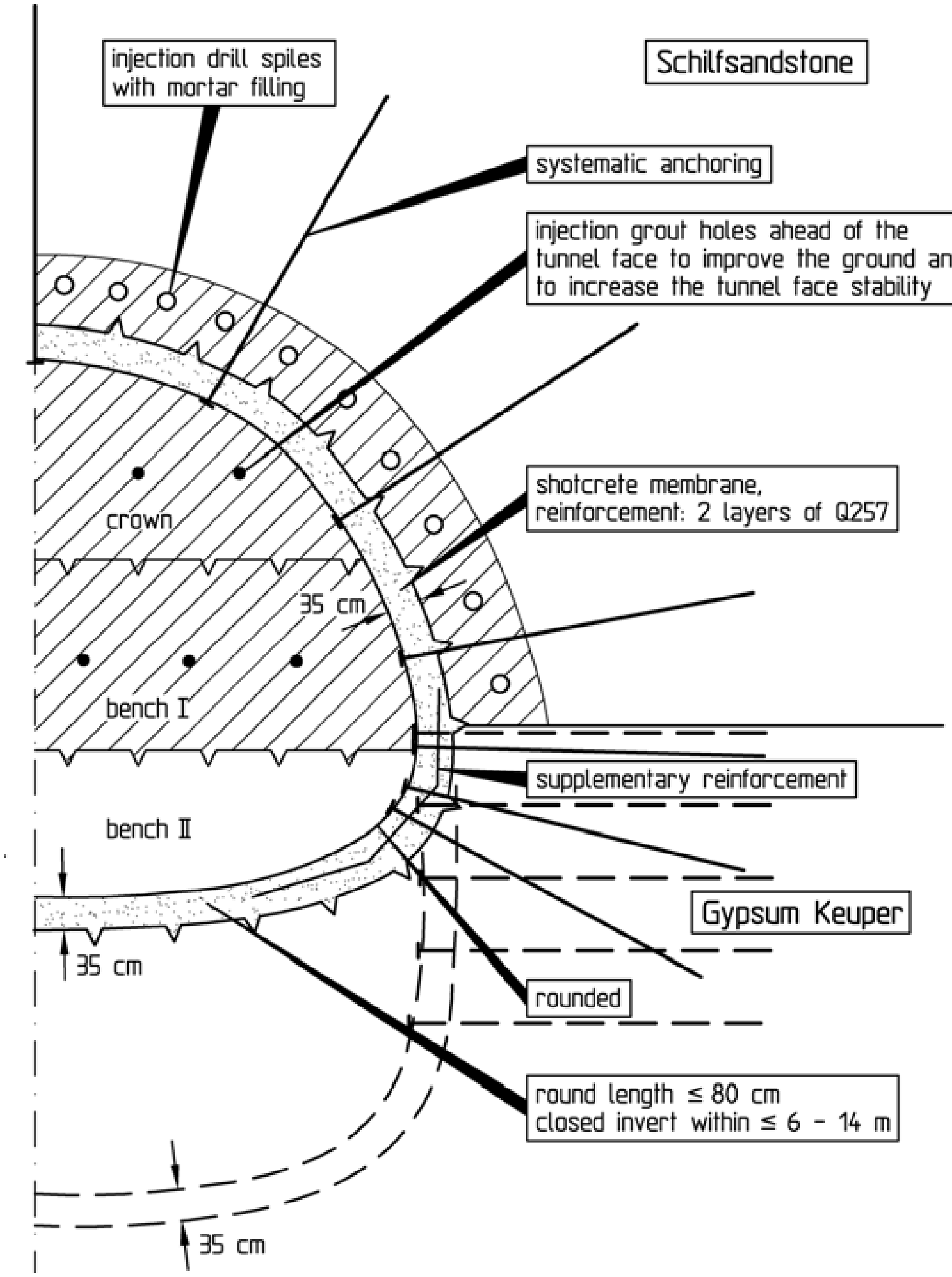


Fig. 4.66: Excavation and support within the area of enlarged cross-section, construction

In addition, advancing injections were carried out to improve the ground ahead the tunnel face and above the crown, and injection drill spiles with mortar filling were installed.

The rounds were carried out with lengths of ≤ 80 cm. The temporary crown invert was supported in the beginning at a distance of 9 to 14 m to the tunnel face. In the area of the undercrossing of the

Gäubahn and in the immediately following part of the heading, the support was closed at the invert at a maximum of 6 m behind the tunnel face (Beiche and Kagerer, 1993).

4.3.7 Monitoring

To measure the tunneling-induced displacements, measuring cross-sections with extensometers and inclinometers were installed in the area of the undercrossing of the Gäubahn and of Zamenhof Street, among other locations. In the tunnel, measuring cross-sections for levelings as well as stress measurement cross-sections with rock mass pressure and concrete pressure measuring cells were installed. In addition, levelings were carried out on the grout surface, on the tracks of the Gäubahn and on structures (Fig. 4.67).

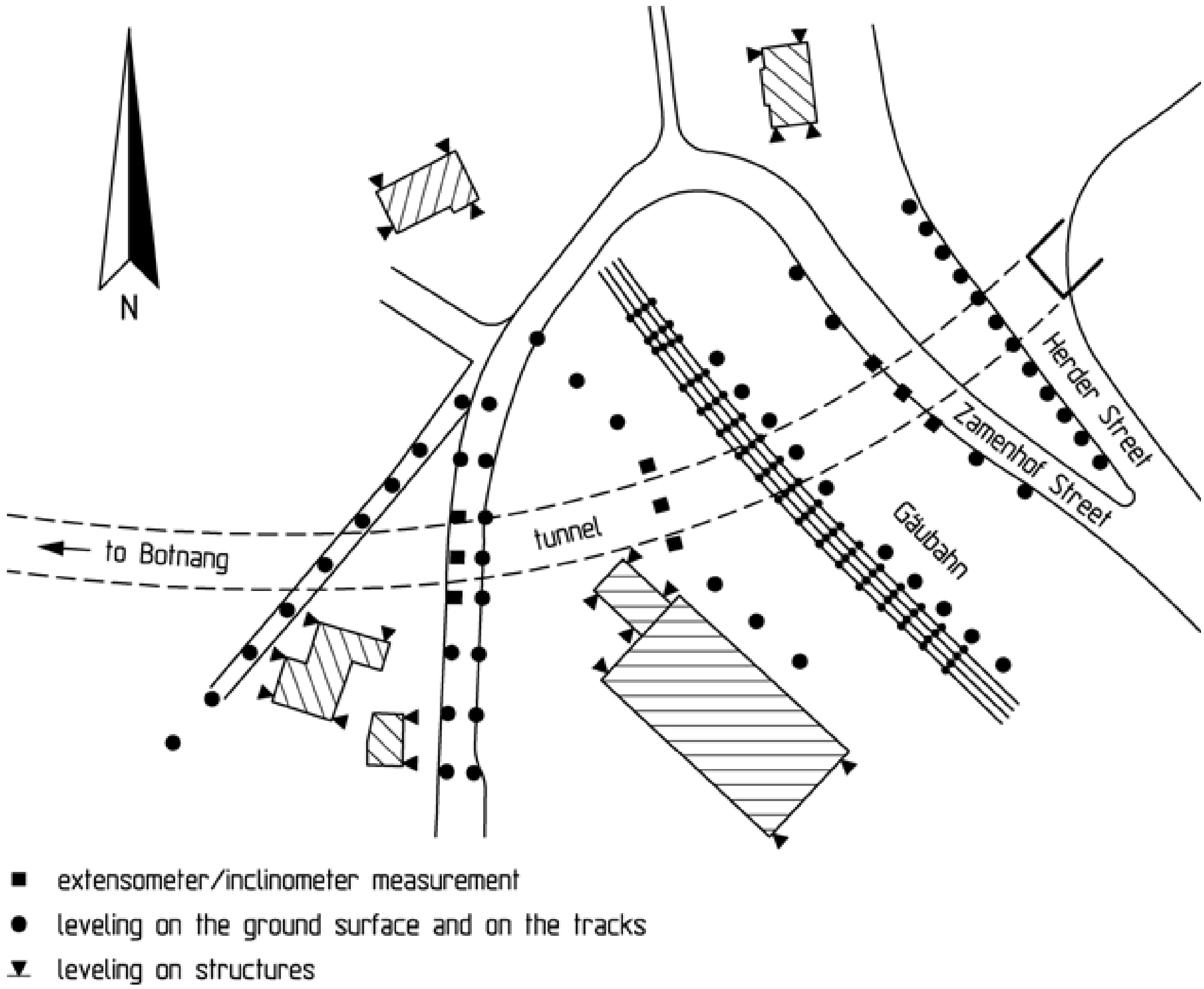


Fig. 4.67: Monitoring program

The subsidence of the ground surface due to the crown heading amounted to approx. 25 mm at the beginning of the excavation and reached 51 mm at the edge of the Gäubahn embankment (Fig. 4.68). Up to this time the temporary crown invert was supported 9 to 14 m behind the tunnel face. To reduce the subsidence, especially during the undercrossing of the Gäubahn, the support was closed at the invert in the further course of the heading at the minimum possible distance of 6 m to the tunnel face, as mentioned above. It was thus possible to reduce the subsidence, which then amounted to only 34 mm in the area of the tracks of the Gäubahn (Fig. 4.68). The magnitude of this value is in reasonable agreement with the subsidence of 27 mm computed under the assumption of an early closing of the invert for the analysis cross-section 2 (see Fig. 4.63).

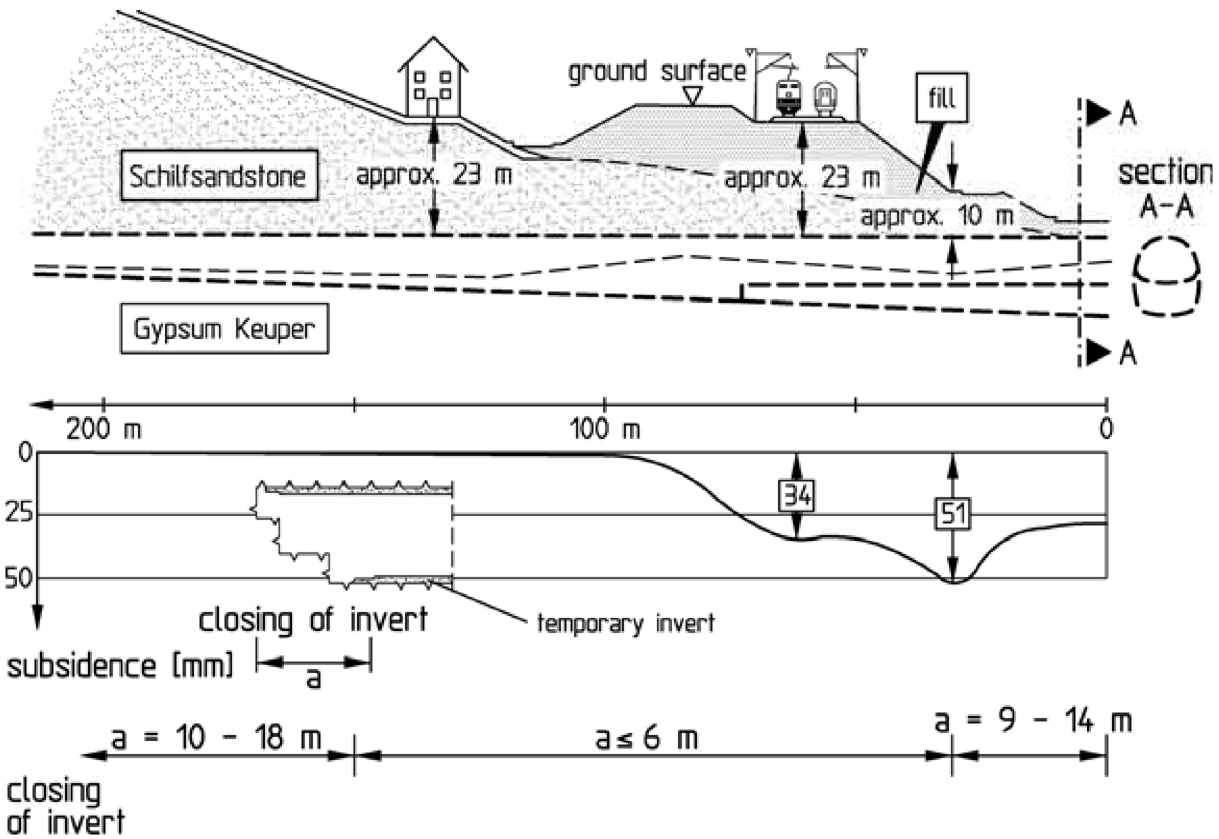


Fig. 4.68: Measured ground surface subsidence, longitudinal section through the tunnel axis

In the further course of the heading the tunneling-induced subsidence decreased to very low values due to the more favorable geotechnical conditions (Beiche and Kagerer, 1993).

4.3.8 Conclusions

During the construction of the Stuttgart city railway tunnel to Botnang, the tracks of the Gäubahn and built-up areas had to be undercrossed (see Fig. 4.49). Since in the area of the undercrossing of the Gäubahn the ground had a high deformability and a low strength, special measures had to be taken to limit the ground surface subsidence due to tunneling and to avoid interference with railway operations and damage to the buildings.

A crown, bench and invert heading with closed support at the invert and a following lowering of the invert was chosen. With an early closing of the invert the tunneling-induced subsidence could be limited to admissible values. It was possible to reduce the loading at the transitions from the sidewalls to the temporary invert decisively by rounding the temporary crown invert. The loading of the high sidewalls after the enlargement of the cross-section could be clearly reduced by a systematic anchoring (see Fig. 4.65).

With these measures it was possible to limit the ground surface subsidence during the undercrossing of the Gäubahn to an admissible value of some 3 cm. Railway operations were not interfered with and damage to the structures undercut and to the railway facilities did not occur.

The FE-analyses contributed essentially to the specification of the excavation and support measures, such as the early closing of the invert, the curvature of the temporary crown invert as well as the systematic anchoring of the sidewalls.