7. Heading under the protection of jet grouting columns

7.1 Road tunnel for the federal highway B 9 in Bonn-Bad Godesberg, Germany

7.1.1 Introduction

In the city of Bonn–Bad Godesberg, Germany, the federal highway B 9 was relocated into a tunnel over a length of approx. 1200 m (Fig. 7.1). The tunnel undercrosses buildings as well as tracks of the German Rail (Deutsche Bahn AG) and the city railway.

The tunnel cross-section is located in gravel sand. The groundwater table lies above the tunnel's invert. To guarantee the stability and to limit the subsidence due to tunneling, the tunnel was headed by the NATM under the protection of advancing jet grouting columns (DIN EN 12716, 2001).

7.1.2 Structure

Two tunnel tubes with two lanes and a width of approx. 11 m each were excavated over a length of approx. 1000 m (Fig. 7.1 and 7.2). The overburden of the tunnel tubes amounts to approx. 6 to 8 m (Fig. 7.2). Approximately in the middle of this section the tunnels undercross the ICE/IC (Intercity Express/Intercity) line Cologne-Koblenz of German Rail as well as a city railway tunnel. The latter is located at the construction pit Moltke square (Fig. 7.1).

This tunnel section is followed by the construction pit Van-Groote square (Fig. 7.1). In the approx. 200 m long tunnel section south of the construction pit Van-Groote square the four lanes run through a tunnel tube which is divided into three sections (Fig. 7.1b and 7.3). This tunnel tube has a total width of approx. 30 m and an overburden of approx. 7 m (Fig. 7.3).

A mouth-shaped profile was selected for the cross-section of the two two-lane tunnel tubes. Fig. 7.4 shows the geometry of the 11.2 m wide and 10.1 m high standard profile. The excavated cross-section amounts to approx. 92 m². The shotcrete membrane is 25 cm thick. The cross-section was excavated in two parts, as detailed in Chapter 7.1.4. The temporary invert of the partial excavation was rounded with R = 13.3 m and supported by a 20 cm thick shotcrete membrane. The reinforced concrete interior lining was constructed 40 cm thick (Fig. 7.4).

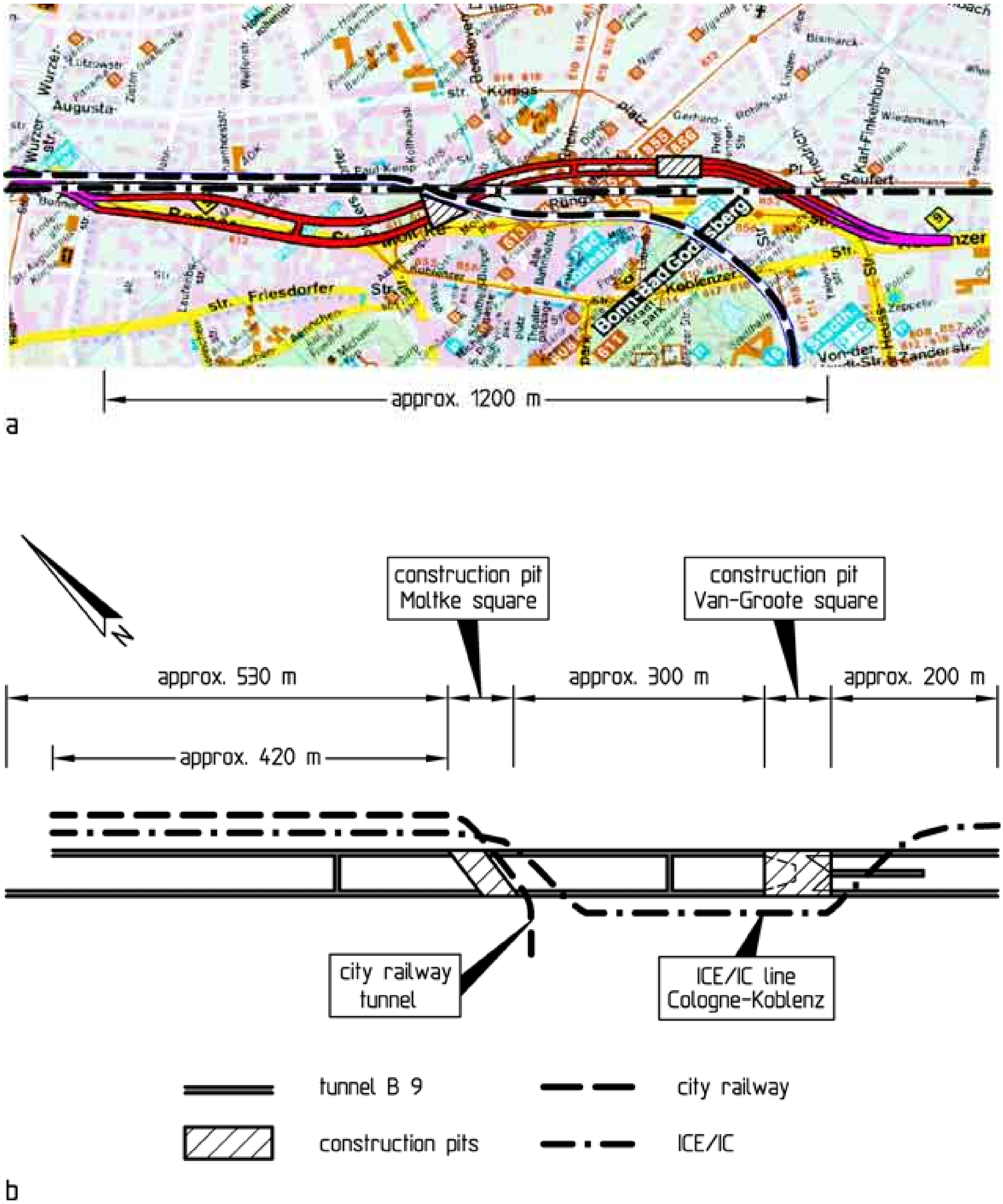


Fig. 7.1: Road tunnel Bonn–Bad Godesberg: a) Site plan; b) schematic representation of the structure

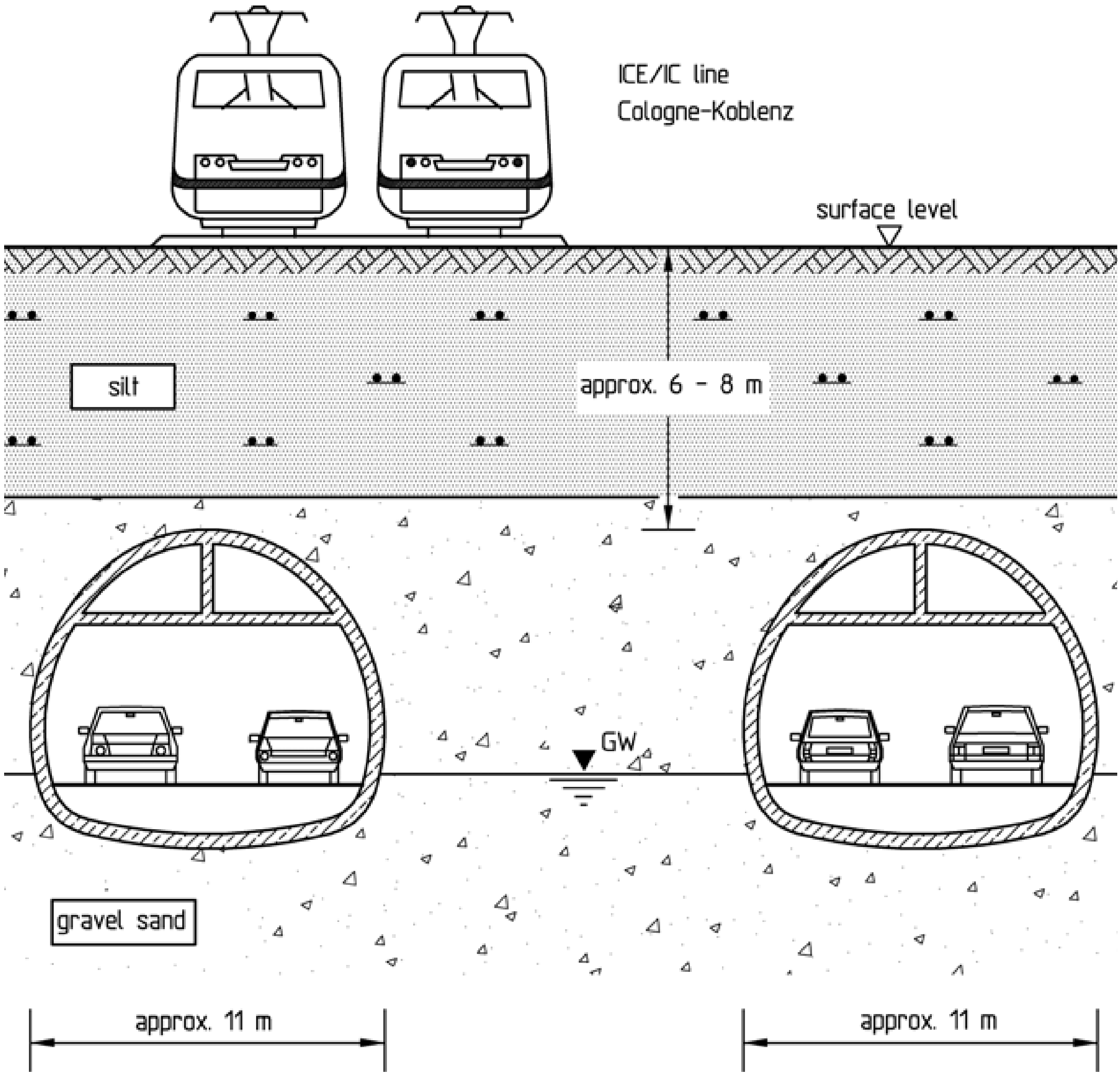


Fig. 7.2: Two-lane tunnel tubes, cross-section

Relatively large radii of curvature were selected for the shotcrete support in the roof and sidewall areas with R = 4.726 m and R = 6.426 m. As a consequence the loading of the shotcrete membrane by bending moments and shear forces is small in these areas, as shown below.

The transitions from the sidewalls to the temporary invert and the permanent invert, respectively, were constructed with relatively small radii (Fig. 7.4). This leads to bending moments and shear forces in the shotcrete membrane in these areas. As shown below, at a radius of 1.2 m supplementary reinforcement is required at the transitions from the sidewalls to the temporary invert in addition to the planned reinforcement with inside and outside steel fabric mats Q188. At the transitions from the sidewalls to the permanent invert, however, no additional reinforcement is required at a radius of 1.8 m (see Chapter 7.1.5).

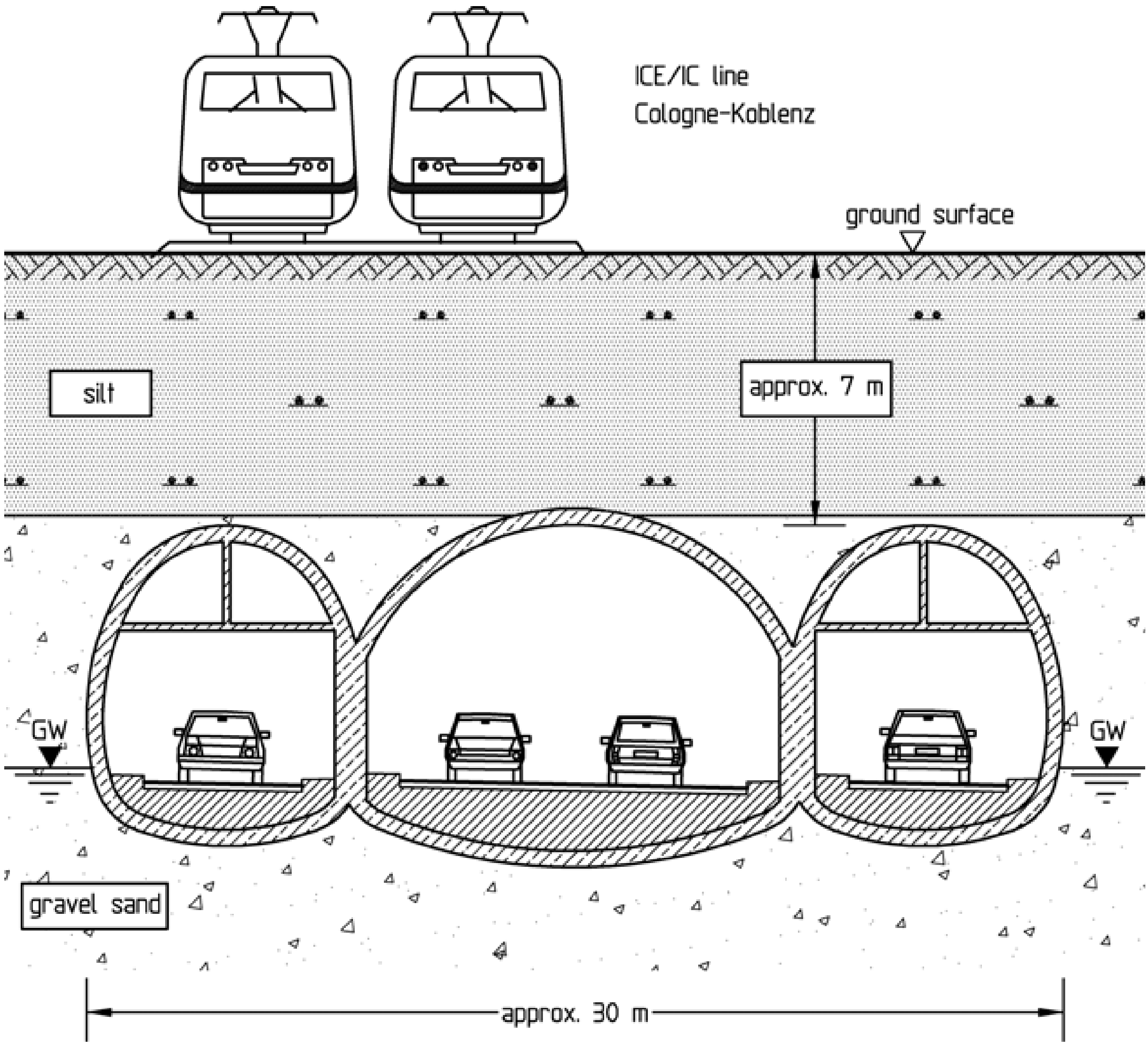


Fig. 7.3: Tunnel tube divided into three sections, cross-section

The invert was slightly rounded with a radius of curvature of R = 15.4 m because of its loading by the water pressure (Fig. 7.4). A greater curvature would lead to a smaller amount of reinforcement in the interior lining, but also to additional excavation and would therefore not be economical.

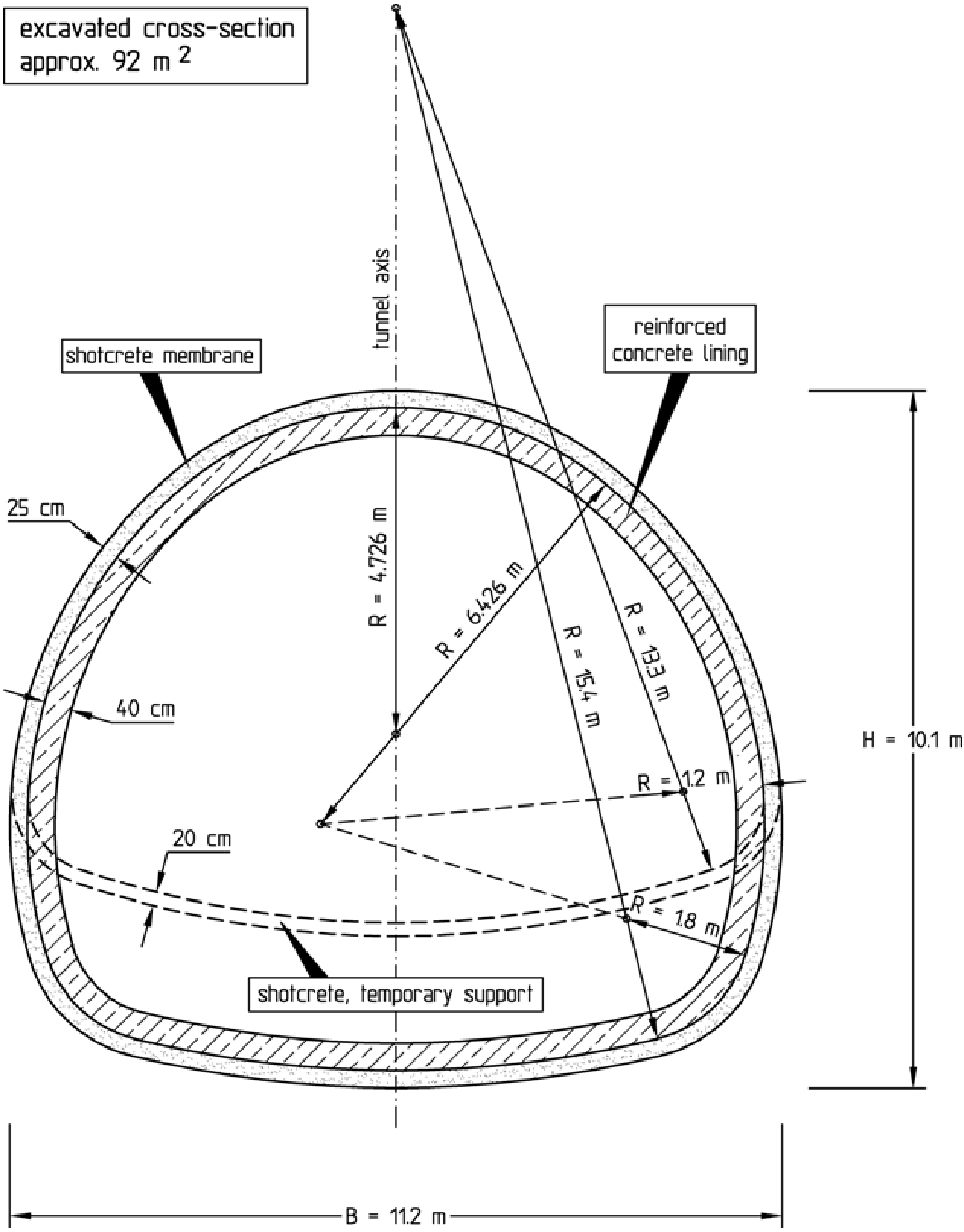


Fig. 7.4: Two-lane tunnel tube, standard profile

7.1.3 Ground and groundwater conditions

The tunnels were headed in the mostly sandy and gravelly soil layers of the gravel deposits of the lower terraces typical for the

Rhine valley in the area of Bonn. Below the tunnel invert silt lenses are sporadically embedded in the sand and gravel layers which will be termed gravel sand in the following (Fig. 7.5).

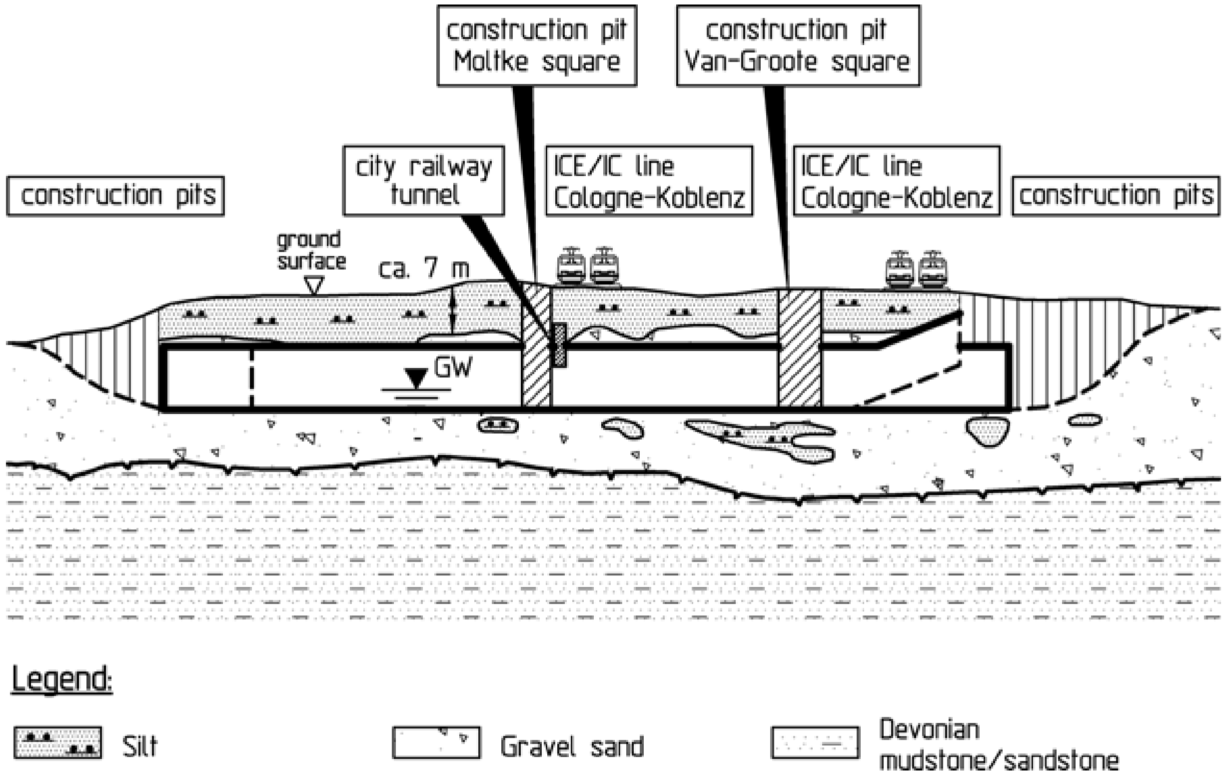
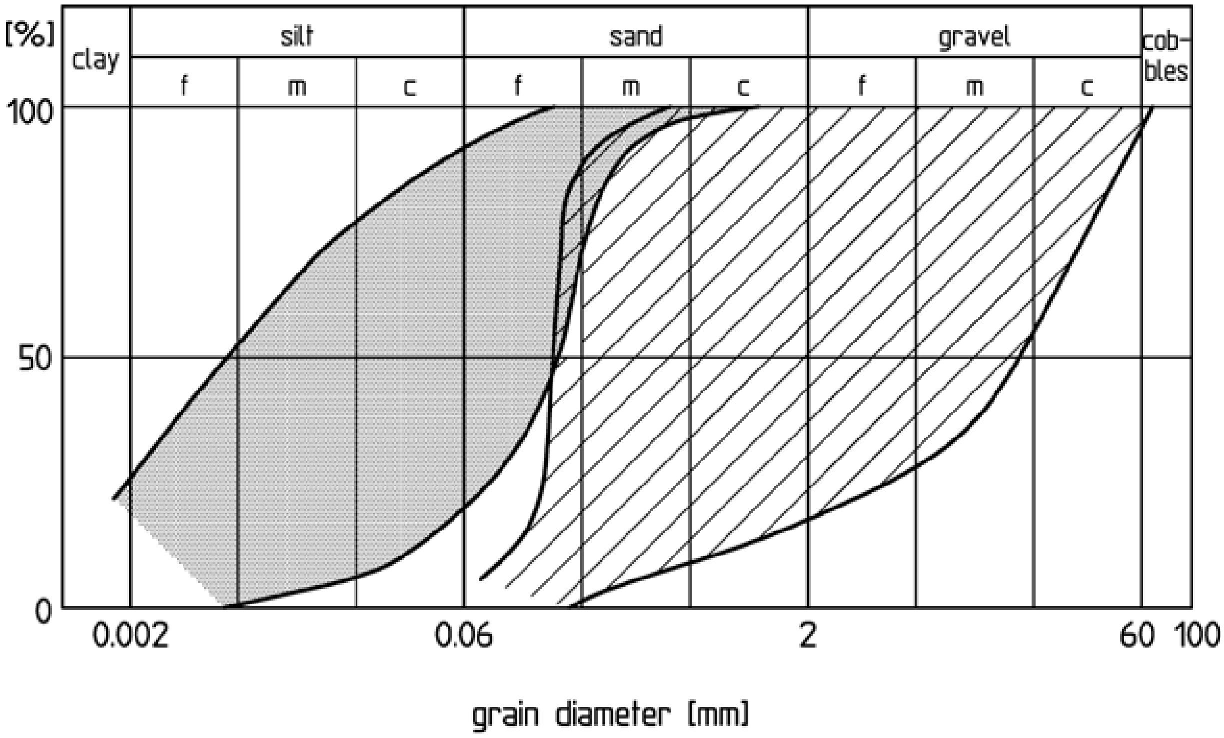


Fig. 7.5: Road tunnel Bonn-Bad Godesberg, longitudinal section with ground profile

The gravel sand is covered by an up to 7 m thick silt layer extending to the ground surface. Below the gravel sand is the Devonian base rock consisting of mudstone and sandstone layers (Fig. 7.5).

Fig. 7.6 shows the grading ranges of the encountered gravel sand and the silt, together with the parameters derived from the subsoil exploration results.

The gravel sand has a high porosity and permeability. It constitutes an aquifer connected to the Rhine river. The groundwater table is therefore influenced by the water levels of the Rhine. On average the groundwater is encountered approx. 3 m above the tunnel's invert (Fig. 7.5).



	E_S [MN/m²]	φ' [°]	c' [kN/m²]
silt	4 - 20	17.5 - 25	25 - 10
gravel sand	20 - 150	30 - 37.5	0

Fig. 7.6: Grain-size distribution of the soils and soil mechanical parameters

7.1.4 Design and construction

The sequence of excavation and the support measures for the heading of the two two-lane tunnels are shown in Fig. 7.7 and 7.8 in cross- and longitudinal section.

First the part of the tunnel cross-section located above the groundwater table was excavated. For statical reasons the temporary invert was rounded and supported by shotcrete (① in Fig. 7.7). Regularly spaced gravity wells were drilled from the temporary invert. With these wells, the groundwater table was lowered to the final tunnel invert (② in Fig. 7.7). Protected by this groundwater drawdown the tunnels were excavated in stages down to the invert and supported (③ in Fig. 7.7).

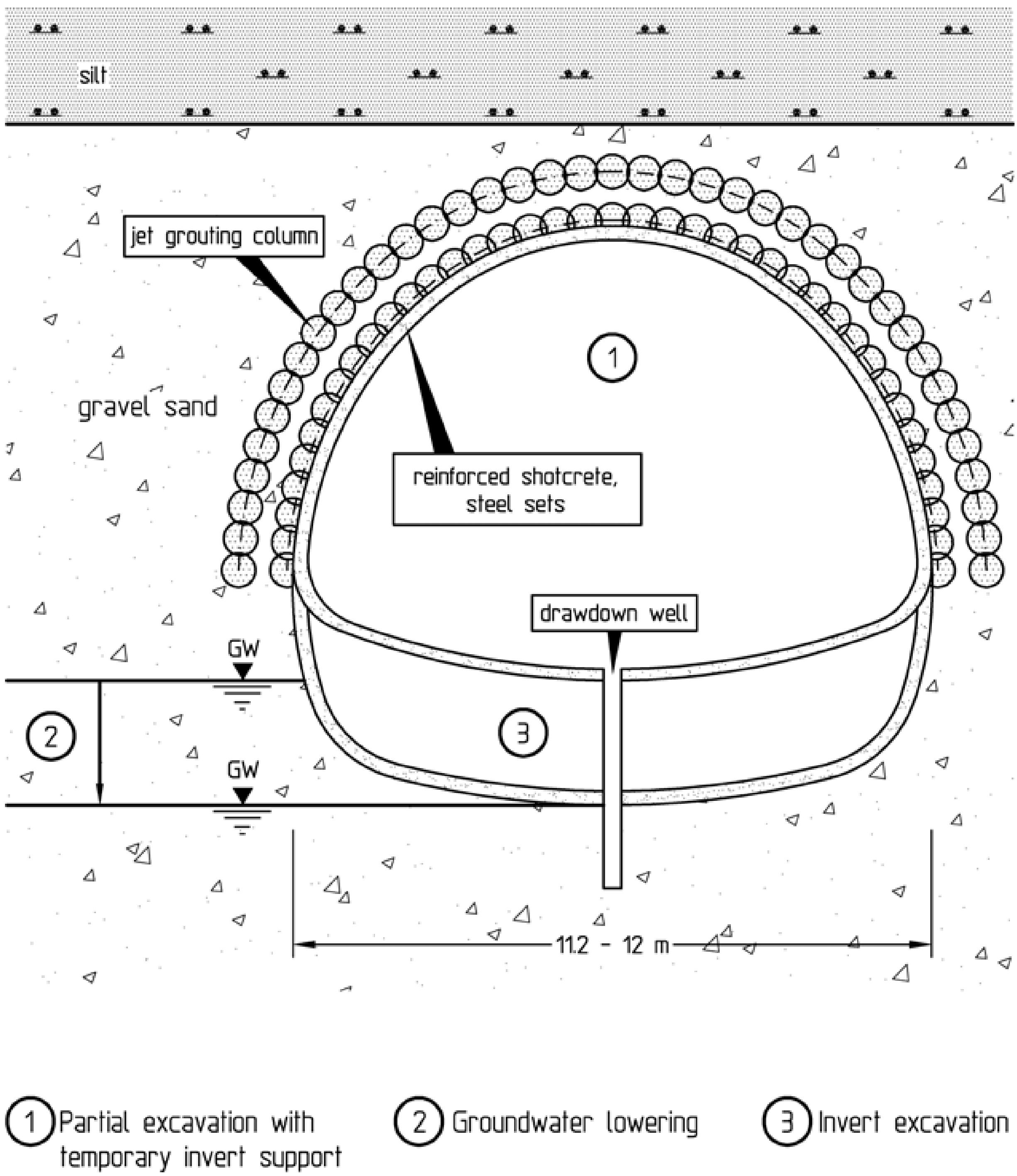


Fig. 7.7: Heading of the two-lane tunnels, excavation and support, cross-section

The partial excavation above the groundwater table was subdivided into crown, bench and invert and carried out with a stepped tunnel face. The round lengths of the partial excavations amounted to 1 m each. To limit the subsidence the temporary invert was closed 6 to 8 m behind the roof excavation. This lead to an inclination of the tunnel face of 60° (Fig. 7.8). Because the tunnel face inclination exceeds the angle of friction of the gravel sand (see Fig. 7.6), the tunnel face was not stable if the apparent cohesion was not taken into account. Since the latter quickly vanishes with the soil drying up, only small sections could be excavated in one step. These sections were immediately sealed with reinforced shotcrete.

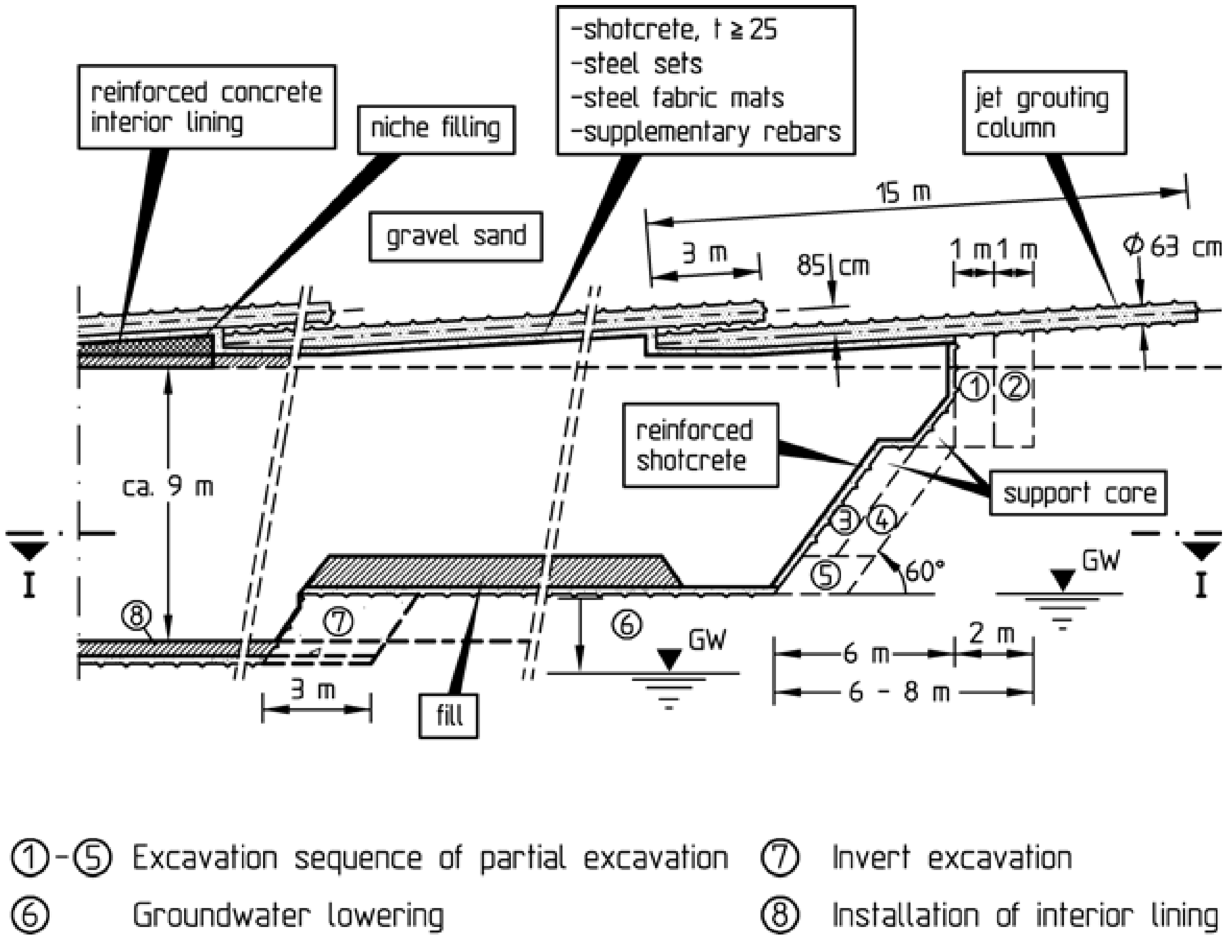


Fig. 7.8: Heading of the two-lane tunnels, excavation and support, longitudinal section

The excavation contour was supported using reinforced shotcrete and steel sets (Fig. 7.7 to 7.9).

As already mentioned, the tunnel tubes were excavated under the protection of advancing jet grouting columns forming a jet grouting vault (Fig. 7.7 and 7.8). This jet grouting vault transfers loads in transverse and longitudinal tunnel directions (Fig. 7.10). Thus the green shotcrete close to the tunnel face as well as the tunnel face area were less strongly loaded. Furthermore, with the jet grouting columns, the subsidence is limited, collapses are avoided, and the safety of the tunneling staff is thus ensured as well.

Fig. 7.9: View of the temporary tunnel face

The jet grouting columns are approximately horizontal, 15 m long and have a design diameter of 63 cm. Two successive jet grouting vaults overlap by 3 m (Fig. 7.8 and 7.11).

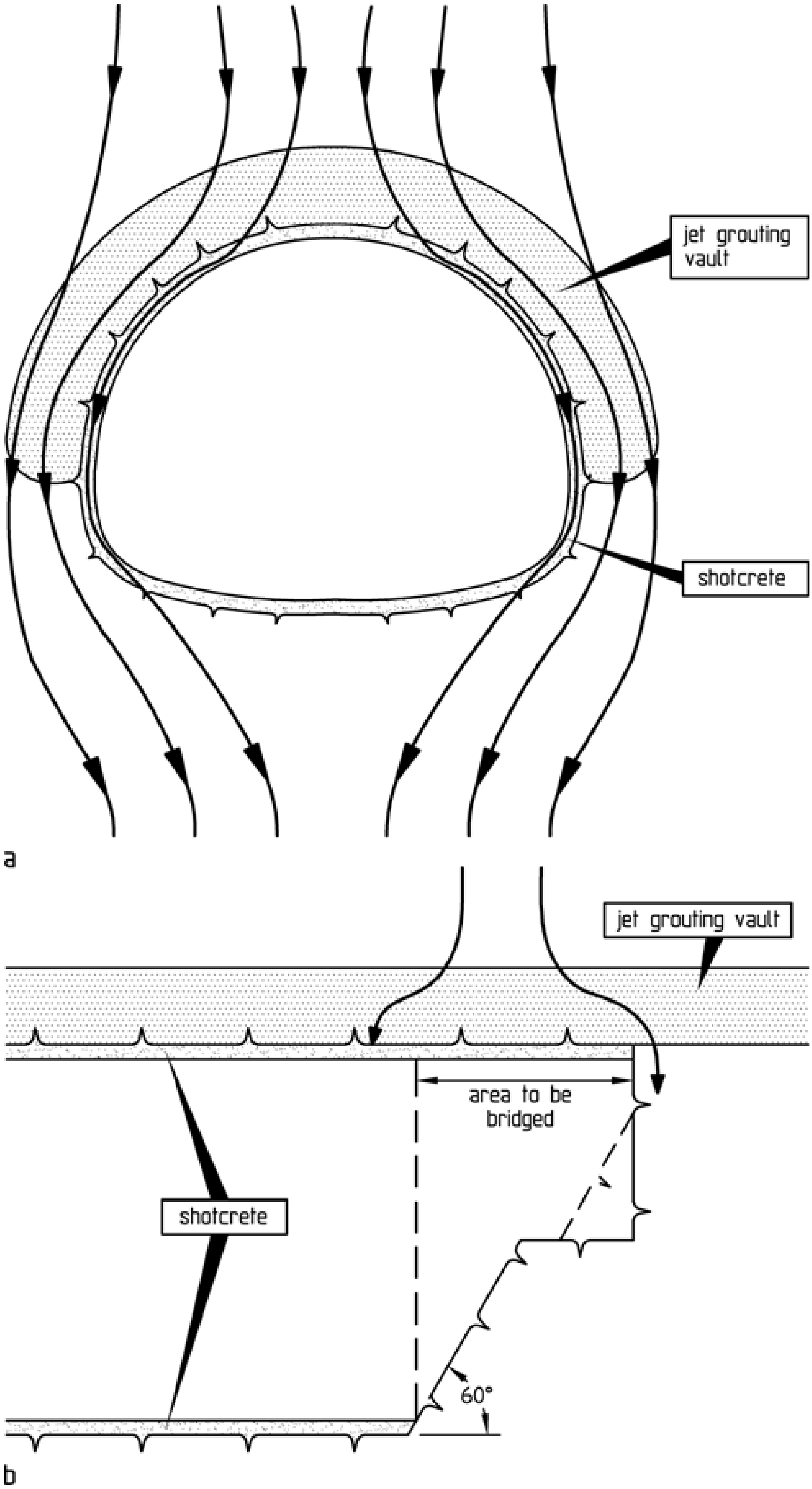


Fig. 7.10: Load transfer by the jet grouting vault and the shotcrete membrane: a) Cross-section; b) longitudinal section

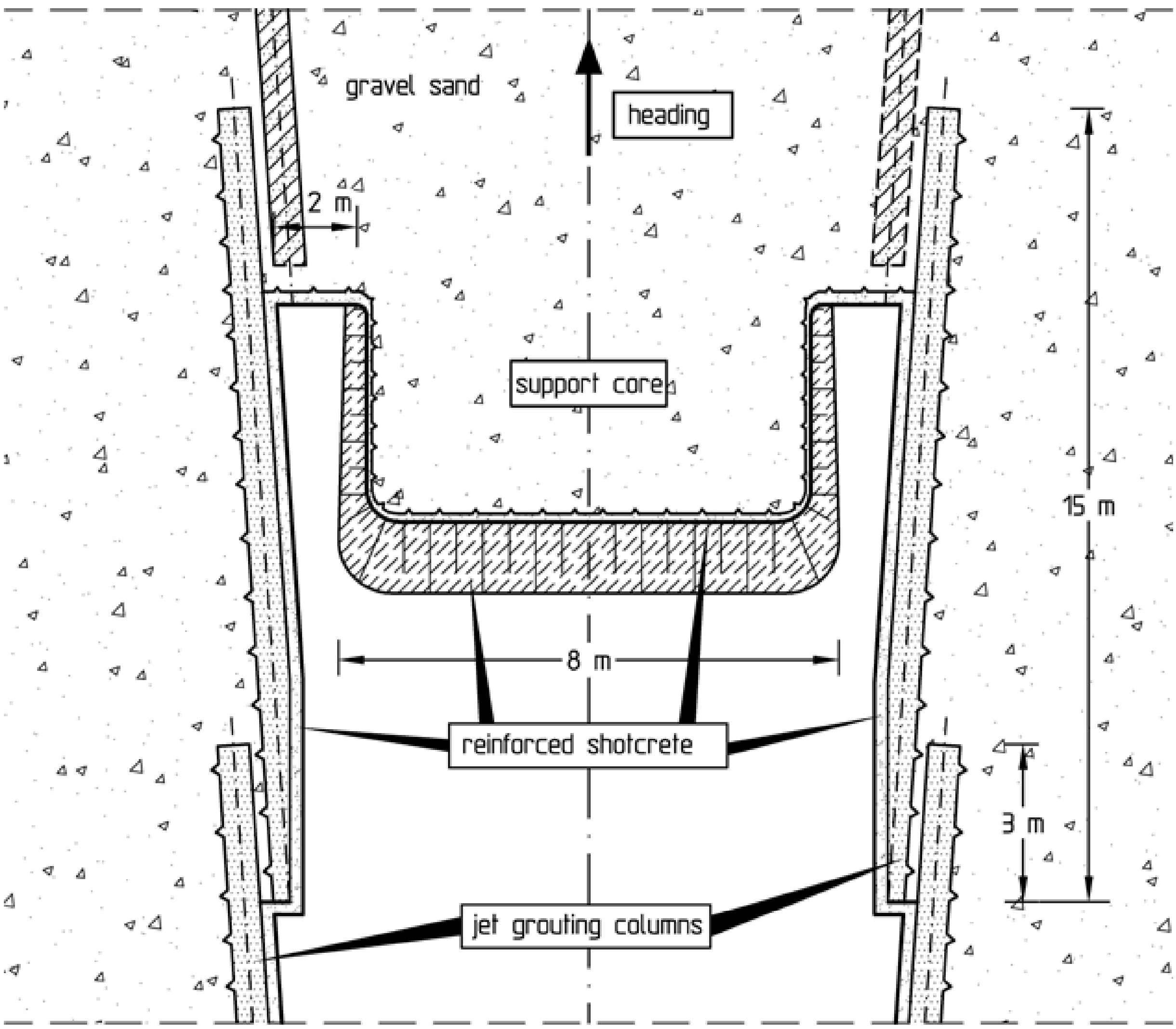


Fig. 7.11: Heading of the two-lane tunnels, excavation and support, plan view (section I-I, see Fig. 7.8)

In order to comply with the tunnel clearance, the jet grouting columns were not constructed horizontally but rather at a slight outward slant (Fig. 7.8 and 7.11). The excavation had thus to be widened in a trumpet-shaped way (Fig. 7.8, 7.11 and 7.12).

An additional stabilization of the tunnel face was achieved by the support core shown in Fig. 7.11 and 7.12. Beside and above this support core was enough space for the jet grout drill carriage (Fig. 7.11 and 7.13). The tunnel face was supported in sections using reinforced shotcrete (Fig. 7.8 and 7.11).

The jet grouting columns were constructed by the single-phase method (DIN EN 12716, 2001), according to which in a borehole cement based suspension is injected under high pressure (approx. 500 bar) into the soil via a rod and a nozzle at its end.

Fig. 7.12: Tunnel face with support core and niche for the construction of the jet grouting columns

Fig. 7.13: Jet grout drill carriage in operation

The nozzle rotates with the rod which is slowly pulled out of the borehole. In this way a column develops due to the mixing of the suspension with the ground. After hardening of the cement, this column possesses a high strength in comparison with the undisturbed soil. The surplus mixture of suspension and soil exits as backflow through the annular gap between borehole and rod. Fig. 7.13 shows the jet grout drill carriage in operation.

To optimize the production parameters six test columns were constructed and dug out. Column diameters ranging from 60 to 90 cm were obtained. The production parameters are listed in Table 7.1. The parameters in the lines marked with arrows in Table 7.1 were selected for the construction of the jet grouting columns. Column diameters between 60 and 70 cm were achieved with these parameters (Wittke et. al, 2000).

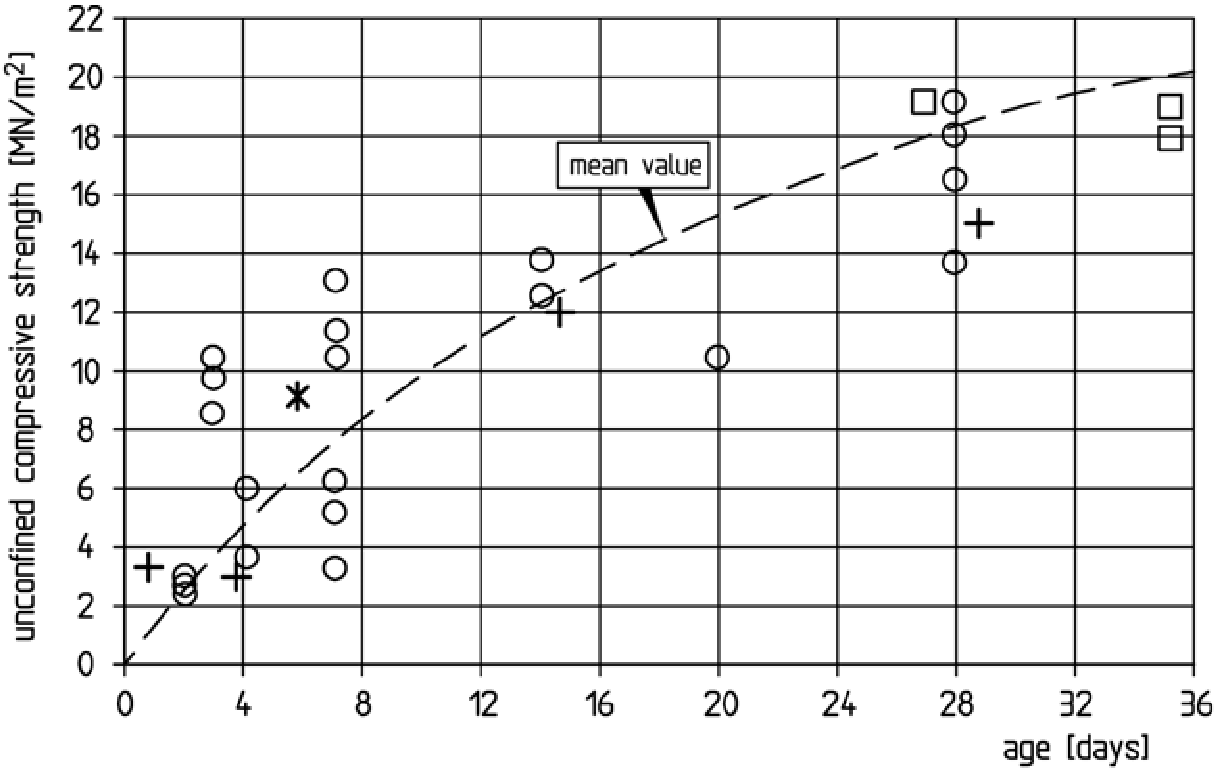
Retracting rate [cm/min]	Injection pressure [bar]	Water/ cement ratio [-]	Cement quantity [kg/m]	Column diameter [cm]
➔ 30	500	1.05	251	60 – 70
"	"	1.0	260	"
➔ 27	500	1.05	283	60 – 70
24	"	"	313	"
"	"	1.0	325	"
20	"	"	510	80 - 90

➔ Parameters selected for the construction of the jet grouting columns

Table 7.1: Production parameters of six test columns

A comparatively small unconfined compressive strength of $\sigma_D = 0.75$ MN/m^2 was demanded for the jet grouting columns and accounted for in the stability analyses (see Chapter 7.1.5). This strength was achieved already after a short time and the idle time causing high cost and long construction times could be minimized.

Fig. 7.14 shows the unconfined compressive strength measured on drill cores taken and on samples from the backflow as a function of the sample age (Wittke et al., 2000). The diagram also includes the unconfined compressive strength determined on drill cores and backflow samples of jet grouting columns constructed during the heading of the city railway tunnel "Killesberg-Messe" in Stuttgart (see Chapter 7.2). According to this diagram, values of σ_D = 0.75 MN/m^2 are achieved already after one day.



	backflow	drill cores
tunnel B 9 Bonn-Bad Godesberg	*	+
tunnel Killesberg-Messe, Stuttgart (see Chapter 7.2)	○	□

Fig. 7.14: Unconfined compressive strength measured on backflow samples and drill cores versus sample age

The construction of the jet grouting columns included comprehensive quality management measures. The success of these measures became apparent during the excavation. No defects were found in the jet grouting vaults.

To determine the influence of the vibrations due to tunneling as well as railway and road traffic on the stability of the tunnel face, the vibration velocity was measured during tunneling. The occurring vibrations turned out to be small. As an optional position it was planned to additionally stabilize the tunnel face using jet grouting columns (Fig. 7.15). To reduce the strength a portion of the cement in the suspension would have been replaced with bentonite to facilitate the later demolition of these columns. The construction of tunnel face columns did not become necessary, however.

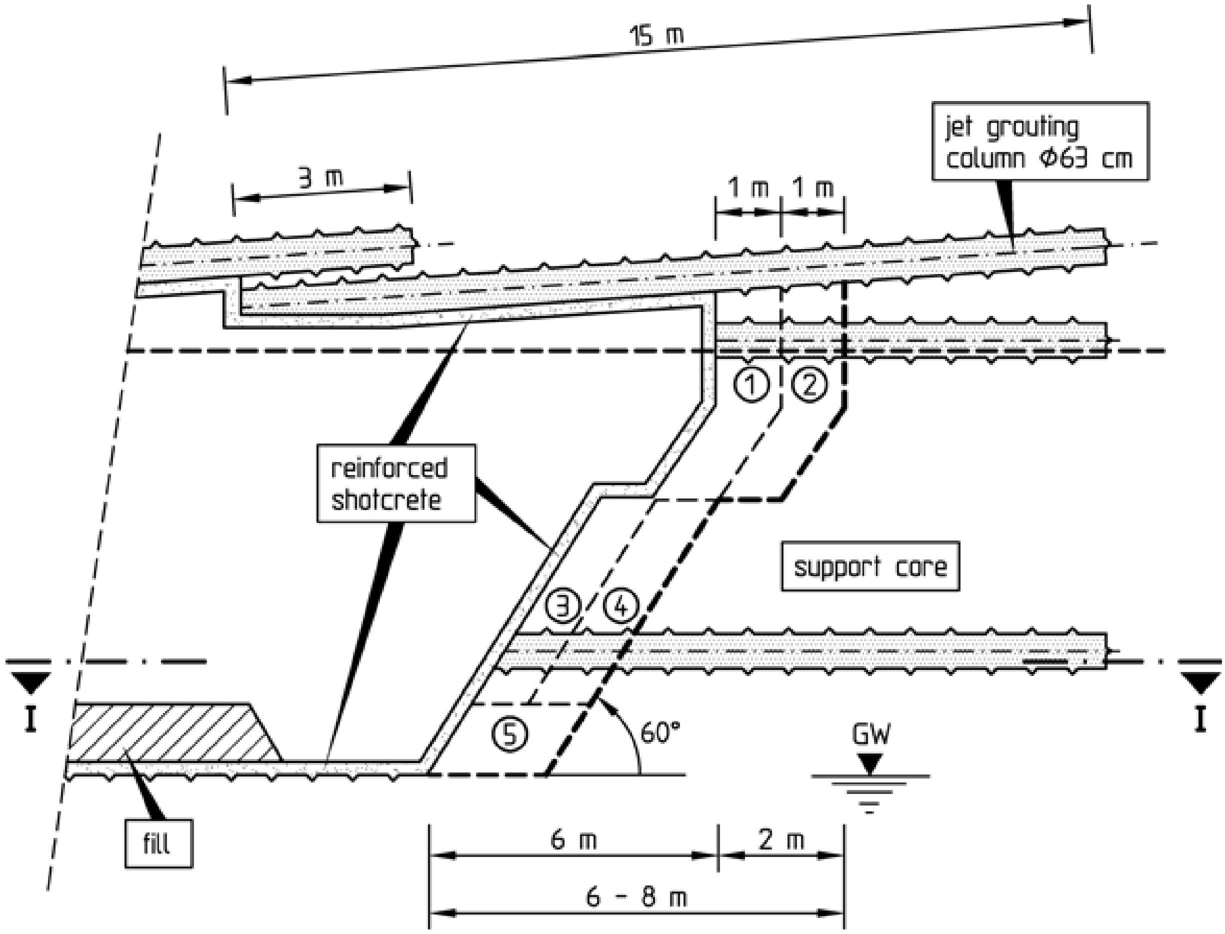


Fig. 7.15: Stabilization of the tunnel face using jet grouting columns (not carried out)

In the area where the tunnel tube is divided into three sections, the side tubes were excavated first under the protection of jet grouting columns and supported. The central tube was only excavated after the interior lining had been installed in both side tubes. A crown excavation with trailing bench and invert was carried out here. The shotcrete membrane of the central tube hereby is supported by the interior linings of both side tubes (Fig. 7.16).

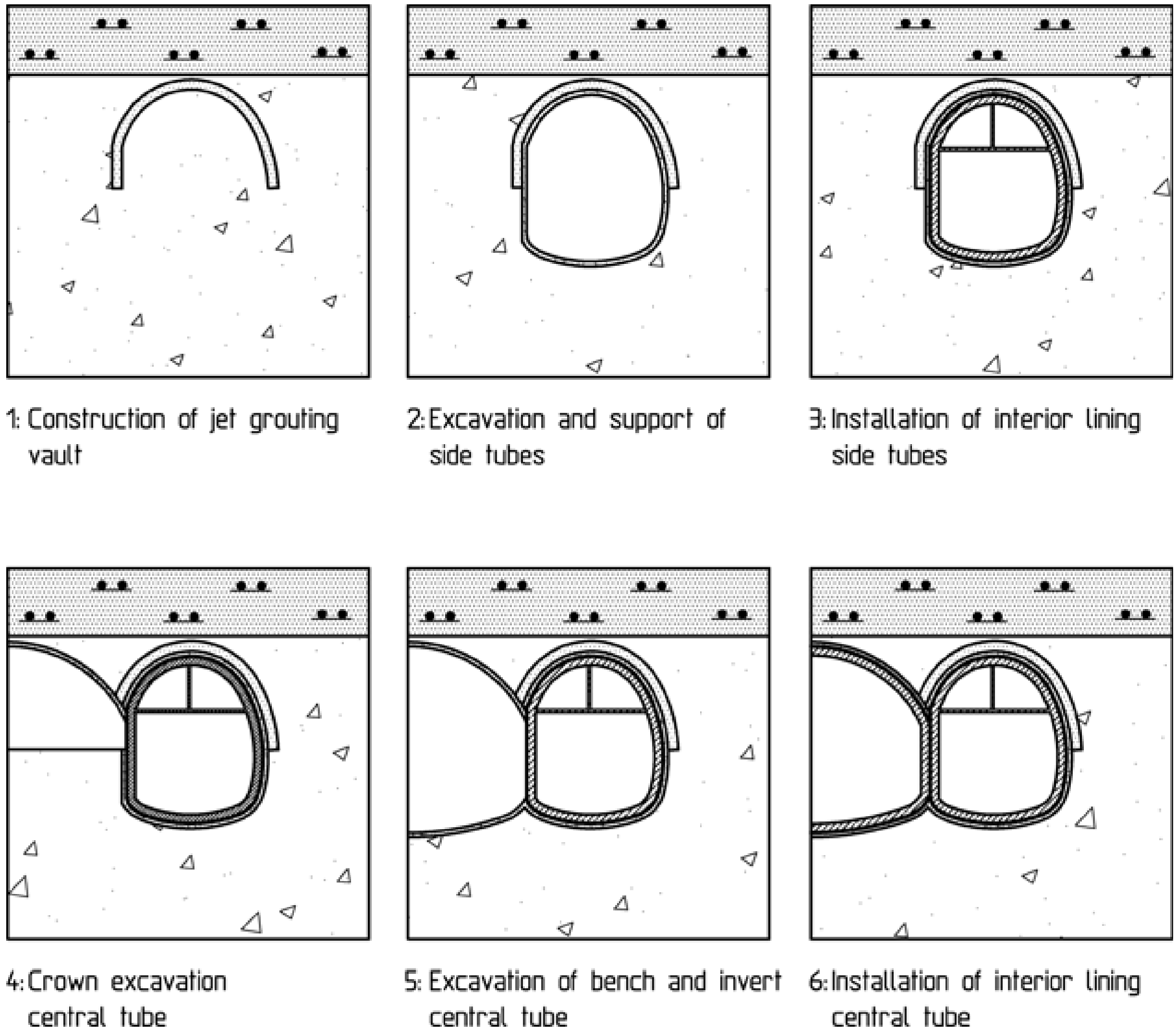


Fig. 7.16: Excavation sequence/construction stages for the tunnel tube divided into three sections

7.1.5 <u>Stability analyses for the design of the shotcrete support</u>

To design the shotcrete support two- and three-dimensional FE-analyses were carried out with the program system FEST03 (Wittke, 2000). Fig. 7.17 shows the location of the 10 analysis cross-sections (AC 1 to AC 10) in the geological cross-section which are investigated in the design analyses.

Fig. 7.18 shows exemplarily the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters for a three-dimensional analysis.

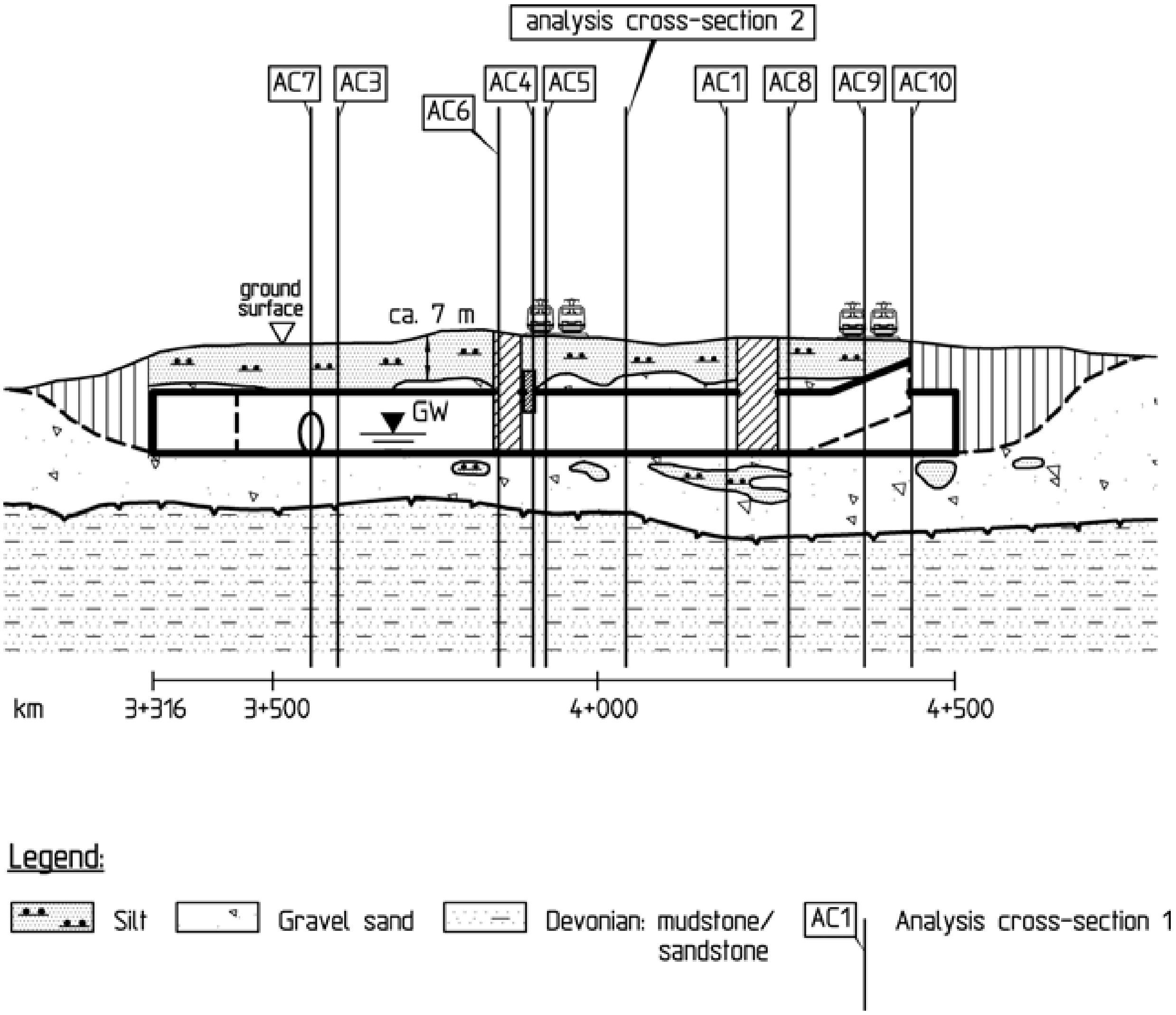


Fig. 7.17: Location of the 10 analysis cross-sections

In the following the results of a two-dimensional analysis for analysis cross-section 2 are presented. Analysis cross-section 2 is located in the section of the two two-lane tunnel tubes (see Fig. 7.17).

Fig. 7.19 shows the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters this analysis was based upon. The computation section consists of a 25 m wide, 30.5 m high and 1 m thick slice of the ground. The FE-mesh was subdivided into 669 three-dimensional isoparametric elements with a total of 797 nodes.

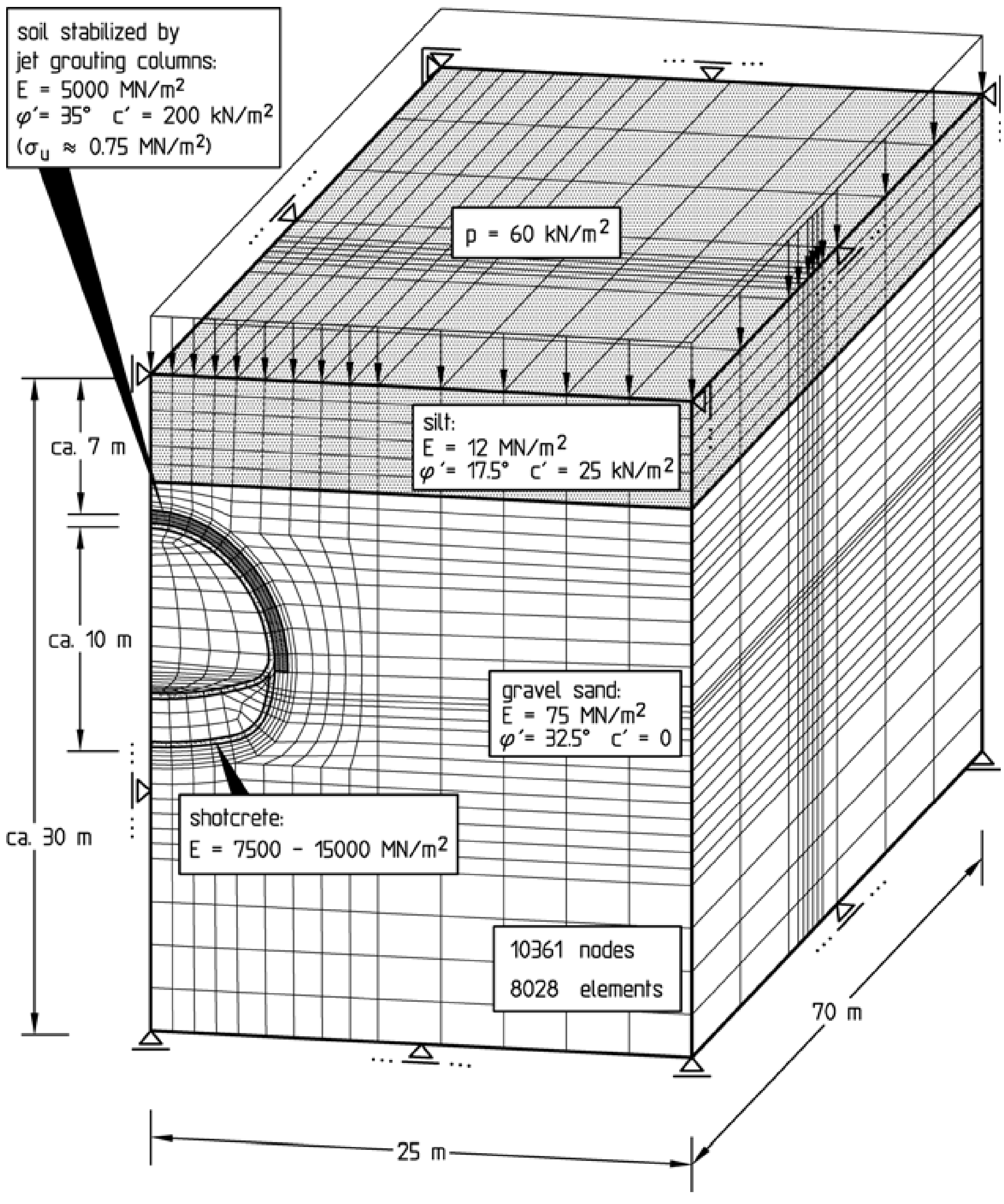


Fig. 7.18: Three-dimensional computation section, FE-mesh, boundary conditions, ground profile and parameters

For the nodes on the lower boundary (z = 0), horizontally sliding supports were assumed as boundary conditions. Vertically sliding supports were introduced for the nodes located on the vertical lateral boundary planes (x = 0 and x = 25 m) (Fig. 7.19). All nodes were assumed fixed in y-direction. The loading due to buildings that acts on the analysis cross-section was represented by a surface load (p = 60 kN/m^2).

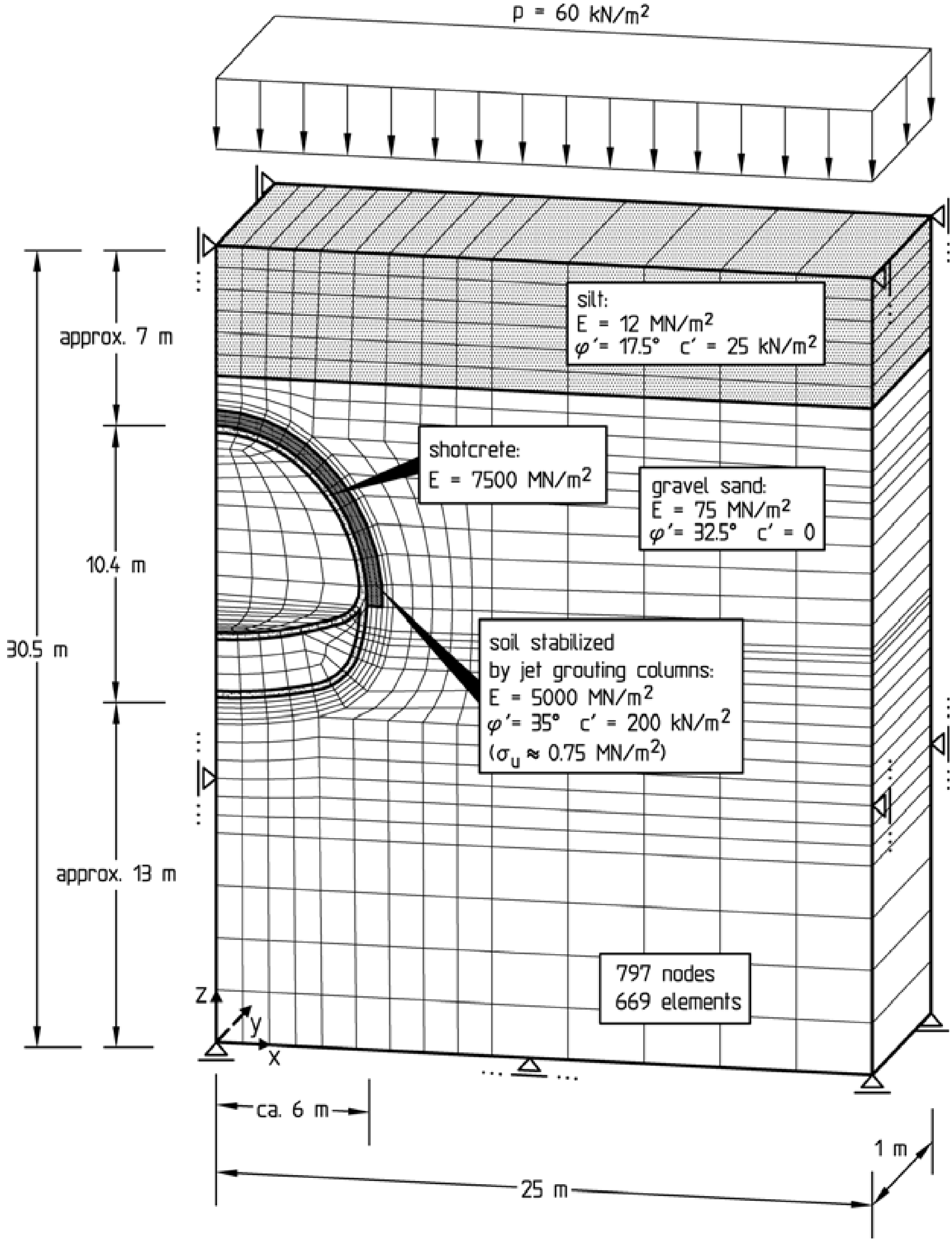


Fig. 7.19: Analysis cross-section 2, FE-mesh, boundary conditions, ground profile and parameters for two-dimensional analyses

In the stability analyses for analysis cross-section 2 the heading of only one tunnel tube was investigated. Because the distance of the two tunnel tubes amounts to more than one tunnel diameter in this area, the two tubes influence each other only to a small degree. Since symmetry exists with respect to the tunnel axes, the ground profile and the cross-sectional shape of the tunnel tubes, only one half of a tunnel tube was modeled. The vertical section through the tunnel axis constitutes the plane of symmetry (Fig. 7.19).

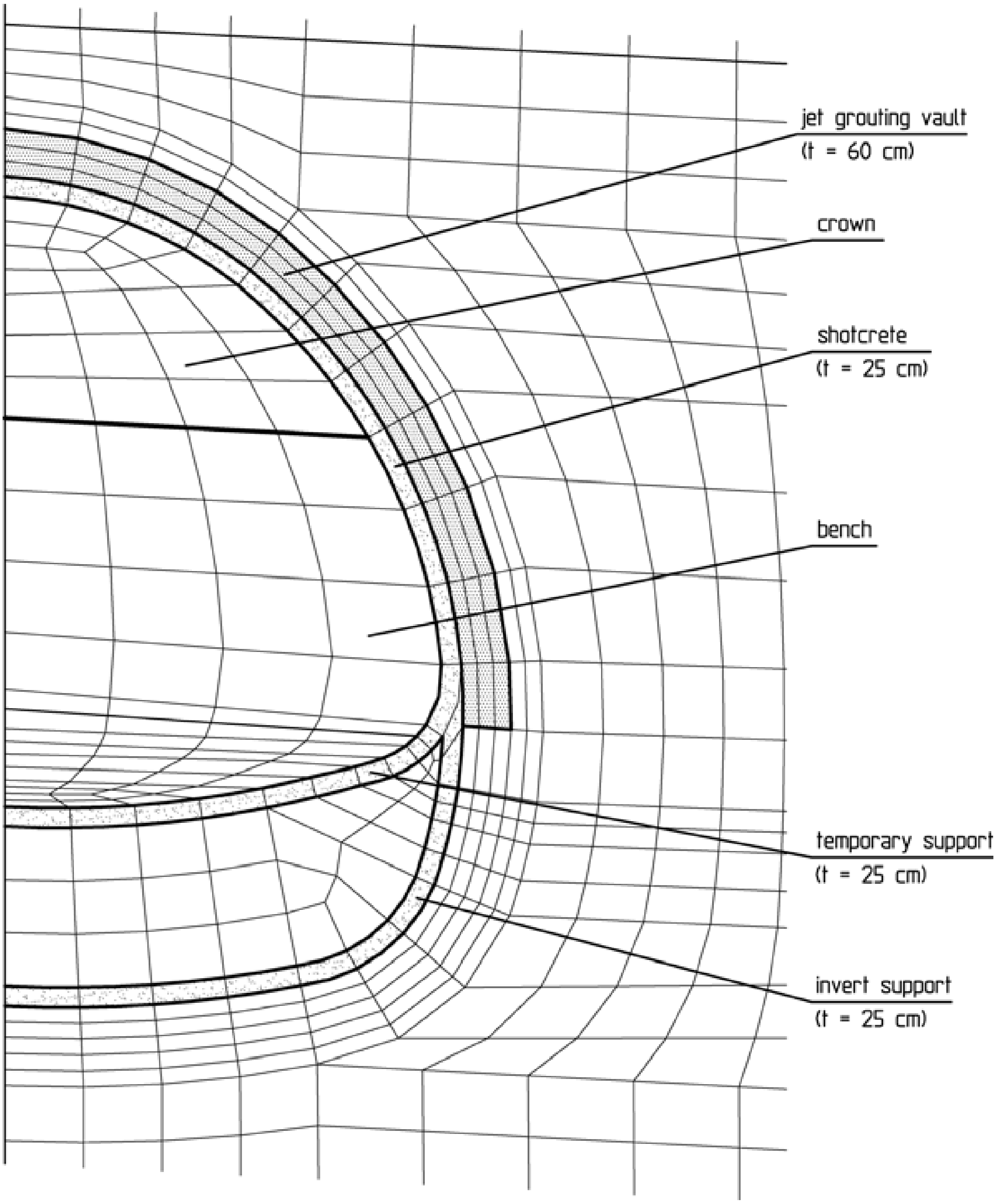


Fig. 7.20: Analysis cross-section 2, FE-mesh, detail

The shotcrete membrane with a thickness of d = 25 cm was modeled by one element layer. Three element layers with a total thickness of 60 cm were selected for the simulation of the jet grouting vault (Fig. 7.20).

The parameters chosen for the undisturbed soil and the soil stabilized with jet grouting columns were determined on the basis of the exploration results by the parties concerned during technical discussions. They are shown in Fig. 7.19.

The deformability and the strength of the soil stabilized by the jet grouting columns develop with time (see Fig. 7.14). In agreement with the parties concerned, values of E = 500 MN/m^2, φ' = 35° and c' = 200 kN/m^2 were specified for Young's modulus and the shear strength parameters of the jet grouting vault. These shear strength parameters correspond to an unconfined compressive strength of σ_D = 0.75 MN/m^2. According to Fig. 7.14, these values are attained after a few days already.

Young's modulus assumed for the shotcrete was varied in the stability analyses. In the following the computation sequence and the results of an analysis are presented, in which a modulus of E = 7500 MN/m^2 was selected for the shotcrete (Fig. 7.19). This relatively small value reflects the development of strength and deformability and the creep properties of the shotcrete (see Chapter 2.1). In the case presented here the shotcrete is loaded at a very young age due to the early closing of the shotcrete support approx. 6 to 8 m behind the crown excavation (see Fig. 7.8).

Fig. 7.21 shows the eight computation steps applied to simulate the excavation and support of the tunnel. In the 1st computation step, the state of stress and deformation resulting from the dead weight of the soil and the loading due to buildings (in-situ state) is determined. In the 2nd computation step the installation of the jet grouting vault is simulated. A preceding stress relief of the soil in the area of the crown excavation is modeled in the 3rd computation step (Wittke, 2000). The reduced Young's modulus of the gravel sand of E_{red} = 45 MN/m^2 corresponds to a stress relief factor of a_V = 0.6 according to (4.1). The 4th computation step represents the crown excavation and its support using shotcrete. In the 5th and 6th computation steps, the preceding stress relief of the soil in the area of the bench excavation as well as the bench excavation with temporary invert support using shotcrete are simulated.

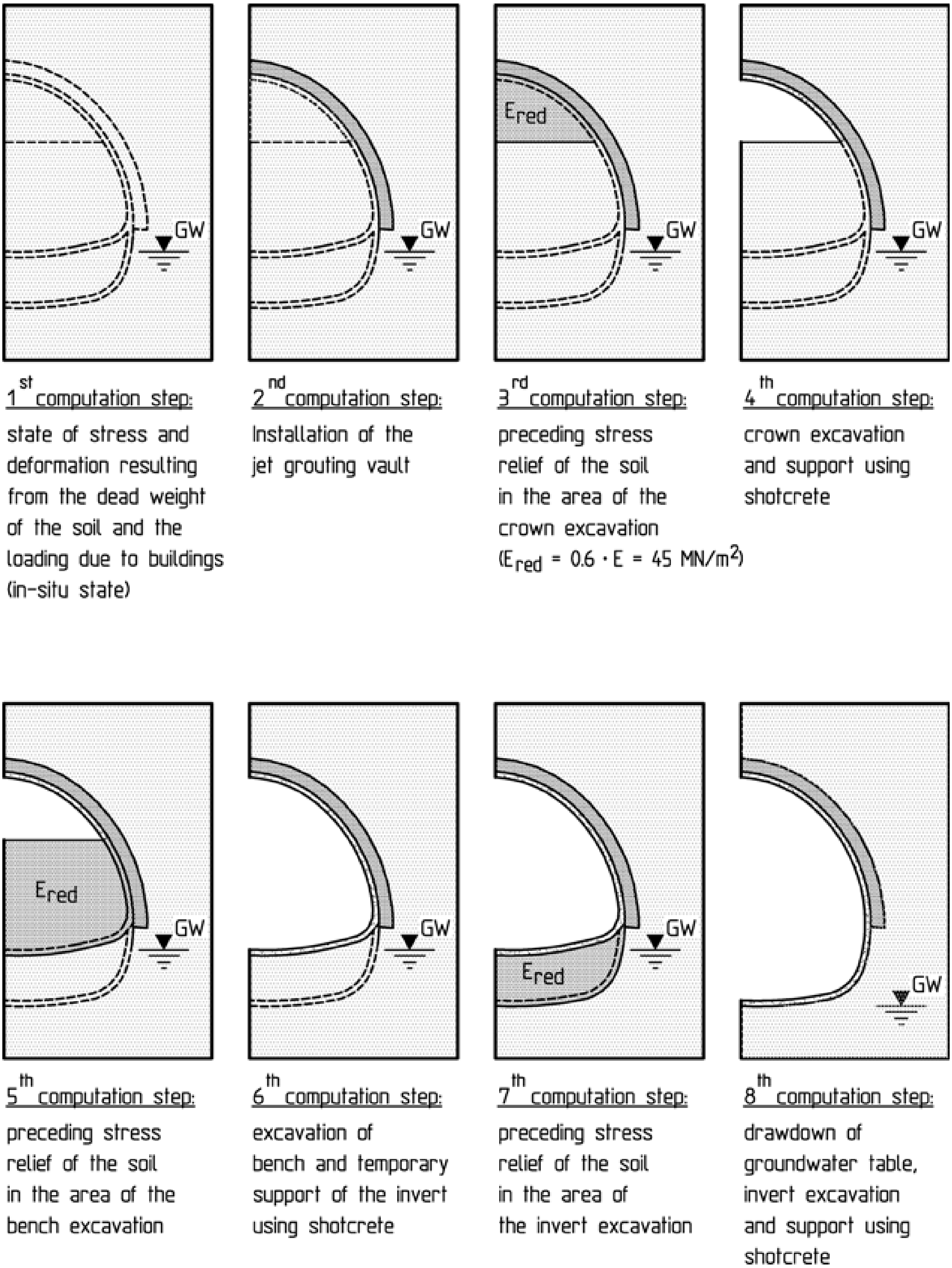


Fig. 7.21: Analysis cross-section 2, computation steps

After the preceding stress relief of the soil in the area of the invert excavation in the 7th computation step, the drawdown of the groundwater table to the invert level and the excavation and shotcrete support of the invert are simulated in the 8th computation step.

In Fig. 7.22 the nodal displacements computed for the 8th computation step related to the in-situ state (1st computation step) are shown in horizontal sections above the tunnel roof. The subsidence of the ground surface above the tunnel roof amounts to 33 mm.

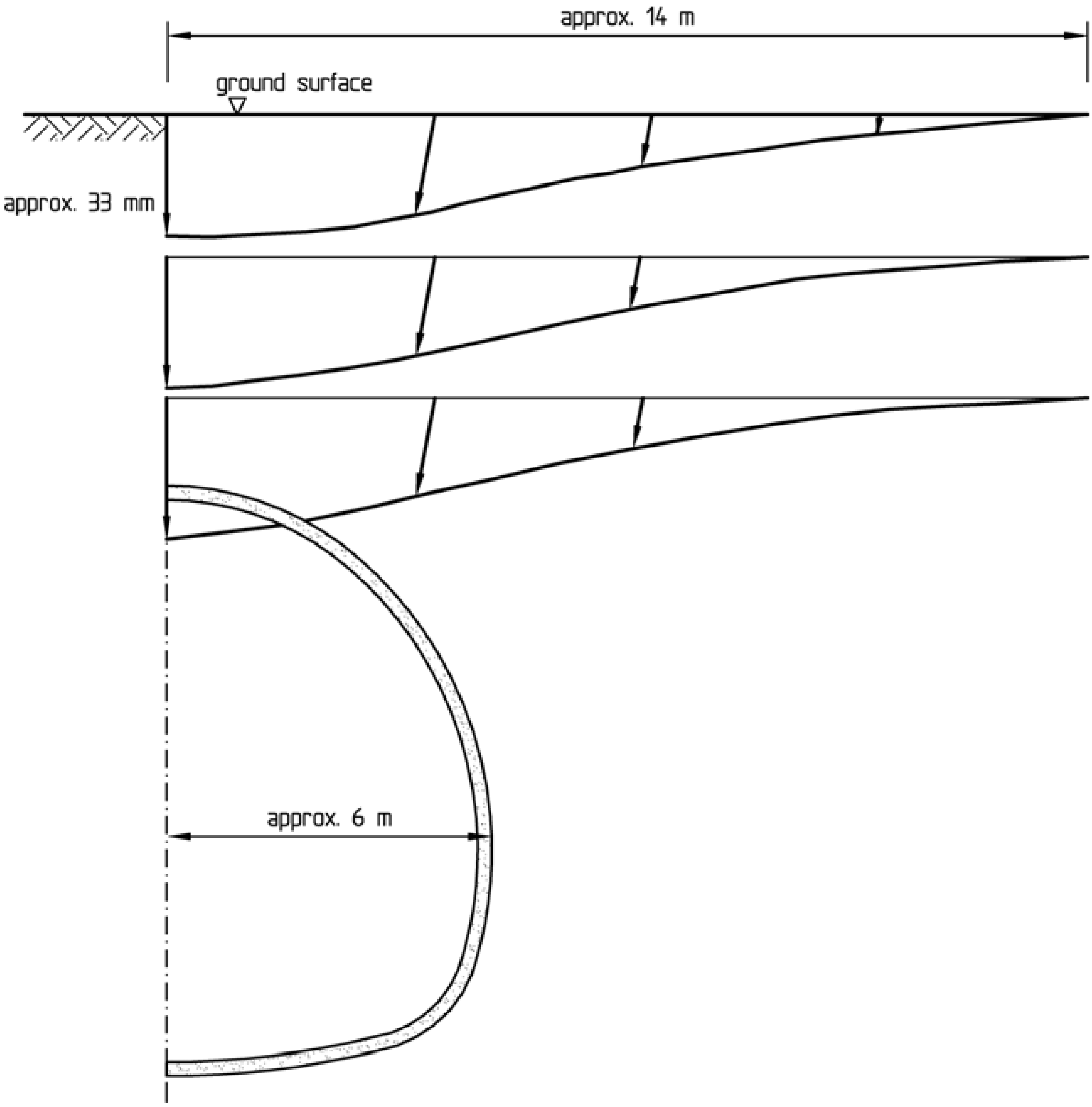


Fig. 7.22: Analysis cross-section 2, displacements, 8th – 1st computation step

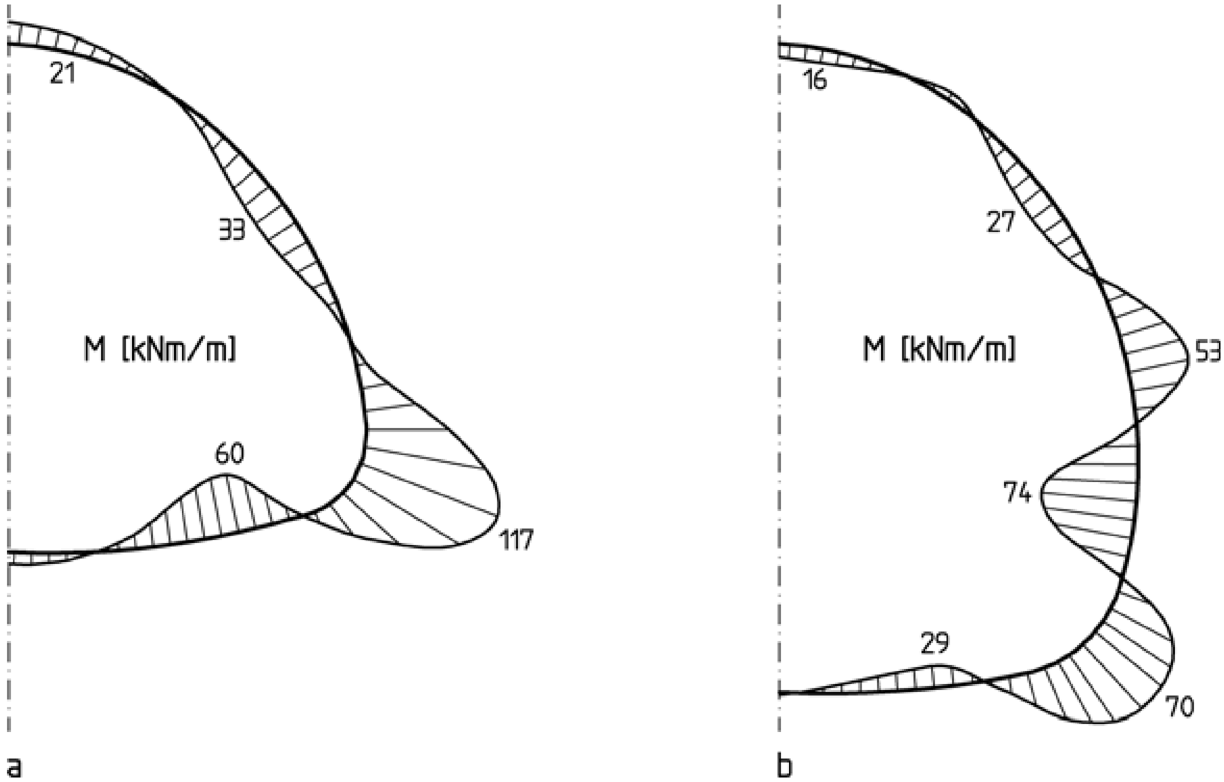


Fig. 7.23: Analysis cross-section 2, bending moments in the shotcrete membrane: a) 6th computation step; b) 8th computation step

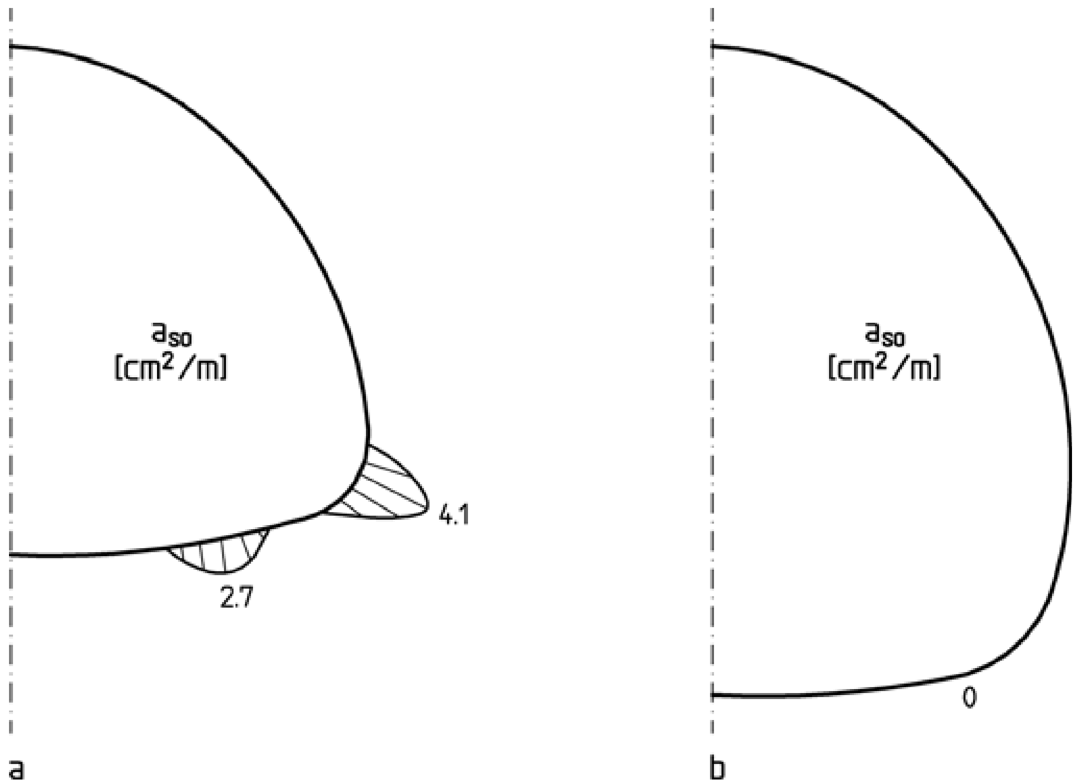


Fig. 7.24: Analysis cross-section 2, statically required outside reinforcement of the shotcrete membrane: a) 6th computation step; b) 8th computation step

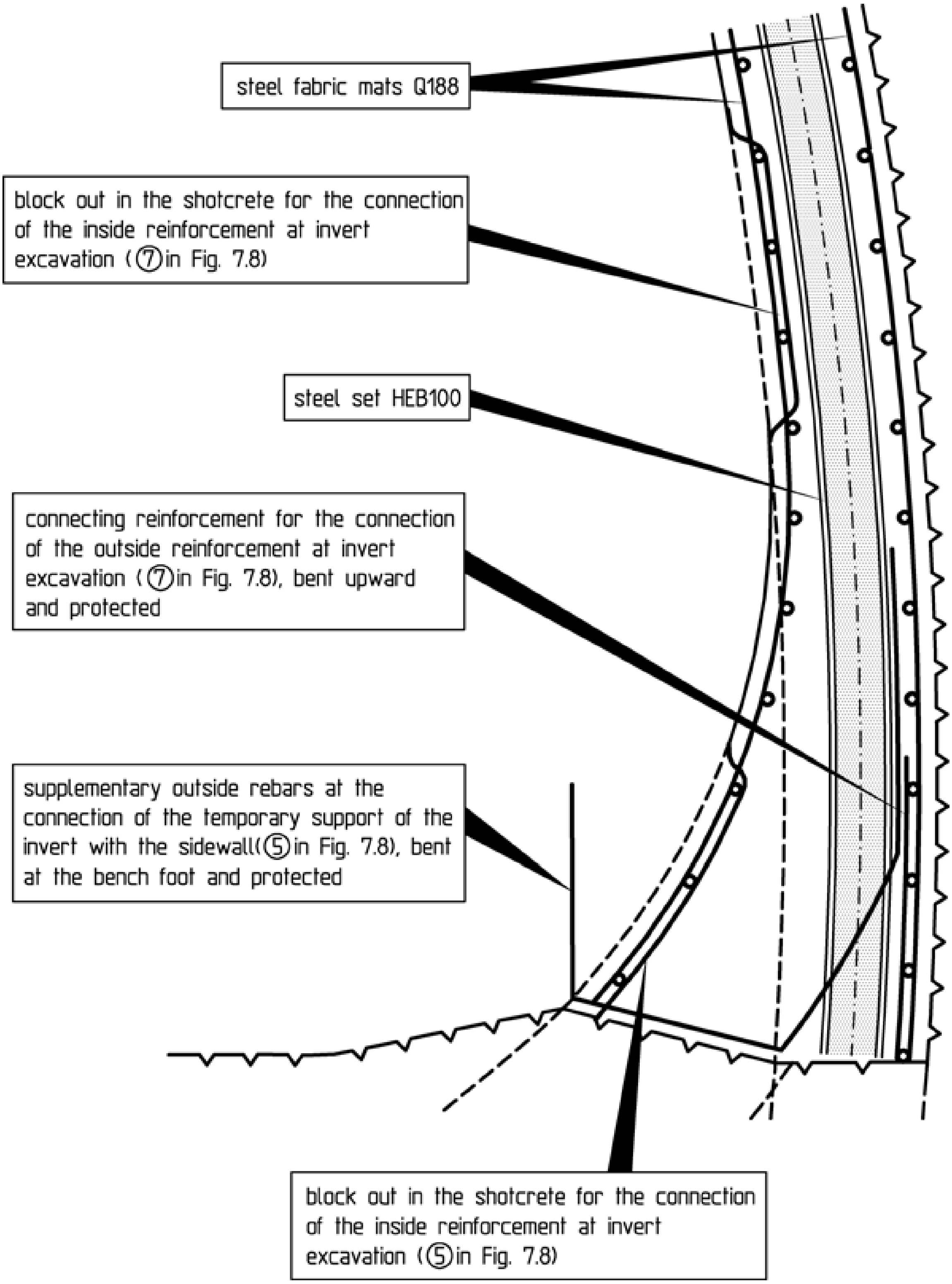


Fig. 7.25: Design of the support in the area of the bench's foot

Fig. 7.23 shows a comparison of the bending moments in the shotcrete lining computed for the 6th and the 8th computation step. Due to the small radius of curvature of R = 1.2 m in the area of the connection of the temporary invert to the sidewalls (see Fig. 7.4), comparatively large bending moments occur at this location in the 6th computation step as previously mentioned (Fig. 7.23a). In addition to the planned steel fabric mats Q188 supplementary outside reinforcement becomes necessary in this area (Fig. 7.24a). In the 8th computation step the radius amounts to R = 1.8 m in the area of the connection of the permanent invert to the sidewalls (see Fig. 7.4). Smaller bending moments are computed for this step (Fig. 7.23 a and b). They can be carried by the shotcrete membrane without any reinforcement for an assumed safety factor of 1.35 (Fig. 7.24b).

Fig. 7.25 shows the design of the support in the area of the bench's foot.

7.1.6 Monitoring

The subsidence due to the heading was measured by leveling using a closely spaced raster of measurement cross-sections. In addition, subsidence and convergency measurements were carried out in the tunnel tubes.

Fig. 7.26 shows the maximum values of the ground surface subsidence measured during the heading between the two construction pits Moltke square and Van-Groote square. The largest subsidence of approx. 6.5 cm occurred close to the construction pit Moltke square in an area where the tunnel tubes were headed by sidewall adit excavation without jet grouting columns. In the other areas the maximum subsidence of the ground surface ranged between approx. 15 mm and approx. 45 mm (Fig. 7.26). The maximum ground surface subsidence of 33 mm computed for analysis cross-section 2 is in good agreement with the measured values (see Fig. 7.22 and 7.26).

7.1.7 Conclusions

With the relocation of the federal highway B 9 in Bonn-Bad Godesberg into a tunnel the whole tunnel cross-section was located in cohesionless gravel sand. Buildings and railway facilities had to be undercrossed with little overburden. Therefore, the subsidence of the ground surface due to the heading had to be limited to small values.

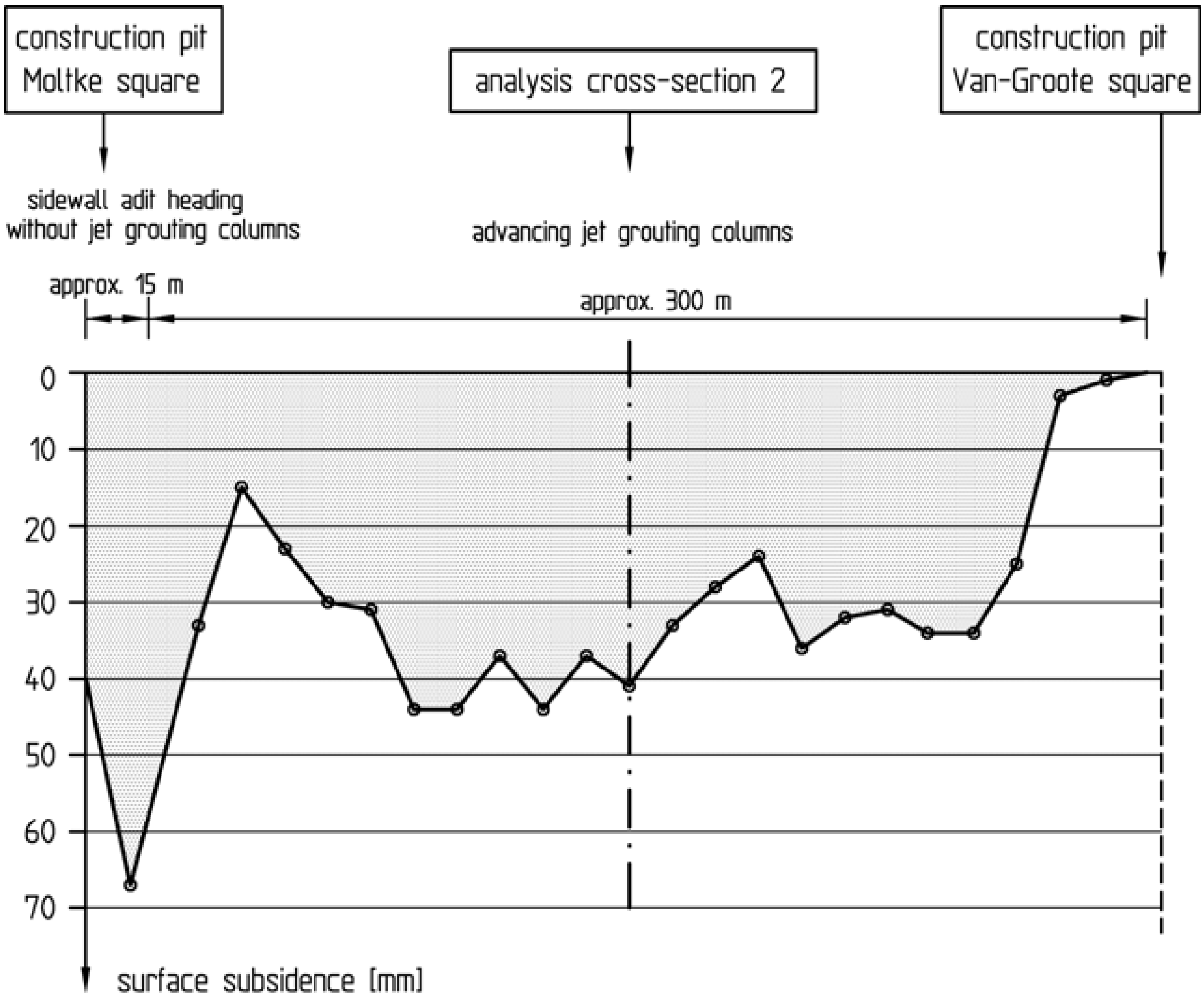


Fig. 7.26: Maximum values of the measured ground surface subsidence, longitudinal section between the two construction pits Moltke square and Van-Groote square

The tunnel was excavated by the NATM under the protection of advancing jet grouting columns. With this measure a part of the overburden load could be transferred in lateral and longitudinal tunnel direction (see Fig. 7.10). Thus the green shotcrete in the tunnel face area and the tunnel face itself was less strongly loaded. Furthermore, with the jet grouting columns, the subsidence was limited, collapses were avoided, and the safety of the tunneling staff was thus ensured as well. The tunnel was excavated with a steeply inclined stepped tunnel face, short round lengths and a fast closing of the invert support (see Fig. 7.8). To guarantee the stability of the tunnel face, an immediate tunnel face support in sections using reinforced shotcrete was necessary.

With these measures, a stable excavation of the tunnel was feasible, and the subsidence of the ground surface could be limited to 2 to 4 cm (see Fig. 7.26). No damage occurred on buildings or railway facilities.

The FE stability analyses carried out for this project represented an essential contribution towards the design, the statics and the design of the excavation and support measures.

7.2 City railway tunnel "Killesberg-Messe" in Stuttgart, Germany

7.2.1 Introduction

Between July 1990 and April 1991 the "Killesberg-Messe" city railway tunnel was constructed in Stuttgart, Germany. A 64 m long section of the tunnel alignment is located immediately adjacent to the State Academy of Art and Design (Academy of Art, Fig. 7.27 and 7.28). In addition, the tunnel was driven through a quarry fill in this area. To keep the subsidence due to tunneling small, the tunnel was headed in this section under the protection of an advance support constructed by jet grouting columns (EN 12716, 2001).

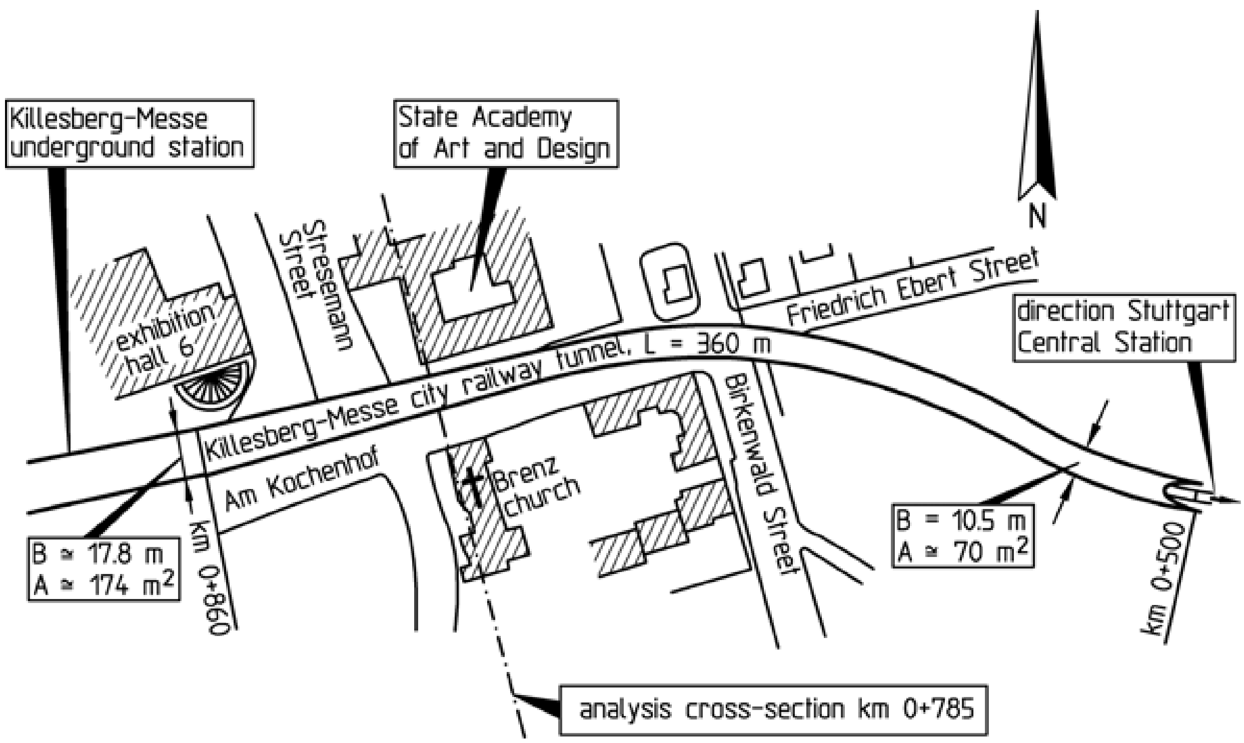


Fig. 7.27: Killesberg-Messe city railway tunnel, site plan

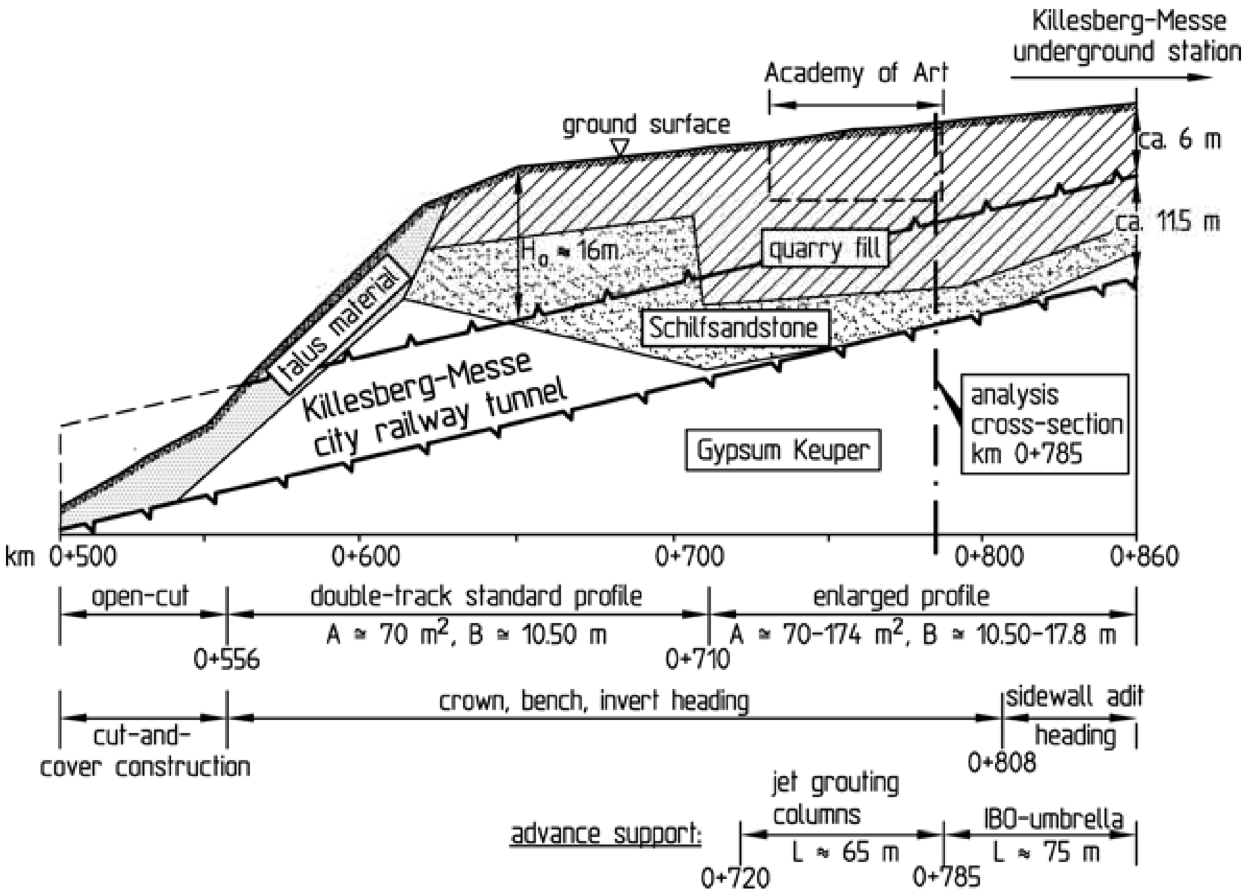


Fig. 7.28: Killesberg-Messe city railway tunnel, longitudinal section with ground profile, excavation and support measures

7.2.2 Structure

With the "Killesberg-Messe" city railway line in Stuttgart, the so-called Trade Fair Line, a fast and convenient rail transit connection was constructed between the central junction Stuttgart Central Station and the Killesberg heights with the hill park. Over a length of approx. 360 m up to the Killesberg-Messe station the city railway line runs in a tunnel (Fig. 7.27). The overburden of the tunnel first rises to approx. 16 m, then decreases and amounts to approx. 6 m at the beginning of the underground Killesberg-Messe station (Fig. 7.28).

From km 0+500 to km 0+710 the double-track standard profile was excavated. With a width of approx. 10.5 m, the excavated cross-section amounts to approx. 70 m² in this section. In the further course up to the Killesberg-Messe station the tunnel widens to a width of approx. 17.8 m. The excavated cross-section at the beginning of the station amounts to approx. 174 m² (Fig. 7.27 to 7.29).

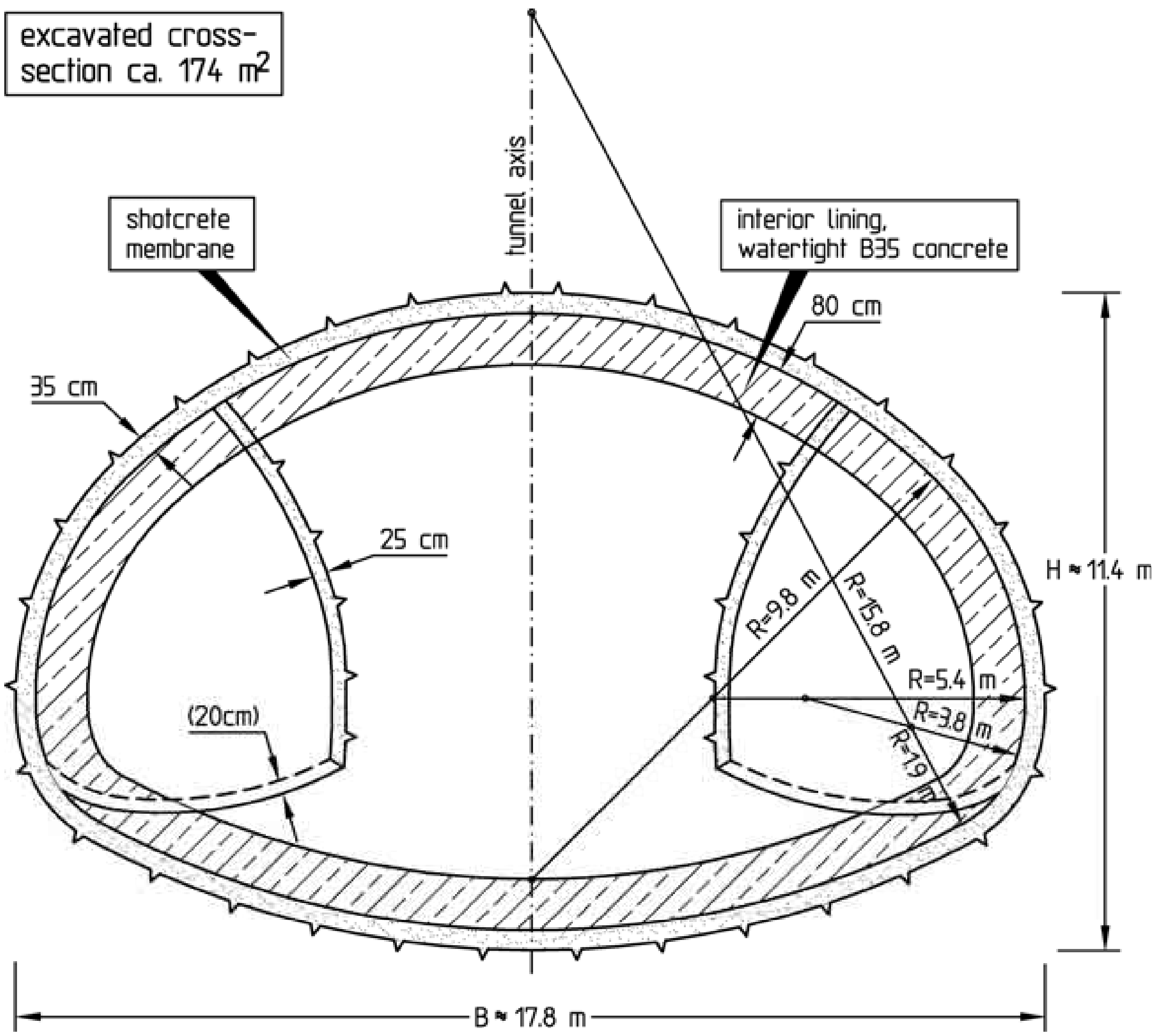


Fig. 7.29: Enlarged cross-section at the beginning of the Killesberg-Messe station (km 0+860)

A mouth-shaped profile was selected for the tunnel cross-section. The geometry of the enlarged cross-section at the beginning of the Killesberg-Messe station (largest cross-section) is shown in Fig. 7.29. The shotcrete membrane has a thickness of t = 35 cm, the inside sidewalls of the two sidewall adits had a 25 cm thick shotcrete membrane. If required the shotcrete membrane of the sidewall adits was planned to be closed at the invert with t = 20 cm. The interior lining was constructed 80 cm thick with watertight concrete of grade B35.

In the roof and sidewall areas of the largest cross-section radii of curvature of R = 9.8 m and R = 5.4 m, respectively, were selected for the shotcrete support. The transitions from the sidewalls to the invert were constructed with the comparatively small radii R = 3.8 m and R = 1.9 m. Because of the water pressure loading of the interior lining the invert was slightly rounded with a radius of curvature of R = 15.8 m (Fig. 7.29).

7.2.3 Ground and groundwater conditions

The plateau of the Killesberg heights is formed by the Schilfsandstone, which was mined in numerous quarries in the past. These quarries were later closed and backfilled 80 to 90 years ago with quarry fill (Fig. 7.30). Below the Schilfsandstone the layers of the Gypsum Keuper are encountered (Fig. 7.28).

At the portal (km 0+500) the city railway tunnel cuts into talus material and the upper layers of the Gypsum Keuper (Estherien layers). Starting at km 0+650 the Schilfsandstone enters into the tunnel profile from the roof. From km 0+700 to the end of the tunnel at km 0+860 the roof and the upper part of the sidewalls are located in the quarry fill. The lower part of the sidewalls and the tunnel invert cut into the Schilfsandstone. The boundary to the Gypsum Keuper is located in this area at the level of the tunnel invert or slightly below (Fig. 7.28).

The ground conditions were mainly derived from the results of core drillings. In addition, test pits were excavated and dynamic probing were carried out. The evaluation of old aerial photographs and seismic measurements served to localize the quarry edges. In front of the Academy of Art a test shaft 4 m in diameter was sunk. Here samples for soil mechanical laboratory tests were taken and plate loading tests were carried out to determine the deformability.

The quarry fill is very heterogeneously composed. It consists of sandstone blocks of different sizes with edges up to 80 cm long. Partly the pieces of rock lie in a sandy silt matrix, partly they constitute a pure sandstone deposit without any filling material (Fig. 7.30). According to the geological report the porosity ranges between 3 and 35 %. By means of density measurements on large-scale samples a content of 10 % of large voids or cavities, respectively, was estimated. The grain size distributions determined for five samples are shown in Fig. 7.31. According to this, the fill consists of smoothly graded gravel/sand (GW), gravel/silt (GU) and sand/silt mixtures (SU).

Fig. 7.30: Quarry fill

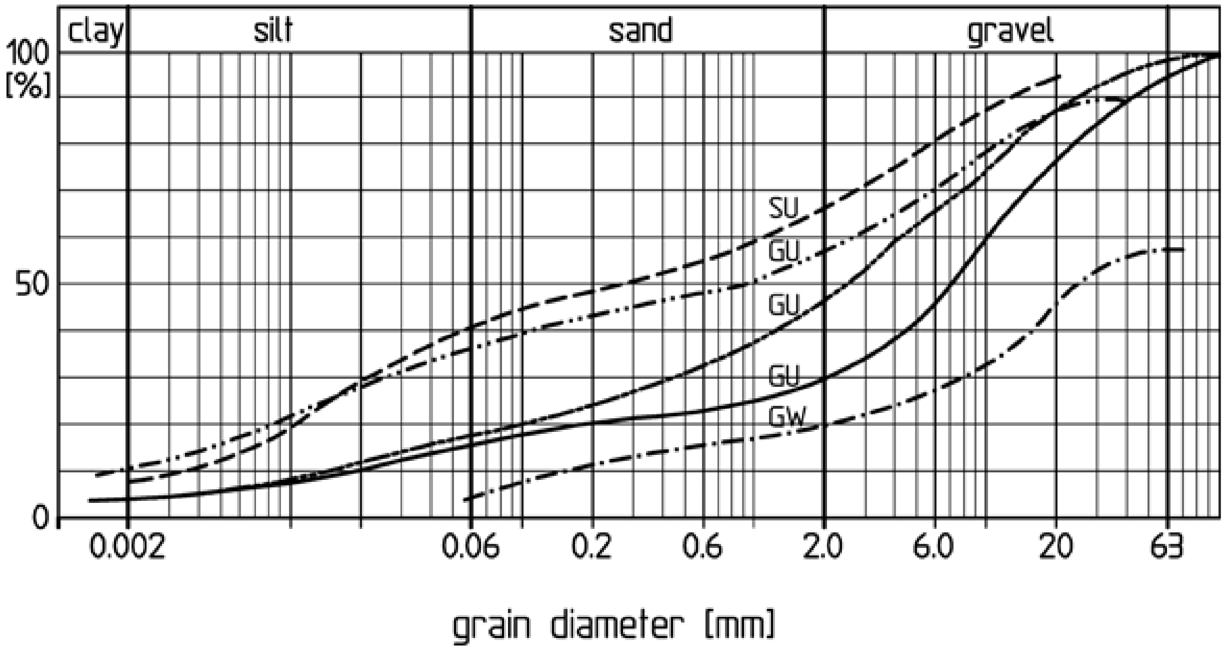


Fig. 7.31: Grain size distribution of samples from the quarry fill

The undisturbed Schilfsandstone consists mostly of hard sandstones with clay flasers and a marked horizontal bedding. The banks are between 10 and 50 cm thick. The sandstone is mostly vertically jointed with a medium to wide joint spacing.

The Gypsum Keuper layers in the tunnel invert area belong to the White and Grey Estherien layers. Anhydrite or gypsum deposits were not encountered in the course of the ground exploration down to approx. one tunnel diameter below the invert. Swelling phenomena due to the tunneling were therefore not to be expected in the ground. No leaching cavities were drilled into either in the area specified above.

The soil and rock mechanical parameters of the different ground layers determined or estimated from the exploration results are listed in Table 7.2.

layer	**Young's modulus E [MN/m²]**	**Pois-son's - ratio ν**	**unit weight γ [kN/m³]**	**intact rock**		**discontinuity sets**			
						bedding		**jointing**	
				φ [°]	**c [kN/m²]**	**φ_B [°]**	**c_B [kN/m²]**	**φ_J [°]**	**c_J [kN/m²]**
talus material	7	0,40	21,5	25	25	-	-	-	-
quarry fill	6	0,25	19	30	0	-	-	-	-
Schilf-sandstone	2000	0,20	25	40	3000	40	30	40	0
Gypsum Keuper	100	0,33	23	30	50	30	20	30	20

Table 7.2: Soil and rock mechanical parameters

Five core drillings were equipped as observation wells. The measured water levels show that the ground water table lies approximately on the level of the quarry base and thus above the tunnel's invert. The boundary between the permeable Schilfsandstone and the Gypsum Keuper with its low permeability forms a preferred groundwater horizon. Above the quarry edge seepage water has accumulated. An amount of 1 l/s of water was pumped out of the test shaft mentioned above over a period of several days.

7.2.4 Excavation and support

Between km 0+556 and km 0+808 the city railway tunnel was driven as a crown heading with trailing bench and invert excavation (Fig. 7.32a).

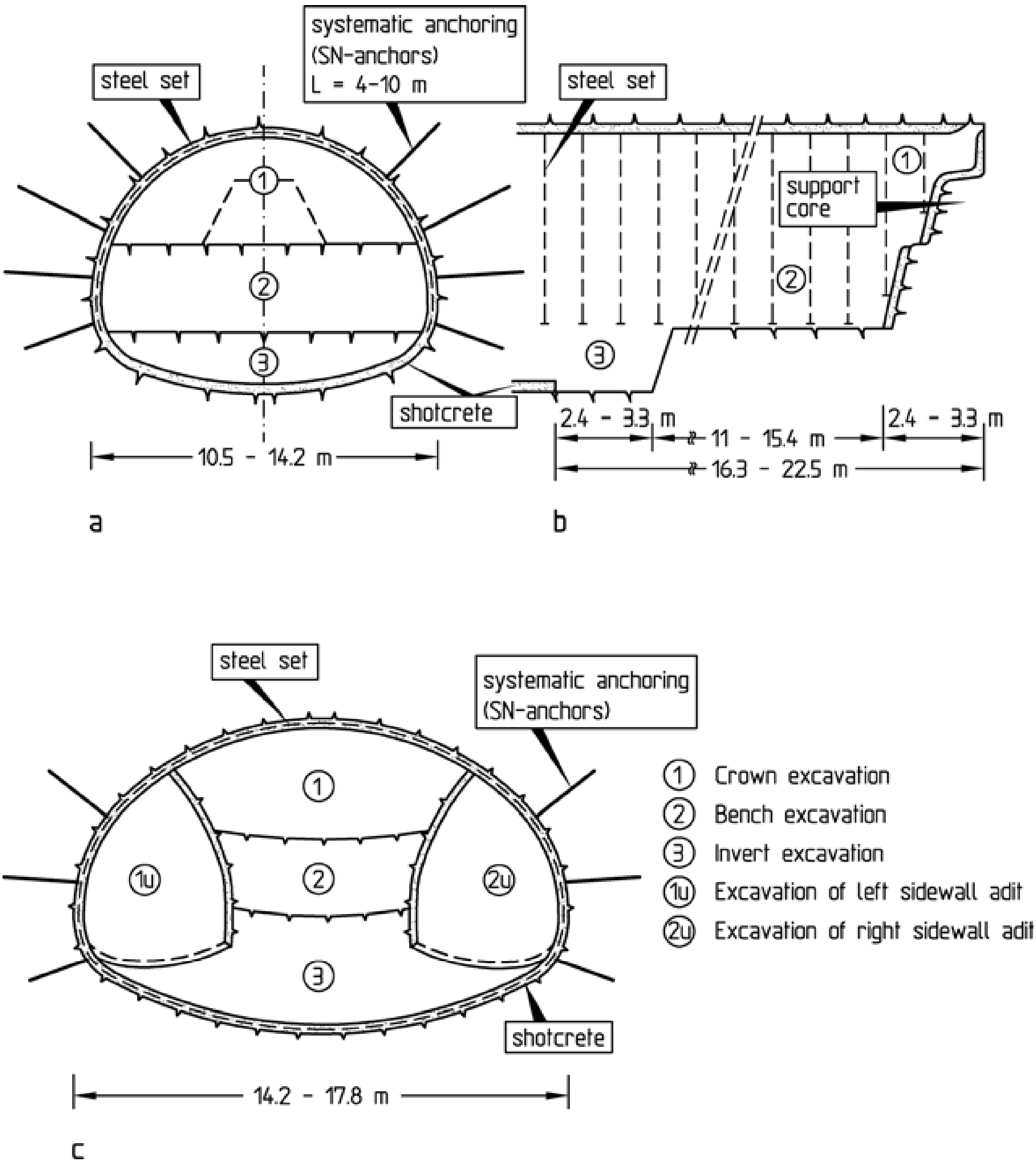


Fig. 7.32: Excavation and support: a) Crown heading, cross-section; b) crown heading, longitudinal section; c) sidewall adit heading, cross-section

Crown and bench were excavated with a stepped tunnel face and a support core in the crown area and with round lengths of 2.4 to 3.3 m. To ensure the stability of the temporary tunnel face, it was supported in sections with reinforced shotcrete. The invert was excavated 11 to 15 m behind the bench. The distance between the crown face and the closing of the support at the invert amounted to between 16 and 22 m, depending on the ground conditions encountered (Fig. 7.32b).

Because of the large cross-section at the end of the widening segment the tunnel was driven in the further course up to Killesberg-Messe station (km 0+808 to 0+860) by sidewall adit excavation (Fig. 7.28, 7.29 and 7.32c). The excavation profile was supported with reinforced shotcrete, steel sets and a systematic anchoring using SN-anchors, the length and spacing of which were determined as required.

The foundations of the Academy of Art are located at close distance from the tunnel (Fig. 7.33a). In this section of the alignment the tunnel roof and the upper sidewalls are located in the quarry fill. Because of the high deformability and low strength of the fill, the close distance of the foundations of the main building of the Academy of Art to the tunnel and the large tunnel cross-section, it was feared that tunneling-induced subsidence would lead to damages to the main building of the Academy of Art.

To limit the subsidence, the tunnel was excavated in the area of the quarry fill under the protection of an advance support by jet grouting columns. To this end, from km 0+710 to km 0+785 seven jet grouting vaults were constructed in sections by the single phase method described in Chapter 7.1 (Fig. 7.33b). The jet grouting columns were constructed with a length of 11.0 to 12.75 m, measured from the tunnel face. At the end of each excavation section the columns extended 3 m beyond the tunnel face. With 1 m of non-grouted borehole length, this results in an overlap of the columns of 2 m (Fig. 7.34b).

The drillings were directed with a 6 degree outward slant with respect to the tunnel axis. Thus, at the end of the enlarged excavation section there was enough clearance to construct the columns for the following section (Fig. 7.34b).

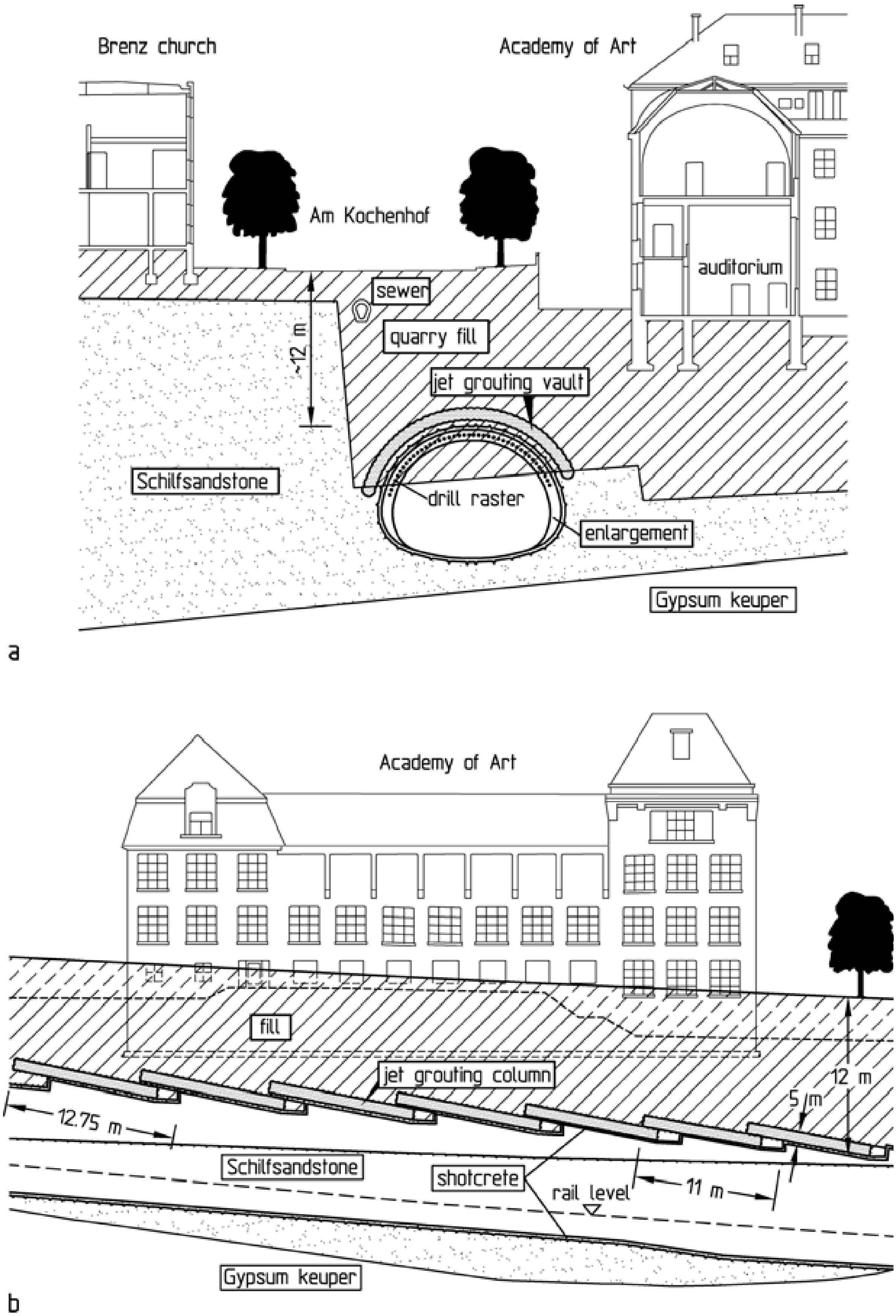
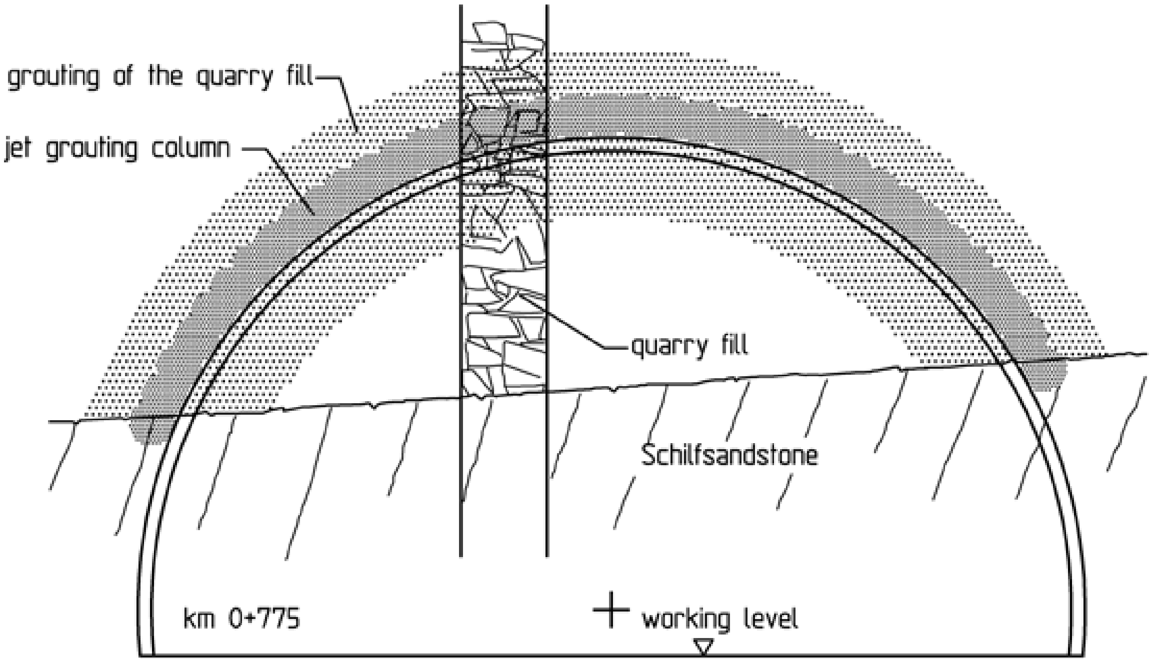
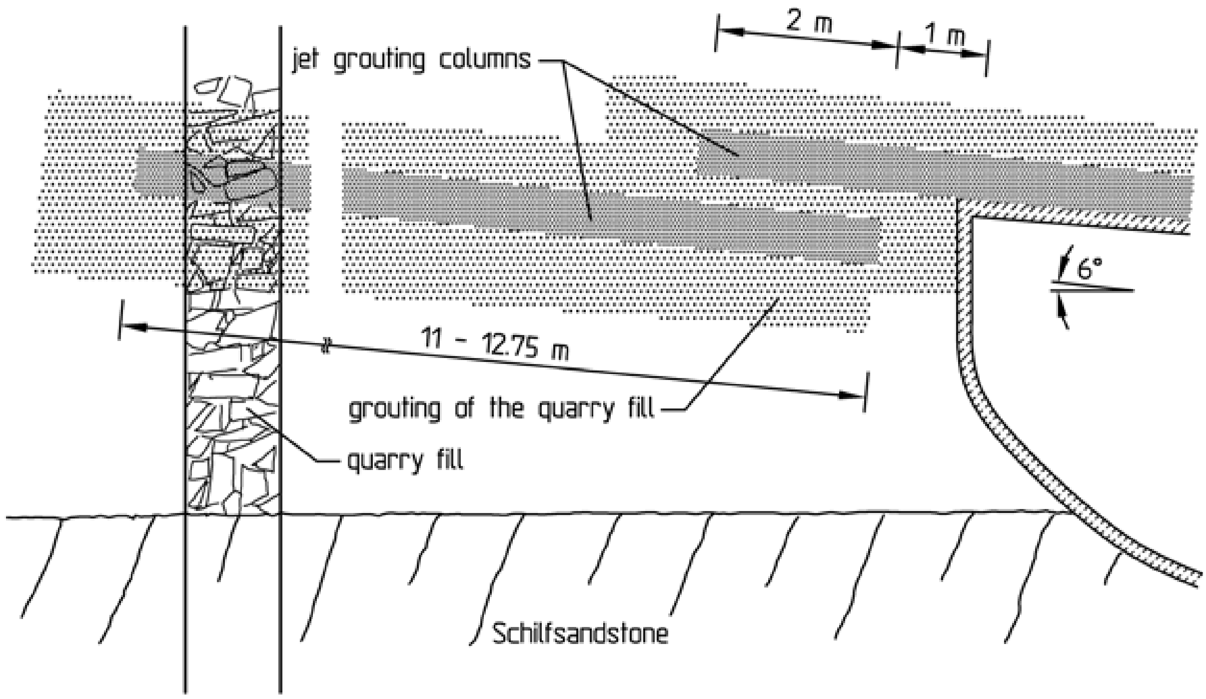


Fig. 7.33: Advance support by jet grouting in the area of the Academy of Art: a) Cross-section; b) longitudinal section



a



b

Fig. 7.34: Advancing grouting of the quarry fill and jet grouting columns: a) Cross-section; b) longitudinal section

The nature of the ground in the area of the jet grouting columns to be constructed required particular measures. To prevent the cement based suspension from seeping into large voids or cavities in the quarry fill, and in order to ensure that the columns could be constructed according to plan with respect to diameter, continuity, strength and position, the quarry fill was grouted in advance to fill existing cavities (Fig. 7.34).

To this end drillings 114 mm in diameter with air flushing were sunk, equipped with PVC sleeve pipes and grouted with packers in steps of 1 m from the bottom up with a cement-bentonite grout (250 kg cement and 40 kg bentonite for 1000 l) which is stable with respect to sedimentation (Table 7.3). The purpose of this was to achieve a void-free matrix filling potential cavities in the quarry fill which would not impede the construction of the jet grouting columns and the tunnel excavation.

		Per section	7 sections 64 m
Boreholes	number	14 - 25	152
Drilling/sleeve pipe	m	283	1950
Grouted volume	m^3	52	361
Cement	t	14	99
Covered volume of soil	m^3	325	2280
Achieved grouting volume	%	16	16

Table 7.3: Amount of grouting of the quarry fill

Water/cement (PZ 35 F) ratio of grout	-	0,8
Pump pressure	bar	400
Pump capacity	l/min	95
Retracting rate	m/min	0,41
Rotational speed	min^{-1}	20
Grout flow rate	l/min	233
Cement quantity	kg/m	205

Table 7.4: Operational parameters of the jet grouting

Only a few hours after the completion of the grouting of the quarry fill the construction of the jet grouting columns started. With a planned minimum diameter of the columns of 0.5 m, the drillings were placed at a distance of 0.35 m to the tunnel contour. To confine temporary decrease in strength in the soil locally, the columns were constructed first with a spacing of 1.4 m, then halfway between two columns each, and finally in the gaps between two adjacent columns. This way a vault supported by the Schilfsandstone was constructed made up of intersecting or, in the area more widely fanned out, touching columns. The operational parameters of the jet grouting and the amount of work done are given in Tables 7.4 and 7.5 (Beiche et al., 1991).

		Per segment	7 vaults 64 m
Columns	number	28 - 44	268
Drilling	m	308 - 561	3280
Grout	m^3	103	723
Cement	kg	91000	637000

Table 7.5: Amount of jet grouting work performed

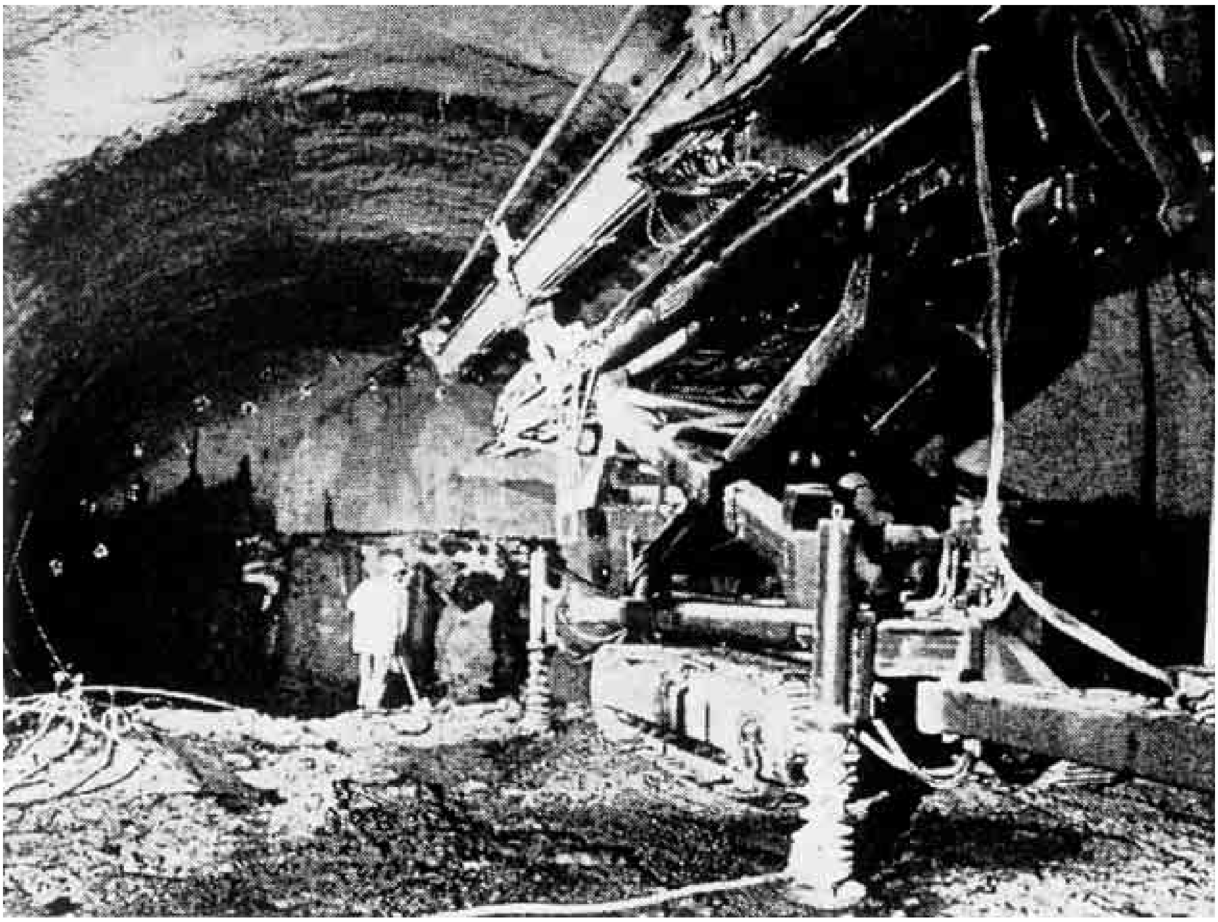


Fig. 7.35: Special construction equipment SR 510

Fig. 7.14 shows the unconfined compressive strength measured on backflow samples and drill cores as a function of the sample age (Wittke et al., 2000).

The heading generally recommenced 12 to 15 hours after the completion of a jet grouting vault. At this time the columns were between 12 hours and 2 days old.

An essential technical requirement for advance support by jet grouting is the ability to construct the single structural elements – the soil-concrete columns – in a self-contained continuous operation. Suitable technical equipment must therefore above all possess a feeding length at least equal to the length of a jet grouting column.

For economic reasons it is important that the drill mount can be quickly positioned as desired over the entire excavation profile.

Fig. 7.35 shows the construction equipment SR 510 used for the Killesberg-Messe city railway tunnel in operation. The boreholes for the grouting of the quarry fill were drilled with the same equipment (Beiche et al., 1991).

Jet grouting vault	1	2	3	4	5	6	7
Borehole length	11,00 m	11,00	12,00	12,75	12,75	12,75	12,75
Heading segment length	8,00 m	8,00	9,00	9,75	9,75	9,75	9,75
Days	10	20	30	40	50	60	70
Systematic advance filling Number of boreholes	14 (s = 0,8 m)	20	22	23	24	24	25
Construction time							
Number of columns	28 (s = 0,33 m)	34	38	40	42	42	44
Construction time							
Heading							

Table 7.6: Construction of the jet grouting vaults

The grouting of the quarry fill and the construction of the jet grouting columns had to be worked into the heading scheme. Since the columns could not be constructed in parallel with the heading, the established heading operation in ten-days periods was generally changed over for the area with jet grouting columns to a continuous operation on demand for reasons of time (Beiche et al., 1991).

The length of the heading section protected by jet grouting columns was 64 m in total. This section was constructed in 68 days. 24 days thereof account for the advance grouting of the quarry fill, 18.5 days for the construction of the columns and 25.5 days for the heading (Table 7.6).

7.2.5 Stability analyses

For the design of the shotcrete support and the interior lining two-dimensional FE-analyses were carried out using the program system FEST03 (Wittke, 2000). Eight analysis cross-sections were investigated in total, differing with respect to

- the geometry of the tunnel cross-section,
- the ground conditions and overburden height,
- the construction stages and/or
- the support installations.

Two of these analysis cross-sections are located in the area of the Academy of Art. In the following a stability analysis for the analysis cross-section km 0+785 (see Fig. 7.27 and 7.28) is exemplarily presented (see Beiche et al., 1991).

Fig. 7.36 shows the computation section, the FE-mesh, the boundary conditions, the ground profile and the parameters for this analysis cross-section.

The upper part of the tunnel cross-section lies in the quarry fill. The edge of the former quarry is located approx. 2 m away from the tunnel sidewall opposite to the Academy of Art. The lower part of the tunnel's cross-section is situated in the Schilfsandstone. The boundary to the Gypsum Keuper is encountered approx. 1.2 m underneath the tunnel's invert (Fig. 7.36).

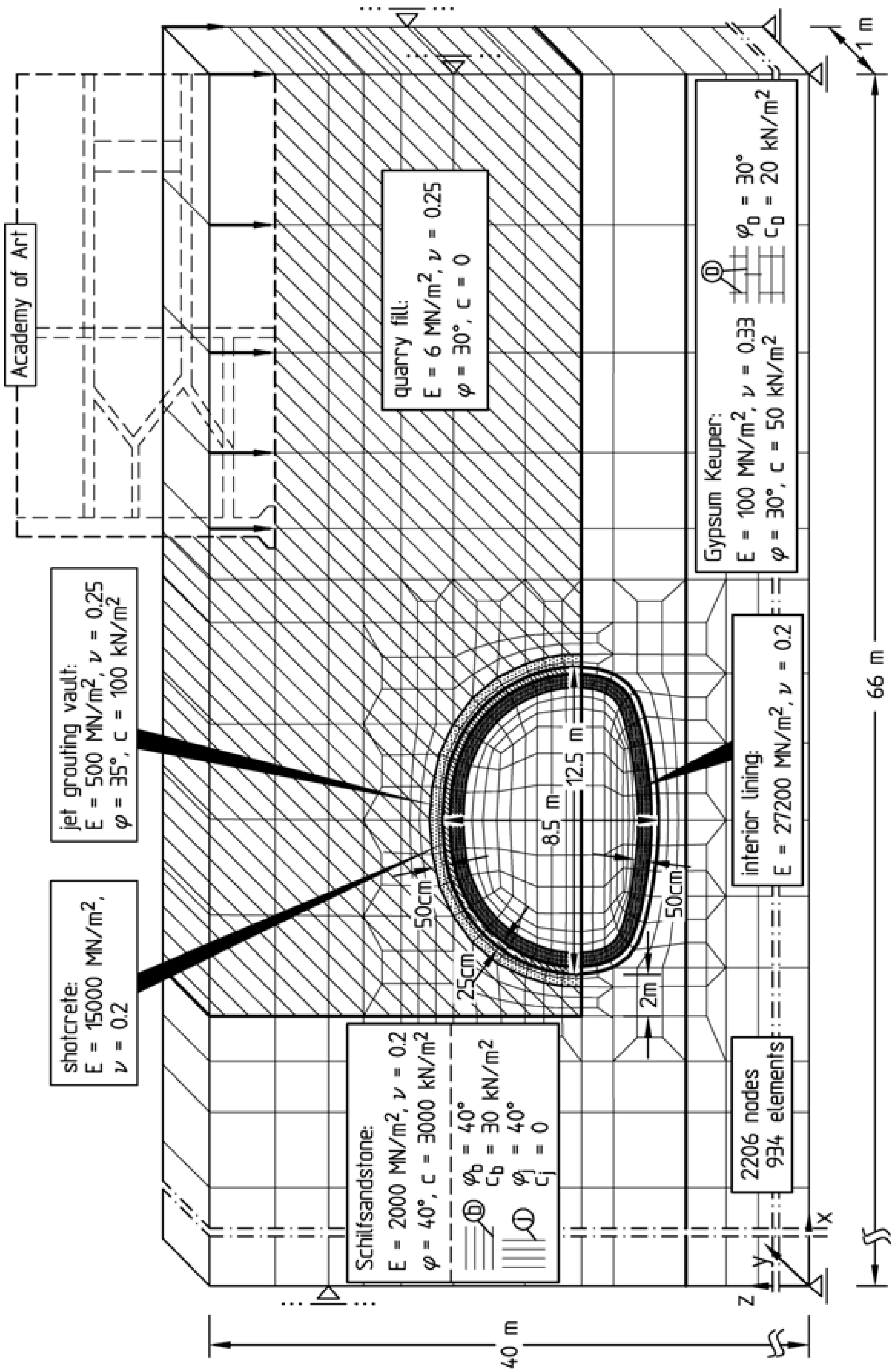


Fig. 7.36: Analysis cross-section km 0+785, FE-mesh, boundary conditions, ground profile and parameters

The computation section consists of a 1 m thick, 40 m high and 66 m wide slice including the tunnel's cross-section as well as the surrounding ground. It is divided into 934 isoparametric elements with a total of 2206 nodes. Vertically sliding supports are selected as boundary conditions for the nodes on the vertical lateral boundary planes (x = 0 and x = 66 m), while horizontally sliding supports are chosen for the nodes on the lower boundary plane (z = 0) (Fig. 7.36). All nodes are fixed in y-direction.

The loading resulting from the Academy of Art building to the right of the tunnel is simulated by a surface load. The surface load is applied to the corresponding nodes of the FE-mesh in the form of point loads (Fig. 7.36).

The shotcrete membrane (t = 25 cm) is simulated by one row of elements, the interior lining (t = 50 cm) by three and the jet grouting vault (t = 50 cm) by two element rows. The effect of a separating non-woven material preventing the transfer of tensile and shear stresses between shotcrete support and interior lining is simulated by a thin row of elements with no stiffness assigned and by radially arranged truss elements of high stiffness (pendulum rods), which only allow the transfer of normal compressive stresses (Fig. 7.37).

The soil and rock mechanical parameters of the different layers as well as the parameters assigned to the shotcrete, the reinforced concrete of the interior lining and the jet grouting vault are given in Fig. 7.36.

It was not possible to carry out advance tests to determine the mechanical parameters of the quarry fill improved by jet grouting. These parameters therefore had to be estimated for the stability analyses.

The deformability and the strength of the soil improved by jet grouting develop with time (see Fig. 7.14). Since tunneling recommenced already approx. 15 hours after the completion of each jet grouting vault, Young's modulus and the strength of the improved soil are still low when the cross-section is excavated. Young's modulus and the shear parameters of the jet grouting vault were therefore assumed comparatively low and thus conservative in the analyses with values of $E = 500\ MN/m^2$, $\varphi = 35°$ and $c = 100\ kN/m^2$. These shear parameters correspond to an unconfined compressive strength of σ_u = 0.5 MN/m^2. According to Fig. 7.14 this value is reached after one day already.

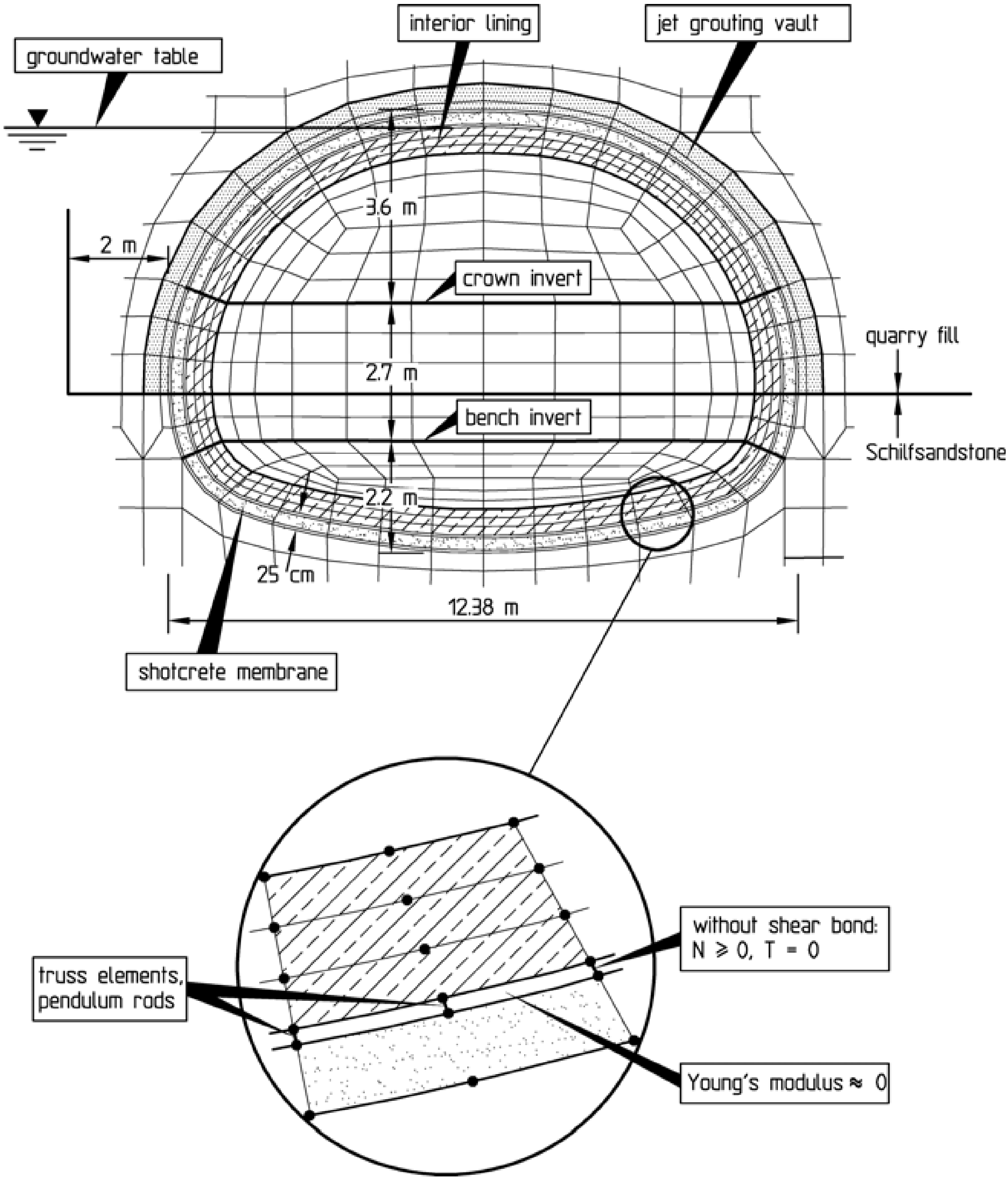


Fig. 7.37: Analysis cross-section km 0+785, FE-mesh, detail with shotcrete membrane and interior lining

A three-dimensional analysis was not considered necessary in this case, since a three-dimensional arching cannot develop in the quarry fill due to the low strength.

Although the jet grouting vault constructed in advance reaches down to the quarry base and is thus supported by the Schilfsandstone, it cannot transfer any significant loading during crown excavation. The reason is that the time span between the construction of the jet grouting vault and the crown excavation is very short. The vault has therefore only reached a small fraction of its final strength at this stage. It is only after the bench excavation that the shotcrete support reaches down to the Schilfsandstone and a setting process has taken place in the vault. Both means of support are then ready to carry loads. It was therefore determined that the crown heading must not be ahead of the bench by more than twice of the lattice girder spacing (2.4 to 3.3 m) (see Fig. 7.32b).

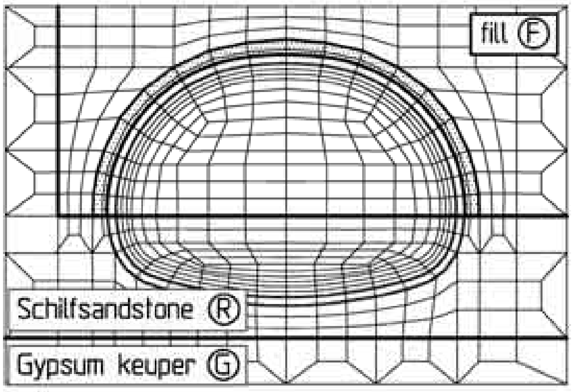
Fig. 7.38 shows the seven computation steps simulating the in-situ state and the construction stages, which are the installation of the jet grouting vault, crown, bench and invert excavation, installation of the interior lining and rise of the groundwater table to roof level.

Fig. 7.39 to 7.41 show the analysis results for the 4th computation step (crown and bench excavation).

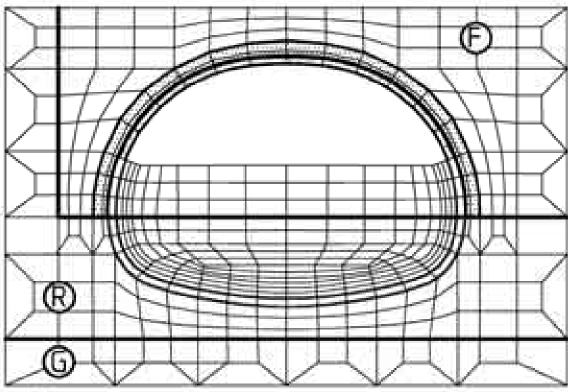
Above the right half of the tunnel practically no arching develops in the fill due to its low strength and large deformability. This can be recognized in Fig. 7.39a from the fact that the major principal normal stresses are almost vertically oriented.

Above the left half of the tunnel in the area of the vertical quarry wall the major principal normal stress deviates from the vertical. The vertical stresses are lower in the fill and higher in the Schilfsandstone than the overburden weight. This result is due to the different Young's moduli of the Schilfsandstone and the fill. The quarry fill is thus "hung" on the edge already in the stage before the tunnel excavation and the construction of the jet grouting vault. This effect should be less pronounced in reality than in the analysis, since the fill was placed in layers rather than in one step as simulated in the analysis.

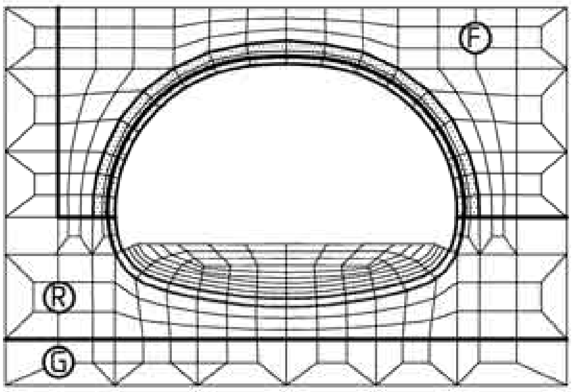
1st computation step: state of stress and deformation resulting from the dead weight of the ground (in-situ state)



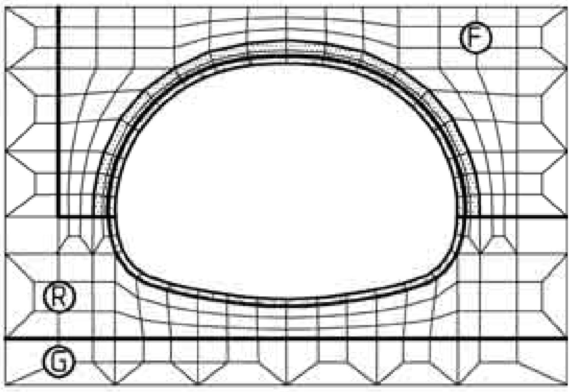
2nd computation step: construction of the jet grouting vault



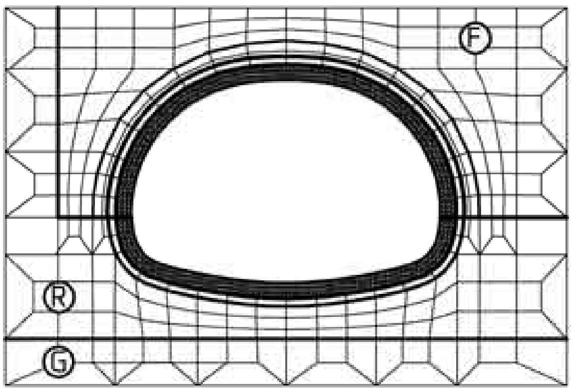
3rd computation step: crown excavation and shotcrete support



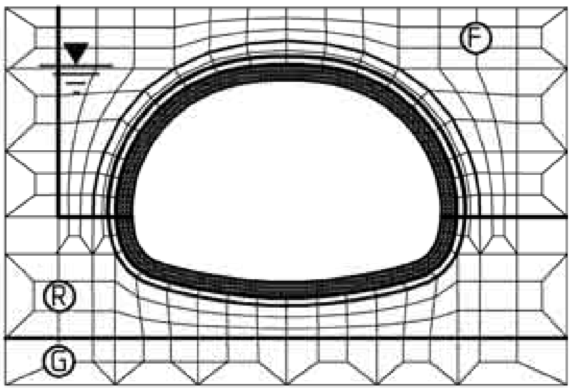
4th computation step: bench excavation and shotcrete support



5th computation step: invert excavation and shotcrete support



6th computation step: installation of the interior lining, shotcrete support and jet grouting vault become ineffective (decomposed)



7th computation step: simulation of rise of groundwater table to tunnel roof level

Fig. 7.38: Analysis cross-section km 0+785, computation steps

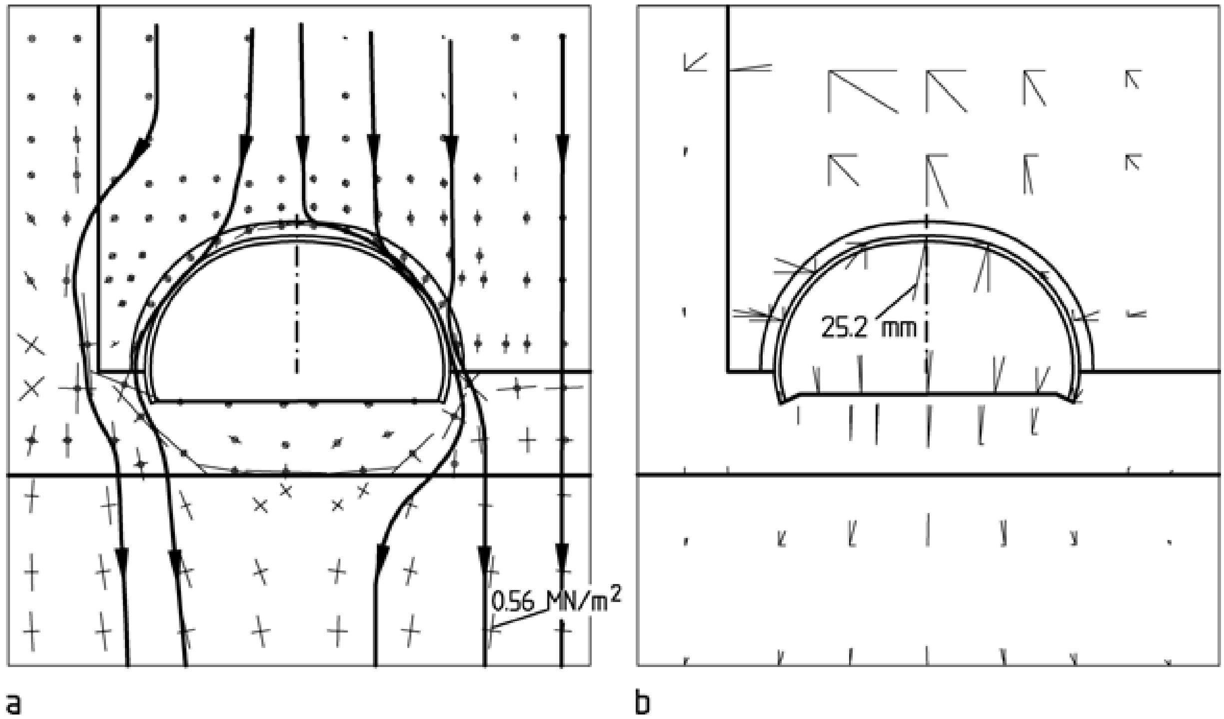


Fig. 7.39: Analysis cross-section km 0+785: a) Principal normal stresses, 4th computation step; b) displacements, 4th – 1st computation step

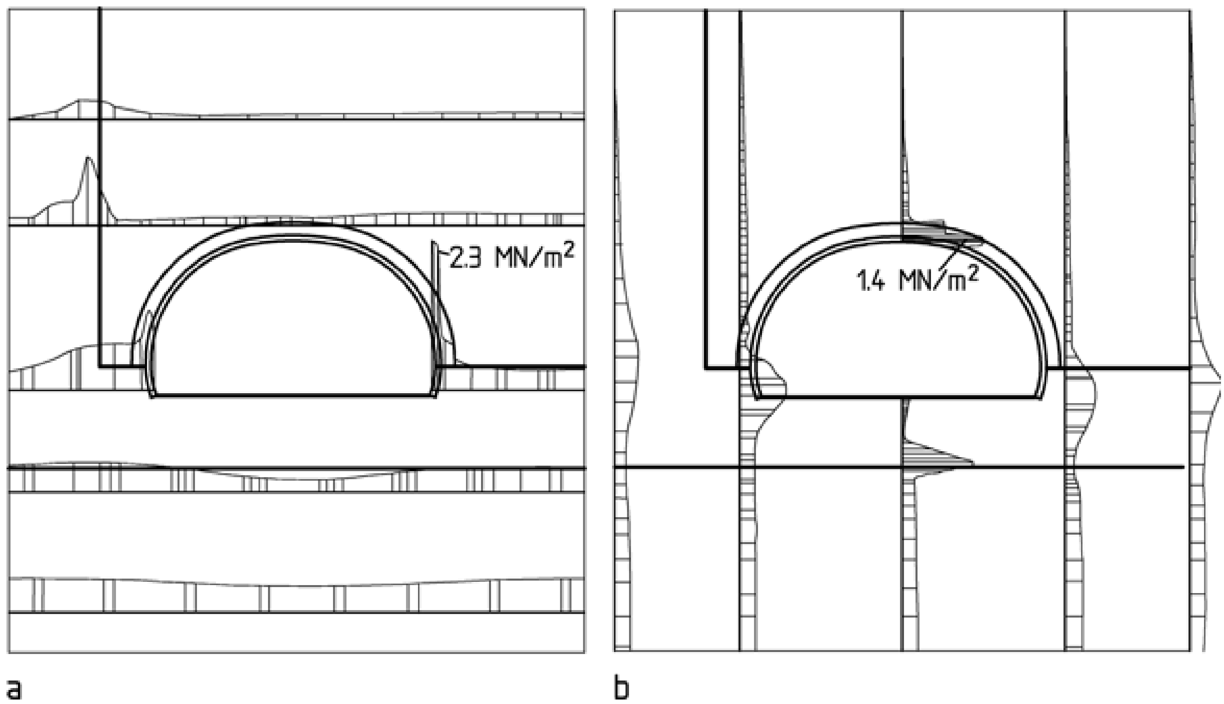


Fig. 7.40: Analysis cross-section km 0+785, 4th computation step: a) Vertical stresses in horizontal sections; b) horizontal stresses in vertical sections

The computed roof subsidence amounts to approx. 2.5 cm. Heave occurs at the bench base (Fig. 7.39b).

An arch develops in the jet grouting vault above the crown. Due to its bond with the shotcrete membrane the latter is strongly loaded as well and stress concentrations result in the area of the bench base. To illustrate the load transfer described above, the horizontal and vertical stresses are shown in sections in Fig. 7.40. Stress concentrations in vertical as well as in horizontal direction are apparent at the base of the bench and of the jet grouting vault. The loading of the shotcrete membrane exceeds the one of the jet grouting vault. This is due to Young's modulus of the shotcrete being markedly higher than the one of the jet grouting columns (see Fig. 7.36).

Fig. 7.41 shows the computed bending moments and normal thrust in the shotcrete membrane. Large bending moments together with a comparatively small normal compressive thrust occur in the roof and the lower sidewall areas.

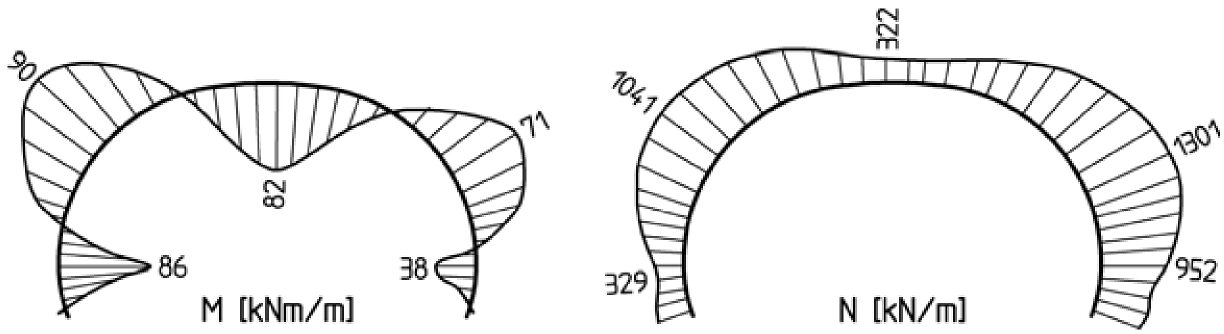


Fig. 7.41: Analysis cross-section km 0+785, bending moments and normal thrust in the shotcrete membrane, 4th computation step

These results change only slightly with the invert excavation and the immediate closing of the support (5th computation step).

The design of the interior lining was based on the following load cases:

- Dead weight of the interior lining,
- dead weight of the interior lining and rock mass pressure,

- dead weight of the interior lining, water pressure (groundwater table at roof level) and rock mass pressure,

- dead weight of the interior lining and water pressure (groundwater table at roof level).

In the load cases with consideration of the rock mass pressure (Fig. 7.38, 6th and 7th computation step) it is assumed that the jet grouting columns and the shotcrete membrane are decomposed and are thus not effective any more. The rock mass pressure generally leads to a great normal compressive thrust in the interior lining with favorable effects on the dimensioning for bending and normal thrust.

In the load cases without rock mass pressure it is assumed that the support effect of the jet grouting vault and the shotcrete membrane remains intact. The surrounding rock mass and the shotcrete membrane are assumed weightless. A bedding of the interior lining is given, however, since for the shotcrete as well as for the bedrock Young's moduli listed in Fig. 7.36 are effective. The load case dead weight and water pressure is generally decisive for the bending reinforcement of the interior lining at the invert and the sidewalls.

The statically required cross sectional areas of reinforcement for the interior lining range between 0 and 10.5 cm^2/m.

7.2.6 Monitoring

To supplement and verify the results of the stability analyses, the tunnel stability and the ground surface subsidence were monitored by comprehensive measurements during construction.

One extensometer measuring cross-section each was positioned in front of and behind the Academy of Art building. Surface measuring points every 8 m on the tunnel alignment, bolts on the outside and inside of the buildings and elevation measuring points on the ceilings facing the tunnel completed the measuring points on and above ground surface. In addition, convergency measuring cross-sections were installed every 10 m in the tunnel, with roof bolts in the middle between them.

Fig. 7.42 shows the subsidence of a selected surface measuring point in the area of the 4th jet grouting vault as a function of the heading location. It can be seen that the total subsidence of approx. 30 mm can be divided into three parts. Approx. one third of the subsidence is preceding subsidence caused by the approaching heading. Another third occurs during the construction of the jet grouting vault while the heading is stopped. At this stage, the tunnel face is located 3 m away from the measuring point. The last third of the subsidence occurs as a trailing impact after the undercrossing of the measuring point (Beiche et al., 1991).

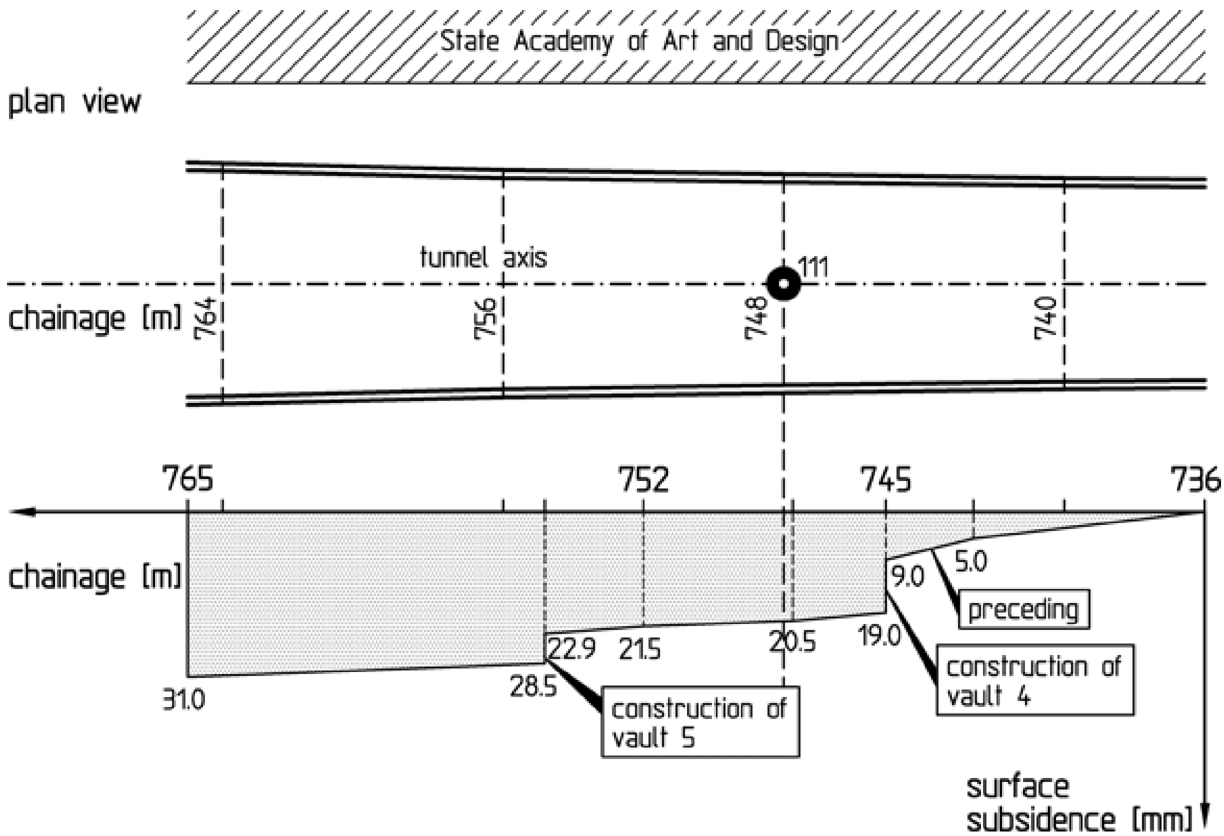


Fig. 7.42: Subsidence of surface measuring point 111 vs. heading location

In Fig. 7.43 measured and computed subsidences in the area of the Academy of Art are compared. In the FE-analysis subsidences in the order of up to 5 mm were computed for the area of the Academy of Art. An expert's report on the Academy's actual state and sensitivity to settlements lead to the result that subsidences of this magnitude would not cause any damages. The computed subsidence was confirmed by the monitoring results. A subsidence trough from the front to the back side of the building was hardly recognizable. The subsidences measured in the tunnel and on the ground surface above the tunnel could be reproduced by the FE-analysis as well (Fig. 7.43, Beiche et al., 1991).

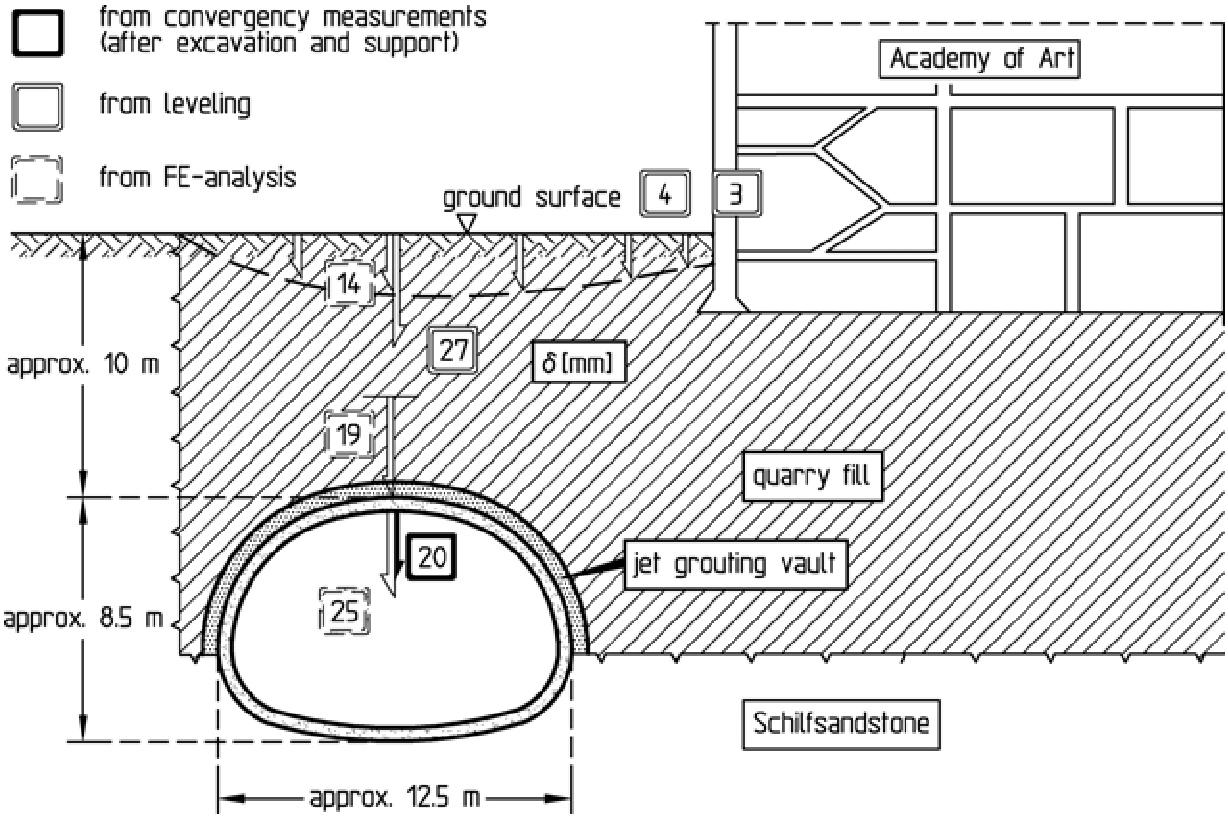


Fig. 7.43: Comparison of measured and computed subsidence in the area of the Academy of Art

7.2.7 Conclusions

The Killesberg-Messe city railway tunnel crosses partly through a quarry fill. In this area the tunnel alignment is located immediately adjacent to the Academy of Art, which is founded on the quarry fill and sensitive to settlements. To limit the heading-induced subsidence, the tunnel was driven in this area under the protection of an advance support, constructed by the jet grouting method. To do this it was necessary to fill the large voids and cavities existing in the quarry fill in advance by cement grouting. This way during jet grouting the cement based suspension was prevented from seeping into the cavities.

The tunnel was driven by crown heading with trailing bench and invert excavation. It was constructed with a steep tunnel face, short round lengths and a fast invert support closing (see Fig.

7.32b). To guarantee its stability, the temporary tunnel face was supported in sections using reinforced shotcrete.

With these measures it was possible to limit the tunneling-induced subsidence at the Academy of Art building to a maximum of 4 mm (see Fig. 7.43).

The FE-analyses carried out have been an important tool for the design, the statics and the specification of the excavation and support measures. The comparison of the FE-analysis results with the measured displacements in the ground showed good agreement (see Fig. 7.43).