Tunnel Design for the Vienna Underground Construction Lot U1/9

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1. Summary

The construction lot U1/9 with "Altes Landgut" station is part of the extension of underground line U1 to the South from Reumannplatz station to Oberlaa station. The U1 will grow by 4.6 kilometres and five stations in forthcoming years. By 2017, it will be the longest underground line in Vienna (19.2 km). The Construction lot U1/9 consists of two app. 25 m and 30 m deep cut & cover shafts, two mined app. 120 m long platform tunnels, four mined single track running tunnels, several cross-passages and one emergency shaft. The tunnels are located below residential buildings, main roads and the very critical "Laaerberg Tunnel" of the A23 freeway. Main focus of the design of the sequential excavation method (NATM) for the tunnels was the minimisation of settlements and achieving high safety level. Prior tunnelling the buildings at surface and the Laaerberg Tunnel were improved partly regarding their structural capacity. The tunnels run mainly in the Miocene silt/clay layers which have a relatively homogeneous structure. They basically comprise of slightly sandy, grey, dark grey to grey-blue silts/clays, turning semi-solid to solid with increasing depth. A thick mixed layer of fine sand or sandy silt layers, 5 m thick on average, runs through the construction lot from approx. 20 m below ground level. It was necessary to pre-drain this layer by filter wells from surface. Extensive monitoring was foreseen to control the performance of the excavation works. The monitoring devices comprise bolt gauges for 3-D trigonometric shotcrete shell displacement measurements, surface settlement points, liquid levelling system on A23 tunnel, horizontal automatic inclinometers above the tunnel tubes underneath the A23 and in above the platform tunnel in places. The observed deformations during construction were within the predicted range.

2. Introduction

The extension of underground line U1 is planned from Reumannplatz station to Oberlaa station, which includes construction lot U1/9. Starting from Reumannplatz, the U1 will be extended by 4.6 kilometres with five new stations in the forthcoming years. By 2017, it will be the longest underground line in Vienna (19.2 km).

The planning group PCD – FCP – iC in cooperation with Architektengruppe AGU is carrying out the design for getting permission, tendering and construction Lot U1/9. PCD, the lead planning partner, is responsible for the design of the cut-and-cover method, whilst iC is responsible for the design of the NATM tunnel sections. Preliminary geotechnical work, main geotechnical investigations and geotechnical support during construction were undertaken by the municipal department MA 29 (bridge construction and ground engineering). The design for obtaining the building permit in accordance with the Austrian railway law was carried out in 2009 and 2010 and approval was given in January 2011. Tenders were first drafted in 2010, and were published in the summer of 2011. The tender was awarded to the company STRABAG. Construction started in the spring of

2012. Tunnel excavation commenced at the end of January 2014 and was completed in October 2014. Works for the secondary lining started in September 2014 and are expected until summer 2015.

3. General arrangement

The entire U1/9 construction lot traverses at a deep level. The running tunnels (construction section B and C) run from the interface of construction lot U1/8 – U 1/9 in Favoritenstrasse, curving left under the Favoriten ring road and the A23 Laaerberg tunnel up to Altes Landgut station (Ref. [2]). The two tracks each run in single track tunnels with a centre to centre distance of c. 9m at the beginning of the construction lot and widening to a distance of c. 31m at the approach to the station. The Altes Landgut station is arranged along a straight section and comprises the two station shafts "Altes Landgut" (construction section L) and "Katharinengasse" (construction section K) with the 115 m long station tunnels in between (construction section S).

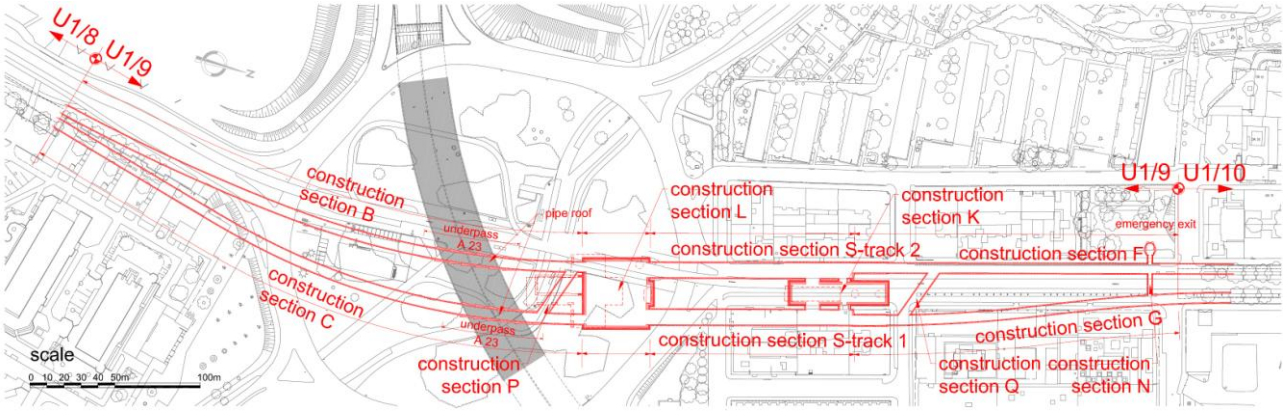


Fig. 1 *Layout (Ref. [2])*

The "Altes Landgut shaft" is located in the Favoriten ring road and the "Katharinengasse shaft" in Favoritenstrasse.
The tunnels between Altes Landgut station and the interface of lot U1/9 – U 1/10 run under the Favoritenstrasse, with the spacing between the lines reducing from c. 31m to c. 13m (construction section G); track 2 runs in a straight line (construction section F). A cross-passage (construction sections P and Q) is located immediately before and after the station at c. 60° to the alignment axes. These cross-passages serve to reduce wind pressure loading and wind speeds caused by trains entering the Altes Landgut station to provide utmost comfort to the waiting passengers.
The Maria-Rekker-Gasse emergency exit is located just near the northern end of lot U1/9 (construction section N). The emergency exit is a shaft construction located to the west of the two running tunnels. A cross-passage links the two running tunnels to the emergency exit.

4. Ground conditions

The project area is located in the 10th district of Vienna at Laaerberg. Below made ground (locally refilled brick quarries) are the gravels of the "Laaerberg terrace" underlain by Miocene inter-bedded layers which are divided with the U1/9 area being in four different layers (Ref. [2]).

The upper weathered Miocene groundmass type GA IV is underlain by the unweathered upper Miozene (GA IVa), both consisting of clayey silts with soft to stiff consistency. The groundmass type IVb – which is most relevant for the tunnel excavation – consists of poorly graded coarse silts and fine sands. The range of its consistency is between stiff and semi-solid. Within this layer, with horizontal permeability up to 5 x 10^{-5} m/s, perched groundwater with low yield but hydrostatic pressure of 15 m above the tunnel crown is prevailing. This layer GA IVb was most relevant for assessment of face stability due to the hydro-geological boundary conditions. In particular, the fine sands showed high potential for suffusion during the execution of long-term pumping tests. The lowest layer of interest comprises the ground mass type IVc, to be expected at the invert level,

which consists of clayey silt with at-least semi-solid consistency with a permeability of about 10^{-8} m/s. Due to this technically impervious layer a full de-watering of the upper GA IVb is not feasible and residual pore pressures will remain.

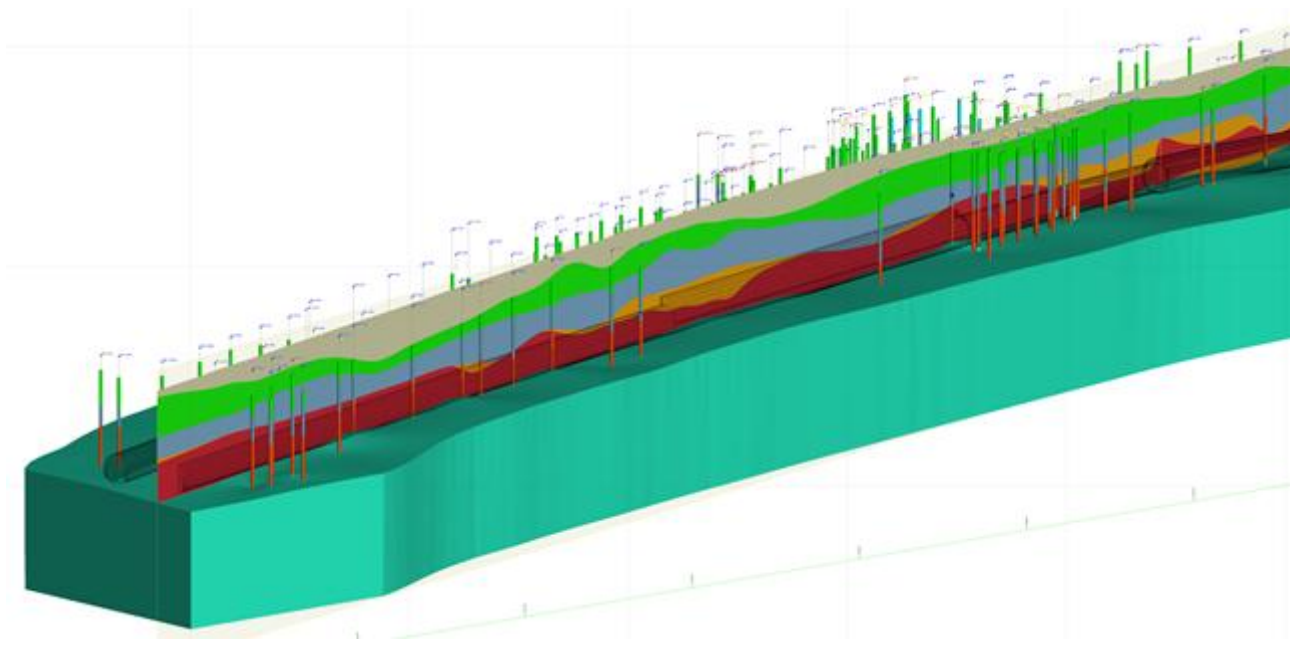


Fig. 2 *Geological model for track 1 (Ref. [2])*

Based on extensive geological and geotechnical site investigations, including pressure meter tests in various depth and soil mechanics laboratory test, the following geotechnical parameters were derived for the relevant groundmass types:

Table 1 *Geotechnical characteristic parameter (Ref. [2])*

Parameter	Abbr.	Unit	GM I	GM II	GM IV	GM IVa	GM IVa/ IVb	GM IVb	GM IVc
Unit weight	γ	[kN/m³]	19	19	20	20	20	20	20
Poisson ratio	ν	[-]	0.3	0.33	0.35	0.35	0.34	0.33	0.35
Elastic modulus	E	[MPa]	20	30	40	60	90	120	100
Friction angle	φ	[°]	25	27.5	23	23	25	27.5	23
Cohesion	c	[kPa]	0	0	30	30	20	10	50
Permeability vertic.	k_v	[m/s]	-	5E-5	1E-9	1E-9	2E-9	5E-6	1E-9
Permeability horic.	k_h	[m/s]	-	5E-4	1E-8	1E-8	2.5E-5	5E-5	1E-8
Lateral pressure	k_0	[-]	0.58	0.43	0.61	0.61	0.58	0.54	0.61

Note: Values for elastic moduls are related to un/re-loading (overconsolidated)

The tunnels are located mainly in the Miocene silt/clay layers which have a relatively homogeneous structure. They basically comprise of slightly sandy, grey, dark grey to grey-blue silts/clays, becoming semi-solid to solid with increasing depth. They may in part contain white precipitates or concretions and hardened layers. From Maria-Rekker-Gasse to the centre of the Favoriten ring road a thick mixed layer of fine sand or sandy silt layers, 5m thick on average, runs through the construction lot from approx. 20m below ground level; this mixed layer is also found in construction lot U1/10. About 95% of groundwater infiltration into the tunnel occurred on this level. Fig. 2 shows the surveyed geological conditions over a longitudinal section through track 1.

5. Design of tunnels

5.1 Design for preparatory works

Existing residential buildings were inspected and assessed at early design stage. Based on the condition of the building and its foundation – construction measures for strengthening of the building structure were decided and designed. Main measure was the installation of a reinforced concrete slab at foundation level for about 30% of the buildings at Favoritenstraße. The purpose of the slab was to increase the stiffness of the building structure in order to enforce more even settlements of the building. The slabs were designed that the foundation achieved sufficient safety against base failure under application of EC 7. The slabs were installed under the Contract U1/9 in advance of the tunnel excavation.

The owner of the freeway A23 decided to refurbish during the years 2011 and 2012 the Laaerberg Tunnel (which was built in 1972). These refurbishing was unfortunately prior of the tunnel exaction for the subway, which had to be carried out in 2014 just below the Laaerberg Tunnel. The structure of the Laaerberg Tunnel consists of 3 reinforced walls parallel to the freeway and of the ceiling girders with thin reinforced concrete slab on top. There exists no base slab. In order to minimise the risk for damaging the newly refurbished Laaerberg Tunnel, the subway designer was allowed to design some special preparatory works, which were carried out during the refurbishing. These preparatory works included jet-grouting at the central diaphragm wall of the Laaerberg tunnel. This D-wall reached below the invert of the subway tunnel, consisting of panels c. 6 m long and gaps c. 2m in between. Scope of the jet-grouting was to close the gaps with stronger material in order to achieve an arching effect during tunnel excavation. The new concrete pavement of the freeway has been reinforced at the crossing area in order to get some bridging effect in case of local cavities caused by the tunnel excavation. Extensive analyses have been carried out on the Laaerberg Tunnel structure in order to evaluate its allowable differential settlements.

5.2 Running tunnels

The overburden over the tunnel crown is between c. 16 and 20m and c. 6.5m up to the road surface of the A23. Fig. 3 shows a cross-section of the running tunnels in the area of the A23 Laaerberg tunnel. The clear internal area of the standard cross-section types of running tunnels is 24.12 m^2. The load-bearing water pressure resisting 40cm thick secondary lining is made of fire-resistant reinforced C25/30-WDI fibre concrete. The excavation area amounts to 37.74m^2 for the 25cm shotcrete thickness, with 3 cm extra for deformation (Ref. [2]). The scheduling provided for excavating the entire excavation cross-section of the single tunnel tunnels with short top heading and fast ring closure (5 to 6 m), divided into the partial cross-sections of top heading, bench and invert.

The running tunnels underpass the existing A23 Laaerberg tunnel over a length of c. 35.1m (track 1) and 33.7m (track 2). A thicker reinforced secondary lining is provided in this area and also extends for several adjacent metres at both ends. Tunnelling is carried out underneath a pipe roof umbrella. The surfaces of the sawtooth-like excavation amount to 42.24 m^2 min. / 54.18 m^2 max. in the case of a 45 cm thick cast-in-situ secondary lining and a 40 cm thick shotcrete primary lining with 3 cm extra for deformation. The primary lining underneath the A23 is fully electrically insulated isolated from the remaining structure via GRP armouring and FPO foil (Ref. [2]) .

The pipe roof umbrella comprises 12m long steel pipes (dia. 114mm, t = 6mm) with a spacing of normally 300mm at the bore face and a longitudinal overlap of at least 4m. The pipes and annular space of the tunnel bore are grouted with a cement suspension from the driving end of the pipe, avoiding heaving. The axes of the A23 tunnel and the subway tunnel are angled at ca. 60°, enabling the diaphragm walls to be separated into short sections during undercutting and to be underpinned with a reinforced shotcrete ring. A reinforced shotcrete primary lining with all-round lattice girders was constructed in the entire underpass area (Ref. [2]). The fine sand layers were drained from above ground using vacuum filter wells and vacuum lances were also deployed from the driving side where necessary. Systematic face anchoring and short ring closure were provided in order to minimise settlements.

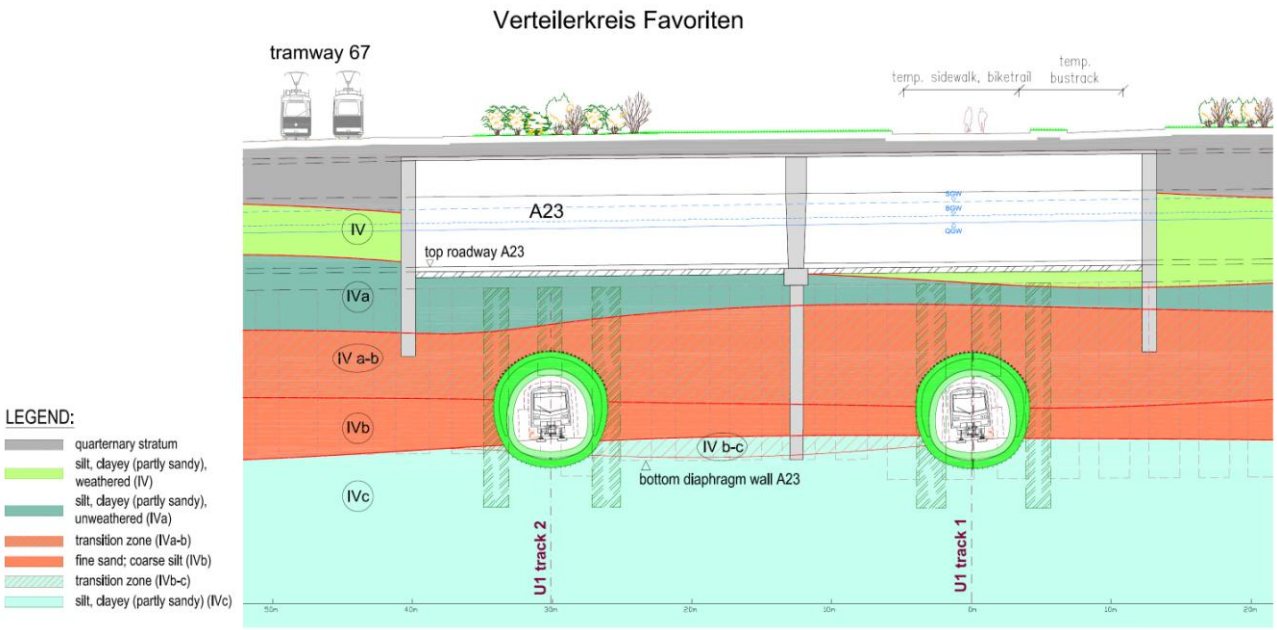


Fig. 3 *Running tunnel cross-section under the A23 Laaerberg tunnel (Ref. [2])*

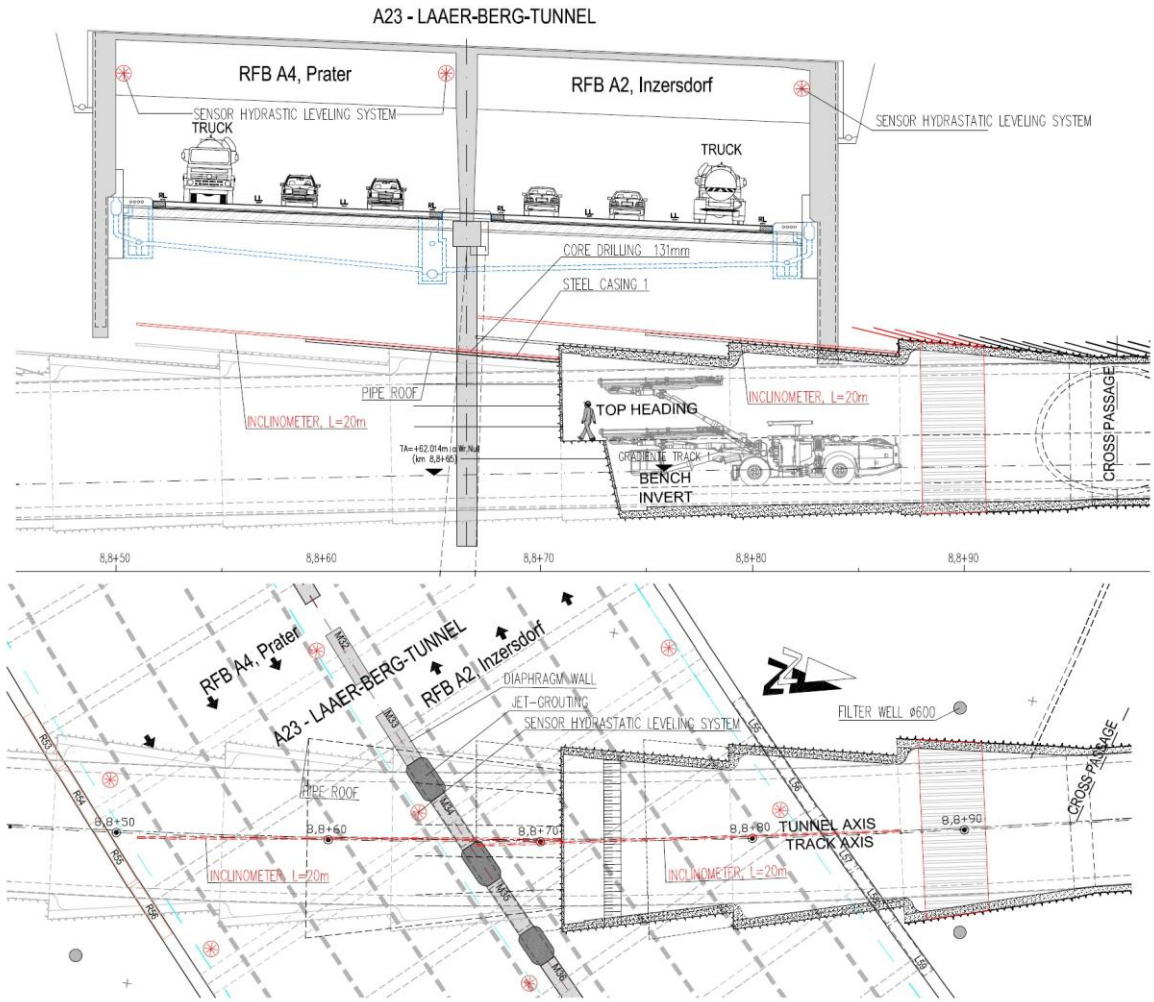


Fig. 4 *Running tunnel cross-section under the A23 Laaerberg tunnel (Ref. [2])*

5.3 Platform tunnels

The section "Altes Landgut station shaft up to c. the centre of Katharinengasse station shaft" will have mined platform tunnels. The tunnels are below the ring road and Favoritenstrasse connecting in the north, under buildings. The overburden over tunnel crown is between c. 16 and 17m to the terrain surface and c. 14m to the foundation base in the built-up area (**Fig. 5**). The clear internal area of the platform tunnels is 55.47m^2. The 40 cm thick secondary lining comprises fire-resistant reinforced C25/30-WDI fibre concrete and can resist the full water pressure. The excavation area measures 77.60m^2 for 30cm shotcrete primary lining thickness with 3cm extra for deformation (Ref. [2]).

In order to limit the size of the heading – the excavation sequence with one side drift and one enlargement was applied – see **Fig. 6**. The thickness of the C25/30 shotcrete lining was in general 300 mm. The shotcrete is reinforced with 2 layer of wire mesh with dia. 6 mm, c/c=100mm and below buildings with dia. 7 mm, c/c=100mm respectively. The round length is 1 m, lattice girders (90/20/30) were foreseen in top heading and bench, every other girder also in the invert in order to have full-round girders for geometry control. For the inner side wall of the side drift TH21-girders were used. Face bolts with ultimate load of 250 kN were applied systematically.

Short top heading and fast ring closure (max. 5 m) was foreseen, divided into the partial cross-sections of top heading, bench and invert. Top heading and bench excavation used a round length of max. 1 m. The invert was excavated in sections with a maximum length of 2 m. Fore poling is used for pre-support. The inner wall was demolished shortly prior the invert excavation of the enlargement.

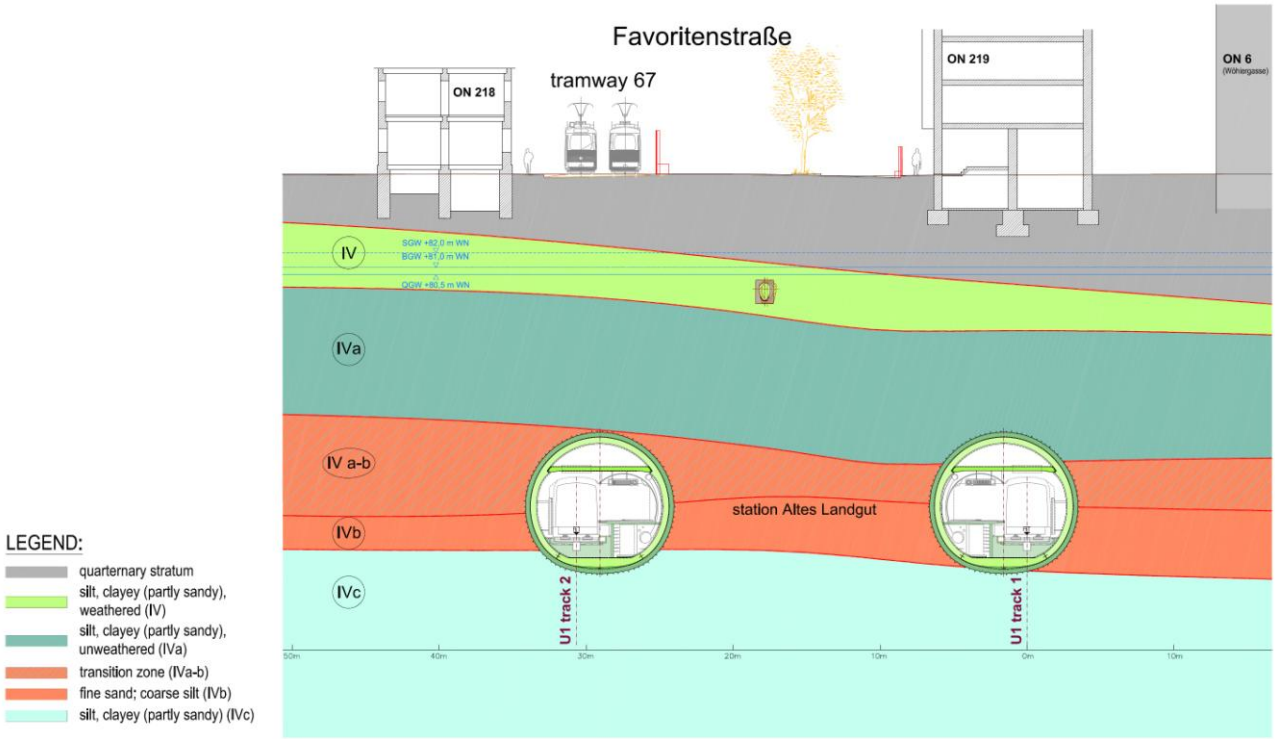


Fig. 5 *Cross-section, platform tunnel (Ref. [2])*

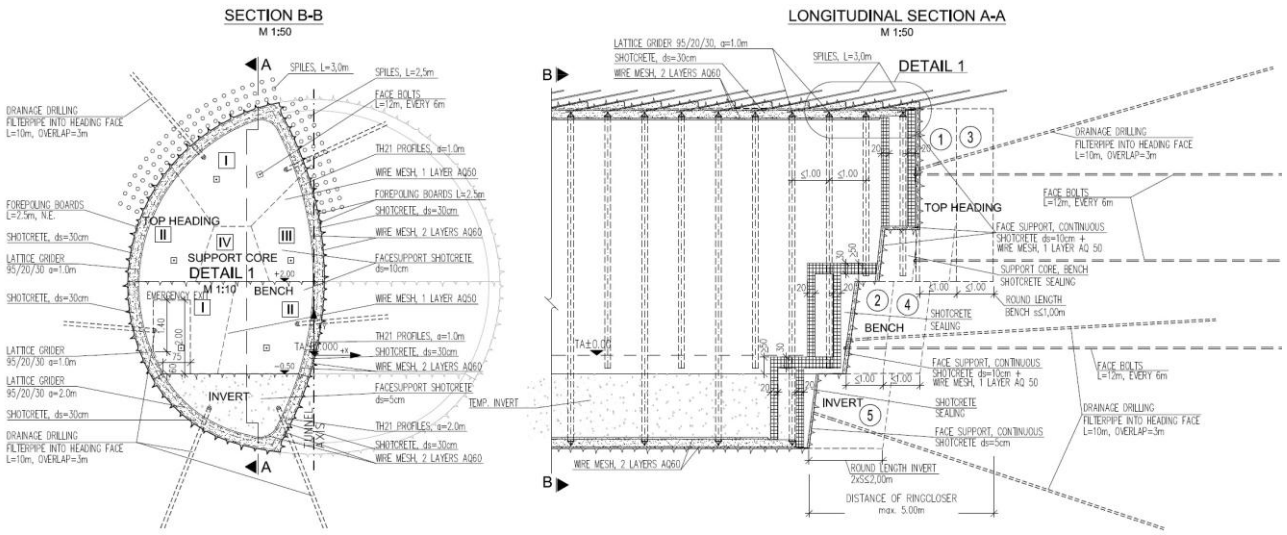
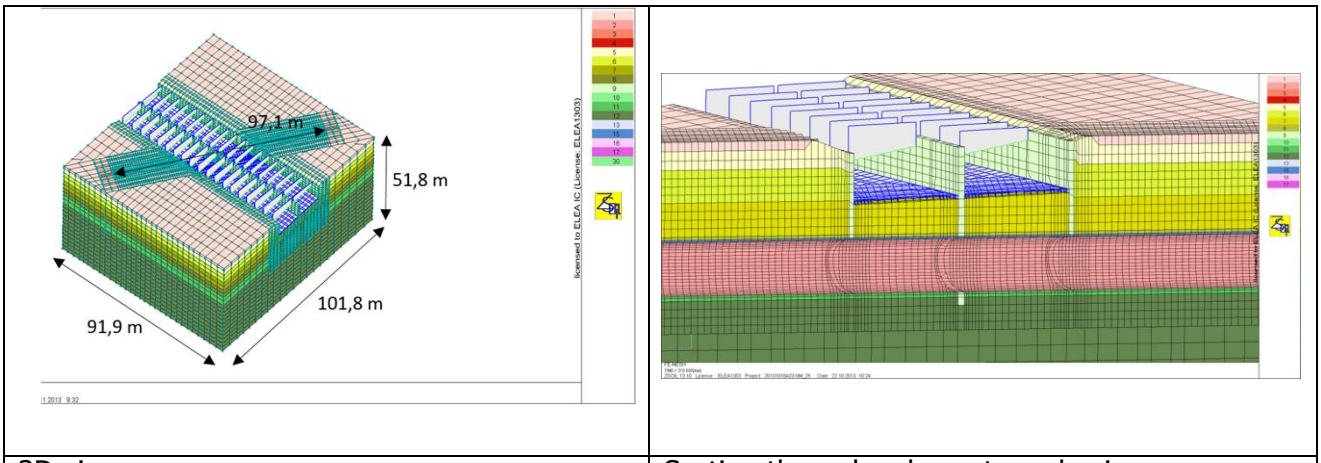


Fig. 6 *Outer lining and excavation sequence for platform tunnel / side drift*

5.4 Structural analyses

In order to predict the ground deformations, ground stresses, pore pressures and the sectional forces of the shotcrete lining – detailed 2D FE effective stress analyses using Phase 2 (Ref. [3]) and/or Sofistik (Ref. [6]) were carried out to model the excavation of all tunnels. The three dimensional stress-state during the different construction stages was modelled by a series of two-dimensional finite difference runs. The applied pre-deformation – by reduction the effective stress at excavation area by 10% - considered the deformations occurring ahead of the heading face due to the inward deformations of the heading face. The related radial deformations could occur before the support elements were installed.

For underpass section of the freeway A23 Laaerberg Tunnel - 3D FE analyses were carried out using Z_Soil (Ref. [5] as a coupled effective stress consolidation analyses in "real time" (days). The calculated effective stress, pore pressures and sectional forces were obtained as a function of the time (days). The shotcrete lining was considered permeable in relation to the surrounding low permeable ground – thus the tunnel acts as a "drain". **Fig. 7** shows the 3D mesh for the structural analysis for the underpass of the A23 Laaerberg Tunnel.

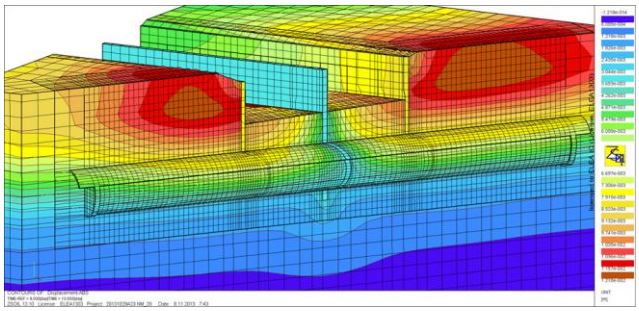


3D view	Section through subway tunnel axis

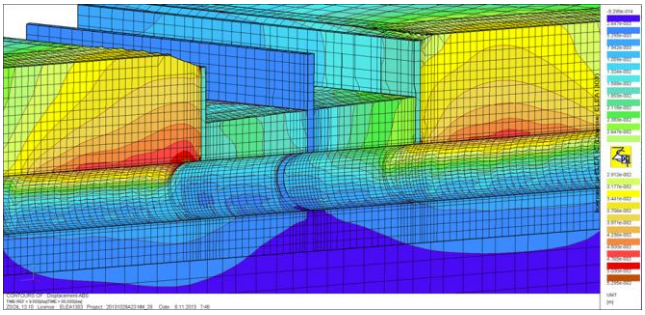
Fig. 7 *3D-FE model for underpass of A23 Laaerberg Tunnel*

The predicted surface settlements are 12 mm after completion of the de-watering with wells from

surface. After completion of the tunnel drive, which is occurring on day 85 in the analysis, maximum surface settlements reach 36 mm. The settlements of the freeway pavement are 10mm after de-watering from surface and reach 24 mm after completion of the excavation. The additional settlements caused by the tunnel excavation are 14 mm.



Settlements due to groundwater lowering from surface, 12 mm



Settlements after tunnel excavation (85 day), 36 mm at surface

Fig. 8 *Predicted settlements at A23 Laaerberg tunnel*

The magnitude of the calculated vertical ground displacements were in the range of 30 to 50 mm, the horizontal ground displacements were in the range of 5 to 20 mm. A yielded zone of several metres in thickness develops adjacent the side walls of the tunnel excavation. The pore pressures were significantly reduced around the shotcrete lining for the entire period until installation of the inner lining. Seepage occurred towards the shotcrete lining. The calculated normal forces, bending moments and shear forces varied also versus time. The normal forces were up to 1200 kN/m, the bending moments reach 80 kNm/m. The following figure (**Fig. 9**) presents the predicted settlements versus time of a selected roof girder of the Laaerberg Tunnel. The girder with biggest settlement on the third diaphragm wall is selected.

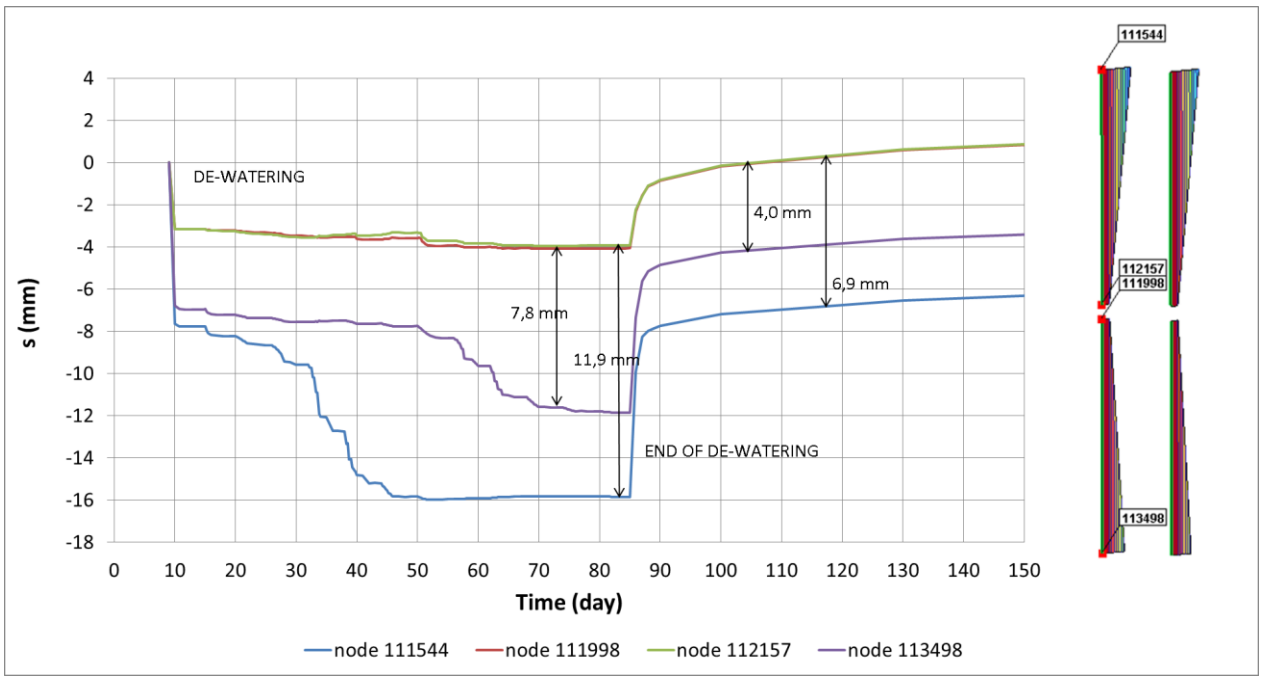


Fig. 9 *Settlement roof girder of the Laaerberg Tunnel versus time*

The structural design of the shotcrete lining was performed as a load factor approach according to the Austrian standard ÖNORM EN 1992-1 for members subjected to normal force and bending moment. The numerical analyses were executed with characteristic values of the considered parameters and loads. Therefore, in the numerical model all material parameters and all loads are considered with a safety factor of 1.0. For the reinforcement design the sectional forces, were multiplied by a safety factor 1.2 according to RVS 9.32 (Ref. [1]), the applied safety factors for the concrete was 1.50 and for steel 1.15, respectively. The considered minimum reinforcement was

0.10% of lining cross area for the inner and outer face, a value which is required actually for the inner lining according to the RVS 9.32. The actual applied reinforcement was at-least 2.83 cm^2/m per face (dia. 6mm, c/c = 100 mm).

6. De-watering from above ground

Two long-term pumping tests were carried out for verification of the best de-watering measures to achieve pore pressure relief within the GA IVb layer. Different types of filter wells were undertaken with variety of boring methods (cable tool or percussion core drilling), drilling diameter (DN 640, DN 324), filter-tunnel diameter, grain size distribution of filter and width of filter slots. The pumping was done with and the without use of a vacuum, each for 3 weeks.
Based on the results of the pumping tests it could be confirmed that the various layers were hydraulically connected and could be designated as a sort of aquifer. The success of pore pressure relief depended on well installation type and duration of pumping. The filter wells DN 640 showed better success in pore pressure relief. Due to the low yield (< 1 litre/sec) additional vacuum was quite beneficial to reduce pore pressures. Further information is provided in Ref. [2].

7. Tunnel construction

Hydrogeological conditions demand that the water-bearing strata be drained or pore pressure-relieved for tunnel driving by means of above-ground measures.
Starting from the South shaft (construction section L), the running tunnels are excavated southwards and the platform tunnels northwards in soft ground in cyclical tunnel driving in accordance with NATM principles and applying the ON B 2203 Part 1 standard. A flexible construction time model was put to tender for tunnel driving.

Fig. 10 Running tunnel track 2 – start of excavation

Fig. 11 Running tunnel track 1 below A23 – pipe roof installation

In general, construction should ensure minimum settlement in built-up areas and traffic zones (Ref. [2]). This requires rigid construction with short ring closure as well as face anchors to reduce pre-relief. Subsidence due to ground water stress relief is due to increase of the effective stress in the ground (no hydrostatic uplift). Drainage was carried out especially in secondary sand layers, thus anticipating rather minor subsidence and expansive flat subsidence basins with little effect on the existing buildings. The measures were specified based on structural calculations and permissible deformation of existing structures.

8. Geotechnical survey

The tunnel driving had been started by the end of January 2014. The entire tunnel driving has already been successfully completed and the installation of the secondary lining is also completed meanwhile. During excavation, the ground is temporarily stable. The monitoring equipment comprises bolt gauges for 3-D trigonometric shotcrete shell displacement measurements, surface settlement points, horizontal automatic inclinometers above the tunnel tunnels underneath the A23 and above the platform tunnel in places. The supporting structure of the A23 was constantly monitored using a liquid levelling system.

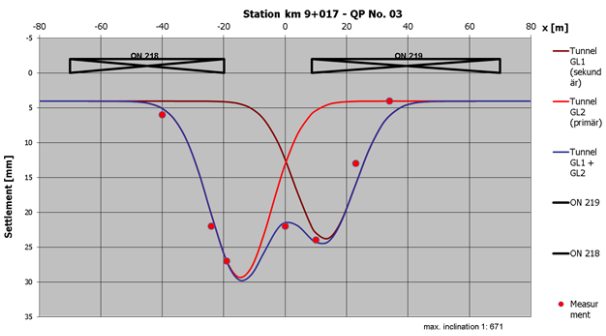
Control values such as advance warning, warning and alarm values were defined for all monitoring

parameters considered. The horizontal inclinometers and tunnel levels generate automatic messages when the control values are exceeded. During excavation in the vicinity of the A23, shotcrete lining displacements were less than 10 mm, subsidence of the A23 road surface was in the region of the predicted 15 mm and the settlements of the diaphragm walls were below 10 mm. The probably most critical specification for the project was the twisting of the joists spanning the centre diaphragm wall with an advance warning ratio of 1:1000 and an alarm ratio of 1:500. Measured twisting did not exceed c. 1:2000, due to more favourable characteristics encountered. The survey of the remaining tunnel excavation was within the prediction – settlements less than 30 mm and trough inclinations flatter than 1:500.

9. Back-analyses of surface settlements

Some monitoring sections have been selected to carry out a back-analysis for the volume loss and trough parameter applying the calculation approach according to Ref. [3].The calculated settlement troughs are shown in the following **Fig. 12**. This figure shows also the assumed volume losses and trough parameter in order to match with the monitoring results for the final stage.

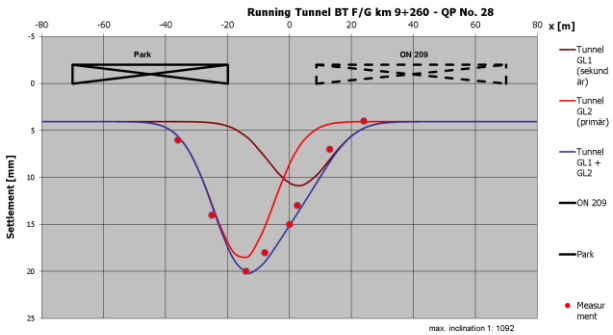
Station tunnels BT S km 9+017 – QP No. 03



Volume loss first tunnel: VL = 0.87 %
Volume loss second tunnel: VL = 0.66 %

Trough parameter first tunnel: k = 0.46
Trough parameter second tunnel: k = 0.45

Running tunnels BT F/G km 9+260 – QP No. 28



Volume loss first tunnel: VL = 1.01 %
Volume loss second tunnel: VL = 0.48 %

Trough parameter first tunnel: k = 0.45
Trough parameter second tunnel: k = 0.45

Fig. 12 *Back analysis of settlements using volume loss approach (Ref. [3])*

The volume loss is in the range between 0.48% and 1.01%. The excavation of the first tunnels results in larger volume loss compared for the second tunnel. The trough parameter is practically 0.45.

10. References

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