Project Modernizace trati Praha-Výstaviště – Praha-Veleslavín

Expert Assessment

on behalf of Správa Železnic

Part II –Assessment of Alignment Variants

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1 Introduction and Objective

1.1 Project Introduction

As part of the train connection between the centre of Prague and Prague International Airport and the City of Kladno, respectively, a modernisation and expansion of the existing railway track between Praha-Dejvice and Praha-Veleslavín (hereafter referred to as Project) is currently investigated.

Five variants have been taken into consideration in a current feasibility study: (1) a widening of the existing single-track at-grade alignment; (2) a close-to-surface underground alignment following the existing alignment (CUT-AND-COVER variant); and (3/4/5) three different bored tunnel variants (NORTH, CENTRE, and SOUTH) that will deviate from the current alignment and will be located deep below the ground surface.

For reasons of potential interference with the surrounding residential areas along the railway track, the at-grade variant has mostly been ruled out. At present, four remaining variants (CUT-AND-COVER, NORTH, CENTRE, and SOUTH) are still in consideration (see Figure 1).

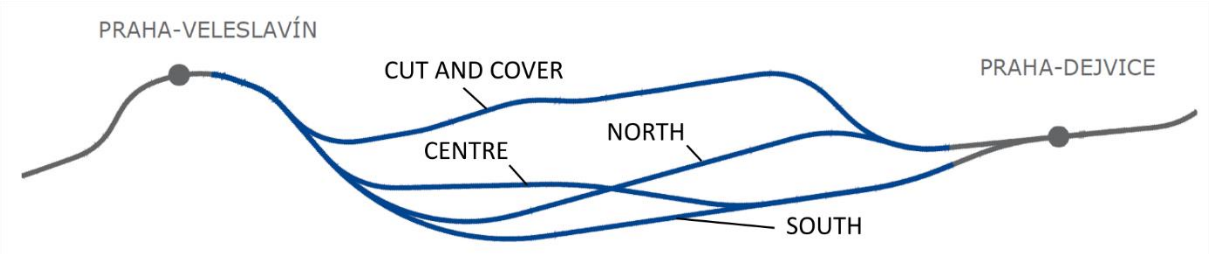

Figure 1: Alignment variants (sketch taken from [2], markers added)

The CUT AND COVER variant follows the current alignment of the rail track. The bored tunnel variants follow three different alignments south of the existing rail track.

1.2 Objective and Scope of this Report

The Authors of the Report were selected by Správa Železnic, the Owner of the Project to provide an expert assessment of the currently considered four tunnel variants for the railway track between Praha-Dejvice and Praha-Veleslavín.

In particular, the Authors have been asked to provide their opinion regarding the following three items:

1) The sufficiency of the ground investigations and their interpretation so far conducted for the preliminary design stage and for the selection of the preferred tunnel alignment option.

2) A technical assessment of all four variants regarding geotechnical risks, impact on the surroundings, and technical suitability. As a results of this assessment, a preference for one of the variants is to be provided.
3) An answer to several questions on specific topics that have been provided by the Owner.

Part I of our Report addressed the first of the aforementioned items by assessing the geotechnical investigations regarding their feasibility and sufficiency. The further two items are covered in this second part of our Report.

The focus of the assessments is on the impact of the different tunnelling variants on the surroundings. Therefore, particular emphasis was put on the numerical analysis of settlements in twelve cross-sections along the four alignment variants.

2 Reference Documents

2.1 Design Documents of the Feasibility Study

[1.1] Studie proveditelnosti Železniční spojení Prahy, Letiště Ruzyně a Kladno, Metroprojekt Praha, 2016 (in Czech language), Document ID: 16 7021 007 01 03 00 000.

[1.2] Modernization of the line Prague-Výstaviště (excl.) - Prague-Veleslavín (excl.) - Study "Comparison of variants of tunnel solutions in section Praha-Dejvice - Praha-Veleslavín", Metroprojekt Praha, 02/2020, English version of Document ID: 18 7461 22 01 00 00 000.

[1.3] Amendment of preliminary design 03/2009 Modernization of Prague - Kladno line with connection to Ruzyne airport, Phase I, Joint venture Metroprojekt Praha and SUDOP for PRaK Phase I, 03/2009, Parts D (Geotechnical Exploration) and E (Construction Part) for CUT AND COVER variant.

[1.4] Modernizace trati Praha-Výstaviště (mimo) – Praha-Veleslavín (mimo), dílčí plnění předkládající technické řešení a geotechnický průzkum pro variantu raženou SEVER, „Společnost MP+SP – Výstaviště-Veleslavín“, (02/2019, in Czech language) and English translation of Geotechnical Survey report for NORTH variant (03/2019), Document ID: 18 7461 06 08 01 004.2.

[1.5] Geotechnical exploration report for bored tunnel, SOUTH variant (08/2019), English translation of Document ID: 18 7461 04 02 01 07 006.01.

[1.6] Geotechnical exploration report for ventilation shaft, SOUTH variant (11/2019), English translation of Document ID: 18 7461 04 02 01 07 009.01.

[1.7] Three geological cross-sections through all alignment variants (scale 1:1000/500) including a situational plan, per e-mail by Metroprojekt on May 18, 2020.

[1.8] Clarification of questions concerning regional geology of Prague, Metroprojekt, 05/2020, Document ID: 18 7461 23 00 00 00 000.

[1.9] Table summarising laboratory and field testing carried out in each of the boreholes - for variants bored tunnel SOUTH and NORTH, Metroprojekt, 05/2020.

[1.10] Prague-Kladno Railway Route Modernization with a connection to Ruzyně Airport, phase 1, E. Construction part, E.1 Engineering structures, E.1.7 Railway tunnels, Construction lot 06 Dejvice – Veleslavín, 08 Veleslavín – Liboc. Technical Report, Metroprojekt Praha, 03/2009, English version.

[1.11] Geotechnical exploration report for bored tunnel, CENTRE variant (07/2020), Metroprojekt Praha, English translation.

[1.12] Geotechnical longitudinal section, CUT AND COVER variant, Metroprojekt, Document ID: 07 4477 002 05 01 07 001.

[1.13] Geotechnical cross-sections for numerical analyses, provided by Metroprojekt.

2.2 Relevant Codes, Standards and Recommendations

[2.1] DIN EN 1997-1:2014-03, Eurocode 7: Geotechnical design – Part 1: General rules; German version EN 1997-1:2004 + AC:2009 + A1:2013.

[2.2] DIN EN 1997-2:2010-10, Eurocode 7: Geotechnical design – Part 2: Ground investigation and testing; German version.

[2.3] ČSN EN 1997-1, Eurokód 7: Navrhování geotechnických konstrukcí - Část 1: Obecná pravidla.

[2.4] ČSN EN 1997-2, Eurokód 7: Navrhování geotechnických konstrukcí - Část 2: Průzkum a zkoušení základové půdy.

[2.5] DIN 4020:2010-12: Geotechnical investigations for civil engineering purposes – Supplementary rules to DIN EN 1997-2.

[2.6] DIN 18196:2011-05: Erd- und Grundbau – Bodenklassifikation für bautechnische Zwecke (Earthworks and foundations – Soil classification for civil engineering purposes).

[2.7] DIN 18312:2016-09: German construction contract procedures (VOB) – Part C: General technical specifications in construction contracts (ATV) – Underground construction work.

[2.8] ITA Report on Strategy for Site Investigation of Tunnelling Projects.

[2.9] Deutscher Ausschuss für unterirdisches Bauen e. V. (German Tunnelling Committee (ITA-AITES)): Empfehlungen zur Auswahl von Tunnelbohrmaschinen (Recommendations for the selection of tunnel boring machines, in German), 2020.

[2.10] Deutsche Gesellschaft für Geotechnik (DGGT): „Empfehlungen des Arbeitskreises "Numerik in der Geotechnik" – EANG“, ISBN: 978-3-433-03080-6 (2014).

[2.11] Burland, J. B., and Wroth, C. P. (1974). “Settlement of buildings and associated damage.” Proc., Conf. on Settlement of Structures, Pentech Press, London, England, 611-654.

3 Description and Characteristics of Alignment Variants

The characteristics of the four variants to be assessed in this Report (see Figure 1) are briefly described below.

3.1 Variant CUT AND COVER

The CUT AND COVER variant follows the existing rail track along its original alignment in a shallow tunnel stretch. The technical design for this variant [1.10] lists various Advance Sections (sections of different construction methods) that employ cut and cover methods, the New Austrian Tunnelling Method (NATM) or the traditional Czech “Tortoise” method. The Advance Sections and their respective construction methods, as defined in the geotechnical longitudinal section [1.12], are listed in Table 1.

Since the alignment follows the existing rail tracks, no vulnerable structures directly above the tunnel are to be considered. However, there are some buildings directly adjacent to the alignment, which require particular attention regarding excavation-induced ground movements. The construction of the reinforced concrete structures of the tunnel and the stations is not in the scope of this Report. Thus, the focus of considerations is put on the NATM parts and on the trench support methods, and here in particular those designed as bored pile walls with and without staggered soldier pile walls (methods a and b). These trenches are most challenging and associated with the highest risk compared to other stretches.

Table 1: Advance Sections, CUT AND COVER variant

Section	Chainage [km]	Length	Construction method (see Ch. 3.1.1/3.1.2)
SO 06-171-001	4+030 – 4+760	730 m	Cut and cover, trench support method a/b
SO 06-172-001	4+760 – 5+080	320 m	NATM
SO 06-172-002	5+080 – 5+235	155 m	“Tortoise” method
SO 06-172-003	5+235 – 5+750	515 m	NATM
SO 06-171-002	5+750 – 6+602	852 m	Cut and cover, trench support method c/d
SO 17-046-001	6+602 – 6+807	205 m	Cut and cover station, not part of the scope of this Report.
SO 06-172-004	6+807 – 7+000	193 m	NATM
SO 06-171-003	7+000 – 7+672	672 m	Cut and cover, trench support method d
SO 07-171-001 SO 07-141-001 SO 07-146-001 SO 07-141-002 SO 07-171-002	7+672 – 7+675 7+675 – 7+704 7+704 – 7+849 7+849 – 7+853 7+853 – 7+855	3 m 29 m 145 m 4 m 2 m	Cut and cover station, not part of the scope of this Report
SO 08-171-001	7+855 – 8+070	215 m	Ramp, not part of the scope of this Report

3.1.1 Cut-and-cover sections

For the cut-and-cover sections, one or a combination of the following construction methods is planned as trench support method:

a. Anchored bored pile walls according to pre-design (see Figure 2).
b. Anchored bored pile walls with staggered soldier pile walls (see Figure 3).
c. A combination of bored pile walls with a nailed rock slope, secured by sprayed concrete (see Figure 4).
d. A combination of soldier pile walls and a nailed rock slope, secured by sprayed concrete (see Figure 5).

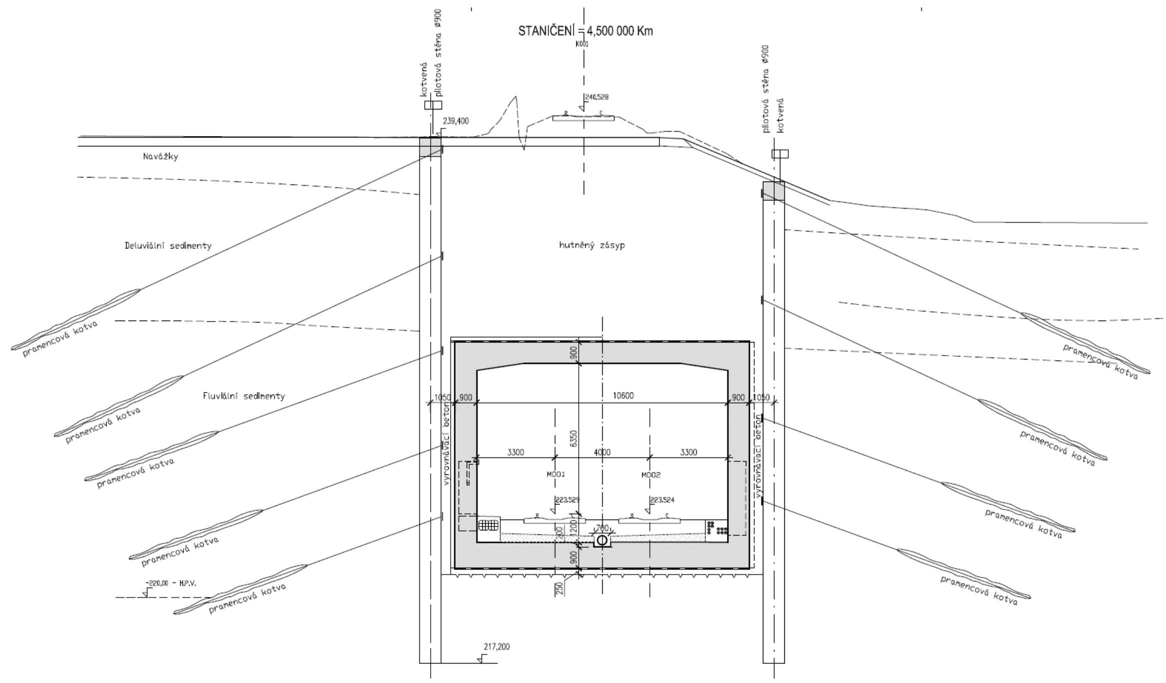

Figure 2: Trench support method a: anchored bored pile walls.

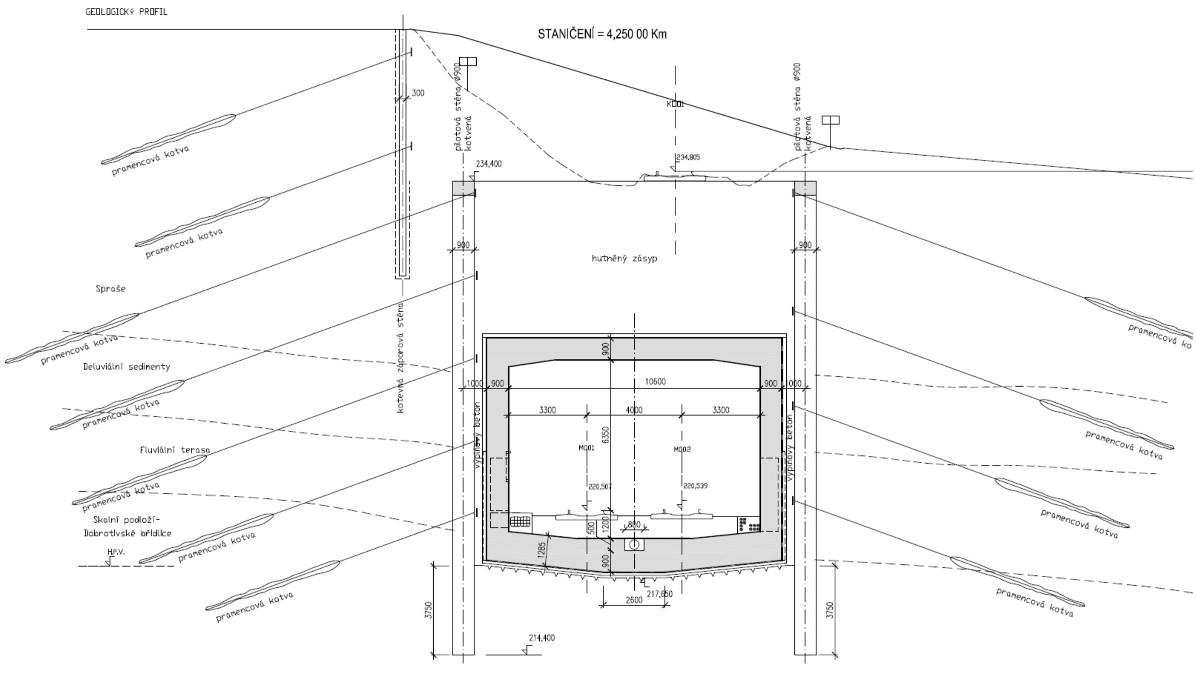

Figure 3: Trench support method b: anchored bored pile walls with staggered soldier pile wall (left).

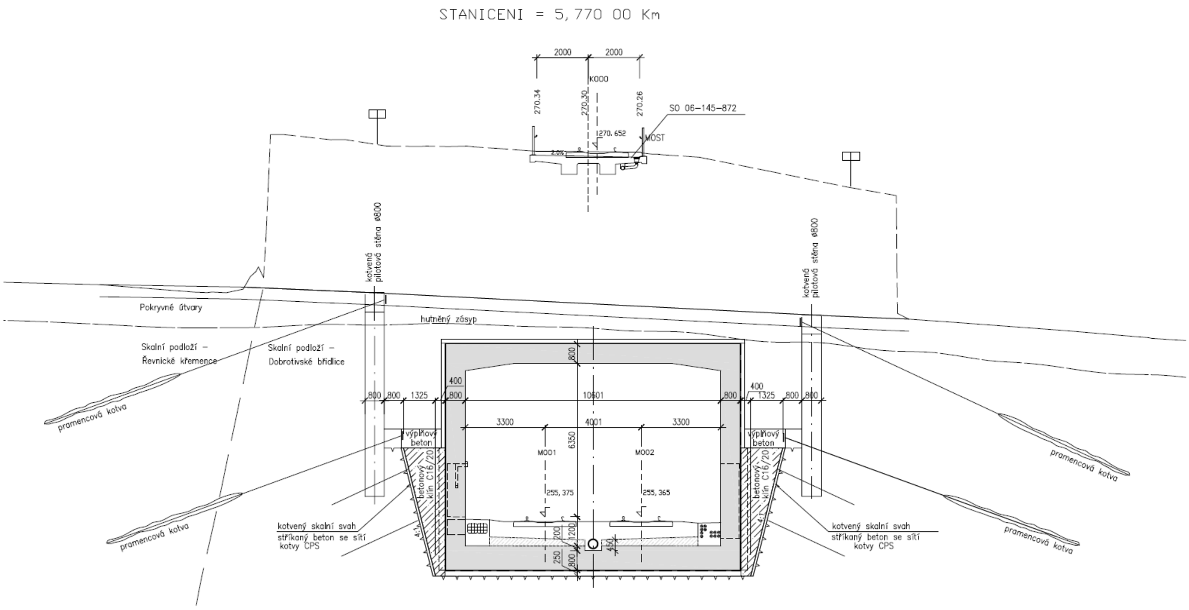

Figure 4: Trench support method c: bored pile walls at the top, anchored rock slope with sprayed concrete at the bottom.

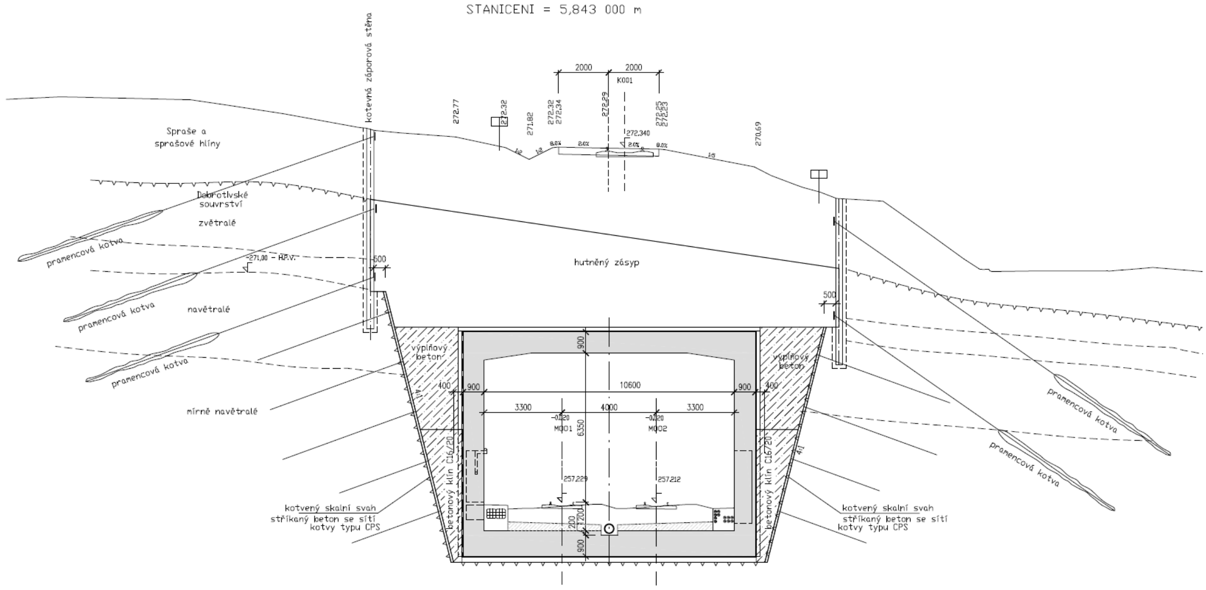

Figure 5: Trench support method d: soldier pile walls at the top, anchored rock slope with sprayed concrete at the bottom.

3.1.2 NATM and "Tortoise" Sections

Three advance sections are designed using the New Austrian Tunnelling Method (NATM). A typical cross-section for these sections is shown in Figure 6. According to [1.10], full-face or horizontally sequenced excavation steps (top heading – bench – invert) are expected in most cases. Where required, a vertical separation of the tunnel face (side wall drifts) is planned. Additional measures of stabilising the ground are not explicitly given in the design and need to be incorporated as required by the local conditions. The ground properties described in the geotechnical investigation report indicate the necessity to first excavate the top heading with a separate, temporary invert slab. This method has been accounted for in the numerical analyses of Chapter 5.

For the excavation of ground, mechanical excavators or blasting is proposed in [1.10]. A dual-layer tunnel lining with an intermediate sealing layer and a reinforced concrete inner lining is planned.

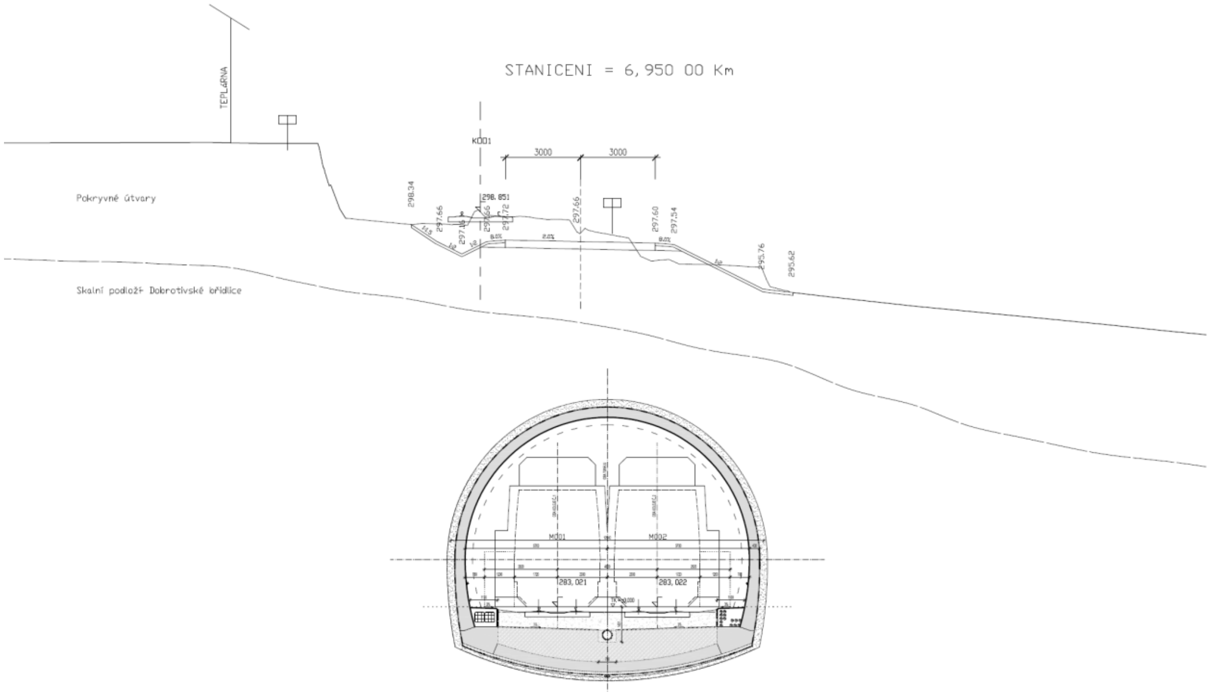

Figure 6: Typical NATM cross-section.

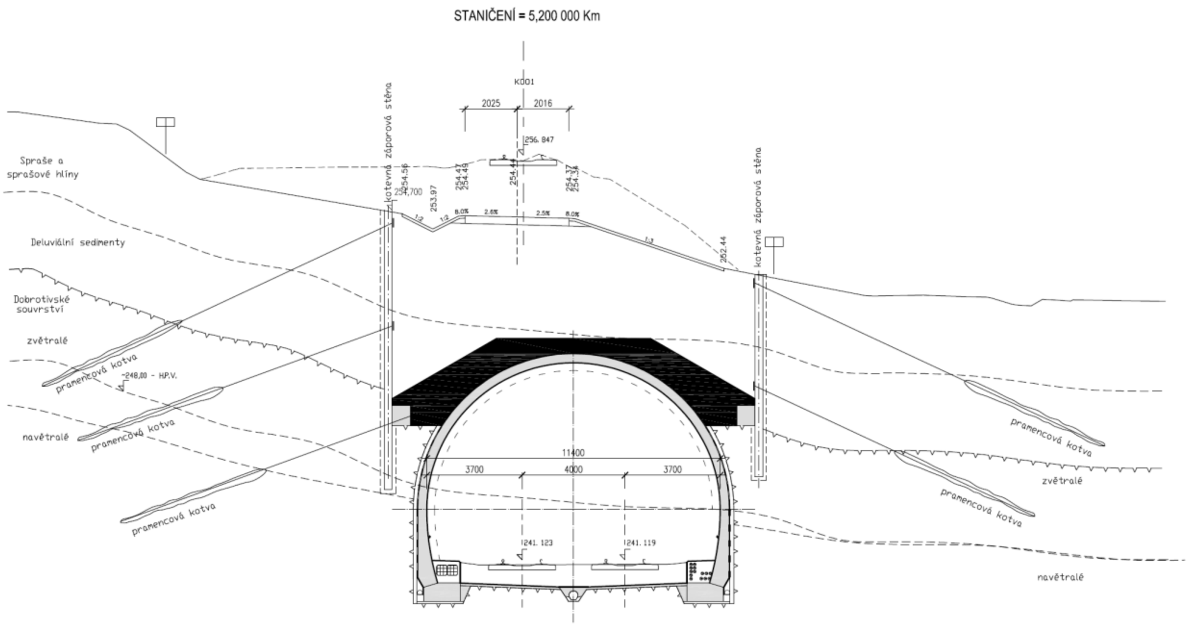

Figure 7: Typical cross-section of the „Tortoise” method.

The “Tortoise” method (typical cross-section shown in Figure 7) combines the cut-and-cover method and the top-down method with the NATM in that the top heading of the mined tunnel is covered by a concrete slab prior to the excavation, which is constructed inside a trench supported by anchored soldier pile walls. This method is foreseen in one advance section of 155 m in length.

3.1.3 Surroundings

The existing railway track runs through the neighbourhoods of Dejvice, Ořechovka and Veleslavín. The planned tunnel of the CUT AND COVER variant would follow the existing railway alignment close to the surface. Towards the eastern portal, the tunnel is next to the Bruska water storage facility with its underground storage tanks in the immediate vicinity of the tunnel. Further along the track the alignment passes next to allotments and garages. Further towards the west, residential buildings, including several heritage buildings, are located in the immediate vicinity of the alignment, which are potentially vulnerable to ground deformations. Finally, the Veleslavín heating plant is located directly to the left of the alignment.

3.2 Bored Tunnel Variants

The three bored tunnel variants would be constructed using earth pressure balanced tunnel boring machines (EPB TBMs). All three variants are deep tunnels running underneath the rock ridge of the Střešovice area. Each variant also has a ventilation shaft at a central location with high rock cover. Horizontally, three different alignment variants, NORTH, CENTRE, and SOUTH, are proposed.

3.2.1 Variant NORTH

The NORTH variant shares the alignment with the CUT AND COVER variant on the first approx. 500 m, before diving under the residential area of Ořechovka. Starting in alluvial soils, the tunnel dips into the shale rock as the overburden becomes much higher in the Střešovice area. Towards the west portal, the alignment passes south of the Veleslavín heating plant before hitting the original railway alignment at Veleslavín Station.

Along the NORTH alignment, the Bruska water storage facility, residential buildings in Buštěhradská Street, the Institute of Physics of the Czech Academy of Sciences, and the Military Hospital are located.

The easternmost stretch of the NORTH variant is designed as a cut-and-cover tunnel (where it shares the alignment with the CUT AND COVER variant). The bored tunnel is to begin as the alignment dips underneath the residential areas. The cut-and-cover stretch has not been designed yet. Hence, only general aspects the cut-and-cover method are considered in the assessment of this variant.

3.2.2 Variant CENTRE

The SOUTH and CENTRE variants share their alignment in the eastern part, where their path deviates from the existing railway track directly after Dejvice Station. The eastern portal is located before Svatovítská Street. Here, the tunnel alignment crosses one of the entrance ramps of the Blanka road tunnel. This crossing is a complex design task that has been excluded from the present scope of this Report. In the subsequent advance section, the tunnel passes close to residential buildings at Pod Hradbami Street and underneath a tram depot. Here, the CENTRE and SOUTH alignment separate. The CENTRE alignment follows the streets Střešovická and Na Petřinách while diving under the Střešovice plateau. There, the alignment runs along the northern slope of the plateau and just north of the Military Hospital, before joining the other alignments close to the heating plant.

3.2.3 Variant SOUTH

From their separation point close to the tram depot, the SOUTH alignment variant dips under the Střešovice plateau, where it underpasses St. Norbert church and the protestant church. After passing the Military Hospital at a high overburden, the alignment turns north, where it joins the other alignments close to the heating plant.

Both CENTRE and SOUTH variants are purely bored tunnel variants, where the exact design of the portal areas is subject to detailed design in course of the project.

4 Tunnelling Methods and their associated risks

4.1 Cut and Cover

Cut-and-cover tunnels are constructed by excavating an open trench from the ground surface, building the tunnel structure inside the trench and then refilling the trench. The tunnel is comparable to a deep excavation pit for a building and the construction risks associated with this method are basically the same as for any deep excavation and mainly related to stability and deformations of the retaining walls supporting the trench.

Comments on the specific construction methods used in the different cut-and-cover stretches are given below.

4.1.1 Section SO 06-171-001 (km 4+030 to 4+760)

<u>Construction methods</u>

The section is characterised by deep excavations between 12.5 m at the beginning of the stretch and maximum depths of 23 m below the ground surface. Anchored bored pile walls (∅ 80/90 cm) with multiple anchor layers are used to support the trench in the shallower sections. The deeper areas use staged walls with anchored soldier pile walls in the first 6 m followed by anchored bored pile walls of up to 17 m depth.

The ground water table is below the bottom of the trench. Buildings are partly located aside of the trench.

Risks associated with bored pile walls

The construction of bored pile walls is a well-established and approved method. However, as with every complex construction method, problems may arise. The most common issues expected in using bored pile walls for a cut-and-cover tunnel are:

- Due to the narrow trench, the requirements on verticality and horizontal placement of the piles are very high.
- Increased tool wear due to ground properties, leading to higher maintenance costs and increased construction time.
- The nature of the trench being a long structure requires the construction of many piles. Any systematic prolongation therefore leads to a significant increase of the complete construction time.
- Noise emissions along the full stretch of the trench will remain for a long time, thus will have a significant impact on the local residents.
- Prior to the construction of the piles, a paved work plane needs to be built. This may cause additional work especially in the slope areas.

In contrast to other deep excavations, the ground water table is expected lower than the bottom of the trench. Hence, water handling is limited to precipitation and the trench does not need to be sealed against water inflow.

Risks associated with anchors

Anchors are a common construction method as well bored piles. Risks to be expected in the current project are mainly

- Extremely high anchor forces: In order to maintain structural stability of the trench, both a large number and high anchor forces are required. If in-situ tests do not approve these high forces, additional auxiliary measures will become necessary. The installation of additional anchors is geometrically difficult, since the anchors require a certain distance from each other.
- Failure in the anchors or their grout bodies will lead to an immediate failure of the trench retaining system.

- In locations where the anchored walls are not sufficient by themselves, a bracing structure may be required that forms an obstacle for the construction of the tunnel structure.
- Design risk: In case longer anchor lengths are required that cause anchors to reach beneath private property, potential delays and additional costs for permits may arise.

4.1.2 Section SO 06-171-002 (km 5+750 to 6+602)

Construction methods

The trench in this advance section is to be stabilised by double-anchored bored pile walls (∅ 80 cm) in the upper part and a nailed rock slope, secured by sprayed concrete, in the lower part. The bored pile walls are secured by anchors in the top (for wall stability) and bottom (to secure the footing).

In areas of more competent ground conditions, soldier pile walls are used instead of bored pile walls. Partly, both methods are applied on either side of the trench.

Risks associated with the construction methods

The general risks described above for the bored pile walls apply to this advance section as well. However, the reduced depth of the bored pile walls helps with reducing both deformations and anchor forces.

Soldier pile walls bear the following typical risks in the present setup:

- Being relatively flexible, the deformations are higher than in bored pile walls. Respective deformations are to be considered in the design and need to be met with matching spacing and beam profiles.
- The drillability of the ground may partly be reduced due to quarzites in the ground. This may render driving the soldier piles more difficult.

The rock nails to be used in the bottom part of the trench are a well-established and common method that is expected to limit the wall deformations to approximately 1 to 3 % of the wall height. The most important factor here is the exact determination of the bedrock head in order to securely found the upper retaining wall.

4.1.3 Section SO 06-171-003 (km 7+000 to 7+672)

The construction methods in this advance section are the same as in Section SO 06-171-002. The general comments and the potential risks are therefore the same as in the previous section.

4.1.4 Summary on Cut-and-Cover Methods

The most critical cross-sections in Section SO 06-171-001 are those employing staged retaining walls in combination with relatively weak ground properties. Here, additional design works and probably also design changes will be required in order guarantee structural stability and admissible deformations.

Apart from this, all construction methods foreseen in the cut-and-cover sections are well-established and approved. The main risk lies in the bearing capacity of the anchors. Here, further investigations of the bearing capacity of the ground-anchor system are recommended.

A positive aspect of the CUT AND COVER alignment is the ground water table, which is assumed below the bottom of the trench throughout all advance sections.

In all cut-and-cover parts, an in-depth analysis of the deformation behaviour is required. This holds in particular for the staged retaining walls of soldier piles and nailed rock walls. Especially the beam profiles and the spacing of the soldier piles play an important role in the deformation characteristics.

4.2 Conventional (NATM) Tunnelling

4.2.1 Construction Method

The mined advance section of the CUT AND COVER variant are to be built using the New Austrian Tunnelling Method (NATM), a method that relies on the observation of ground behaviour and respective reaction with auxiliary measures to control both stability and ground deformations. The tunnel is excavated in flexible steps, either full-face or in stages. After each excavation step, the tunnel face and the intrados are temporarily secured by sprayed concrete. Additional measures like anchors, pipe umbrellas or injections are possible to stabilise the ground ahead of the tunnel face or radially around the tunnel. The benches can be footed on micro piles and a temporary invert slab in the top heading can be used to increase the stability and to reduce the settlements.

In the present project, the NATM tunnelling under the given boundary conditions is challenging from the following points of view:

- very shallow tunnels with low overburden,
- large cross-section due to a double-track tunnel,
- soft ground with low shear strength,
- an urban environment, where sections of critical and settlement-sensitive assets in the vicinity of the tunnel are present.

On the other hand, NATM is a very flexible construction method that allows to adapt the auxiliary measures to changing situations and volatile ground conditions based on observations. The ground and its deformations are directly visible at the tunnel face, can be interpreted by geologists and therefore used to determine the required amount of auxiliary measures such as anchors, foot piles, temporary invert slabs, drainage drilling or a reduction of the excavation length.

4.2.2 Associated Risks

The following risks are associated with NATM tunnelling:

- Instability of the tunnel face (including local instabilities in transition zones between bedrock and soft ground). Potential consequences are severe third-party damage, threat to human lives, and long standstills (delay of construction time and increase cost). As a countermeasure, a sufficient face stability assessment and a robust design of auxiliary measures, e.g. face bolting, a support core or ground improvement injections, where applicable, may be required.
- An important part of managing the stability and deformations in NATM tunnelling is the selection of partial excavation of the cross-section. The tunnel face can be vertically or horizontally divided, e.g. top heading/bench/invert or core/side drifts.
- A general risk of conventional mining is the instability of the ground over the unsupported span (crown and/or walls) in the excavation area. As a countermeasure, auxiliary supports need to be employed, such as a pipe umbrella or forepoling. Another solution may be the reduction of round length (to approx. 1m).
- A reduction of the round length is also an option when instabilities or excessive deformations are observed while excavating the benches and the invert.
- Inadmissible settlements of the ground surface and existing infrastructure may be a result of large deformations at the tunnel face and intrados due to:
    - tunnelling works (stress relief and redistribution processes in the ground associated with tunnel excavation)
    - time dependent consolidation processes in cohesive ground (pore presssure dissipation)
- Failure of the primary shotcrete lining by exceeding its load-bearing capacity. Countermeasures are a robust design with conservative calculation assumptions and constructive design, an adequate execution, site supervision and control of the execution quality and the applied materials as well as monitoring of ground and primary lining behaviour.
- Instability of the temporary tunnel lining: local instability of the footings of the top heading primary lining. Countermeasures are:

  - A temporary invert lining for the excavation of the top heading (settlement reduction and increase in safety)
  - widening of the calotte feet and/or micropiles
  - rapid closure of the temporary shotcrete lining for stability and settlement reduction.
- Formation of an “hour-glass effect” where loose material ripples through the gaps between umbrella pipes.
- The general risk of encountering supply pipes, cables or other infrastructure, which may not be fully documented. This is a general risk of shallow tunnelling.
- Unforeseen water ingress in case the ground water table is higher than expected.
- Deterioration of ground properties (cohesive ground) due to contact with water or humidity.
- Excavation through dill and blast may not be possible due to seismic and noise emissions. This may have an adverse impact on the advance rates.

4.2.3 Summary NATM

Given the boundary conditions of this project, a safe conventional tunnel construction with low impact on the surroundings tunnel is considered feasible only with extensive application of additional auxiliary measures, reduced round length and intensive monitoring. This is likely to result in comparably low advance rates in the range of approximately 1 m per working day, especially in critical sections. Some issues that need to be addressed in a potential design phase are the possibilities to perform blasting with respect to seismic emissions and noise and the vibrations stemming from hydraulic excavation tools.

4.3 “Tortoise” method

The “Tortoise” method is a traditional tunnelling method in the Czech Republic, which, with a certain similarity to the top-down method, combines open trenches and mined tunnelling: a slab is constructed at the level of the top heading from an open trench. This slab serves as a stable roof for a conventionally mined tunnel.

The advantage of this method is that the depth of the trench is much less than for cut-and-cover construction while the conventional mining does not suffer from weak soils above the tunnel.

The disadvantage is mainly an increased effort, combining both the surface disruption and effort of the trench construction and the effort of underground mining. Regarding general risks, both the risks of the cut-and-cover method and of the NATM need to be considered. Yet, the reduced depth of the trench and the slab-secured top heading significantly reduce these risks. As an additional risk, the foundation of the slab needs to be considered. Being located at the bottom of the trench retaining walls, its construction simultaneously disturbs the footing of the retaining walls.

As an alternative to the “Tortoise” method, the NATM alone can be used with respective auxiliary measures that replace the slab.

4.4 Bored Tunnelling (EPB)

4.4.1 Construction Method

The bored tunnel variants are to be constructed by means of an earth pressure balanced tunnel boring machine (EPB TBM). EPB TBMs are designed to provide an active tunnel face support by means of pressurised earth muck in their sealed excavation chamber. Provided a well-maintained consistency of the earth muck, the pressure can be controlled by the interaction of advance rate and material extraction rate, whereas the extraction of muck from the excavation chamber is done by a screw conveyor, which provides the required pressure gradient between the atmospheric conditions in the tunnel and the excavation chamber.

The intrados of the excavation profile are secured by a steel shield, which also houses the area of the lining construction, which consists of precast segments with gaskets. The remaining gap between the tunnel intrados and the segmented lining is backfilled by grouting material injected through the shield tail. This principle allows for a secured excavation and lining process, which is continuously pressurised to retain ground water and loose ground.

In areas where the ground water does not need to be retained and the tunnel face is stable, EPB TBMs can be operated in an open mode without active face support, where the muck level in the excavation chamber is just high enough to allow for extraction of material through the screw conveyor. Retaining of water in case of permeable yet structurally stable ground can also be done by compressed air in the excavation chamber with a low level of muck in the so-called transition mode.

Their flexibility in operation modes and their suitability for all predominant ground types expected in the project area renders EPB TBMs feasible for each of the bored tunnel variants.

4.4.2 Associated Risks

The EPB method, if applied and monitored correctly, is a well-established and safe construction method for tunnels, which can deal with a large variety of ground conditions. Risks associated with bored tunnels in general and the EPB method in particular are:

- Abrasivity of the ground may lead to excess primary and secondary wear (of the tools and structural parts of the TBM). For mitigation, a conditioning concept, the observation of the excavation parameters and regular maintenance interventions are required.

- Some ground materials are prone to clogging if not conditioned well. Clogged tools and cutterhead openings significantly reduce the excavation performance and may cause a disruption of operations, increased wear in other parts, and difficulties in maintaining support pressures and material extraction. A ground conditioning concept is required to prevent clogging.
- Due to malfunction or maloperation of the ground conditioning system, the consistency of the earth muck may suffer. This reduces the ability to maintain a homogeneous material required for keeping the support pressure and efficient excavation and extraction of material.
- Insufficient support and grouting pressures may cause deformations ahead and above the TBM, leading to large settlements. The same holds for overexcavation of material.
- The annular gap between excavation intrados and the segmental lining needs to be completely filled with grout material. Failure to achieve a completely filled gap may result in increased settlements and to an insufficient bedding of the lining segments, which in turn may cause inaccuracies in the positioning, cracks and damages, and leakage of ground water through the gaskets.
- In squeezing ground conditions or in case of excessive wear of the gauge cutters of the TBM, the shield may become stuck in the tunnel intrados. In the worst case, this may lead to a complete loss of a TBM if it cannot be freed by itself. As a countermeasure, apart from adjusting the overcutting gauge to the expected ground conditions and maintaining the tools, the TBM needs to be equipped with sufficient thrust force and possibilities for shield lubrication.
- Deviations from the design alignment in both horizontal and vertical direction are possible, in particular when operating in open mode or if parts of the tunnel face have a significantly higher strength than other parts. Operation parameters need to be adjusted and monitored accordingly.

As can be seen, the risks are mainly of procedural nature and rarely affect the surroundings in deep tunnels. Only shallow tunnels in soft ground suffer a certain risk of excess settlements in case of operational problems. With a close monitoring of operation parameters (process controlling), the volume loss can be controlled and kept at acceptable levels.

5 Numerical Settlement Analyses

In order to estimate the impact of tunnelling-induced ground deformations on the surroundings, three representative cross-sections have been analysed by means of finite element analyses for each alignment variant.

In the CUT AND COVER variant, different construction methods are employed. Therefore, the selected cross-sections comprise one representative analysis of the cut-and-cover method and two analyses of representative NATM sections. The short section using the "Tortoise" method has been exempted, since its situation was not found decisive for the feasibility of the whole variant. Firstly, because the trench is less deep than in most cut-and-cover sections and secondly, because the general behaviour of NATM sections without overarching slab is of more concern than with the slab.

For the bored tunnel variants, a total of nine cross-sections have been selected, which cover the most prominent aboveground buildings as well as the representative situations for all three alignment variants. This allows to extrapolate the analysis results for the complete alignment of all variants.

5.1 General Modelling Approach

The numerical analyses were performed by means of the finite element software PLAXIS 3D, version 2019.0, but considering a slice with 1 m thickness (corresponding to 2D plane strain conditions). For the cut-and-cover section, a slice of 2 m thickness was user, instead (see respective model description below).

The respective geometry of each simulation, i.e. the shapes of the ground surface, ground layers, ground water levels, position of the surface infrastructure, building, retaining walls, etc., is based on calculation cross-sections provided by Metroprojekt. Information on specific surface structures and foundation loads, where required, were as well considered as provided by Metroprojekt.

The ground is discretised by 10-node tetrahedral elements and considered as a homogeneous, linearly elastic, perfectly plastic material obeying the Mohr-Coulomb yielding criterion with a non-associated flow rule.

Rock mass is considered as described above for soft ground, and is thus idealised in a simplified way as a homogenous and isotropic material with "smeared" rock mass parameters according to the respective geotechnical investigation reports. Anisotropy of rock mass (due to stratification of the rock layers as well as rock joints etc.) is not taken in to account in the present analyses.

The in-situ stress field in soft ground was taken as lithostatic, considering a coefficient of earth pressure according to commonly applied relationship $K_{0,nc} = 1\text{-}\sin(\varphi)$ for normally consolidated ground. For rock mass, a bandwidth of possible in situ stress fields were analysed by varying the lateral stress coefficient K between $K = \nu/(1\text{-}\nu)$ and K = 1.0. However, it was found that the latter assumption of K = 1.0, which is a reasonable value for rock mass with high overburden, leads to lower surface settlements. The focus in the performed calculations was therefore set on the more conservative assumption of $K = \nu/(1\text{-}\nu)$, which generally holds for an elastic material undergoing 1D-compression with prevented lateral expansion. It has to be mentioned that the in-situ stress state can deviate considerably from this assumption, especially for heterogeneous rock mass or zones of previous tectonic activity (fault zones, rotation of principal stresses). However, for sections with high overburden, its influence on settlements at the ground surface is considered to be rather negligible. Taking into account the smeared (reduced) rock mass parameters and with regard to the present project phase with generally limited geological and geotechnical knowledge, the results can be considered reasonable and on the safe side.

The 2D-simulation of the construction process was carried out stepwise, starting with the calculation of the initial stress state. Depending on the analysed problem, i.e. trench/cut-and-cover section, NATM tunnel excavation or TBM tunnel construction, different approaches have been applied.

5.2 Cut-and-Cover Method

At km 4+250, a representative cross-section for the cut-and-cover sections was selected. It represents a complex situation using both a secant pile wall, anchored by strand anchors and a staged soldier pile wall, also anchored.

This cross-section features a high complexity for construction (two interacting retaining wall construction methods) as well as increased vulnerability to ground deformations due to a neighbouring building. It is also one of the deepest trenches in the project. The cross-section is shown in Figure 8.

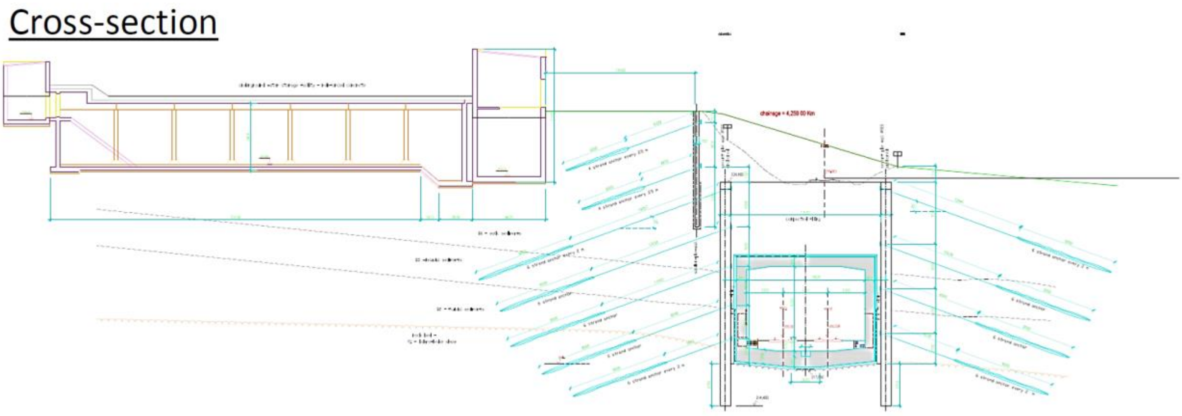

Figure 8: Cut-and-cover cross-section km 4+250.

For the cut-and-cover section, the construction process takes into account several construction stages with excavation levels approx. 1 m below the corresponding anchor row, until the final excavation stage is finalised, i. e. the bottom level of the trench is reached. The installation and pre-stressing process of the strand anchors was simulated realistically and in detail implemented as intermediate steps, always prior to ground excavation to the next excavation level, as this has a major influence on resulting ground deformation.

Details on the simulation stages, the model setup, and the calculation result plots can be found in Appendix A.

As a general observation, the shear strength and the stiffness of the soils in the considered cross-section are very weak. In combination with the adjacent building, which would require low deformations of the retaining walls, this imposes difficulties for the current design of the retaining system. The simulation results indicate that the staged retaining wall, as a whole, is at risk of structural failure. In this case, the tieback of the upper soldier pile wall is completely inside the failure body.

Stability can only be achieved mobilising very high anchor forces in the secant pile wall, which go up to 1000 kN. Even then, the deformations of the wall head are up to 20 cm. The reason for the unfavourable deformation characteristics are very low stiffness of the ground (E_{def} = 5 to 8 MPa) and a large depth of the trench of 23 m. It is therefore recommended to reconsider the choice of the tieback system in favour of a bracing structure with struts inside the trench in course of further design stages.

5.3 NATM Tunnels

The NATM Tunnel construction was simulated stepwise. In the first step, the tunnel top heading excavation and temporary support were simulated. In a second step, the bench/invert excavation and temporary support were modelled. The spatial stress relief and stress redistribution processes were taken in to account in a commonly used way according to the so-called stress reduction method, with implementation of intermediate calculation steps. Furthermore, reduced stiffness of the temporary shotcrete lining due to early loading, cracking and creep processes was considered.

The material properties provided in the geotechnical investigation report for the CUT AND COVER variant were applied in the simulation. Note that in order to compensate the insufficient consideration of the unloading behaviour in the Mohr-Coulomb model, the stiffness of the ground directly below the tunnel has been increased by a factor of 3. For further design stages, in particular for detailed settlement analyses, the use of more sophisticated material models is recommended. This requires the elaboration of additional material parameters, however.

Two cross-sections were modelled (see Figure 9):

- km 5+675: Shallow overburden underneath two retaining walls with buildings immediately adjacent to both sides.
- km 6+950: Tunnel adjacent to the heating plant with high foundation loads.

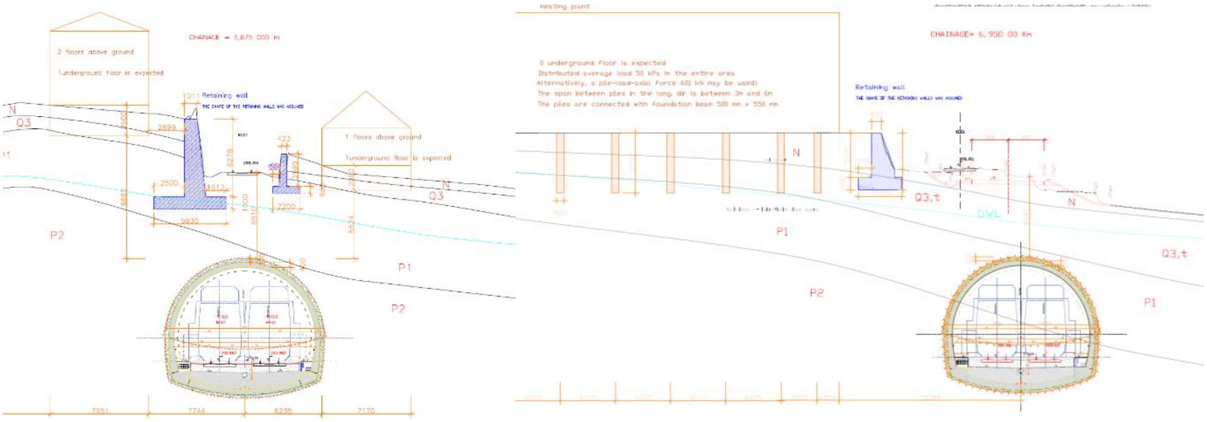

Figure 9: NATM cross-sections km 5+675 (left) and km 6+950 (right).

Details on the simulation stages, the model setup and assumptions, and the calculation result plots can be found in Appendix A.

According to the pre-design of the NATM sections, a conventional shotcrete tunnelling method is planned, using a top heading and bench/invert excavation sequence. The temporary shotcrete lining is designed at 30 cm thickness.

<u>Assessment of simulation results km 5+675</u>

Without auxiliary measures, the deformations are relatively high and may negatively affect the buildings next to the tunnel. To reduce ground deformations and prevent loosening up of the ground ahead of the tunnel face (and thus to reduce overall settlements at the ground surface), a short round length, fast closure of the primary top heading lining (e. g. by a temporary invert lining), forepoling and adequate face stabilisation measures, e.g. face bolting or a support core should be considered.

Assessment of simulation results km 6+950

Without temporary invert lining for the top heading, the results show indications of instability of the temporary lining during excavation and support of the top heading. To ensure tunnel stability and to reduce ground movement and surface settlements, a temporary invert lining for the top heading was applied for the calculations. A settlement-reduced construction without the temporary invert lining or other additional measures is not possible. Even with a temporary invert lining for the top heading, the deformations are still relatively large.

5.4 Bored Tunnels

5.4.1 Modelling Approach

The determining factors for the tunnelling-induced ground movements in closed shield tunnelling are:

- The active support pressure at the tunnel face,
- the overcut and taper of the shield, determining the steering gap,
- the effectiveness in terms of pressure and volume of the annular gap grouting,
- and the control of excavated material.

In 2D FEA, all these influence factors can only be approximated in a generalised way by calculating an intermediate step of stress relief and redistribution in the ground before installation of the segmental lining, in which the degree of stress relief (ratio of residual stress to initial stress $\beta = p/p_0$) is chosen in such way, that the assumed ground volume loss around the tunnel is reproduced in the calculation. As a result of the calculation, the influence of tunnelling on the ground surface and on buildings can be estimated. This is sufficient for an approximate assessment of vulnerability of adjacent structures (stage-1 damage assessment) in order to determine those buildings that should be further investigated in course of the design process.

The three bored tunnel variants, NORTH, CENTRE, and SOUTH generally pass the same geological zones from east to west: shallow-cover soft ground, high-overburden sound rock, and tectonic fault zones. For TBM tunnel construction different approaches were applied to simulate the influence of the tunnel drives on ground deformations, depending whether soft ground tunnelling, tunnelling through a fault zone or tunnelling in competent rock was analysed. The following model assumptions were made for the different cases:

Excavation in competent rock (open mode EPB)
No active support pressure is modelled in this case. The ground deformations are immediate after excavation and will be finalised before the lining is installed. Hence, the excavation step uses a reduction factor for relief of the in-situ stress by around 80%, expressed in terms of the residual stress to be taken by the primary lining ($\beta = 0.2$).

Excavation in soft ground (closed mode EPB)
A comparably high, active support pressure is assumed, which ensures face stability and reduces ground movements ahead of the tunnel face. Ground deformations occur mainly along the shield and in the tail gap. The ground deformations are is simulated assuming reasonable values for volume loss (VL) during the shield passage. Note that by the overcut and

the taper of the shield, some ground deformations can occur. Based on experience, a prudent estimate of

- VL < 0.25% (typical case for good EPB operation)
- VL = 0.5% (characteristic base case) and
- VL = 1.0% (worst case)

can be made. In the assessment of tunnelling impact for this Report, the characteristic base case and the worst case have been applied. Note that EPB tunnelling with negligible settlements is possible and more often than not the regular case.

<u>Excavation in loose rock/fault zones (closed mode EPB)</u>
An active support pressure is assumed to ensure stability and prevent shield jamming, yet has an insignificant influence on tunnel-induced settlements. Calculations are performed with reasonable stress reduction factors corresponding to the assumed support pressure. Fault zones are considered to have no major extent (smaller than the shield diameter) and are not predicted to run parallel to tunnel axis. Since 2D analyses represent a plain stain situation, any explicitly modelled fault zone would extend infinitely in axial direction. Therefore, the calculation results can be regarded to be significantly on the safe side.

5.4.2 Simulation Results

The model setup with screenshots from the discretised models and the result plots can be found in Appendix B. In Table 2, the simulation results are summarised. Those buildings that requires additional stage-2 assessment of settlements are marked with an asterisk.

Table 2: Simulation results, bored tunnels

Cross-section	Description	Relevant surface settlements
N 4+634	Below buildings in Buštěhradská Street, shallow cover, soft ground	VL = 0.5%: 14 mm VL = 1.0%: 30 mm *
C,S 4+285	Below Pod Hradbami Street, first building in the influence zone.	VL = 0.5%: 4 mm VL = 1.0%: 15 mm *
C,S 4+335	Below Pod Hradbami Street, first private building in the influence zone.	VL = 0.5%: 0 mm VL = 1.0%: 3 mm
N,C,S 7+300	Next to heating plant, shallow cover, fault zone	using support pressure: SP = 1.3 bar: 13 mm *
C,S 4+793	Below tram depot, rock with fault zone	β = 0.5/0.8: 5 mm
S 5+600	Deep below St. Norbert Church	β = 0.2: 6 mm
S 6+820	Deep below Military Hospital	β = 0.2: 7 mm
N 6+820	Deep below Military Hospital	β = 0.2: 7 mm

C 6+820	Deep below Military Hospital, close to slope, fault zone in Tunnel 1.	β = 0.8/0.2: 2 mm (10 mm above tunnel at slope)

Based on the simulation results for the bored tunnels, there are only three cross-sections that exhibit relevant surface settlements at the location of vulnerable structures: the NORTH variant at km 4+634 while passing underneath the buildings of Buštěhradská Street, the SOUTH and CENTRE variants while passing underneath the buildings of Pod Hradbami Street (yet with less severe settlements) and all alignment variants while passing the heating plant towards the west portal.

5.5 Assessment of Simulation Results

The CUT AND COVER variant employs different well-proven and established construction methods that allow for a design adjustment to meet any deformation criteria if required. However, given the present maturity level of design, the analysis results indicate very large deformations and partly stability issues that would need to be addressed when the design moves on.

It is to be expected that additional auxiliary measures such as the installation of temporary top heading invert lining, forepoling, or pipe umbrellas will lead to a significant effort in the NATM sections. For the cut-and-cover trenches, a massive increase of tieback measures or an internal bracing structure will probably be required. In either case, this will lead to additional effort and, thus, increase in costs and construction time.

Also the design of bored tunnel variants will need to deal with ground deformations, particularly towards the portal areas. The EPB method allows for tunnel excavation with very little volume loss, if carried out correctly and with continuous monitoring.

Based on the building damage assessment scheme after Burland and Wroth [2.11], a stage-1 assessment was performed using the simulation results as representative green field settlements. As assessment criteria, a total displacement of 12 mm at the level of the building's basement and a maximum inclination of η = 1/800 were chosen. Locations and buidings that exceed either of these values need to be investigated in a future stage-2 assessment, which incorporates increased detailing in modelling as well as more sophisticated material models.

Those locations that are recommended for a more detailed stage-2 assessment of settlements in course of the further design are:

- The east portal area for the NORTH variant (cut-and-cover section close to Bruska water plant).
- The buildings at Buštěhradská Street for the NORTH variant.
- The crossing of the Blanka tunnel ramp for the CENTRE and SOUTH variants (consideration is not part of this Report).
- The first building at Pod Hradbami Street (km 4+285) for the CENTRE and SOUTH variants (to a lesser extent than the portal area of the NORTH variant).
- The western portal with passage of the heating plant (for all variants).

In all other investigated areas, the total settlements, even with a worst-case volume loss of 1.0 % will be below 12 mm at the location of aboveground structures and the inclination of the settlement trough is expected less than 1/800.

In order to provide information on the expected settlement impact on the complete project area, the calculation results have been interpolated and extrapolated by means of a risk classification. Therefore, all buildings along the alignment are assigned to either of the following categories:

- Cat. A: Engineering required for risk mitigation
- Cat. B: Potential risk from settlements
- Cat. C: Low risk from settlements
- Cat. D: Influence of tunnelling probably negligible

For buildings in Categories A and B, it is recommended to further assess the influence of tunnelling during further design. Where required, engineering solutions for risk mitigation should be developed. The respective zones where buildings of each category are located are shown in a map of the project area in Figure 10.

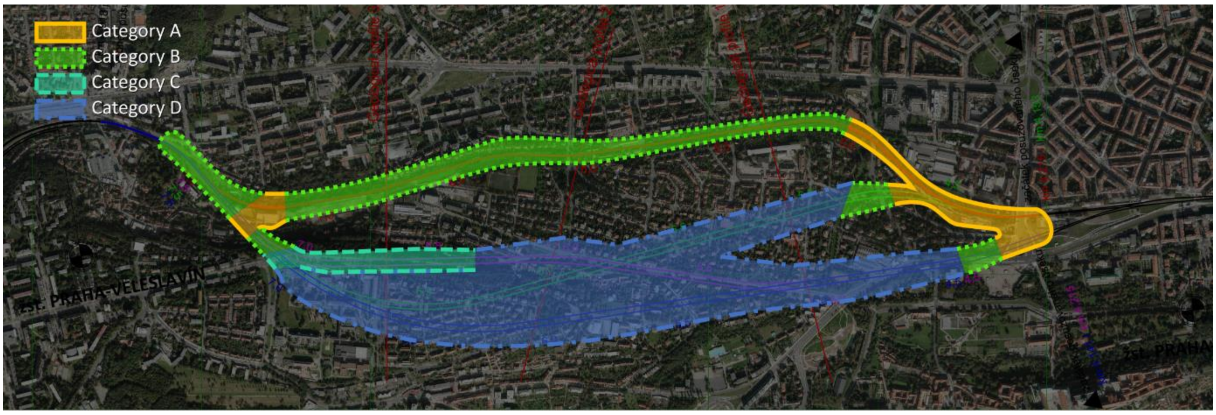

Figure 10: Risk categories of buildings in the project area.

6 Additional Questions on Specific Topics

6.1 TBM Passage of Fault Zones and Potential Connection of Aquifers

The presence of expected zones of higher permeability in tectonic fault zones has raised concerns about the potential connection of two distinct aquifers. To prevent this, it is important to mitigate drainage of ground water along the tunnel alignment.

Since also the mechanical properties in the fault zones require an adaptation of excavation mode and parameters, it is therefore recommended to perform exploration drilling or other prediction methods to detect fault zones ahead of arriving there. Upon mining through a fault zone, the EPB closed mode needs to be selected and an active support pressure is to be applied. If detection of a fault zone is impossible, once water inflow is detected, the screw discharge gate needs to be temporarily closed until the support pressure can be kept. A pre-emptive operation in transition mode is also possible, where compressed air in the excavation chamber is used to retain the ground water.

For the prevention of drainage along the finished tunnel, the annular gap between lining and ground needs to be properly grouted. It is further encouraged to consider injection ports in the lining segments, which can be used to perform a secondary sealing injection (either prophylactic or upon detection of leakages).

6.2 Construction of Ventilation Shafts

The mined ventilation shafts that will cross both aquifers need to be sealed upon sinking the shafts. This can be achieved by generating injection barriers between the aquifers. For this, a suitable injection pattern in at least two different levels needs to be designed, which depends on the injectivity of the ground.

In an example from a reference project, each barrier layer consists of at least four injection rows around the circumference of the shaft with an alternating offset of 0.75 and 1.0 m. Using jacket pipes and precast injection ports in the shaft lining, the injections can be conducted in a controlled and water-sealed manner. The length of the drillings depends on the ground permeability and needs to be sufficient to form an impermeable barrier. Typical drilling lengths are in the range of 3 to 4 m.

Typical materials for the injections are cement slurry or acrylate gel. In either case, the environmental safety needs to be checked. The actual pattern and materials need to be selected in the upcoming design process.

To ensure the quality and effectivity of the barrier injections, a continuous recording of injection pressures and volumes is required. Re-sealed test drillings between two barrier levels can be used for in-situ tests of the effectivity of the barriers.

If the barrier injections are conducted in good quality, the risk of connecting aquifers is low.

6.3 Additional Ground Investigations in Design Phase

In the design phase of the bored tunnel variants, it is recommended to perform additional ground investigations, particularly laboratory tests, on parameters that are relevant for EPB tunnelling. Figure 11, an excerpt from the latest revision of the DAUB recommendation for the selection of TBMs [2.9] gives an overview of the relevant parameters for EPB machines.

DAUB

Anhänge zur Empfehlung zur Auswahl von Tunnelbohrmaschinen

Anhang 3.5 Einsatzbereiche und Auswahlkriterien EPB

Geotechnische Kennwerte Geotechnical parameters	**Erddruckschildmaschine (EPB)** Earth Pressure Balanced shield (EPB)		+ Haupteinsatzbereich / main field of application O erweiterter Einsatzbereich / extended application – Einsatz eingeschränkt / application limited		
	Lockergestein (Soil)				
Feinkornanteil (< 0,06 mm) DIN 18196 Fines content (< 0,06 mm)	<5 %	5 – 15 %	15 – 40 %	>40 %	
	–	O	O +	+	
Durchlässigkeit k nach DIN 18130 [m/s] Permeability k [m/s]	sehr stark very high $> 10^{-2}$	stark high $10^{-2} – 10^{-4}$	durchlässig permeable $10^{-4} – 10^{-6}$	schwach low $< 10^{-6}$	
	–	–	O	+	
Konsistenz (I_C) nach DIN 18122 Consistency (I_C)	breiig very soft 0 – 0,5	weich soft 0,5 – 0,75	steif stiff 0,75 – 1,0	halbfest very stiff 1,0 – 1,25	fest hard 1,25 – 1,5
	O	+	+	O	O
Lagerungsdichte nach DIN 18126 Relative density	dicht dense	mitteldicht medium dense	locker loose		
	+	+	+		
Stützdruck [bar] Confinement pressure[bar]	0	1 – 4		4 – 7	7 – 15
	+	+		O	–
Quellpotential Swelling potential	kein none	gering little	mittel fair	hoch high	
	+	+	O	–	
Abrasivität (äquivalenter Quarzanteil) äQu [%] Abrasivity (equivalent quartz content) [%]	0 – 5	5 – 15	15 – 35	35 – 75	75 – 100
	+	+	O	O	–
	Festgestein (Rock)				
Gesteinsfestigkeit [MPa] Unconfined compressive strength [MPa]	0 – 5 | 5 – 25	25 – 50	50 – 100	100 – 250	> 250
	O | O	O	–	–	–
Bohrkern- Gebirgsqualität [RQD] Core sample - rock quality designation [RQD]	sehr gering very poor 0 – 25	gering poor 25 – 50	mittel fair 50 – 75	gut good 75 – 90	ausgezeichnet excellent 90 – 100
	+	O	O	–	–
Rock Mass Ratio [RMR] Rock Mass Ratio [RMR]	sehr schlecht very poor < 20	schlecht poor 21 – 40	mäßig fair 41 – 60	gut good 61 – 80	sehr gut very good 81 – 100
	+	O	O	–	–
Wasserzufluss je 10 m Tunnel [l/min] Waterinflow per 10 m tunnel [l/min]	0	0 – 10	10 – 25	25 – 125	> 125
	O	O	O	O	O
Abrasivität (CAI) Abrasivity (CAI)	kaum abrasiv hardly abrasive 0,3 – 0,5	schwach abrasiv slightly abrasive 0,5 – 1	abrasiv abrasive 1 – 2	stark abrasiv very abrasive 2 – 4	extrem abrasiv extremely abrasive 4 – 6
	+	+	O	O	–
Quellpotential Swelling potential	kein none	gering poor	mittel fair	hoch high	
	+	+	O	–	
Stützdruck [bar] Confinement pressure[bar]	0	1 – 4		4 – 7	7 – 15
	O	+		O	–

Figure 11: Appendix 3.5 of the DAUB recommendation for TBM selection [2.9] (EPB criteria)

The following parameters are crucial for EPB operation and may have a significant impact on the machine design and the operation parameters:

- Required support pressure: depending on the permeability of the fault zones and the amount of water inflow in these areas, the required support pressure may reach the "extended range" of application according to [2.9]. In this case, the TBM has to be designed to withstand respective pressures (e. g. gaskets, compressors, etc.).

- Consistency and clogging potential of the material in soft ground areas as well as of weathered rock and crushed material from the rock areas: This is required for the conditioning concept and the design of the conditioning system. This includes plasticity testing (Atterberg values) of all relevant soils and re-worked samples of those rocks, which contain clay minerals. Since plasticity testing according to geotechnical standards is only performed with soils, it is important to consider that clay-bearing rocks (mostly sedimentary) may be changed to a soil-like appearance upon excavation.
- Abrasivity: the abrasivity of the rock mass is of decisive importance in the design of the maintenance concept. Interventions in water-bearing areas may be difficult due to high pressures.
- Rock strength: Besides influencing the tool wear rate, also the excavation performance is strongly affected by the rock strength. The expected penetration rate is important in designing the supply chain, the segment production and the allocation of staff and plant.
- Elastic behaviour of the rock mass in all ground layers: for a detailed settlement assessment of the potentially affected surroundings, the un- and reloading behaviour should be assessed in further investigations.

6.4 Satellite-Based Surface Monitoring

Large-area surface monitoring by means of satellite-based radar interferometry is a proven method to complement the terrestrial tachymeter monitoring. While the resolution in both time and space is not sufficient to be used as the only monitoring system, it can be used to increase the survey area, to provide additional measuring points and thus increase the conservation of evidence and public acceptance of the project.

If specific radar reflectors are placed at different locations in the monitoring area, which are also equipped with tachymeter targets, the data can be correlated and the quality of measurements can be further increased.

6.5 Vibrations from TBM Tunnelling

Depending on the rock properties, the low-level vibrations that are exerted on the rock mass by excavating the ground using a TBM attenuate within a certain distance from the cutterhead. The permissible immission of vibrations is covered by ISO 2631 and its Czech national implementation CSN ISO 2631-2, which deal with low-frequency vibrations of 1 to 80 Hz.

According to the regulations, the most important aspect is the duration of disturbance. Temporary vibration immissions are permissible. The passage of a TBM is a temporary source of vibrations in any case: Either the excavation is fast, then the TBM will have passed quickly and the disturbance is short; or the TBM is in longer standstill. In this case, the vibrations will stop once the TBM does not excavate.

Along the bored tunnel alignments, there are two main sections regarding vibration assessment: the soft-ground shallow-cover area close to the portals (especially the east portal) and the rock areas with high overburden along most of the central part of the alignments.

With respect to vibration immissions, none of these areas is of high concern:

- In the shallow section, the soft ground has damping properties that attenuate the vibrations at small distances.
- In the rock section, the distance between the cutterhead and the ground surface is generally high.
- Experience from comparable projects shows that vibrations from TBM operation were usually not problematic.

It is recommended to install vibrations measurement devices along the tunnel alignment for the conservation of evidence. Along with transparent communication of the construction activities and the associated noise and vibration immissions, public acceptance can significantly improve. Depending on actual immissions, compensation or temporary accommodations of residents outside their homes are typical procedures.

Note that the considerations above hold only for the TBM itself. Vibrations and noise stemming from the aboveground site installations are not considered here. Also the tunnel operation after completion is not considered in the aforementioned statements.

6.6 TBM Operation for Minimisation of Settlements

As already described in Chapter 4.4, the safety and efficiency of TBM excavation depends on design and application of feasible operation parameters. The TBM needs to be well-maintained with functioning tools to efficiently excavate the ground at the tunnel face. It must be kept from clogging of the tools, the cutterhead openings, the excavation chamber, and the conveyance systems in order to maintain a controlled support pressure and to operate efficiently. The tail gap grouting must be controlled in terms of pressures and volumes to ensure minimal volume loss during excavation. Finally, the segmented lining needs to be installed according to the design, without leaking gaskets, damages and misplacements.

TBMs are equipped with a data acquisition system that continuously records hundreds of sensor values of all sub-systems of the machine. It is therefore possible and of crucial importance to observe, analyse and assess these measurements to obtain and keep detailed information on the excavation process and the TBM operation.

Using an integrated data management platform, surface monitoring data can be displayed and evaluated alongside with the TBM data to improve the level of information. Potential effects of operation decisions can be directly observed and the operators and shift engineers have access to all information they need for improved operation.

7 Comparative Assessment of Alignment Variants

7.1 Impact of tunnelling on specific structures in the area

In the appointment brief and scope, several prominent structures are identified that are to be assessed regarding their vulnerability to tunnelling-induced settlements. This list of buildings and infrastructure has been an important criterion in selecting the analysis cross-sections of Chapter 5. The estimated impact of green field settlements interpolated from numerical calculations is listed in Table 3 below.

Table 3: Impact of tunnelling-induced ground settlements on prominent structures

Structure/Building	**Affected by variant**	**Estimated green field settlements**	**Comment**
Institute of Physics of the Czech Academy of Sciences	NORTH	$S < 10$ mm $\eta < 1/800$	Note: vibrations from tunnel operation are exempted from our investigation.
Military University Hospital Prague	NORTH, CENTRE, SOUTH	$S < 12$ mm $\eta < 1/800$	High rock cover, negligible impact on ground surface
Water supply facilities Bruska	C+C, NORTH	$S > 10$ mm	Stage 1 analysis indicates necessity for second stage investigations
Blanka road tunnel	CENTRE, SOUTH	-	Note: Exempted from our investigations, see notes in text.
Střešovice tram depot	CENTRE, SOUTH	$S < 10$ mm $\eta < 1/800$	medium rock cover, negligible impact on ground surface
St. Norbert church	SOUTH	$S < 10$ mm $\eta < 1/800$	High rock cover, negligible impact on ground surface
Evangelic church Střešovice	SOUTH	$S < 10$ mm $\eta < 1/800$	High rock cover, negligible impact on ground surface

Veleslavín heat plant	All variants	S > 10 mm	Stage 1 analysis indicates necessity for second stage investigations
Heritage buildings in Proboštský Dvůr area	C+C	S > 10 mm	Vulnerable buildings at high risk of settlements directly next to retaining walls, Stage 2 investigation required
Buštěhradská St.	NORTH	S > 10 mm	Stage 1 analysis indicates necessity for second stage investigations
Pod Hradbami St., first building in tunnel influence zone (km 4+285)	CENTRE, SOUTH	S > 10 mm for higher volume loss	Stage 1 analysis indicates necessity for second stage investigations
Pod Hradbami St., first private building (km 4+335)	CENTRE, SOUTH	S < 5 mm η < 1/800	

7.2 Risks associated with each alignment variant

While the numerical settlement analyses and their predicted impact on structures along the alignment are the core of the assessment, other technical and organisational aspects also play an important role. Lacking detailed cost estimates in the current stage of design, the risk assessment in the following is based on a simple, non-weighted matrix that lists each risk aspect and a broad assessment whether the respective risk is high, medium or low for a given variant.

Table 4 contains this risk matrix for each alignment variant. Therein, only technical or general considerations are given. Based on the current level of design, construction effort and costs can only be qualitatively considered. It is assumed that the technical feasibility, the safety, and the potential for large public acceptance of the project outweigh the detailed monetary aspects. A '+' indicates low risk of a given variant, a 'o' indicates medium risk, and a '-' indicates high risk of a given variant. Positive and negative scores are assigned to each '+' and '-', respectively. At the bottom of Table 4, a total score for each variant is calculated, assigning a -1 for each high risk, a +1 for each low risk and a zero for each medium risk. Note that the individual aspects are not weighted and therefore, a weighted risk analysis may lead to other score values.

Table 4: Risk assessment of each variant

Risk (+ = low, o = medium, - = high)	CUT AND COVER	NORTH	CENTRE	SOUTH
Investigation and analysis effort				
Additional boreholes	+	-	-	-
Additional laboratory tests	o	-	-	-
Additional settlement investigations	-	o	o	O
Design risks				
Underpass Tunnel Blanka	+	+	-	-
Protection of buildings	-	-	o	o
Protection of heat plant	-	o	o	o
Sealing of aquifers	+	-	-	-
Shaft design	+	-	-	-
Cut-and-cover design	-	o	+	+
Construction effort				
Auxiliary measures in NATM	-	+	+	+
Auxiliary measures in C+C	-	+	+	+
Auxiliary measures for shaft construction	+	o	o	o
Auxiliary measures for EPB	+	o	o	o
Space for site area	o	o	o	o
Blasting required?	-	+	+	+
Geological risks				
Ground stability insufficient	-	+	+	+
Influence of ground water table	+	o	o	o
tunnelling parallel to slope	+	o	-	o
Deviation of ground parameters	-	o	o	o
Tunnelling risks				
Amount of soft ground tunnelling	-	o	+	+
TBM risks	+	-	-	-
NATM risks	-	+	+	+
Cut-and-cover risks	-	o	+	+
Impact on surroundings				
Settlements	-	o	+	+
Connection of aquifers	+	o	o	o
Vibrations from construction	-	o	+	+
Noise from construction	-	o	+	+

Risk (+ = low, o = medium, - = high)	**CUT AND COVER**	**NORTH**	**CENTRE**	**SOUTH**
Contamination of ground water	o	+	+	+
Portal areas (nuisance for neighbours)	-	-	o	o
Drainage of ground water	+	+	+	+
Summary				
Total Score, non-weighted	**-5**	**1**	**6**	**7**

7.3 Comparative assessment

As can be seen from the non-weighted score of Table 4, the CUT AND COVER variant is associated with the least favourable score. This is partly attributed to the incomplete design in the current stage. Given the technical possibilities to improve the design of retaining walls, the adverse impact on surface settlements can be handled in most cases. However, the inherent surface disturbance of the open trench construction remains. A large number of auxiliary measures required to handle the stability issues and deformations will render both the cut-and-cover and the NATM sections tedious and expensive. Furthermore, noise and vibrations from the construction will cause nuisance for residents along the complete alignment, instead being limited to the portal areas.

The NORTH variant inherits some of the cut-and-cover risks in its eastern portal area, where the passage along the Bruska water facilities is to be built in an open trench. Shortly afterwards, the bored tunnel passes underneath residential building at a very shallow cover in weak ground. This area, while technically manageable, bears the highest settlement risk of all bored tunnel alignments. With respect to public acceptance, there are concerns with directly underpassing the Physics Institute, which can be most easily met with an alignment off its premises.

The detailed design of the CENTRE and SOUTH variants will have to deal with the crossing of the Blanka Tunnel ramp. This aspect has been excluded from the considerations of this Report. Apart from this section, no large settlements are expected. A few locations (Pod Hradbami Street and western portal) will require more detailed settlement analyses with more sophisticated modelling in course of the further design. Yet these areas are less vulnerable than the building encountered in the NORTH variant.

After separation of the CENTRE and SOUTH variants, both alignments are deep tunnels in mostly sound rock. Fault zones are expected in all alignments. Their exact extent and orientation is unknown and has therefore not been distinctively considered in the risk assessment. However, the CENTRE alignment runs parallel to the slope of the Střešovice plateau, where one fault zone is expected. This provides a slight advantage for the SOUTH variant. Finally, the number of curves in the alignment is lower in the SOUTH alignment than in the CENTRE alignment, thus allowing to smoother rail operations.

As a result of the assessment, a preference for the SOUTH alignment variant is indicated. In the second place, the CENTRE alignment follows. The NORTH alignment is the least favourable of the bored tunnel variants, whereas the CUT AND COVER variant is the overall least favourable variant, given technical difficulties as well as disturbance of neighbours along the alignment.

8 Concluding Remarks

Summarising the alignment variant assessment, it can be stated that the depth of the design and the amount of ground investigations is very mature for the given design stage between feasibility study and pre-design. That said, it is clear, however, that several important aspects remain to be investigated as the design stages move on. This includes construction details specific locations such as the portal areas and the passage of some buildings, the design of retaining walls and NATM auxiliary measures in case the CUT AND COVER variant should be selected, and the design of the TBM and lining as well as the ventilation shafts for the bored tunnel variants.

If a bored tunnel variant is selected, additional ground investigations will be recommendable to determine ground parameters that are specifically important for the EPB design and operations. Furthermore, the ground properties with respect to injectivity and response to barrier injections need to be further investigated, in order to design the sealing of the aquifers for both bored tunnel and shaft construction. Further ground investigation may also detect further tectonic fault zones, knowledge of which may be employed in the improvement of design.

An integrated data management system for the combination of measurements from the surface monitoring, potential satellite monitoring, TBM data, logistics, site plant, and further data sources is strongly recommended to allow for tight supervision, conservation of evidence, improved internal and external communications and adequate response to potential hazards. It is further recommended to appoint independent expertise in EPB tunnelling and data management already in an early stage of the project to ensure a smooth tendering and procurement process. It is further advised to appoint independent data analysis and assessment expertise for the TBM, site, and monitoring data.