Kralovopolske Tunnels in Clays: Deformation’s Impacts on the Surface Buildings - Response to Excavation

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1. Kralovopolske Tunnels – General information

1.1. Technical information – description of the tunnels

Kralovopolske Tunnels are located in the town of Brno, Czech Republic, Europe. Low overburden, excavation mostly in clays, and middle sized cross section establish conditions for a remarkable settlement of the surface. On top of these unfavourable conditions, the area is quite densely populated with almost 160 buildings and engineering structures directly affected by the excavation.

Basic data: Kralovopolske Tunnels are named after the city ward Kralovo Pole in Brno, which is the second largest town in the Czech Republic. The tunnels form a main part of project called Dobrovskeho B municipal road ring. This project constitutes a part of the outer motorway city ring and is also an important element of the regional transit motorway system I/42 and of the international road E461.

Apart from the tunnels, this project includes two fly-over crossings adjacent to cut and cover sections of the tunnels and several additional connected constructions.

The client is the Road and Motorway Directorate. The contractor is a group of firms comprising OHL ZS, Subterra and Metrostav. The project has been designed by Amberg Engineering Brno. Gotechnical monitoring is provided by GEOtest Brno and ARCADIS Geotechnika companies.

Horizontal and vertical alignment: The key parts of the Kralovopolske Tunnels are two parallel two-lane tunnel tubes. Tunnel I is straight in the whole length under Dobrovskeho and Veleslavinova streets. Tunnel II is curved at both ends of driven parts in order to increase the axial spacing of the tubes from 15 meters at the beginning of the driven section to 90 meters in the central part. The tunnel tubes are interconnected by a total of four emergency cross passages.

The overburden thickness above the tunnels ranges from 4 to 21 meters. There is a sag 100 m away from the driven portal where pumping sumps for the tunnel drainage are located. From this point, the tunnel rises to both sides with gradient of 2.3% to 4.5% (Tab. 1).

Tab. 1: Basic technical data

<table>
<thead>
<tr>
<th>Length [m]</th>
<th>Cross section [m²]</th>
<th>Geological structure in tunnel profile</th>
<th>Overburden [m]</th>
<th>Driving</th>
</tr>
</thead>
<tbody>
<tr>
<td>total / driven / cut and cover parts</td>
<td>regular / emergency bay</td>
<td>prevailing / subsidiary</td>
<td>in whole / of clays</td>
<td>method / number of sections / excavation</td>
</tr>
<tr>
<td>Tunnel I</td>
<td>1239/1053/134+52</td>
<td>124 / 132</td>
<td>Neogene clay / quaternary deposits (sand and gravel)</td>
<td>4 - 21 / 0 -13</td>
</tr>
<tr>
<td>Tunnel II</td>
<td>1258/1060/146+52</td>
<td></td>
<td></td>
<td>4 - 21 / 0 -13</td>
</tr>
</tbody>
</table>

Cross section and tunnelling method: driven tunnels are double-shell structures with closed intermediate, hydrostatic pressure resistant waterproofing consisting of welded PVC membrane. The thickness of the primary lining is 350 mm while the minimum thickness of the secondary
lining is 500 mm. The tunnel excavation face is divided vertically and horizontally into six partial headings (Fig. 1). The basic rule says that the mutual horizontal distance between partial headings must not be shorter than 6 m during any excavation phase. Primary lining consists of shotcrete and steel, with highly rigid support elements. Rigid steel support elements are combined HEB beams and lattice girders. The basic excavation principle is to establish a very rigid lining and not to allow any ground deformations.

**Work schedule:** In advance to the main tunnel excavation, 3 geological exploratory galleries were excavated in 2002 and 2003. Two of them were excavated in tunnel II (2x831m) as two partial sections 'I' and 'II' of the main tunnel (Fig. 1). One gallery with the length of 365 m was excavated in tunnel I as partial section I. Because of some administrative and partially also technical problems, construction of the main tunnel started as late as at the beginning of 2007. Since this time, the excavation has been going without any serious problems (October 09). Till this time, 80 % of the total length have been excavated and both upper and lower vaults of the secondary lining are already being constructed. The excavation is to be finished in early spring of 2010 and the tunnels should be commissioned in the middle of 2011.

1.2. Geological conditions
The sequence of strata in the tunnel cross section is relatively very monotonous, i.e. secondary loess and anthropogenic fills with thickness between 3 – 10m, locally with layers of saturated gravel to sand terraces. The sub-base of the terraces consists of Neogene clay with the thickness ranging in tens of metres (the bedrock was not found even by boreholes over 60m deep). Groundwater is bound in gravel sand layers on the upper horizon of the Neogene in the form of aquifers in local depressions. The consistence of the Neogene clays is stiff, locally hard. In terms of plasticity, Brno clay is highly plastic; in combination with water, it is swelling and squeezing. The overburden thickness is approximately the same for both tunnels. It ranges from 6m to the maximum of about 21m.

1.3. Surface buildings
There is a total of 157 buildings affected by the excavation in the area of predicted subsidence. It is a typically varied urban development including different structures from the perspective of use, size, construction, technical state and ownership. Out of the total number of buildings it was possible to estimate an effect on 126 buildings in the area between Podebradova and Jana Babaka Street, where the major part of deformation response has gone through. Most of the buildings are private properties. Twelve buildings were bought by the investor in advance of excavation. These buildings are situated nearby the temporary portal at the place of the lowest overburden, thus the relatively most affected area. They are rented to the contractor.

2. Measures implemented to reduce negative impacts of excavation
To minimize deformations and the impact on surface buildings, the following technical measures such as micropile umbrellas, jet grouting, chemical and compensation grouting are being applied beforehand and during the excavation. Monitoring and its timely evaluation also contribute to minimizing damage on the surface.

2.1. Micropile umbrellas
This measure works primarily as tunnel face stability protection, but also as additional maintenance for surface settlement reduction. A micropile umbrella consists of 19 steel tubes (D 114/10 mm at intervals of 400 mm) drilled over the "V" partial excavation (Fig. 1). The length of the tubes is 16 m with an overpass of 8 m.
2.2. Jet grouting
Jet grouting works as the building's protection against the settlement impact. Thanks to this measure the width of area of subsidence is reduced and this leads to smaller building impact caused by uneven settlement. Vertical diaphragms from the jet grouting piles were carried out prior to tunnel excavation. Approximately 700 m of diaphragms were drilled under the tunnel tubes. The total of 59 buildings was protected by jet grouting shields.

2.4. Compensation grouting
This measure is carried out to reduce building deformation. Grouting is implemented while the tunnel faces underpasses the buildings. Twenty seven buildings were grouted in this way. Permanent monitoring of building settlement with results available online was a part of the compensation grouting technology.
A total of 28 buildings was protected out of which 23 were protected by jet grouting shields at the same time.

2.5. Monitoring of tunnel lining, surface and buildings
On account of the geological conditions, low overburden and above all the densely populated surface area, extensive geotechnical monitoring was performed. All monitoring data is available online via an internet interface based on an SQL server.

**Monitoring inside the tunnels:** there are two basic tasks of the underground monitoring in the Kralovopolske tunnels. The key one is the *convergence measurement* for both the primary and secondary lining. In the case of the primary lining there are cross convergence profiles in typical distance range of 15 to 30 metres which consist of 6 permanent and four temporary convergence points. In order to control the behaviour of the invert of the tunnel tube, one convergence point is placed in the middle of the primary lining invert. The *geological documentation* of partial tunnel faces is carried out on each upper face and once a day on the lower faces. In monotonous geology the most important is to give information about the consistency of the Neogene clays and even more important is the information about character changes to the clays which can warn against the proximity of the cohesionless geology strata whose presence in specific locations was predicted by the geological survey carried out. Additional underground measurements are those of tension and temperature measurement in the secondary lining and control measurement of the geometric location of the primary lining.

**Monitoring of the surface and buildings:** hundreds of monitoring components are installed to monitor the deformation of the rock mass, the surface and the buildings above tunnel tubes. The most important method of surface measurement is *levelling*. There are levelling points on buildings (usually on the outside walls but also occasionally on inside walls), on the engineering structures (tram lines and pipes) and there are cross levelling profiles usually placed in the lateral street lanes above tunnels. Levelling is naturally used to measure settlement but it is also used for indirect, but very important, determination of inclination of the surface and of the buildings. Close to the tunnel tubes extensometers, inclinometers and piezometers are placed. In some of the buildings chosen by a structural engineer *deformation metres* and *tilt metres* are installed to measure the thickness of cracks in walls. Gas concentration is measured in the air above outside gas pipelines and inside in the buildings using automatically or manually operated apparatus.
However, damage is not just caused by deformation of the surface above the tunnel. That is why *noise, air pollution, seismic and hydro monitoring* is performed as well.

2.6. Buildings: initial conditions and its reinforcement

2.6.1. Types of constructions and its foundations
According to the size and construction we can divide the buildings into three types (Tab. 2).
It is possible to divide these buildings into several groups regarding the type of foundations:

1. reinforced concrete foundation plate, about 4 buildings in group C
2. reinforced concrete strip foundation and foot, 6 buildings in group A, 4 buildings in group B, 38 buildings in group C.
3. bricks or stones strip foundation – remaining buildings

2.6.2. Preventive statically reinforcement

All static and protective reinforcement was designed for tolerable changes in constructions. Use of reinforced components takes into consideration minimal impact to constructions with maximum regard to the use of the buildings. The static reinforcement was designed to be temporary for most of the buildings. It is supposed to be removed or kept if the owner agrees or replaced with a similar less visible version. The removal of reinforcement will be done after a guarantee from experts who will evaluate monitoring data and future safety of the construction.

The reinforcement components used most are steel frames and steel draw rods (Fig. 3). Frames, designed as closed components made from U or L shaped profiles, were installed on edges of weakened wall orifices (windows, doors and lintels). Connections of profiles are welded and frames are fixed on sides with screws and wall plugs. Other reinforcing components protecting the building from splitting open are steel draw rods which stabilize the building in the level under the ceilings.

The rods are based in spread footings which are, if possible, leaned against outside walls. Other installed components of static support are, for example: timbering of vaults, reinforcement of floor girders, reinforcement with steel bars and shotcrete, wooden and steel braces.

Some of the buildings were not reinforced. It was not possible to effectively reinforce swimming pool walls and some of the owners did not agree with standard reinforcement.
3. **Deformation caused by tunnel excavation**

Deformation of the rock mass and surface occurred in two stages, related to the excavation of exploratory galleries and the tunnels. This article mainly focuses on the effect of the main tunnel excavation, but we should keep in mind that most of the buildings were already affected, during gallery excavation. Occurrence of deformations was interrupted and buildings were statically secured in the time between excavating the galleries and the tunnels. Measures for static security were also taken.

3.1. **Convergence measurement**

Convergence measurement in tunnel tubes: total maximum deformation in the plane of the convergence profiles usually ranges from 35 to 70 mm (Fig. 4), whereas the first and the second warning levels are defined (excluded other criteria) by the level of 50 and 70 mm. According to these criteria, the construction is safe and economical. In this point of view the static calculation and mathematical model of the tunnel were correctly enumerated.

3.2. **Deformation of the surface**

**Exploratory galleries:** typical settlement of the surface was between 20 and 50 mm, only once was it 63 mm. Higher settlements in this range were measured in profiles with worse geological conditions in the first part of mined tunnels, which might have been caused by defluxion of gravels and sands and a drainage of underground water.

**Tunnel tubes:** the absolute deformation caused by the excavation of main tunnels was naturally higher than the deformation developed at the time of excavating the galleries. Deformation of the surface ranges from 30 to 90 mm above the tunnel axis and the maximum inclination of the surface is about 1:200. The extent of the subsidence area trough is somewhere between 110 and 170 metres. Inflection is 1.0 to 1.5 of tube diameter, i.e. 12 to 20 metres.

3.3. **Impact of surface deformation on buildings**

Overall we can say that the extent of impact depends on the size of the building, location against the tunnel axis and area of subsidence and type of foundation. But from our experience so far, it mainly depends on the technical state of the buildings. Type ‘A’ and ‘B’ buildings which have smaller or more or less the same ground plans as the width of the tunnel tubes are relatively less influenced by the excavation and settlement than others. Buildings which are on the inflexion line...
were influenced by the excavation more. One of the most affected buildings is D4, where the shrinkage joint between the main building and court part extended. Another example is a house on Po26 which was in a bad technical condition caused by unprofessional construction works. ‘C’ buildings have at least one dimension several times larger than the width of the tunnel tube. Out of these, the ones with a longitude axis oriented perpendicularly to the tunnel axis were most affected by excavation and settlement. As an example: the building of clinic D23 and buildings in the area of military clinic D25A and D27C. In the case of the last two buildings mentioned, the effect of the technical state is related to the level of impact caused by mining. Both buildings are of approximately the same height and the size of the ground plan and their location to the tunnel axis is similar. Nevertheless, the damage to D25A, which was built at a lower quality, are more serious than in D27C. D27C has some cracks as well, but smaller and mainly in dilation locations. P28 is another building strongly affected by the mining. It had not been inhabited for a couple of years and it was in a state of disrepair at the beginning of tunnelling. In spite of significant damage, the building has not collapsed thanks to static reinforcement of the rock mass.

According to the real level of damage, we can divide those 126 buildings into four basic categories:

I – buildings practically undamaged, no repairs needed
II – small damage, hairline cracks up to 1 mm, most of the repairs will be done after stabilizing all the subsidence, repairs will mainly be of an aesthetic character (painting, plastering some cracks)
III – larger damage, cracks above 1 mm, damage to weakened wall orifices. Some of the repairs had to be done during operative maintenance, during excavation mostly without operative static measures, repairs of a static character are not generally expected even during the final reparation
IV – significant damage, operative static maintenance needed, more complex repairs to be done after settling.

Based on this classification, we can say that only six buildings out of all of them are in category IV. To be specific these are Ve1, P28, Po26, D25A (Army polyclinic), D29A (swimming pool) and P11. All these six buildings are situated either right above the tunnel or the closest part of the building is at the most five metres away from the surface tunnel projection. These buildings are relatively big, regarding the categorization mentioned in category C (see 2.2.). All six buildings had been in a bad technical state before the beginning of excavation. Two of the buildings were not being used at all because of being in a state of disrepair, reconstruction had been planned for the buildings of the swimming pool (it was built in the nineteen seventies and no reconstruction work has been done so far). Except for buildings V1 and P11 it was not possible to use measures for reducing the effects of excavations (no jet or compensation grouting were used). The maximum settlement of these buildings ranges from 28 mm to 83 mm, tilts reach levels ranging from 1:160 (an exceptional figure for the building itself) to the lowest tilt 1:750. The only levelled building out of this group was P11, where levelling lowered the total settlement, but the building could not be completely protected from static damage due to its technical state.

On the other hand there are only five buildings in the area of the predicted subsidence area which were not affected by the excavation at all – category of damage I. In this case, the major part is played by a small ground plan and the construction of the buildings. These are, in all the cases, tiny non-residential buildings where the high building solidity results in no damage, even when subjected to a relatively high, uneven settlement.

By far the biggest group are buildings in category II and III – that means buildings affected on a lower or greater scale by excavation, but this damage has not resulted in any additional static measures.

In category II (hairline cracks up to 1 mm, mostly aesthetic repairs after stabilizing settlement), there are 51 out of 126 buildings.

In category III (cracks above 1 mm, orifice damage, and operational repairs) are 61 buildings. In this case it is difficult to state the main criterion as to why the building is damaged to a greater or lesser extent. The key role is played by the distance of the building from the tunnel and the resulting total maximum settlement and tilt of the building. Buildings with less damage (category
II) have on average about 10 mm lower settlement and roughly half the tilt of buildings with more damage (category III). The technical state and measures for reducing the effects of excavation also play an important role.

### 3.3.1. Specific examples of damaged buildings

The below-mentioned examples are buildings in categories III and IV, situated right above the tunnels or in close proximity. None of the buildings mentioned was directly protected by jet or compensation grouting.

**D25A:** this is a six-storey building with one basement and three wings for student accommodation and polyclinic in the area of the University of Defence.

**Basic information:** longitude supporting system, strip foundation, walls of reinforcement concrete in basement, brick walls on upper floors up to a thickness of 0.6 m. Ceilings are made of concrete panels.

**Deformation:** middle and southern parts were affected mostly by tunnel II, and the northern part by tunnel I. Highest settlement: 67 mm and tilt of walls: 1:385 were measured in the middle tract. Damage to the building caused by deformation: mainly in the middle and southern parts of the building, two symmetrical systems of shear cracks in all floors are highly visible. Cracks are in both bearing and separating walls – they are visible better in separating walls. Cracks often progress through several floors with thicknesses of up to 3 mm in bearing walls and 8 mm in separating walls.

**Fig. 5:** Situation of building D25A and the tunnels. Arrows show the direction and ratio of tilt.

**Fig. 6:** Typical damages to building D25A – problematical dilation between northern and middle wings (left and middle) and typical shear crack

**Fig. 7:** Shear cracks in a staircase wall; partially pulled down glass wall due to the danger of ‘glass shooting’; typical shear crack leading from a corner of the window.
D29A swimming pool: a specific example is one of the open air swimming pools which are situated very near to tunnel II.

Fig. 4. Situation of the swimming pool with location of longitude cracks, the tunnel tube and tilts

Deformation: the highest settlement of the pool is 39 mm, the maximum tilt ratio is 1:502. The flex point is about 20 m away from the axis of tunnel II, which approximately matches the location of the main cracks in the wall.

Damage to the pool: the main cracks in the pool are through the whole thickness of the walls and were mainly situated in construction joints, which work as dilations due to improper connection during pools construction. Cracks are oriented along the tunnel; their maximum width is about 10 mm. The excavation was executed at the end of summer with the pool in operation. Temporary taping was applied in the pool to prevent water leaking. Nevertheless some loss of water was documented, which were compensated to the owner of the pool. Further reconstruction work will be needed. The investor is going to support the reconstruction financially even though it had been planned by the owner (the city of Brno) before.

Podebradova 28: The courtyard of this building is right above tunnel II. This part was built in addition to the main four storey building. The construction of this part was done carelessly and was made of many different construction materials. Because of this fact even low settlement and tilt caused serious damage to this building. One of the walls had to be supported by a wooden frame. In comparison to this building, the neighbouring building was well built with foundations made of concrete invert. That’s why this building was only slightly influenced by the settlement.

D23 – polyclinic: This building represents different deformation response to the underpass of the tunnel tubes I and II. The building showed no deformation impact when passing through with tube II. The only important deformation impact was observed during tube I tunnelling. This happened due to the vertical position of the jet grouting piles carried out prior to tunnel excavation. The piles were originally designed to protect Ob40 in the area of the Veterinary Faculty and considerably decreased the deformation impact of tube II excavation. Values of settlement for this building were about 25 mm. The most serious damage was found in the area of dilation.

4. Conclusion
All the construction work on the Kralovopolske Tunnels is performed with respect to minimizing surface deformation and its negative impact on buildings. Despite difficult initial conditions, the excavation has been going on without any significant negative reactions from local inhabitants and building owners. This has been possible thanks to the coordinated effort of the investor, contractor, designer, construction supervisors and monitoring engineers.

So far, we have been repairing only functional, dangerous or unsightly damage. These repairs are operational and very effective, but provisional only. The final repairs and financial compensation will be more technically and financially challenging. This will take place in the final stage after subsiding all deformations caused by the excavation.