Analysis of Deformations Prior to the Brezno Tunnel Collapse in the Czech Republic

A. Rozsypal¹, O. Kostohryz¹
¹ARCADIS Geotechnika, Prague, the Czech Republic

1. Introduction

Primary lining collapse and caving in the length of 80 meters occurred during Brezno tunnel construction in May, 2003. This paper analyzes the deformation behavior of the rock massif – tunnel lining system. Special attention has been paid to analysis of the measured deformations. Possible variants and hypothetical physical causes of the deformation process, which resulted in sudden progressive damage to the rock massif resulting to final tunnel collapse, are analyzed here.

2. Main Information about the Project

Brezno tunnel forms part of Brezno u Chomutova – Chomutov railway by-pass in the north-west of the Czech Republic. Construction of the by-pass with the length of 7.1 km was necessitated due to brown coal mining advance in Libous opencast mine. Brezno tunnel is a single-track railway tunnel with theoretic excavation profile of 63.8 m² and with finished cross-section of 43.7 m². The tunnel is 1,758 meters long, with driven part length of 1,478 meters (km 1.242 through to 2.720). Maximum overburden thickness is approximately 25 meters. The tunnel is partly straight and partly bent with radius of 550 meters. Tunnel’s angle of gradient is between 8 and 10‰, with total superelevation of 130 mm.

3. Mining-Geological Conditions

Brezno tunnel driving using the pre-vaulting support method commenced in from southern (Brezno) portal in February, 2002. In May, 2003, the driving was interrupted due to primary lining collapse and tunnel caving at chainage between 2.030 and 2.106 km. Between September, 2004, and March, 2007, tunnel driving was completed, after 15 months interruption, by conventional sequential method from the northern (Drouzkovice) portal. No blasting was used to loosen the rock during driving. All works (including drilling) were performed as “dry”, with no technological water.

The length of each pre-vault was 5 m. The pre-vaults were overlapped each other by 0.5 to 2.5 m. The depth of the ‘footing’ beneath each side of the arch was approximately 0.5 to 0.7 m. The bottom of the tunnel was temporarily stabilized with road panels and shotcrete. Number of radial bolts was 2 x 5 and the bolts supported the open cut during final invert excavation. The final invert was built under 11 pre-vaults.
Convergence points were installed in profiles, whose spacing was originally 30 m. When abnormal deformation occurred, the points were installed on the each pre-vault. For evaluation of the deformation behavior were used maximum values of vertical deformation in crown and maximum values of horizontal deformation in both side walls.

Driving took place in the North-Bohemian basin, in the environment of Neocene clays and coal. At the southern (Brezno) portal between chainages 1.242 and 1.700 km, the tunnel was driven in a heavily undermined area; old mines were confirmed both in the subgrade and in the tunnel profile during the course of construction.

4. **Threshold Concept and Criteria**

For the purpose of tunnel geomonitoring evaluation, it was necessary to specify alert states in the deformation behavior of the underground structure and surrounding rock. Threshold values were derived from the value identified in static calculation. Geomonitoring design has determined four warning levels.

a. State of acceptable changes. Measured values are stable and do not exceed the deformations envisaged in the implementing documentation applicable to the particular driving phase. Geological conditions also correspond to the implementing documentation assumptions. The face is stable. Criterion consists in achieving approximately 60% of the acceptable limit envisaged in the static calculation.

b. Limit acceptability state. Limit acceptability state is the state between the state of acceptable changes and the critical state. Face is in principle stable, no cracks in the lining occur. Criterion consists in exceeding the value of 100% envisaged in the static calculation. Measured deformations are not stable but they have the tendency to stabilize.

c. Critical state. Critical state means such development in the behavior of the rock - tunnel lining system, which would result in emergency state if no extraordinary measures in driving method were adopted. Deformations are not stable and are characterized by steady speed. Soil blocks fall of the face. Cracks may form in the lining. Criterion consists in exceeding approximately 125% of the acceptable limit envisaged in the static calculation.

d. Emergency state. Deformations start to grow progressively. There is a risk of extraordinary situation in terms of valid legislation at the construction site. It is necessary to proceed in accordance with the approved emergency plan in line with relevant mining regulations.

5. **Geotechnical Characteristics of the Rock Environment**

Four quasi-homogeneous segments were determined during the driving in the area between the southern (Brezno) portal and the point of tunnel collapse.

5.1. **Portal Section between 1.242 – 1.550 km**

Tunnel driving commenced in the environment of grey clays (claystones), partially affected by frost weathering in the upper part of the tunnel and in the environment of coal seam with intercalations of clays in the bottom part of the tunnel, while the coal seam inclined under the invert of the tunnel at the approximate angle of 5°. The coal seam was documented in tunnel profile up to the chainage of 1.380 km. This section was characterized by intensive undermining. Undermining was clearly verified by reached caved and in advance remediated horizontal and vertical mine works in the face of the tunnel. Although the mine works were amply timbered in several cases, no open spaces were reached during the driving, also thanks to the advance remediation works. In addition to old mine works, caving of old mine works, failure of rock layers, etc. were encountered. Despite of that, the face was mostly stable. Prediction of the scope of face influencing by old mine works was very difficult as the system of old mine works was irregular and no archive documentation regarding the massif development was available.
Initial platy structure gradually changed to sheeting or massif structure. No tectonic faults were encountered. Underground water appeared only rarely at the face, usually in form of bleeding in the coal seam affected by historic mining. Otherwise, the face was dry. The face was stable.

Driving was characterized by increased deformations of pre-vaults. Critical state for horizontal deformations (20 mm) was reached in 16 profiles, although the value of horizontal deformation remained below 30 mm. The deformation in vertical direction remained below 55 mm (see Fig. No. 2).

**Fig. 2 Development of vertical and horizontal deformation during initial 30 days after lining construction in section between 1.242 and 1.550 km**

5.2. **Section between 1.550 – 1.920 km**

Driving in this section took place in the environment of monotonous grey claystones with no coal inserts. Position of clayey siltstone colored yellow with limonite (20 – 30 cm thick) occurred in the top part of the tunnel around 1.660 km. Stratification gradually changed from sheeting or massif to platy or even thin-platy.

Tectonic damage of the massif resulted in minor subsidence, overlaps, and flexures with stratigraphic throw height in the range of initial meters. At chainage 1.875 km, a 20 m³ block came out from the face over tectonic line filled with limonite. Otherwise, the face remained stable. The face was dry all over the section in question.

Favorable deformation response was typical for this section. Critical state was not reached. Values of horizontal deformation stayed below 20 mm. Value of vertical deformation exceeded 20 mm in rare cases only (see Fig. No. 3).

**Fig. 3 Course of vertical and horizontal deformation during initial 30 days after lining construction in section 1.550 through to 1.920 km**
5.3. **Section between 1.920 – 2.030 km**

Driving in this section continued in the environment of monotonous grey claystones with prevailing platy or thin-platy stratification. Positions of darker clay together with positions of yellow clay (solid to hard consistence) started pursuant to appear in the top heading area. Positions of clays have gradually spread all over the tunnel profile. Small scale tectonic effects were registered. The face remained stable, with no underground water occurrence.

Highly unfavorable development of deformations was registered in this section, when the values of horizontal deformation exceeded 70 mm and vertical deformations achieved up to 100 mm. Values of the critical state were exceeded by several times. Attempts to stop the deformations propagation by proven procedures (anchoring, remediation boreholes under the tunnel invert, reinforced concrete rings insertion) appeared to fail initially. Development of deformations stabilized after profile closing and final invert construction. This resulted in complete stopping of deformations.

It is necessary to admit that a considerable part of the deformation took place in direct response to remediation measures execution, in particular to insertion of reinforced concrete rings when it was necessary to temporarily lower the invert level in the area of the future ring. Increase of the deformation varied between 20 and 50 mm (see Fig. No. 4). Works on tunnel stability assurance in this section lasted approximately 1 month. Face in chainage 2.034 km stopped for 32 days.

![Graph showing development of vertical and horizontal deformation](image)

5.4. **Section between 2.030 – 2.106 km**

In this section, driving continued in the environment of grey clays with predominating platy or thin-platy stratification. Positions of dark clays with solid to hard consistence alternated with position of yellow clays with solid to hard consistence. Tectonic effects were rare. No underground water was observed.

Deformations development was slightly more favorable than in the previous section. Despite of that, the critical state values in horizontal direction were exceeded at several profiles. For this reason, it was decided to close the tunnel invert and construct the final lining in the invert section. Subsequent progressive growth of deformation did not allow completion of the works. After 2 days from deformation development acceleration, the primary lining collapsed in the length of 80 meters and the tunnel was caved on May 5, 2003.

6. **Detailed Development of Deformations in Section 2.033 – 2.106 km**

After remediation of the prior section, the driving was re-started on April 4, 2003. Driving recommencement also revived the deformations in recently stabilized section between pre-vaults 176 and 171.
The pace of deformation growth in the section between pre-vaults 178 and 184 decreased in several days. Approximately after April 16, 2003, measurements revealed slow but continuous growth of deformations having the character of slight secondary creep. However, absolute values of achieved deformations were below 25 mm (limit acceptability state) until April 25, 2003. Compared to the deformations identified during advance through the previous section between 1.950 and 2.033 km, the present situation was considered to be more favorable (see Fig. No. 4 and 5).

Once it became apparent from to the deformations development that the limit acceptability state would be exceeded in several days, the construction site adopted measures specified in the design documentation to maintain the deformation development within acceptable limits (for example extension of radial anchors from 6 to 8 meters). As these measures proved to be insufficient, the attack length was reduced to 3 meters on April 23, 2003, from pre-vault No. 189. However, this also resulted in reduction of the number of radial anchors per pre-vault from 10 to 8 with length of 6 meters only.

The limit acceptability state in the vault was reached on April 28, 2003, from pre-vault No. 185. With regard to the experience with deformation development gained in the previous section (chainage 1.922 through to km 2.033 km), it was proposed to immediately commence with invert execution. However, this requirement was rejected by the financing organization. Only the number and length of radial anchors were modified (10 pieces, length of 8 meters). For technological reasons, the attack length was increased to 4.4 m from pre-vault No. 193.

On May 2, 2003, the parties to the construction performed a detailed inspection of the construction and analyzed the metering results. They stated that there was no risk of sudden increase of the deformation process speed. However, the proposal to complete the invert between 2.033 and 2.106 km (pre-vaults 176 through to 190) was additionally accepted. Commencement with these works was scheduled to Monday, May 5, 2003.

On May 3, 2003, a sudden significant growth of deformations was registered at pre-vaults No. 182 through to 192. The greatest growth was recorded at pre-vaults 180 through to 191 (see Fig. No. 5). It was immediately decided to complete pre-vault 196 as soon as possible and secure the face by additional face anchors (11 pieces) and shotcrete layer reinforced with wire mesh.

On May 4, 2003, fast growth of deformations was confirmed by convergence measurements. At 4 p.m. of May 4, 2003, the contractor’s personnel reinforcing the pre-vaults registered effects of increased rock pressures on the pre-vaults. At 7 p.m., it was decided to immediately support the pre-vaults 184 – 190 with supporting lattice girders and to brace the lattice girders in the invert. During the night time, it was possible to build the lattice girders under pre-vaults 188 and 187 only and construction of supporting lattice girders under pre-vault 186 just started.

Despite of the effort to reinforce the most endangered areas with supporting lattice girders as quickly as possible, further development of deformations was so quick that it could not be stopped by any technical
measures. At 2 a.m. of May 5, 2003, the tunnel invert started to rise. Supporting lattice girders used for pre-vaults reinforcement started to deform towards the inside of the tunnel. Pre-vaults concrete started to crack off with bold sound effects. Evacuation of technology from the endangered area started immediately. Subsequent deformation development was monitored from a safer place behind pre-vault No. 175. Shortly before 5 a.m., the invert of the tunnel near the face rose by up to 40 cm. Immediately after that, approximately at 5 a.m., the pre-vaults closest to the face collapsed. Further pre-vaults up to the number 176 followed shortly with domino effect. Pre-vault 176 remained stable. However, even this segment settled instantly by 75 mm at the moment of the collapse.

Primary lining was crushed between 2.033 and 2.106 km. Subsidence trough with the length of 60 meters, width of 30 meters and depth of 3 – 4 meters appeared on the terrain surface immediately after the collapse. Volume of the subsidence trough was approximately 5,000 cubic meters. Two craters formed along the tunnel axis – one near the face and the other at the beginning of the caved section. The subsidence trough was backfilled with soil, leveled, and drained immediately after its formation in order to prevent water accumulation.

Fig. 6 Growth of vertical and horizontal deformation on pre-vaults 176 through to 192 immediately before the collapse

Figures No. 5 and 6 show that progressive failure of the entire section between 2.033 and 2.106 commenced almost at once all over the collapsed length. Rise of the progressive failure was very fast and unexpected. No similar development had been registered in any prior section of the tunnel driven in comparable conditions. Particularly surprising was the fact that such a rapid progressive failure occurred from significantly lower levels of initial deformations than those, which were safely managed during the driving of prior sections.

Even on April 29, 2003, the development of deformations still seemed to head towards the stabilization (see Fig. No. 5). Deformations of the last pre-vaults (No. 184 through to 191) did not reach even the limit acceptability state. Subsequent measurement performed on May 3, 2003, surprisingly revealed an apparent acceleration of deformations confirming progressive failure of the rock massif under pre-vaults 178 through to 191 (see Fig. No. 5 and 6).

Last measurement, which was performed on May 4, 2003, only confirmed further acceleration of the settlement and further growth of progressive failure of the rock massif under pre-vault footings. In the afternoon of the same day, the emergency situation became obvious and measures towards minimizing the loss of pending collapse were therefore adopted.

7. Description of the Loss of Stability and Collapse of the Rock – Tunnel Lining System

Neither the parties to the construction nor experts invited to comment on the cause of the collapse mentioned any doubts that the collapse had been directly caused by sinking of simple concrete footings of pre-vaults. Sinking was apparently caused by exhaustion of the load bearing capacity of the lining footings in the rock environment.
Extrapolation of deformation curves revealed that the rise of tertiary creep and phase of progressive failure occurred suddenly during May 2, 2003. Sudden change of the deformation behavior from the trend with very slow deformation growth far from the boundary of the limit acceptability state towards the trend of very fast acceleration of the deformation growth above the limit acceptability state eliminate the theory based on presence of rocks with a considerably lower strength (higher deformation) than those encountered in previous tunnel sections as the only cause of the tunnel collapse. Such rock massif would undoubtedly deform for a considerable longer time without an apparent tendency to deformation stabilization after relatively very short time, which were observed before April 28, 2003 at pre-vaults No. 183 through to 191 (see Fig. No. 5).

The failure process did not start at the face but in the area of pre-vaults No. 187 through to 189. This was in the distance of approximately 30 meters from the face.

It is probable that such a rapid change of pre-vaults deformation character must have been caused by multiple factors.

7.1. **Exhaustion of Load Bearing Capacity of Radial Anchors**

The first probable factor is the exhaustion of load bearing capacity of radial anchors. Radial anchors, which were 6 meters long in the area of the collapse, helped to prevent vertical settlement of the footings at smaller loads. However, their load bearing capacity was gradually mobilized due to settlement and growth of vertical deformations. This contributed to false and temporary stabilization of measured deformations. The load bearing capacity of the anchors was exceeded at certain size of the deformations. Their contribution to the load bearing capacity of the footings thus decreased rapidly. This resulted in very fast growth of progressive failure of the entire system.

7.2. **Lining Loading**

It is probable that one of the factors (although not the most significant one) was the growth of overburden thickness in "shallow" tunnel conditions with driving advance. Loading of the lining could have gradually increased with increased footings settlement, in particular in sections where the tunnel overburden was formed by rock with lower consistence. It is possible that this effect was caused by rock properties deterioration due to original wild coal mining. An unfavorable combination of these factors might have caused the rock vault effect to suddenly disappear.

7.3. **Lining Shape and Original Horizontal Stress**

U-shaped simple concrete primary lining was increasingly loaded by side earth pressure in horizontal direction at more significant settlement. There was no invert to strengthen the structure. In certain situation, the loads could have been transferred by radial anchors or steel rock bolts. However, their load bearing capacity was insufficient with regard to the conditions that occurred. Greater original horizontal stresses (than which would correspond to the overburden weight) were identified in other areas of the same tertiary basin in which the tunnel was driven (due to reconsolidation or tectonic stress). Neither the original nor the additional geological survey was aimed at proving or eliminating this factor. However, existence of greater horizontal stresses is supported by the fact that the measured horizontal deformations were relatively always higher than really measured vertical settlement (compared to the calculation) over most of the tunnel length.

7.4. **Brittle Failure under the Footings**

The footings were established on solid or hard clays. They were described as shattered clays. In the areas of omni-directional stress, the existing cracks do not significantly influence the rock behavior. However, the surface of the tunnel invert, in particular in the area of the lining footings, is characterized by highly negative stress state. The loading by the rock excavated inside the tunnel is missing. This unfavorable state, which considerably influences the course of the load – deformation line , becomes
worse with increasing distance from the face as the stress under the footings is favorably influenced by the rock of the face.

Once the loading of the lining footing reaches certain limit, the character of the deformation damage suddenly changes. Shear failure can instantly change to brittle failure. This is confirmed by a very high growth of the tertiary creep, which was identified by convergence measurement shortly before the collapse.

7.5. Pore Pressure Growth and Shear Strength Decrease to Zero

This process can occur in almost saturated clays. After certain deformation, the pores of the soil become saturated by water, water assumes the entire loading and the shear strength of such soil decreases to zero. This results in a very fast loss of stability. This description also roughly corresponds to the process of tunnel collapse, so it can not be excluded. However, no laboratory tests to prove the previous or this hypothesis, were performed as a part of the additional exploration.

8. Explanation of the Domino Effect Collapse of 100 Meters of the Tunnel

High speed of the collapse and its great scope were significantly supported by the fact that individual simple concrete pre-vaults executed under the driving technology do not mutually cooperate from static perspective. If one pre-vault collapses, the entire loading previously affecting such pre-vault must be taken over by both adjacent pre-vaults. These were not prepared to such sudden load increase. This resulted in "domino effect" when the pre-vaults collapsed one after another from number 189 towards the portal to pre-vault No. 176. This pre-vault was already extensively reinforced and provided with a heavy invert, so it succeeded to stop the domino effect. However, even this pre-vault suddenly settled by almost 10 cm during the collapse.

The above-mentioned mechanism of the rock – tunnel lining system stability loss might have been affected by a number of other factors as well.

9. Discussion about Certain Other Causes of Collapse

The above-mentioned mechanism of the rock – tunnel lining system stability loss might have been affected by a number of other factors as well. Several other possible causes of the collapse were discussed among the involved experts. They comprised in particular:

- sudden change of geological and hydrogeological conditions in the area of driving
- undermining
- gasification
- driving technology
- imperfections during driving or pre-vaults execution
- unfavorable shape of lining cross-section

Causes, why the construction site management did not respond with sufficient speed to different mining and geological conditions than those envisaged in the design consisted in particular in the method of tunnel construction financing and some organizational aspects deriving from this fact. However, this area does not form the subject of this paper.

10. References