ECCOMAS Thematic Conference on
Computational Methods in Tunnelling (EURO:TUN 2007)
J. Eberhardsteiner et.al. (eds.)
Vienna, Austria, August 27-29, 2007

NUMERICAL MODELLING OF THE BREZNO TUNNEL RE-EXCAVATION

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Keywords: Tunnelling, Railway Tunnel, Numerical Modelling, Sprayed Concrete Lining.

Abstract. *An intention to extend the surface lignite mine Libous caused requirement to relocate the railway line section between towns Brezno and Chomutov in Northern Bohemia. 7.1km long relocated railway section required 1.8km long single track tunnel construction. The Brezno tunnel is currently the longest railway tunnel in the Czech Republic. The shallow tunnel construction started using the Pre-Lining Support method (Perforex). The tunnel excavation in complicated geological conditions caused many difficulties which resulted in a significant collapse in 2003.*

A decision has been made to separate a collapsed area into 9m long sections using 16m wide-transversally oriented pile walls constructed from the surface and to re-excavate a collapsed area using Sprayed Concrete Lining (SCL). Also some other measures were done prior the re-excavation (ground improvement, micropile umbrellas embedded into pile walls, etc.). Detailed monitoring has been provided during construction (lining convergences, surface settlement,, etc.). Excavation and primary lining construction was completed in 2006. The tunnel was opened for rail traffic in April 2007.

Presented paper deal with a numerical modelling of the tunnel re-excavation. Calculations of the tunnel re-excavation were provided using 2D finite element method (software RIB). Further calculations to evaluate rock mass behaviour in collapsed area were provided using FEM software Plaxis. 2D calculations were realised to provide sensitivity studies, 3D modelling assisted to evaluate tunnel face stability (impact of the pile walls, ground improvement, etc.). Results of the modelling were compared with the monitoring results (back analysis). The paper also briefly describes a construction experience (technical problems, performance of various support measures, etc.).

1 INTRODUCTION

A construction of the Brezno tunnel with overburden up to 30m started using the Pre-Lining Support method (Perforex) in 2000. A tunnel excavation was realised predominantly in plastic clays and claystones, maximal thickness of quaternary deposits (gravels and sands) was about 6m. The area was also affected by previous undocumented mining activities. Very complicated geological conditions caused many difficulties which resulted in significant collapse in 2003. The collapse occurred when about 860m of the tunnel primary lining was completed. About 77m of primary lining were destroyed (chain effect of pre-vaults) and further 44m of the tunnel was filled with collapsed material. An excavation ceased for several months directly after the collapse.

A decision has been made to separate a collapsed area into 9m long sections using 16m wide pile walls constructed from the surface. The walls were formed from 1.18m diameter piles, the walls reached 3m below the tunnel profile. The collapsed tunnel was separated in longitudinal direction into 7 sections (Fig.1).

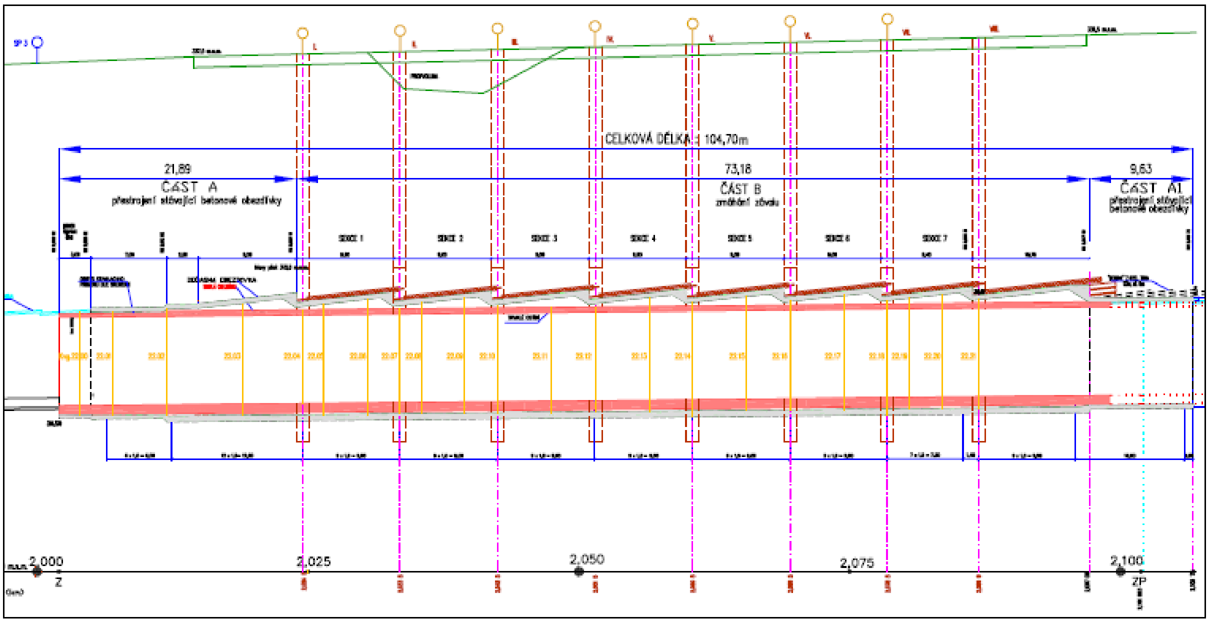

Figure 1: Longitudinal cross-section including separation of the collapsed area using pile walls

For re-excavation of collapsed area Sprayed Concrete Lining (SCL) method was used. The primary lining was designed as sprayed concrete reinforced by lattice girders and meshes, the tunnel face had to be excavated in several stages. The proposed excavation method had to be properly statically evaluated prior its application, all support measures had to be optimised.

Provided calculations were generated using finite element method (FEM). With respect to complexity of the problem, common 2D calculations were also supplemented by 3D calculations to verify some 3D effects (e.g. impact of the tunnel separation by pile walls).

2 INITIAL 2D STATIC CALCULATIONS

2.1 Basic data

Initial static calculations to design a primary lining and excavation sequence were generated using 2D FEM (plane strain model). The rock mass was modelled using linear elasto-plastic Mohr-Coulomb model, software TUNNEL 12.0 (generated by company RIB) was used for calculations. The primary tunnel lining was evaluated in interaction curves according to the Czech Standards (software BETON 2D by company FINE).

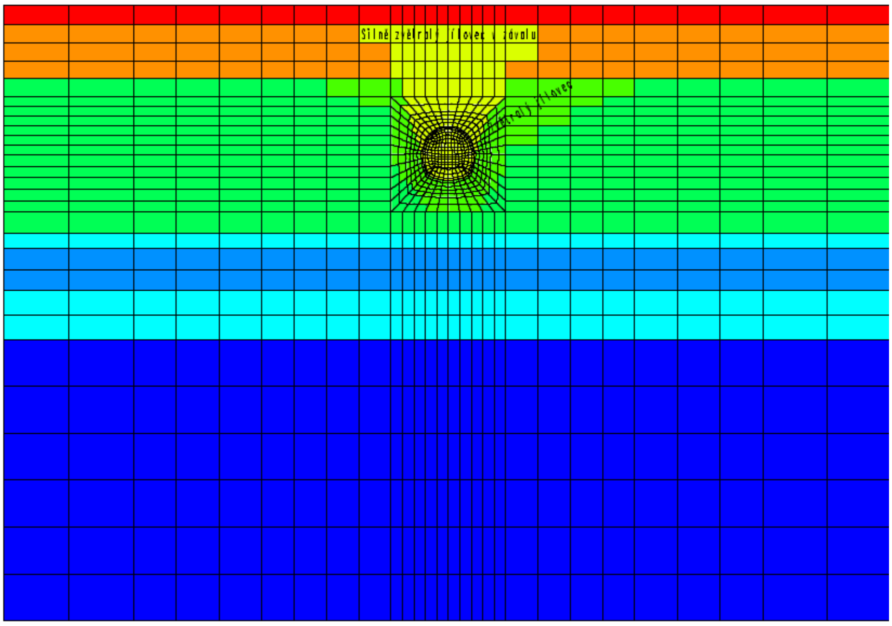

Figure 2: Geometry of the initial 2D model

Initial input parameters for individual geotechnical units are summarised in Tab.1, coefficient of the lateral pressure at rest was used 0.8. The model is presented in Fig.2. The input parameters were derived from a supplemental site investigation realised after the collapse.

Geotechnical unit	Input parameters				
	γ (kN/m^3)	c (kPa)	ϕ (°)	E_{DEF} (MPa)	ν
Quaternary deposits	19.2	11.5	18	17	0.30
Strongly weathered claystone	19.2	11.0	10	19	0.40
Collapsed material	19.2	11.0	8	19	0.40
Weathered claystone	19.5	17.0	19	19	0.40
Claystone A	19.5	36.0	19	32	0.40
Claystone B	19.5	40.0	20	35	0.38
Claystone C	19.5	45.0	25	50	0.38
Coal seam	19.5	30.0	25	60	0.30

Table 1: Input geotechnical parameters

2.2 Primary lining calculations

Generated static calculations modelled an excavation and support installation in several stages (top heading, bench, invert), the model included two types of sprayed concrete – three days old green sprayed concrete (strength 10MPa) and sprayed concrete with final parameters. The lining thickness was 35cm. Top heading lining was expected to be regularly closed by temporary invert which is a crucial measure to reach equilibrium in similar geological conditions. Also of the lining geometry plays a very important role to minimise bending moments (smaller excentricity). Thus lining geometry was optimised.

Calculated maximal axial forces were in the interval from 1500kN to 2450kN depending on stages of excavation, final bending moments are presented in Fig. 3. Evaluation of all results confirmed propriety of 1m top heading advances, bench and invert advances were designed longer. Calculations confirmed that maximum deformations of the primary lining should not exceed 50mm, monitoring during construction generally confirmed these expectations (Tab.6).

The shape of temporary top heading invert was designed as compromise between a static fitness and a space requirement for machinery. The temporary invert was partly designed from in situ cast concrete; requirement for sound connection of sprayed and in situ cast concrete had to be fulfilled (strength should not exceed 50% of the final strength in time of connection). The shape of permanent invert was more appropriate from static view as no compromises were required.

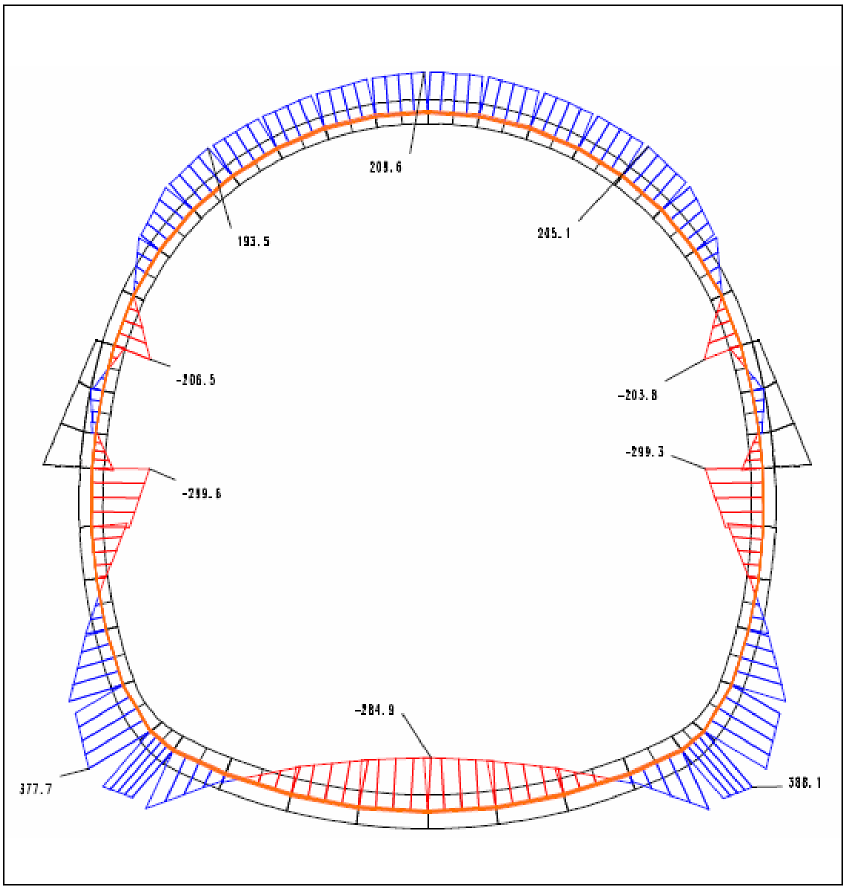

Figure 3: Final bending moments in completed primary lining

3 VERIFICATION STATIC CALCULATIONS

3.1 Basic data

3D calculations were generated using software Plaxis 3D Tunnel. The major aim of this modelling was mainly evaluate an impact of pile walls on excavation and lining. The model was prepared to comply with input of 2D calculations (location of geotechnical units, input parameters, tunnel lining, etc.).

The model was 127m high, 90m wide, and 97m long (see Fig.4). The model included just one half of the tunnel due to symmetry. Excavation sequences were slightly simplified – the bench and the invert were excavated in a one step. One model was generated with pile walls; the second was generated without them.

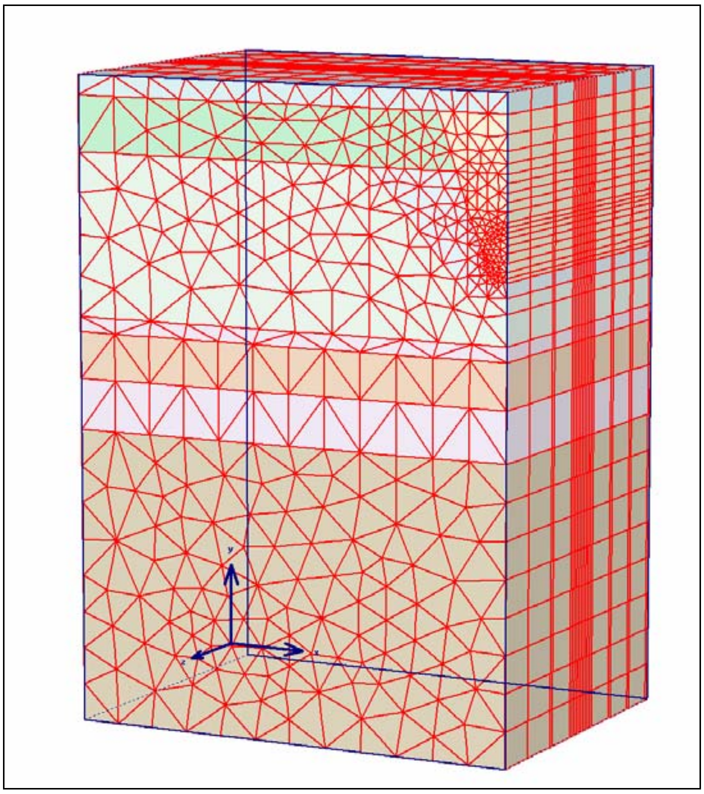

Figure 4: 3D model geometry

3.2 Pile walls impact

The model included pile walls (Fig.5) with spacing 9m. Thickness of the walls was used 1m in the model.

Pile walls were modelled as linear-elastic material, they were separated into two parts (to simulate the real structure):

a) Lower part (in the tunnel area) filled by concrete had parameters: E = 25GPa, $\nu = 0{,}2$
b) Upper part (above the tunnel) filled by suspension had parameters: E = 10GPa, $\nu = 0{,}2$

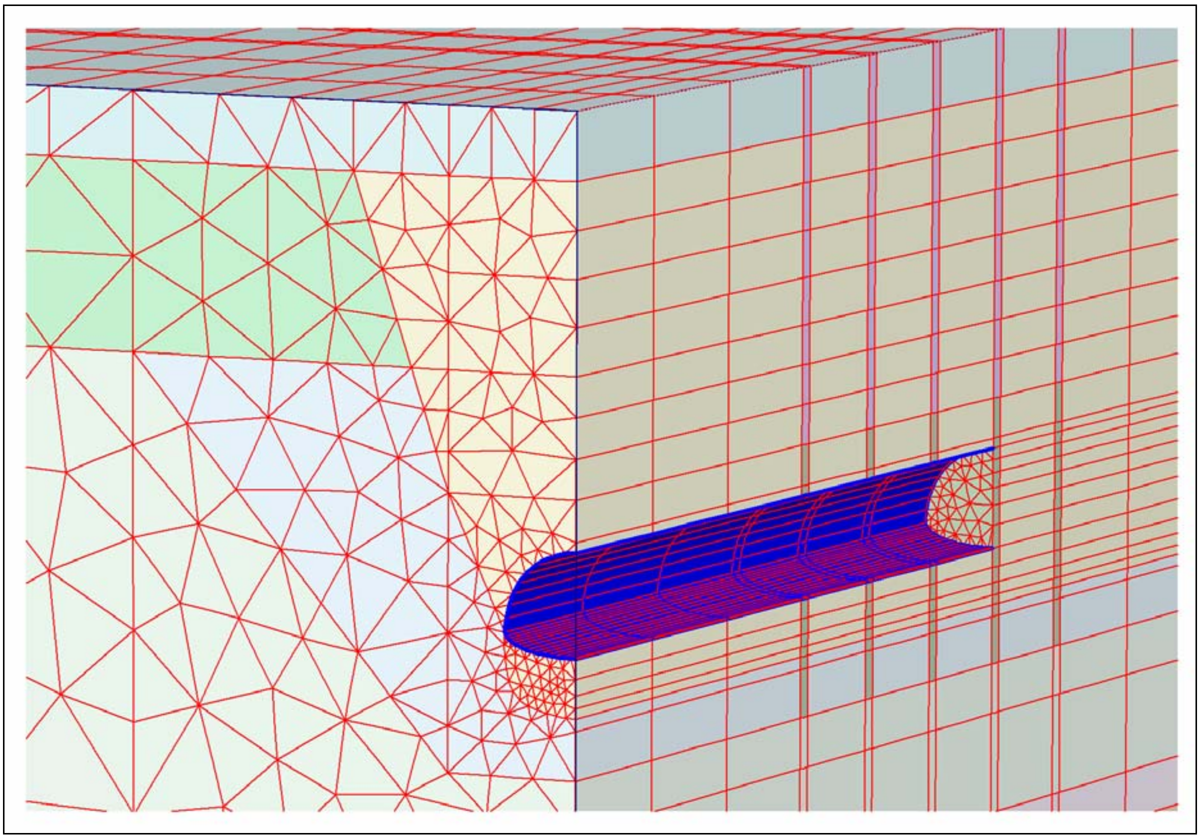

Figure 5: Pile walls in the model

Two calculations were generated: with and without walls. The results of calculations are presented in Tab. 2, they are also compared to 2D results:

		3D – with walls	3D – without walls	2D
Deformations (mm)	Vault	26	116	50
Moments (kNm)	Invert	122	196	285
	Side	120	370	300
	Vault	40	169	200
Axial forces (kN)	Maximum	1610	1770	2450

Table 2: Results – completed primary lining

The results clearly show the stiffening effect of pile walls. The construction of pile walls means significant reduction of deformations and bending moments. Differences between 2D results and 3D results are caused by original estimation of relaxation. The choice of low relaxation (i.e. fast ring closure assumption) in 2D calculations was affected mainly by conservative approach to the primary lining design (to get higher axial forces).

3.3 Impact of bench and invert excavation

The next purpose of 3D calculations was evaluation of bench and invert excavation on the top heading lining performance (i.e. when tunnel invert should be closed). The invert was modelled to be closed in 2m, 4m, and 8m steps (Fig.6). Results of deformations and internal forces in cross direction in top heading lining above excavated bench are presented in Tab.3.

Bench and invert advances (m)	2	4	8
Deformations (mm)	26	29	52
Bending moments (kNm)	61	140	175
Axial forces (kN)	1260	1600	2020
Tunnel lining capacity check	o.k.	o.k.	o.k.

Table 3: Top heading – internal forces in cross direction

Calculations showed that values of internal forces in top heading lining are not a significant problem. More significant problem would be deformations which would be double in case of 8m advances. The next problem would be forces in longitudinal direction and shear forces in the lining close to walls. Thus maximum advance 4m was recommended for the bench and the invert excavation.

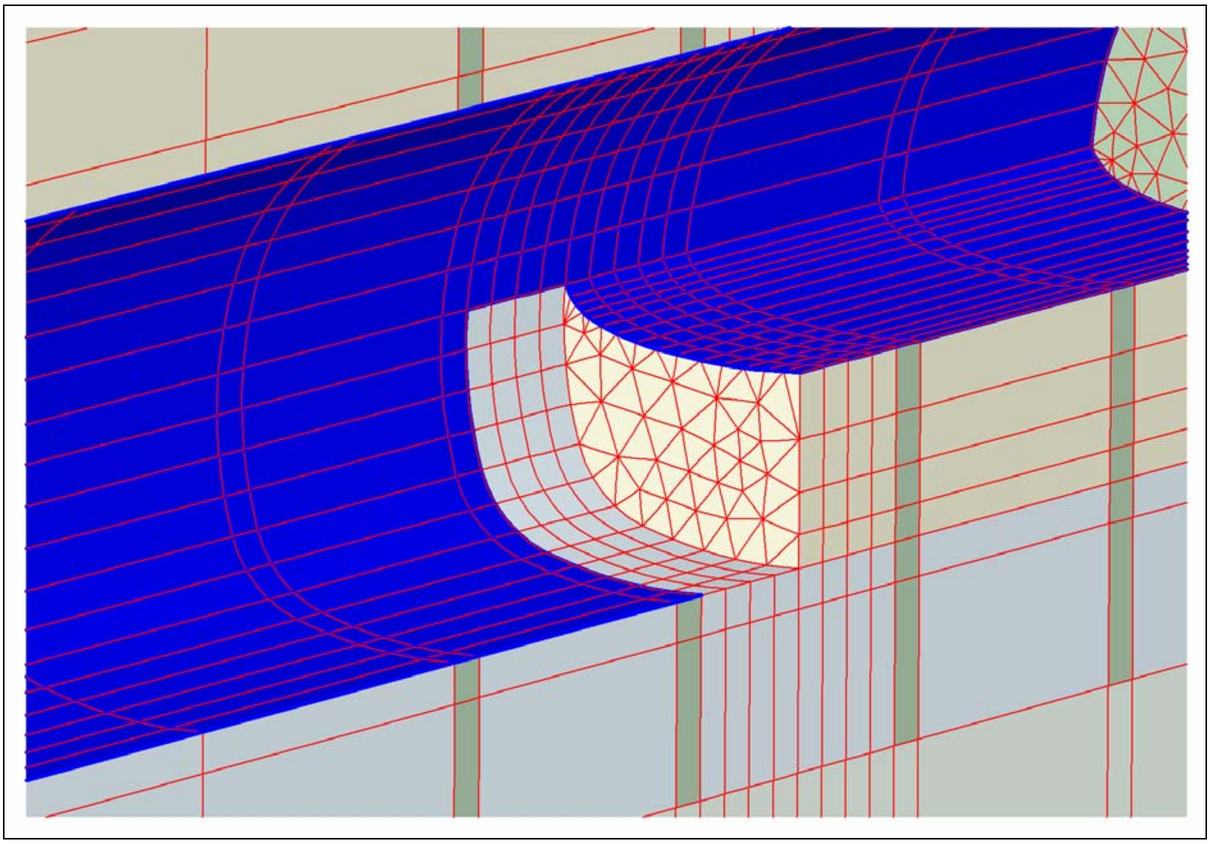

Figure 6: Simulation of the invert excavation

3.4 Top heading face stability

Calculations of the top heading face stability were also generated. Bench and invert excavation was expected to be separated at least by one pile wall to have minimal effect on stability of top heading face. The calculation was done in several stages (installation of pile walls, consequently several excavations and installations of lining). The tunnel face stability was calculated when the face was 2m behind the pile wall and 1m of the excavation was unsupported (Fig.7). The safety factor is calculated in programme Plaxis as ratio of initial and final shear parameters.

Provided calculations showed safety factor very close to 1.0 which means problems of the top heading face stability. However generated calculation did not include designed support measures (support wedge, micropile umbrellas and jet grouting columns, further sequencing of the face, etc.). The principle of possible top heading collapse is shown in Fig.7. The figure clearly shows favourable effect of pile walls to limit propagation of generated deformations.

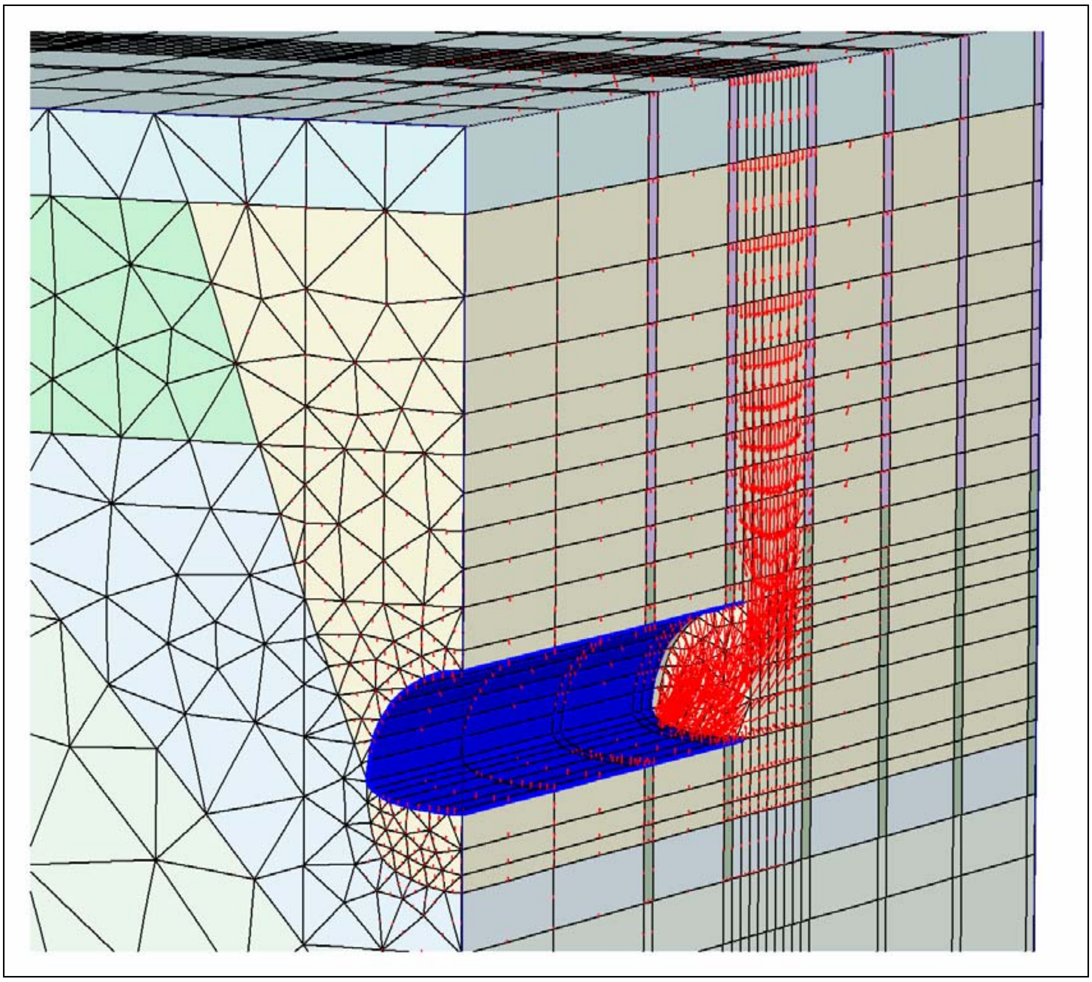

Figure 7: Propagation of the top heading face deformations

4 CONSTRUCTION

There was significant anxiety about ground behaviour prior start of excavation, as the area was significantly disrupted by previous collapse (area in and above the tunnel profile). Thus core drills from the tunnel face were realised prior excavation of each section between pile walls and decision about ground improvement and support measures was done based on results of drilling. In the first section the horizontal jet grouting columns were generated into the face to increase face stability. This measure was used also in the section 3.

Figure 8: Umbrella from micropiles embedded into pile wall

The tunnel profile was regularly protected by micropile umbrellas; micropiles were embedded into the pile walls on the both ends (Fig.8). Some attempts to embed micropiles into horizontal jet grouting columns were done (to increase their stiffness), but similarly to jet grouting columns drilled into the face this approach was finished after the third section.

All excavations were done with advance 1m. Excavated profile was supported by wire meshes, lattice girders a by sprayed concrete. Face stability was regularly increased by a support wedge (ground left in the centre of excavated profile), moreover flash coat of sprayed concrete (several centimetres) was instantly applied on the face and tunnel perimeter after the excavation. Top heading face was sometimes excavated and sprayed in several steps (in cases of local instability). Also temporary top heading invert was closed regularly. Originally it was closed in 2m or 3m steps, later this was even extended. Bench and invert excavation was realised more than 9m behind the top heading face (length of one section). The excavation started at the end of February 2006 and was completed without major problems at the beginning of August 2006.

5 MONITORING RESULTS

Maximal monitored surface settlement reached 28mm (area in the second section). Monitored movement of the tunnel ling are presented in Tab.6. All deformations generally stayed below 40mm, only area in the section 2 had higher deformations. This was caused by local problems which did not affected overall stability of the tunnel. Thus values of monitored deformations reasonably comply with values predicted by the modelling.

Tunnel chainage (m)	**Vault (top)**	**Top heading sides**		**Bench sides**	
	Point 01	**Point 04**	**Point 05**	**Point 06**	**Point 07**
2004	5	7	7	0	0
2007	19	14	16	10	10
2012	21	23	27	10	8
2019	20	30	27	-	5
2025	20	30	36	7	3
2027	27	30	28	7	8
2031	33	27	40	8	10
2034	37	36	40	7	7
2036	50	55	105	11	10
2040	93	65	130	14	-
2043	38	37	34	6	7
2048	39	36	53	-	-
2052	40	42	46	7	8
2057	40	47	53	7	6
2061	40	43	46	3	6
2066	31	32	35	-	-
2070	24	18	23	0	0
2075	26	24	36	-	3
2079	11	18	20	3	3
2081	17	28	27	3	2
2084	8	18	20	-	2
2087	2	7	5	0	0

Table 6: Monitored total movement of the tunnel lining [mm]

6 CONCLUSIONS

The tunnel Brezno had to be excavated in very complicated geological conditions. These ground conditions were significantly worsen by collapse of quite long section of the tunnel lining. To design excavation procedure and appropriate support measures for re-excavation of collapsed tunnel was not a straightforward task.

Static calculations of the tunnel re-excavation were provided using 2D finite element method (software RIB). Further calculations to evaluate rock mass behaviour in collapsed area were provided using FEM software Plaxis. 2D calculations were realised to provide sensitivity studies, 3D modelling assisted to evaluate tunnel face stability (impact of the pile walls, ground improvement, etc.). Results of the modelling were compared with the monitoring results (back analysis). The paper also briefly describes a construction experience (technical problems, performance of various support measures, etc.).

2D and 3D modelling was used to evaluate ground and tunnel behaviour during re-excavation. Provided modelling brought very useful information prior start of construction. The modelling led to tunnel shape and excavation sequence optimisation, the modelling indicated tunnel face stability problems which had to be improved by various measures. Modelling also confirmed a very favourable effect of designed separation of tunnel by pile walls.

Consequent excavation was realised without any significant problems, the construction procedure and support measures were further optimised during construction. The Brezno tunnel construction was successfully completed and the tunnel was opened for traffic in April 2007.

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