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NUMERICAL ANALYSIS OF JABLUNKOV TUNNEL COLLAPSE

NUMERICKÁ ANALÝZA HAVARIE TUNELU JABLUNKOV

Abstract

Jablunkov tunnel is located at the newly renovated track through Jablunkov Pass in Czech republic near Slovak border. The original tunnel was built in 1870. The second tunnel was opened in 1917. When construction was used English modified method of excavation. During the 2nd World War were tunnels damaged. Motion reconstruction allows resetting of the original single-track tunnel to a new modern two track tunnel. Geological conditions were very difficult and the state of both tunnels is unsatisfactory. One of the single-track tunnel was extended about 6 m. The total width of the tunnel is 12,6 m. The tunnel length is 600 m. The excavation carried out in accordance with NATM excavation method. Profile was divided into top heading and bench. During the excavation of bench primary lining collapse the main cause was a marked change in rock.

Keywords

Tunnel, primary lining, cave in.

Abstrakt

Rekonstrukce jednoho ze dvou Jablunkovských tunelů, které vznikaly postupně od roku 1870, byla již nevyhnutelná. Při rekonstrukci se vycházelo z optimalizace celého železničního koridoru, a proto byl navržen nový moderní dvoukolejný železniční tunel na místo stávávajícího jednokolejného tunelu. Při rozšiřování starého tunelu došlo k rozsáhlé havárii. Ražba probíhala v komplikovaných podmínkách. Příspěvek se věnuje zejména modelování situace před havárií a po havárii, kde reaguje na nové výsledky průzkumu po havárii.

Klíčová slova

Tunel, primární ostění, zhroucení.

1 INTRODUCTION

Jablunkov tunnel is located at the newly renovated track through Jablunkov Pass in Czech repuli: near Slovak border. The original tunnel was built in 1870. The second tunnel was opened in 1917. When construction was used English modified method of excavation. During the 2nd World War were tunnels damaged. The old single-track tunnel will be re-build to a new modern two track tunnel. Geological conditions were very difficult and the state of both tunnels is unsatisfactory. One of the single-track tunnel was extended about 6 m. The total width of the tunnel is 12,6 m**.** The tunnel length is 600 m. The excavation carried out in accordance with NATM excavation method. Profile was divided into top heading and bench. During the excavation of bench primary lining collapse. The main cause was a marked change in rock mass properties. The article is mainly dedicated to modeling the effects of changes in rock mass.

The Jablunkov railway tunnels are located in Jablunkov Pass close to the border with the Slovak Republic. This locality is very important for communications between the Czech and Slovak Republics . Two single-lane tunnels were built to negotiate the pass. Their usefulness has reached an end and it was therefore necessary to develop a new solution. As part of the overall modernization of the railway line, one of the existing single-lane tunnels was reconstructed to form a new modern double-lane tunnel. The other existing tunnel will be used as an escape gallery. The Jablunkov No. 1 Tunnel was built in 1870 as part of the single-line Košice-Bohumín Railway. The second tunnel was not opened until 1917. Tunnelling was carried out by the modified English method and the lining was built of stone

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from a nearby sandstone quarry. Both tunnels were damaged at the beginning of World War II, And several further repairs were undertaken after the reconstruction in 1940.

The thickness of the tunnel overburden is between 8 to 24 m. Three stratigraphic formations intersect close to the tunnel, namely the Palaeogene Krosnen Formation, sub-Menilit strata and the IstebnaChalk Formation . Petrographically the whole stratigraphic sequence is composed of alternating sandstone and mudstone layers of flysch origin with either mudstone or sandstone predominating in different localities. In exploration boreholes immediately above the tunnels the pre-quaternary bedrock surface was found at depths of approximately 1.9–6.5 m below the surface, with predominant predominantly mudstone present in the boreholes, but sporadically a predominance of siltstone and/or sandstone. The Quaternary overburden is predominantly composed of alluvial deposits that typically have a thickness of approximately 0.8–3.2 m, rarely up to 6.1 m. Hydrogeological conditions in the area are complicated not only with respect to the complex geological structure but also due to the complex tectonic relations. A fault at the contact of Istebna and Krosnen Formations provides a pathway for rgroundwater migration, especially as a result of intense cataclasis within the fault zone. The piezometric level/water table in all exploration boreholes was found to be in the depth range of 0.25–6.0 m below the surface. It is a typical confined intergranular aquifer in Quaternary overburden and outcropping of Tertiary rocks.

Reconstruction of the Jablunkov railway tunnel was undertaken by widening and enlargement of No. 2 tunnel tube of the old tunnel system which constructed approximately 100 years ago. The length of the tunnelled section of the tunnel is 576 m. The lining of the excavated tunnel is designed as two shells with an intermediate water-proofing layer. The minimal thickness of the secondary tunnel lining is 400 mm. Excavation works employed the New Austrian Tunnelling Method (NATM). With regard to found IG conditions, the considered rock disintegration is mechanized or using blasting operations and mechanized scaling. The tunnel profile is horizontally divided into top heading, bench and invert. For geotechnical conditions found within the detailed geotechnical research, 3 basic technological support classes of excavation by NATM were determined. Primary lining is made of sprayed concrete C 16/20 designed in thicknesses of 150, 200 and 350 mm. The thickness of primary lining depends on the excavation class. Furthermore, reinforcing steel lattice girders, reinforcing steel welded mesh, anchors and drilled steel needles are used. As the primary lining the section of walled lining of the existing single-line tunnel is used which will be equipped with a layer of sprayed concrete reinforced by steel welded mesh and secured by PG anchors of the length of 3 m and the space behind lining will be grouted. The tunnelled section of a new double-line tunnel was built in five stages: - the first stage includes securing the side of the existing single-line Jablunkov tunnel No. 2 by sprayed concrete and PG anchors, supplemented with grouting behind lining using these anchors. - in the second stage the tunnel top heading will be tunnelled which will be immediately secured by primary lining. The part of the top heading of the existing single-line tunnel will be dismantled as well. - in the third stage the predominant part of the old tunnel will be gradually demolished together with a simultaneous excavation of the bench and the invert. Subsequently, the entire profile will be secured by primary lining of sprayed concrete. - in the fourth stage the closed intermediate water-proofing will be executed. - in the fifth stage the secondary tunnel lining will be built. [2]

On 15.11.2009 in morning hours the collapse of the tunnel occurred during which approximately 96 m of the work reinforced by closed primary lining totally caved in. The sudden collapse occurred in excavation of the bench and the invert of the tunnel and in a predominant part it spread to a Jablunkov tunnel portal.

In order to find out the causes of the current collapse it is necessary to state that from the period of tunnelling and reinforcing this single-line tunnel almost no documents remained preserved that could be used to specify geological and geotechnical conditions. The found documents (Municipal office Jablunkov) give evidence of the fact that the conditions in tunnelling in the area of the current eastern tunnel portal were complicated and tunnelling was very difficult. This is also indicated by observations of the disposed old lining of the tunnel that registered a significant increase of the thickness of old lining and relatively extensive rupture of rock environment (partly covered by subsequent long-term consolidation of clay rocks). The collapse occurred by excavation the tunnel bench and the invert. During excavation a predominant part of the top heading bottom and left support of the old tunnel is removed. The top heading bottom was again hinge-connected on the right side to the lower part of the left support.

The collapse itself occurred by obvious exceeding of the bearing capacity of lining of shotcrete in the left part of the support whereas the image of the failure was, according to the photograph taken several minutes before the collapse (photo 1), the same as at the end of the collapse on a Jablunkov side, i.e. it indicated obvious exceeding of the bearing capacity of lining by the formation of a shear crack owing to a high vertical load.

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Photo 1: Photo from the point of origin of the collapse - shear failure in the right side of the lining

Photo 2: Photo from the point of the origin collapse - formation of the surface crater (author MF Dnes)

Fast progress of the collapse also indicates that in the place of its origin a sudden change of physical and deformable qualities of the rock pillar due to the changes of natural conditions from the viewpoint of tectonic, hydrogeological and probably also tension conditions in the given section whereas even the effects of old and recent slope deformations.

2 GEOTECHNICAL CONDITIONS OF TUNNEL EXECUTION

Geological and geotechnical conditions in the tunnel track were investigated and form a starting basis for project preparation and statistical calculation. Within the geotechnical research no geophysical research was carried out which could bring an increase of forecast reliability. It was shown by an additionally performed georadar measurement indicating very complicated conditions of old fossil landslides in the tunnel overburden which could have been partly activated due to the enormous increase of rocks damp in the period before the collapse (high levels of rainfall, snowfall).

For the same reasons the original conditions of natural tension were not investigated which is undoubtedly present in a high level in a broader interest area for both the reason of the existence of fossil landslides and a tectonic structure of the Jablunkov ridge. Overall, we can say that the conclusions of the geotechnical research warn of very complicated natural conditions and a very low solidity of the rock environment.

From the borehole realized in the additional geotechnical research 1/2010, which is localized practically in the place of collapse origin, no data on necessary parameters of deformable property were gained, only shear parameters were determined. The rock massif is formed here by completely eroded and corroded claystones, structurally strongly faulted and completely water-saturated. The input data by boreholes interpretation are given in the table No. 1 and 2. Individual properties of rocks were taken or completed from different stages of geological researches.

The additional research in 2008 suggested the following characteristic properties of the rock massif to the designer that reflect also the properties of the space of the P1 tunnel portal (table 1). The rock massif was divided into three geotechnical types: -heavily, -medium and -s slightly eroded claystone (tab No. 1). See figure 1 a.

Mohr-Coulomb		heavily weathering claystone	medium weathering claystone	slightly weathering claystone
γ_{unsat}	[kN/m^3]	20,00	20,00	19,50
γ_{sat}	[kN/m^3]	20,50	21,50	21,50
k_x	[m/day]	0,001	0,009	0,00001
k_y	[m/day]	0,001	0,009	0,00001
E_{ref}	[MPa]	**20**	**374**	**420**
ν	[-]	0,420	0,270	0,250
c_{ref}	[kN/m^2]	15,00	25,00	32,00
φ	[°]	20,00	28,30	27,60

Tab.1: Values of physico.- mechanical parameters from 05/2008

Mohr-Coulomb		**deluvium**	**claystone 1**	**backfill**	**clystone 2**	**sandstone**	**flysch belt**	**siltstone**
γ_{unsat}	[kN/m^3]	19,00	19,00	19,00	19,00	22,00	19,00	19,00
γ_{sat}	[kN/m^3]	21,00	21,00	21,00	21,00	23,00	21,00	21,00
k_x	[m/day]	0,086	0,001	0,086	0,001	0,000	0,001	0,001
k_y	[m/day]	0,086	0,001	0,086	0,001	0,000	0,001	0,001
E_{ref}	[MPa]	**6**	**10**	**5**	**8**	**80**	**20**	**20**
ν	[-]	0,400	0,400	0,400	0,400	0,150	0,300	0,400
c_{ref}	[kN/m^2]	12,00	15,00	20,00	5,00	50,00	20,00	18,00
φ	[°]	22,00	22,00	20,00	22,00	25,00	22,00	22,00

Tab.2: Values of physico.- mechanical parameters from 01/2010

The exploration boreholes clarified the geological structure in the interest area and clarified geotechnical parameters of found soils and rocks. Characteristic physical-mechanical properties of soils and rocks were, due to permanent supply of water into the rock pillar, almost completely degraded, in particular physical and deformable properties of rocks (elasticity module 6-15 MPa against 374 MPa in comparison with the research values from 2008). A signifcant increase of the volume weight occurred (table No. 2). Also the finding is important that eroded claystones (a sample from the depth of 10.0-10.2 m) behave contractantly (the material reduces its porosity - volume). The contractant behaviour at water-saturated rocks causes the increase of the porous pressure which causes the reduction of an efficient tension and thus lower shear resistance. In case of fast increase of porous pressures the uncontrolled movement in the potential shear surface can occur, possibly a significant reduction of shear strength after opening the breakup of the centre and a sudden increase of the lining load.

3 NUMERICAL ANALYSIS OF THE COLLAPSE

To evaluate the mentioned statements the static solution was made in 2D and 3D mathematical models by the method of final elements. For calculation we used the program system Plaxis 8.2 and Plaxis 3D Tunnel. The combination of solutions of the same problem in a 2D and 3D design is not usual and it shows the importance of the entire collapse. Both models practically suggest the same results when the three-dimensional model tried to simulate the real situation as much as possible, including the advance steps of excavation and solidification of shotcrete.

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In order to compare the analyses in a plane model both for the designed situation and for the situation which derives from the last additional research executed after the collapse (January 2010). The models were made in the predicted cross section, with the overburden of approximately 12 m. The tunnel was modelled in the direction of the proceeding excavation of the bench, i.e. the rest of the old lining was on the left side of the tunnel (see the figure No. 1). The calculation was divided into four stages. In the first stage the top heading was excavated, in the second stage it was subsequently reinforced by sprayed concrete. In the third stage the excavation of the bench, top heading bottom was modelled and in the last fourth stage the stope of the bench and of the invert was secured by sprayed concrete. Individual stages were also modified from the load point of view. In the stages without the reinforcement we considered a load coefficient of 0.4. Thickness of sprayed concrete were always 350 mm. This division was applied to the 2D and the 3D model as well. The concrete age was taken into account in the stiffness of concrete. In the 3D model the concrete age was assigned in individual advance steps that were 2 m. The bench and the invert was modelled in the 3D model so that it corresponds to the real situation before the collapse. The top heading bottom was removed in advance and the reinforced sections corresponded to the concrete age.

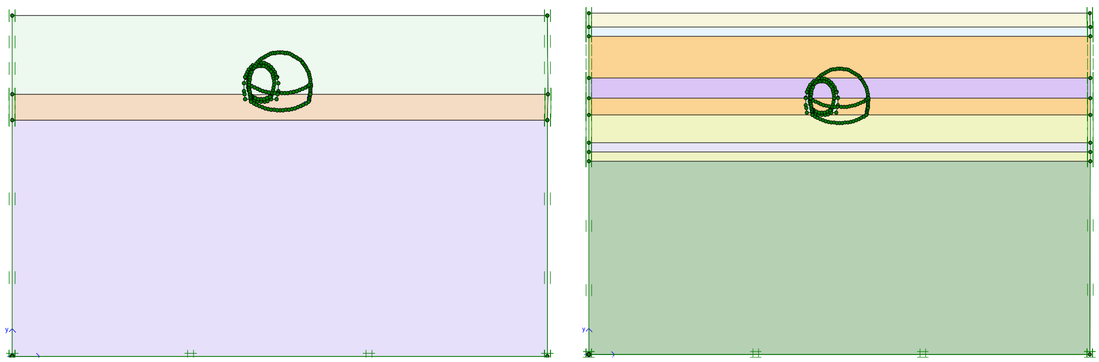

Fig. 1: Comparison of two models for different conditions

In the comparison of both models the clarification of the geological structure of the environment is obvious in the given cross section on the basis of the core hole which was executed after the collapse within the additional research (figure No. 1 on the right). To evaluate and compare both models especially deformation and internal forces were compared. The sizes of porous pressures were not evaluated but in all models the level of underground water was considered and the material was considered to be un-drained.

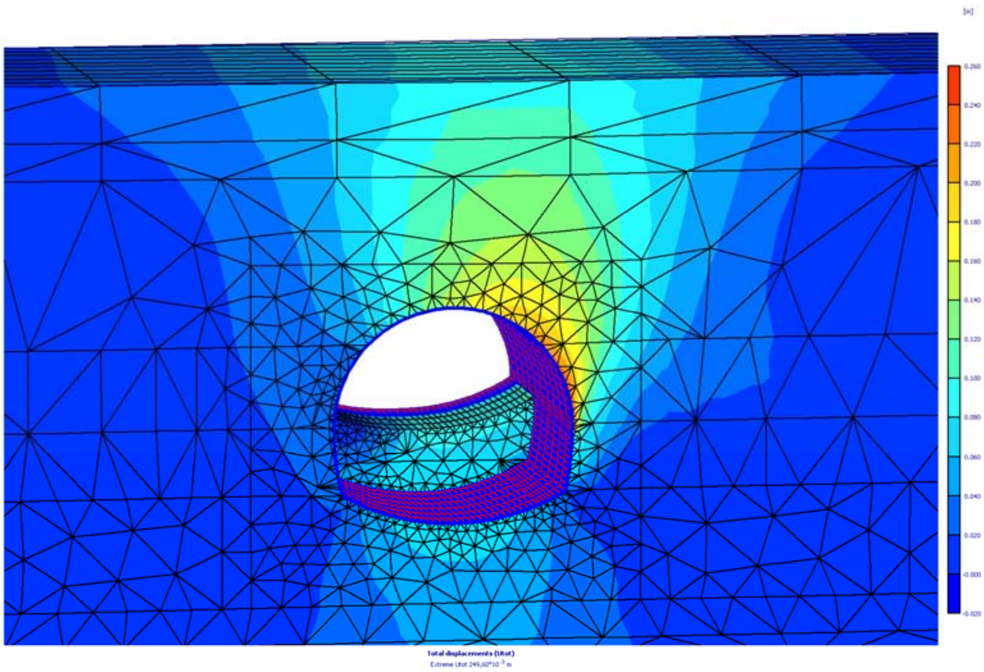

Fig. 2: Displacements in a 3D model

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The resulting values of displacements almost increased by almost double of the original precondition. According to the original calculation the deformations for the 2D model were 117 mm and for the 3D model only 45 mm. For new values the displacements increased at the 2D model to 266 mm and at the 3D model to 250 mm (figure No. 2). The model also confirmed the mechanism of the origin of lining damage (figure No. 3). In the left figure No. 4 there are the total displacements of primary lining. The values of displacements are indicated by the arrows (vectors) of displacements. In the left part of the primary lining there is the remaining part of the old tunnel and deformations are very small here. The right side of the new tunnel demonstrates much bigger deformations which is apparent from the photo No. 1. Next to the figure with the displacements of primary lining there is the detail with the directions of displacements before the installation of the top heading floor, i.e. after excavation the bench. There is a certain drop of primary lining of the tunnel top heading. The tendency of displacements was oriented behind the lining of the top heading floor and the bench. The already finished lining of the lower calotte demonstrated a significant displacement in the direction to the tunnel. This displacement is obvious in the figure No. 5 where the red colour indicates the places with the biggest displacements (the maximal value was 243 mm for the 3D model).

Another evaluated parameter of the carrying capacity of lining and analyses results was internal forces and particularly bending moments on primary lining. It was again necessary to evaluate the size of internal forces for all modelled situations. The values of bending moments are in the chart No. 1. There are the bending moments for the entire closed primary lining. At the 3D model the centre was tunnelled in front of the last ring, but without the support. This condition exactly models the situation before the collapse origin. The biggest bending moments are created in the tunnel top heading. Ripe concrete is considered here for the top heading in contract to the top heading floor where new sprayed concrete is considered. In the 2D model the internal forces were higher than in the 3D model which is probably caused by the load distribution. There is again the increase of the bending moment and again by almost a double compared to original input data. As stated above the top heading was loaded most and as it was obvious from the progress of internal forces the load was not symmetrical and the left part was again influenced by the rests of the old tunnel lining. On the left side of the calotte of primary lining the bending moment were the biggest, only in case of the 3D model for the additional (01/2010) the maximum of the moment occurs on the right side of primary lining. Evaluation of internal forces was performed in an interaction diagram (figure No. 5). The results again confirmed the fact that for the original values, which were used in the design of primary lining, the lining will satisfy but will not satisfy for the newly added parameters.

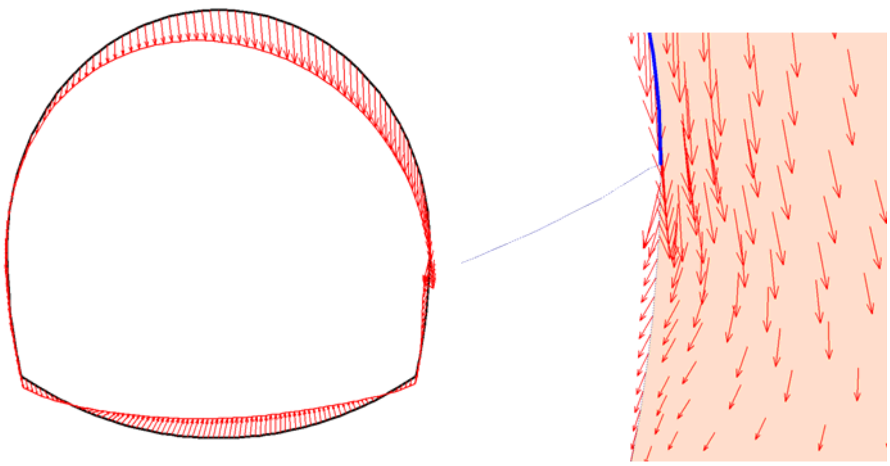

Fig. 3: Vectors of total displacements on the primary lining – 3D model

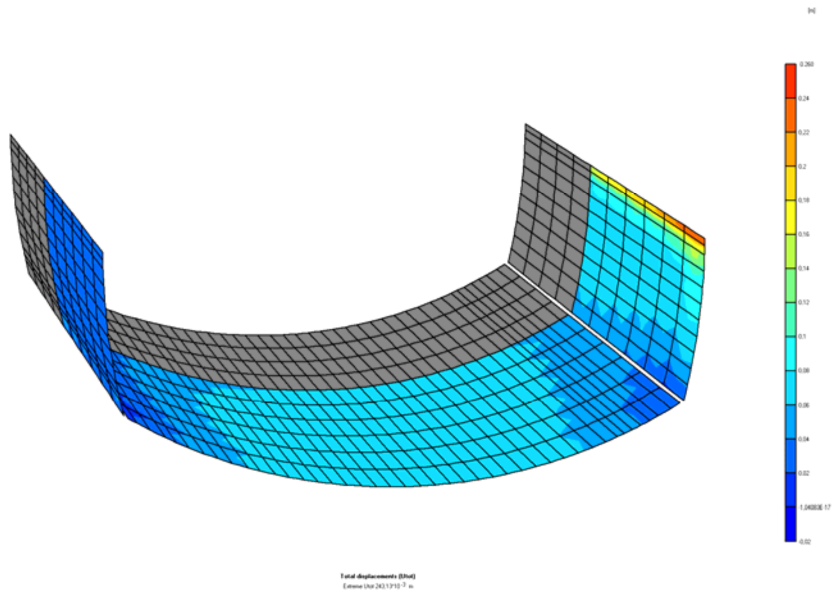

Fig. 4.: Total displacements of the bench and invert primary lining – 3D model

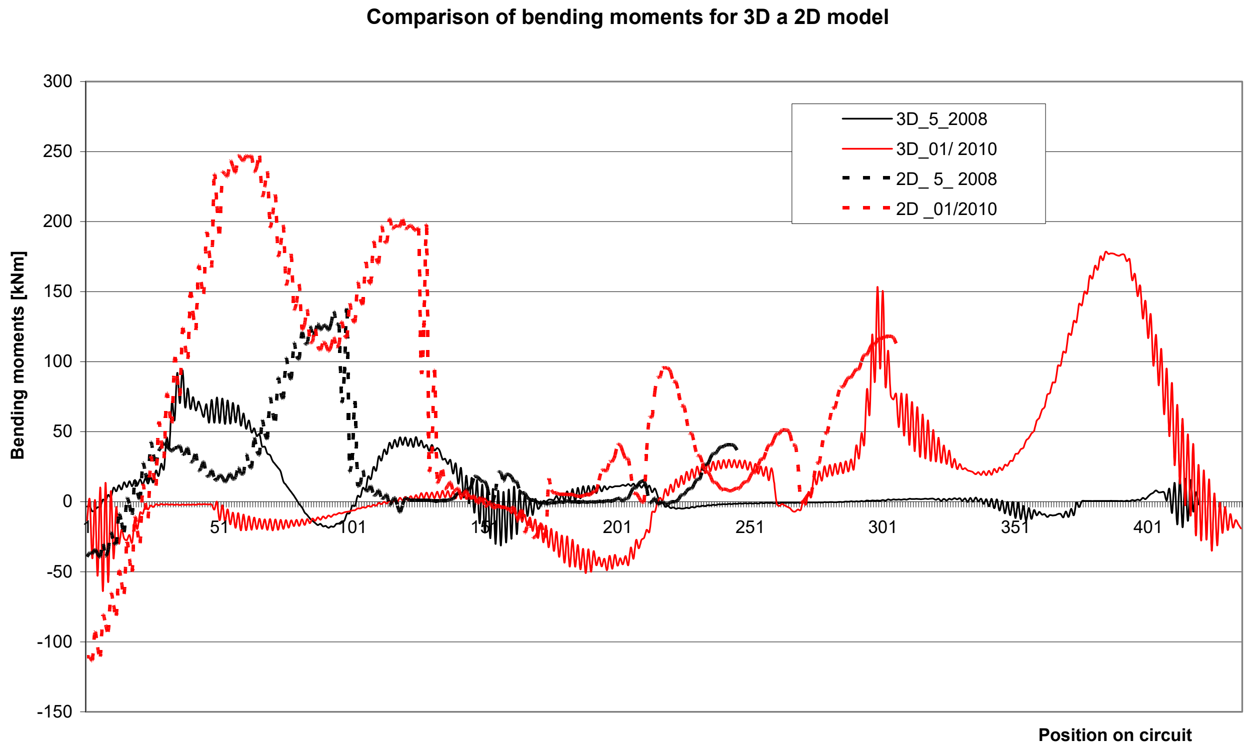

Chart No. 1: Comparison of bending moments for 2D and 3D models

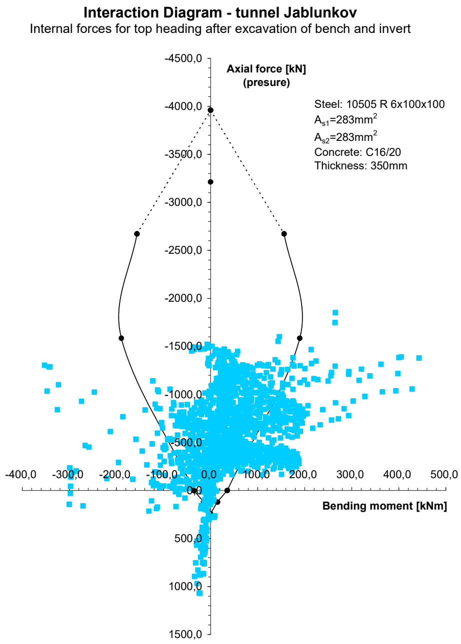

Fig. 5: Interaction diagram for a top heading after excavation of the tunnel bench (3D model)

4 CONCLUSION

Neither the local influence of the damaged surroundings of the old tunnel can be excluded as a factor in the collapse, nor can the changes in magnitude of horizontal tension in the rock environment due to fossil landslides, nor the horst-type tectonic structure, nor the influence of the first collapse that occurred approximately 20 m from the point of origin of the current collapse, nor the influence of water accumulated over a the long period in the in filled crater of the first collapse.

The results of models absolutely and unequivocally indicate the extreme influence of degradation of physical and deformable parameters of the rock environment. In comparison we considered the physical and deformable values of the rock environment assumed by the designer in 2008 and the values determined during the GT research in 2010 specifically at the point of collapse.

The collapse of the lining must have therefore occurred as a result of degradation processes in the rock environment due to the long-term activity of water from rainfall and water flowing into the point of origin of the collapse, where conditions for accumulation of inflows were created over a long period (terrain depression with surface water accumulation, wet terrain). According to the data of the Czech Hydrometeorological Institute, above average rainfall was recorded for October 2009, as for the during which there was 200 % more rainfall in the Moravian-Silesian Region than the long term average. The initiating mechanism for the collapse was tensile changes which occurred in the excavation, the bench and the invert.

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