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Analyisis of shallow tunnels construction in swelling grounds

Analiza gradnje plitvih predorov v nabrekalnih hribinah

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Abstract

Swelling pressures of the rock as a result of chemical and physical processes which are present during construction and operation of tunnels and have the influence on loads and deformations of primary and inner concrete lining. Deep geomechanical analysis of swelling indicates that in practice very often conservative way of calculating load capacity of primary as well as inner lining were used with a goal to keep long-term stability of the tunnel. Particular emphasis was placed on the physical and chemical assessment of the time dependent development of deformation. In the present paper the practical case of tunnel construction in specific swelling clay ground »Sivica« is analyzed. Based on 2D and 3D geostatic analyses, a rigid primary lining was chosen as a final design, because the depth of the tunnel is only about 30 m below the surface. The geotechnical parameters of hoist ground »Sivica« are a result of laboratory and »in situ« tests, which were conducted according to technical standards. During a construction and after it the geotechnical measurements were conducted. The measurement results confirm the correct technical decision in the design stage.

Key words: Road tunnel, support elements, Finite Element Method, primary shotcrete lining, geotechnical measurements

Izvleček

Nabrekalni tlaki v hribinah so rezultat kemijskih in fizikalnih procesov, ki med gradnjo in obratovanjem predorov vplivajo na obtežbe in deformacije, ki se razvijajo tako v primarni kot tudi notranji betonski oblogi. Poglobljene geomehanske analize nabrekalnih pojavov v hribinah so pogosto osnova za konzervativne načine izračuna potrebne nosilnosti primarne in tudi notranje obloge predora z namenom, da se zagotovi dolgoročna stabilnost podzemnega objekta. Posebna pozornost analiz je namenjena fizikalnim in kemijskim procesom v hribinah z nabrekalnim potencialom v odvisnosti od časovno odvisnih deformacij. V tem prispevku je podan praktičen primer gradnje cestnega predora v glinasti hribini »Sivica«, ki ima specifične nabrekalne lastnosti. Končna tehnična rešitev gradnje na osnovi rezultatov geostatičnih analiz 2D in 3D je temeljila na uporabi toge primarne obloge, ker je globina predora le okrog 30 m pod površino terena. Geotehnični parametri »Sivice« so bili določeni z upoštevanjem laboratorijskih in terenskih raziskav, ki so bile izvedene skladno z veljavnimi standardi. Med gradnjo in po njej so bile izvajane geotehnične meritve, katerih rezultati so omogočili primerjavo z izračunanimi parametri in posledično potrditev ustreznosti projektnih rešitev.

Ključne besede: cestni predor, metoda končnih elementov, primarna obloga iz brizganega betona, geotehnične meritve

Introduction

Understanding the basic theory of chemical and physical processes in swelling ground is fundamental condition for successful technical solution of tunnel construction in such circumstances. When tunneling takes place in swelling ground such as anhydrite or swelling clay minerals, i.e. montmorillonite, whole process of construction, including underground water presence and designed technology, needs to be analyzed. This is very important, because in the past in more cases of tunneling in such grounds, after deep analysis were carried out, clear shown the influence of unusable technology which caused and activated swelling potential of present ground layers (Figure 1). It is known that chemical reaction of anhydrite ($CaSO_4$) in the gypsum ($CaS0_4 \cdot 2H_20$) results in the increase of volume up to 61 %. The similar can apply in the shale formations with swelling potential where physic-chemical reaction depends on stress relief and reduction of the chloride concentration by water adsorption (osmotic swelling). The chloride ion diffusion is assumed to be the mechanism for the reduction of the chloride concentration.

Research activities of swelling processes in ground

Numerous calculation methods have been presented in the past to simulate the swelling behavior and the resulting structural response [1–9]. Very important laboratory tests on swelling rocks were done by Barla [10], where swelling gouge from weakness zone in tunnel were investigated.

High sophisticated constitutive laws have been developed to account for the phenomenon of swelling in terms of continuum mechanical models, i.e. [12–15]. Several PhD theses were done on the topic of swelling processes in rocks [10, 16, 17], in which authors were explained fundamentals of physical and chemical base of the such ground behavior. More attention is paid on experimental investigation which was done by [18]. He had shown the compression and swelling behavior of the Callovo-Oxfordian argillite. Two series of oedometric tests were carried out, first one showed that the argillite exhibits a swelling behavior even when fully saturated under stresses higher than the in-situ stress, and the second series of tests showed that the swelling capacity appeared to increase with compression. In both cases, swelling was related either to pre-existing cracks due to sample coring, storage, drying, wetting and trimming or to cracks induced during compression. In this regard, compression was suspected to occur by local pore collapse that created micro-cracks that afterwards swelled when hydrated. Indeed, sample compressed to a higher pressure exhibited higher swelling potential.

A high quality investigation of swelling phenomena was carried out [19] which focused on repair works done in old railway tunnel in Switzerland. Long-term laboratory tests showed that the swelling behaviour occurred in stages and bands of precipitated gypsum were found in samples analysed by mineralogical test after test completion [20]. The swelling processes require water and this could be facilitated by spalling type fractures, brittle failure behaviour with associated extensional fracture development as a potentially controlling mechanism in creating a water conductive zone beneath the tunnel invert. Such brittle fractures were observed during tunnel construction in anhydrite by [21, 22]. Therefore, gypsum crystal growth is most likely to occur where water has access

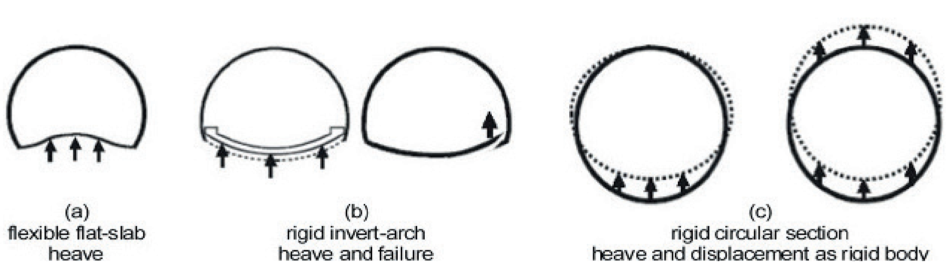


Figure 1: *Typical events associated with swelling in tunnels according to [11].*

and the state of stress is favourable for stress fracturing. On the low confinement side of the spalling limit, water has access through fractured ground and the rock mass is essentially free swelling until the support provides sufficient pressure to prevent further swelling.
One of the important investigation of swelling phenomena was relating to the goal of the presented investigations and that is to describe the phenomenon of self-sealing quantitatively (Figure 2, 3) and to establish a model, by means of which it is possible to take the self-sealing phenomenon into account when designing tunnels in swelling rock [23, 24].
It is expected, that due to the new models a more effective and economic design of tunnels in swelling rock will be possible. Swelling pressures, which had been identified in some old railway tunnels that were built in the 19th and in the early years of the 20th century, manifested in many cases, particularly in the floor inverts. In some cases it was necessary to carry out rehabilitation even twice, because the invert had shallow arch which hadn't enough

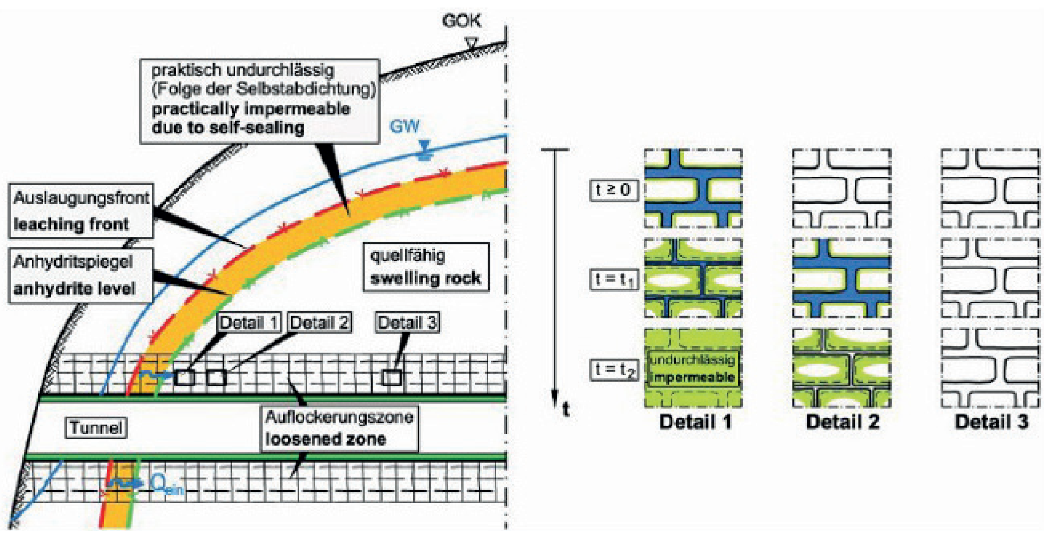


Figure 2: *Mechanisms of self-sealing in SBR [23].*

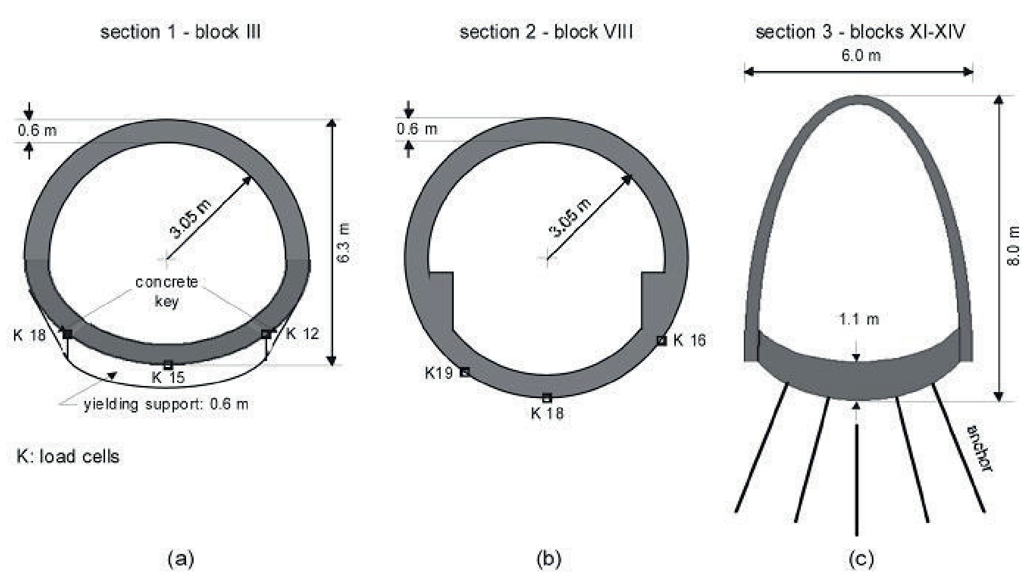


Figure 3: *Characteristics of tests sections in test gallery U1 of Freudenstein tunnel modified [25].*

static resistance of the support ring (Wagenburgtunnel, built in 1957) [26]. From several analyzed cases it can be summarized that the process of swelling is usually intensely present in the floor and in the sides of the underground facility, which represents lower static capacity of the structure.

Special case of tunnel construction in swelling rocks was Engelbergtunnel tunnel, which was built in 1990 [26]. The primary lining has been damaged at the beginning of construction at the connection point between the tunnel side the tunnel invert. At the beginning the original basic design was based on dry rock conditions during construction. Furthermore, in the upper design stage it was planned inner lining of reinforced concrete in the goal to excluding the effect of swelling pressure of the surrounding rock. During the construction phase, the design concept was significantly revised since instead of 30 cm thick shotcrete lining in the invert, 1.5 m thick concrete lining was installed, additionally anchored in the foundation ground. This way sufficient static resistance to swelling pressures was provided. Above anchored and reinforced invert high deformable layer called »Knautschzone« was constructed to prevent possible damages on the inner lining. And finally, the inner lining in invert was installed with a thickness of 3 m, as shown in Figure 4.

In the design procedure is important question when in case of construction of the underground facility the increment of the volume change will be prevented enough with the installation of rigid support system in the goal to reduced deformation in acceptable limit. That requirement in the conclusion still need adequate primary lining with sufficient load bearing capacity to ensure stability during the construction. It should be noted that the time evolution of swelling pressures depends on the amount of water in contact with swelling ground and stiffness properties of the primary lining. This takes into account the principle of long-term stability of the facility with the primary lining, which consists of a standard shotcrete lining, steel arches, steel wire mesh and rock bolts or anchors. A typical example of the combination of swelling and squeezing rocks can be found in Karavanke road tunnel connecting Slovenia and Austria, which was built in the second half of the 1980s and put in operation in 1991. In fact is that deformations, which in some sections had started to develop immediately after completion of construction or even already in the time of construction did not stabilize. The continuous time developed deformations called for the implementation of the rehabilitation of certain sections. Measurements which were carried out in the last years showed that the deformation process after refurbishment not stopped in some sections. Due to difficulties in identifying key parameters for such swelling laws the engineering approach predicting the swelling pressure used back-calculation of data monitored during construction of the tunnel in similar ground conditions. The last investigations in the Karavanke tunnel have shown that installation of a deformable layer in the invert between surroundings rocks and refurbished primary lining can be right technical solution for time stable tunnel structure.

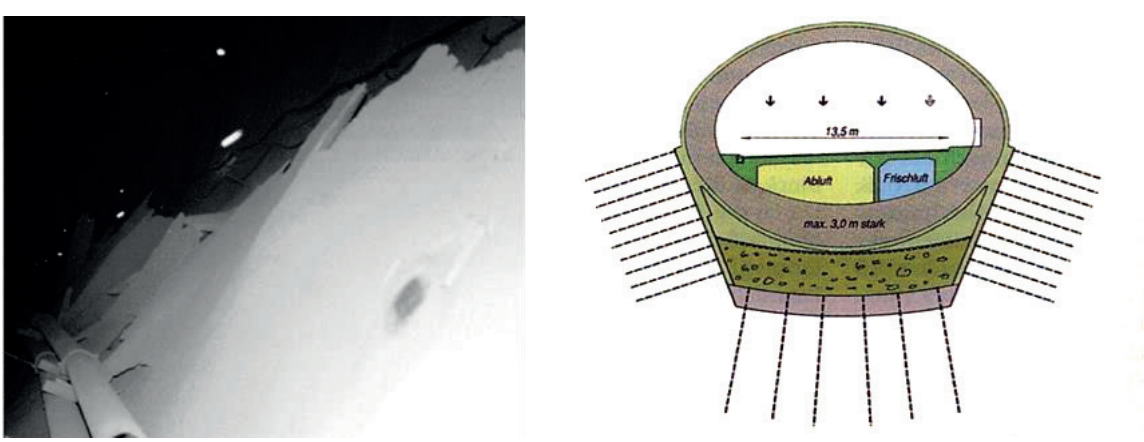


Figure 4: *Construction the Engelbergtunnel* [23].

Basic principles of support system design in swelling ground

The primary mode of support system planning is based on the principle of ensuring a balance between swelling capacity of ground and reacting support pressure. It is exposed to the volume increase that occurs after the excavation step. The relationship swelling ground - support system has important influence on the location of the equilibrium system, as shown in the Figure 5. It should be noted that the release of ground strain is a result of mobilization bearing capacity of the ground and at the same time swelling forces are generated when water or air moisture find the contact with ground surface at the excavation round in the tunnel. Quick and effective prevention of water access indirectly reduces formation of higher swelling pressures relating to formation efficient activation of the self-protection ground layer around tunnel wall. Exactly that self-protective effect has a unique role on reaching equilibrium between the supports system and surrounding ground.

Swelling phenomena aplication in the tunnel Ljubno design

The presented case of the shallow tunnel Ljubno with maximum overburden of about 30 m was built in the ground with swelling potential in the northern part of motorway section A2 Karavanke (Austria) – Obrežje (Croatia), and has shown specific circumstances relating to geotechnical conditions assessment.

The new Tunnel Ljubno was built as the twine road tunnel including reconstruction of the old tunnel tube which is now a part of new motorway (Figure 6). The old tunnel tube was constructed in the 60s of the past century without invert (Figure 6). In more than 40 years that the tunnel was in operation damages such as lifting of the pavement similar as shown in Figure 1–a) occurred. Reconstruction of the old tunnel tube included extension of the current clearance profile according to the standard that requires two lanes with a width of 3.75 m, one intervention lane with a width of 3.20 m and two intervention corridors with a width of 0.8 m which is the same as in the new tunnel which was built 40 m away. The amount of excavated material in the profile is approximately 86 m^2 per running meter of the tunnel.

Results of geological and geotechnical investigation

In order to design construction and reconstruction of tunnel tubes extensive field and laboratory investigations and explorations

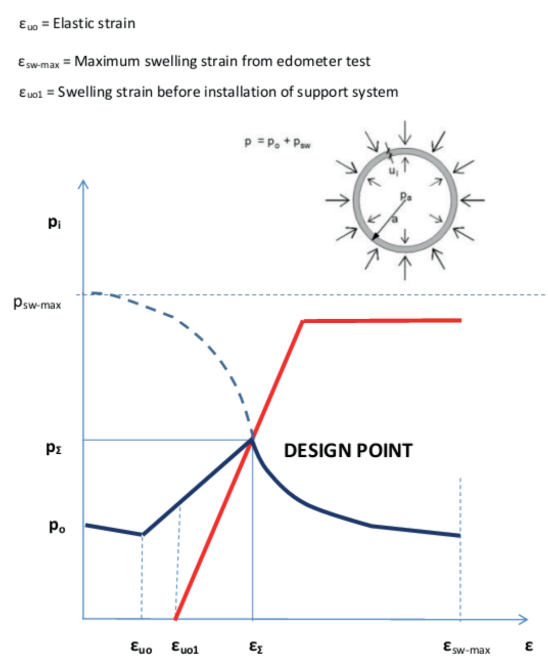


Figure 5: *Basic principle of tunnel support design in swelling ground.*

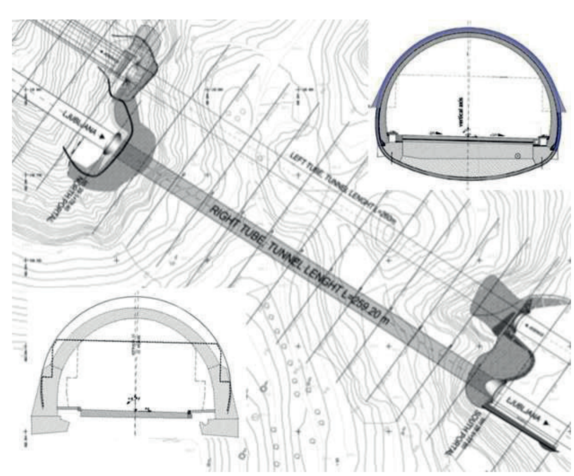


Figure 6: *Location of the new and old tunnel tube with maximum overburden of about* 30 m *and clearance profiles of the old tunnel before reconstruction and after it.*

were carried out to determine geological and geotechnical characteristic of ground materials present in the tunnel area. All investigations and geostatic analysis were carried out before the design of new tunnel tube which was constructed first before the reconstruction of old tunnel tube begun. Special attention was paid to the investigation of hard clay known as »Sivica« in which majority of the excavation works were done. Design of excavation and primary support lining was done in an appropriate way due to extensive amount of information acquired by geological mapping, boreholes drilling, Standard Penetration Tests and Pressuremeter Tests. The prognosis of geological structure is shown on the longitudinal profile in Figure 7. Distance between tunnel tubes is shown in the Figure 8.

The laboratory tests included measurements of moisture content, UCS, Triaxial Shear Tests and particularly attention was paid to the measurements of swelling potential and deformability of »Sivica«. It was found that in a dry environment »Sivica« has solid strength properties, while contact with water causes the relatively high presence of swelling potential. A large number

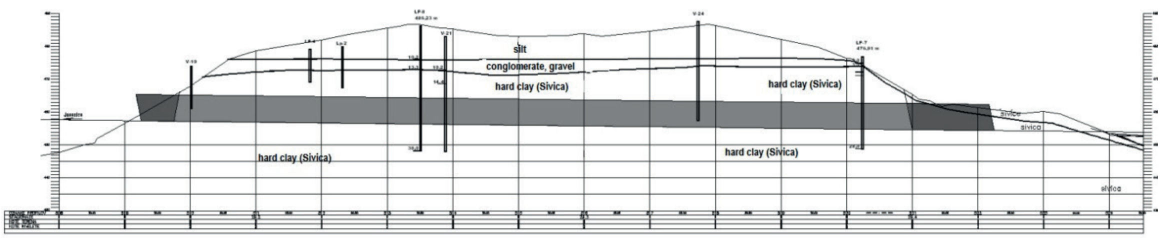


Figure 7: *Longitudinal geological profile of the new tunnel tube.*

Table 1: *Basic geotechnical parameters of the existing ground layers*

Type of ground	Moisture w/%	Unit weight γ/ (kN/m^3)	Youg Modulus E/MPa	Poisson ratio ν	Tensile strenght σ_t/MPa	Unconfined compresive strenght σ_c/MPa	Angle of internal friction φ/°	Cohesion c/MPa
Silty clay	25.0	20.0	5.0	0.4	/	0.001	25.0	0.0
Conglomerat	/	25.0	4 000.0	0.2	/	/	40.0	0.5
»Sivica«	7.0	24.0	200.0	0.3	0.5	5.0	31.0	0.136
Weathered »Sivica«	/	20.0	4.0	0.4	/	/	23.0	0.0

Table 2: *Laboratory determined sweling pressure at prevented strains and swelling strains in the unloading stress conditions*

Depth below the ground (m) (»Sivica«)	Primary vertical ground pressure σ_v/MPa	Swelling pressure p_{sw}/MPa at total prevented strains	Swelling strain ε_{sw}/% at water presece at total first unloading/second unloading conditions
25.0	0.600	0.250	2.0/3.5
20.0	0.480	0.350	2.0/4.0
16.0	0.385	1.45	1.8 /2.0
33.0	0.800	≥ 1.5	1.3/4.0

of laboratory tests to describe rock properties including swelling had been performed. The results were represented in Table 1 and Table 2.
One of the most important investigations was XRD Analysis which was carried out with the goal to determine the mineralogical content of the »Sivica« sediment samples. The diffraction patterns were identified with the data from X'Pert HighScore Plus software ICDD database. The compact gray clay samples were analysed with handheld XRF analyser NI TON GOLDD+ model XL3t He (50 kV) for major and minor element concentrations. The XRF result represents average of four measurements. Analyses were run with helium purge in the XRF in order to determine the degree of improvement in the signal detection attributable to helium's elimination of scattering by atmospheric gases. The analyser uses a 50 keV miniaturized X-ray tube and can quantify elements from magnesium through uranium. Data for all experiments consisted of counts (of X-ray fluorescence) detected per second. Total acquisition times were kept constant at 180 s. Bal variable is balance and incorporates all light elements from H to Na that cannot be detected with this XRF analyser.

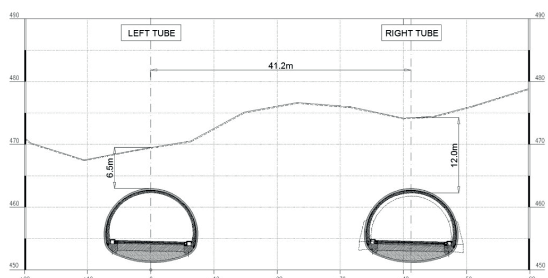


Figure 8: *Vertical cross section through existing (right) and new tunnel tube (left) with low overburden.*

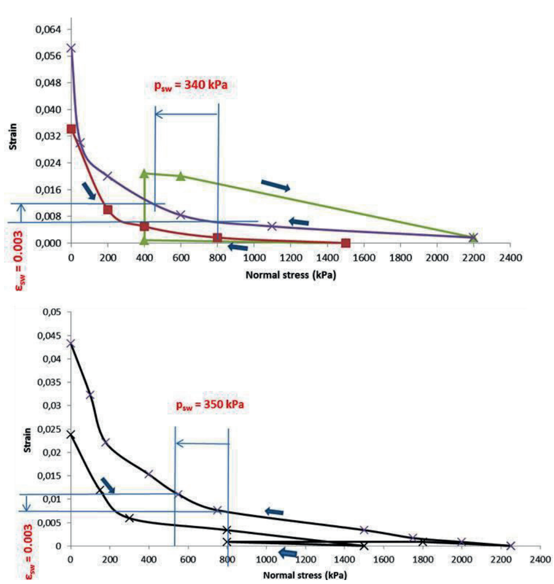


Figure 9: *Laboratory test results of swelling potential of the dark grey hart clay »Sivica« on the sample from the depth* 33.0–33.3 m *below the ground surface which is close to tunnel tubes locations.*

Based on given results as shown on Figure 10 the conclusion has shown that fresh clay sample does not contain any swelling minerals. The clinochlore (chlorite) in the clay sample does not change volume upon solvation with ethylene glycol, but only with the addition of water. When the chlorite mineral in the clay is altered by surface weathering, the alteration in the slate appears to be relatively rapid and complete. Direct surface exposure of the clay leads to the apparent irreversible dehydration of the surface layer of the clay. Weathering procedure is the manner in which chlorite ordinary alters when subjected to an atmospheric environment. It seems that alteration of chlorite leads to formation of mixed-layer chlorite-vermiculite and possibly to montmorillonite (Mg-saponite) which are well known swelling clay minerals.
From the results revealed the presence of swelling potential (Figure 9) and investigation based on XRF analyser (Figure 10), it cannot clear demonstrate the unique possibilities of activation swelling process during construction or reconstruction the tunnel tubes. Taking into account all results of swelling potential analysis, back geotechnical analysis of stability the old tunnel tube (Figure 5) and considerations in the mentioned scientific founding published in adequate journals in the deeper geotechnical design analysis used swelling pressure p_{sw} = 1 200 kPa.

Designed solution for tunnel construction and reconstruction

Due to the expected behaviour of »Sivica« the design provided technological measures to reduce the danger of water influence on the strength of the intact geological material. Immediately after excavation free surface was protected with shotcrete. Drilling and anchoring techniques without water usage were implemented. The purpose of these measures was to avoid contact between ground and any type of water and thus keep »Sivica« in its natural conditions. The basis for determining the swelling ground - support system relationship is a relation between the reactive support pressure and the tunnel wall deformations, as shown in the Figure 11 which is prepared on the basis of assessment usable swelling pressure including in accounting the stiffness of primary shotcrete lining. For assessment of relevant value of design swelling pressure, the elastic pre-deformations $\varepsilon_{ui} = 0.4\ \%$ and stiffness of shotcrete primary lining $k_c = 540\ \text{MPa}$ with adequate $\varepsilon_{sw} = 0.3\ \%$ radial strains were used in the equations based on close form solution method. The geostatic primary vertical pressure in the rock and the additional swelling pressure which was obtained from laboratory tests (Table 2) were included in the geostatic calculations.

The reduction of reaction support pressure was achieved by introducing the flexibility of the primary support system. Reserves in the capacity of proposed support system were established with a safety factor against the collapse. That was applied during the calculation of internal forces and bending moments from the results of 3D geostatic analyses which are described in the next subsection. Determination of the size of the damaged zone around the tunnel lining in the old tunnel tube was determined based on the examination of drilling cores, which have been acquired in the research stage of design.

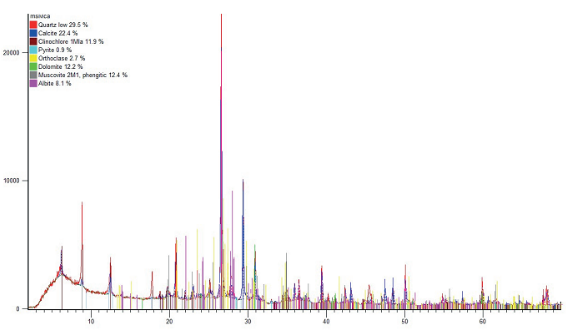


Figure 10: *Result of XRF analysis.*

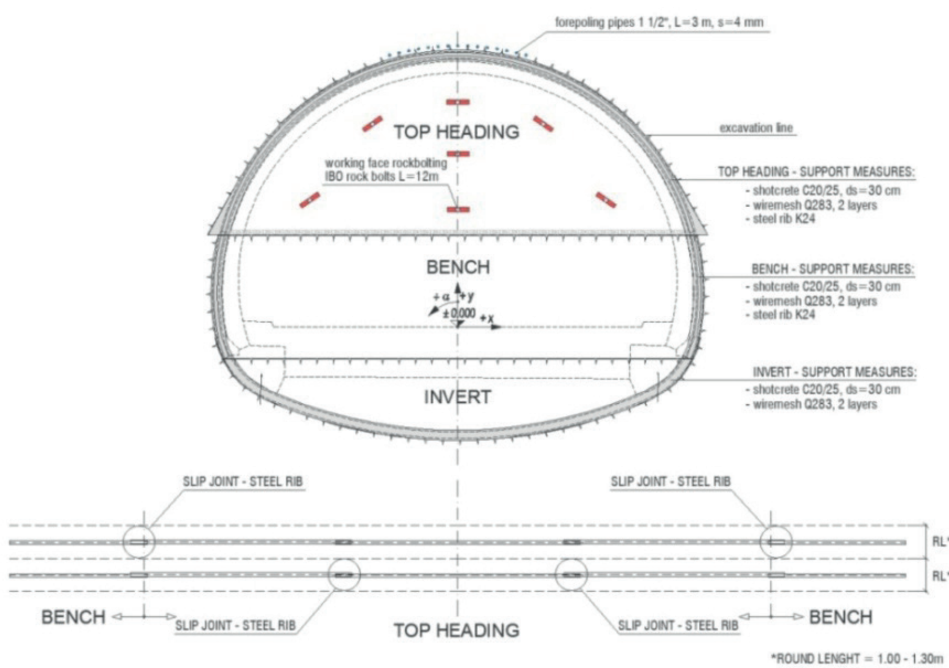


Figure 11: *Designed support measures for essential length of tunnel.*

This ranged from 0.5 m to the on the tunnel roof and around 1.0 m on the tunnel sides and in the floor without invert. During the reconstruction of the old tunnel tubes (Figure 13) the damaged layer around the old tunnel was removed, which is favourable for the establishment of long-term geostatic stability the tunnel system (Figure 12).

Based on the analysis of different theoretical approaches and practical cases of tunnel construction in swelling grounds including the geometrical data and thickness of ground layers cover, the rigid support system was applied. To follow this goal, the standard supporting elements have been considered in the design such as steel arches K24 and sprayed cement concrete thickness between 30 cm and 35 cm with two layers of wire meshes and rock bolts if required (Figure 11). The whole length of the tunnel tubes was scheduled with invert built from shotcrete with the same thickness. Rock classification was made according to Austrian standards OENORM B 2203.

Geostatic analysis of tunnels support measures

To verify the support measures designed as primary lining, extensive 2D and 3D numeric analyses, using Phase2D and MIDAS GTS computers codes, were done. The comparison between load cases without and with considering additional pressures caused by swelling (340 kPa) were carried out for two different design solutions. The swelling pressure was simulated with

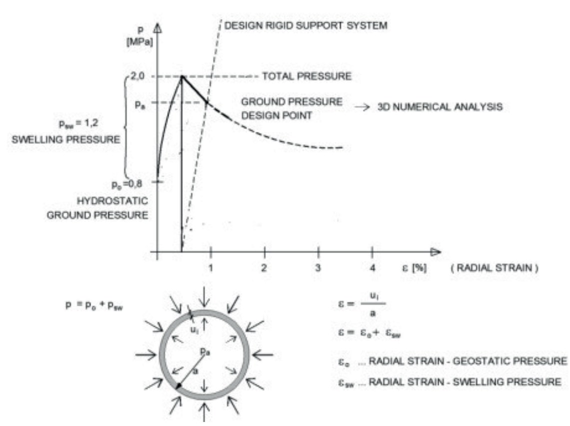


Figure 12: *Reactive support pressure versus radial strain in the swelling ground.*

proportionally higher unit weight of ground. In the Table 3 and Table 4 input ground geotechnical data for both numeric analyses are shown. The excavation of the tunnel was divided into a top heading, bench and invert, while on the longitudinal direction the excavation was simulated in progressive steps of 2 m in length.

The support elements in the calculations were planned with reinforced shotcrete lining with two steel meshes. Figures 14 and 15 shows the results of the calculation displacements without and with swelling pressure considered. In the first case the displacement could reach the value of 3.0 cm in the invert and, while in the second case, displacements were below 6.5 cm. The values of displacement for both calculations are shown in the Table 5. From the same figures the area of influence of tunnel excavation is in the worst case stretched up to 15 m left and right of the planed axis of the new tunnel, which means that the new tunnel tube excavation on the existing tunnel tube practically hasn't effect on stress-strain changes. The same can be concluded from the Figures 16 and 17, where the calculated Strength Factor (SF) is shown. If the values and distribution of main stresses Sigma 1 and Sigma 3, FS and total displacements around tunnel tubes are compared with each other, it is evident that the main concentration are present in the invert area.

This is mostly related to shallow depth of the tunnel locations and stiff static resistivity of primary linings incorporated in the low bearing capacity of surrounding ground.

3D FEM analysis were carried out in the similar way as 2D calculations. The first numerical simulation was done when the excavation procedure of the new tunnel tube was analyzed. Calculated total diplacement which are shown in Figure 18 has the maximum value approx. 8.5 cm in the invert and the minimum value approx. 3.0 cm on the crown. In the next short presentations of calculations results we can find that higher stresses and strains in the primary lining existed in the roof and invert parts of tunnels tubes (Figure 19 and 20). That are comparable with results of 2D analysis which were shown in Figure 14 to 17.

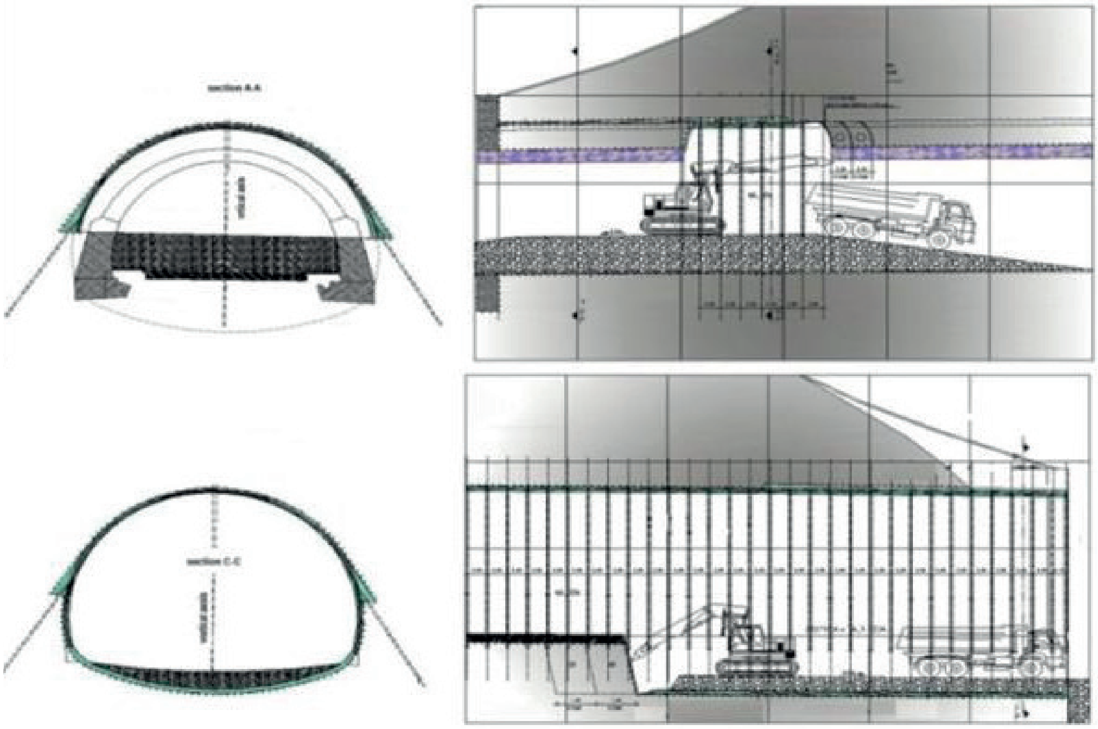


Figure 13: *Designed reconstruction procedure for old tunnel tube.*

Table 3: *Input geotechnical ground data for the analysis*

Type of ground	Unit weight γ/(kN/m^3)	Young Modulus E/MPa	Poisson ratio ν	Cohesion c/kPa	Angle of internal friction φ/°
Silt	20	5	0.4	1.0	25
Conglomerat	25	4 000	0.2	500.0	40
Sivica	24	200	0.3	136.0	31

Table 4: *Mechanical parameters of support elements*

Support element	Unit weight γ/(kN/m^3)	Young Modulus E/MPa	Poisson ratio ν	Thick of primary lining (m)	Wire mesh Q189 (pcs.)
Shotcrete (fresh)	25	3 000	0.30	0.35	2
Shotcrete (final)	25	15 000	0.25	0.35	2

Table 5: *Calculated displacements in the invert and side walls*

Object	Calculated maximum displacements (cm)			
	without swelling		with swelling	
	invert	side wall	invert	side wall
New tunnel tube	3.0	1.1	6.2	2.3
Reconstructed tunnel tube	1.1	0.7	1.4	1.0

The results of 3D simulations of the excavation and primary support installation for new and reshaped tunnel tube with respect to the swelling pressure of the surrounding rock are calculated depending on the depth and characteristics of »Sivica« and the size of the tunnel tubes. For both tunnel tubes calculated total displacements are between 2 cm and 6 cm at the roof and invert (Figure 21).

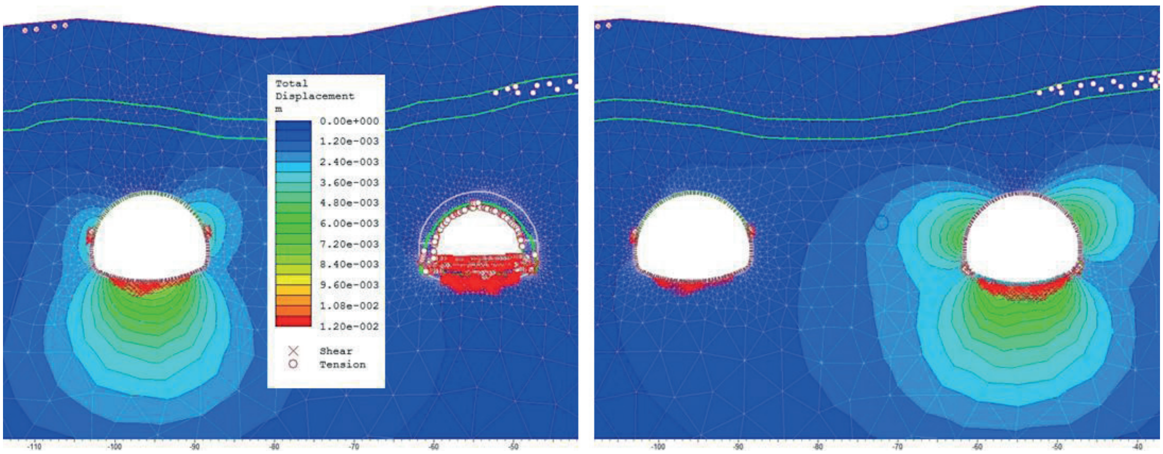


Figure 14: *Calculated displacements in the new and reconstructed tunnel tube without taking into account designed swelling pressure.*

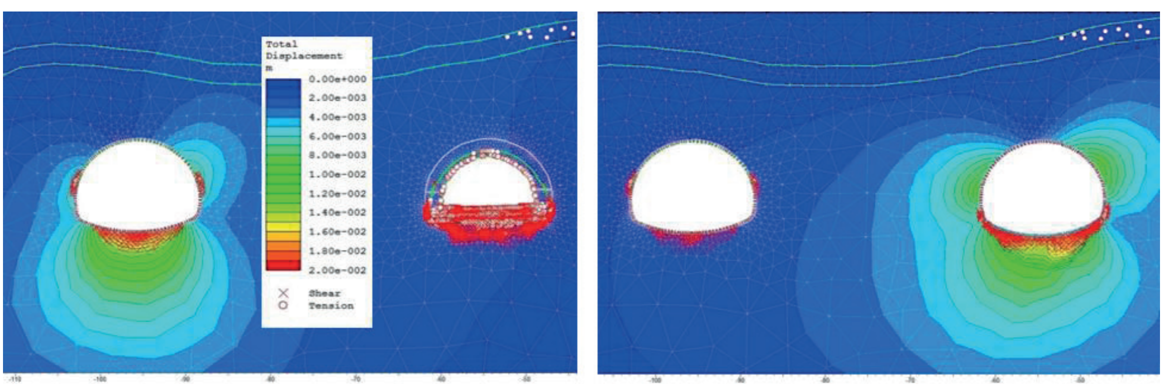


Figure 15: *Calculated displacements in the new and reconstructed tunnel tube with taking into account designed swelling pressure.*

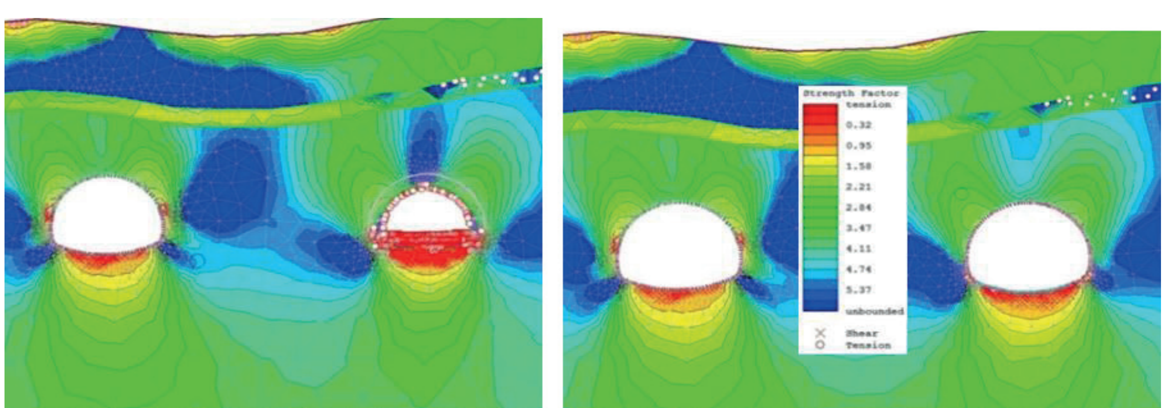


Figure 16: *Calculated Strength Factor (SF) in the ground for the new and reconstructed tunnel tube without taking into account designed swelling pressure.*

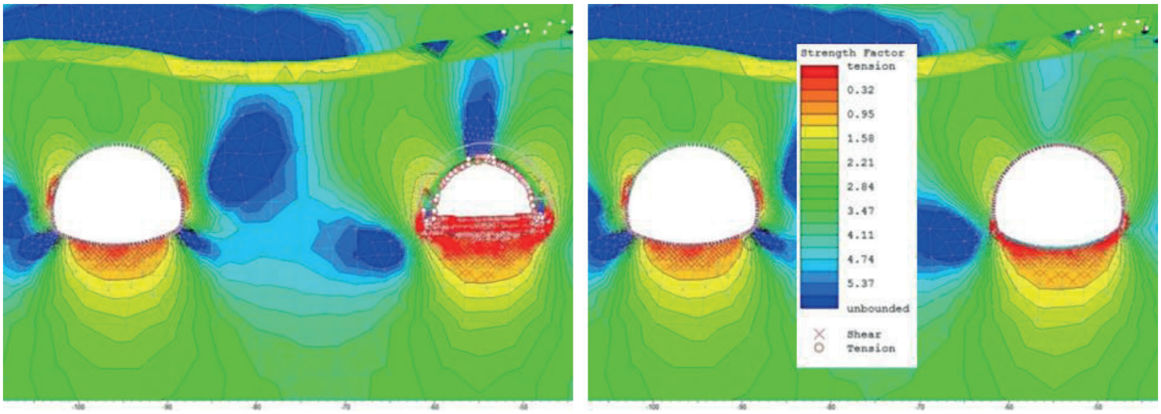


Figure 17: *Calculated Strength Factor (SF) in the ground for the new and reconstructed tunnel tube with taking into account designed swelling pressure.*

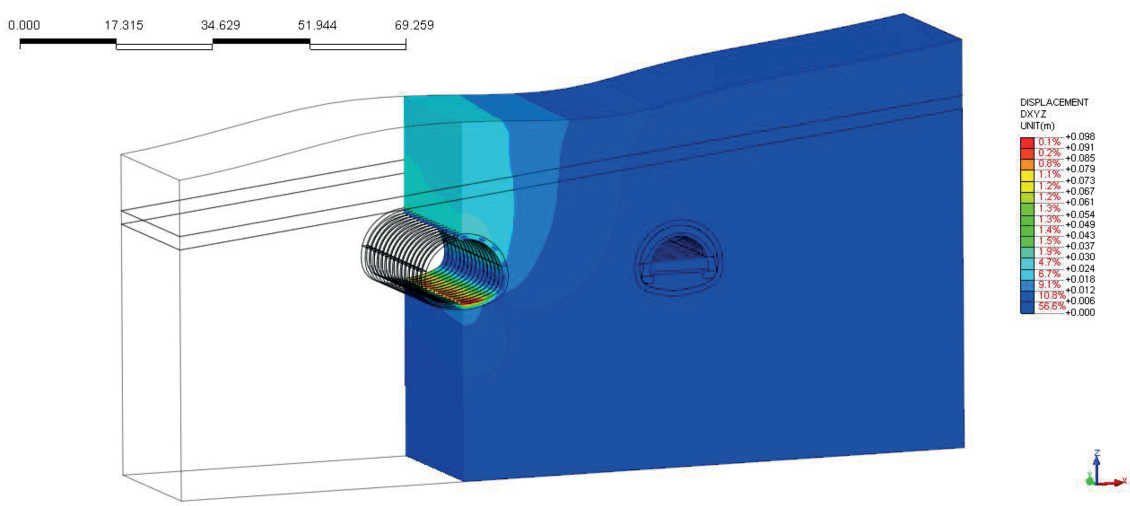


Figure 18: *Total displacements after excavation and support of the new tunnel (left tube direction toward Ljubljana).*

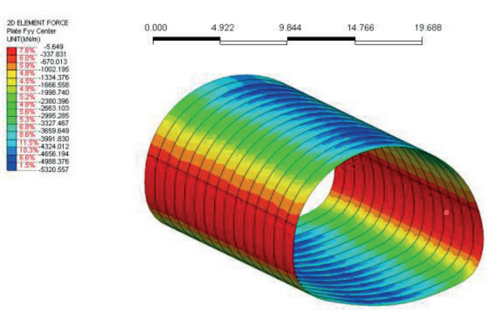


Figure 19: *Tangential axial forces in primary lining – new tunnel (left tube).*

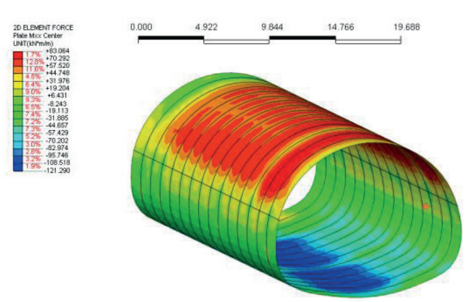


Figure 20: *Bending moments in primary lining – left tube.*

The maximum axial forces in the primary lining were found in the both sides where the calculated values are around –5 000 kN. While the smallest values of calculation results are shown in the roof and in the invert of tunnel tubes which amount to around –300 kN. It is worth noting, as their value is very small and is close to the tension zone, which is which is compensated by a double reinforcement wire mesh, so that provides sufficient safety against

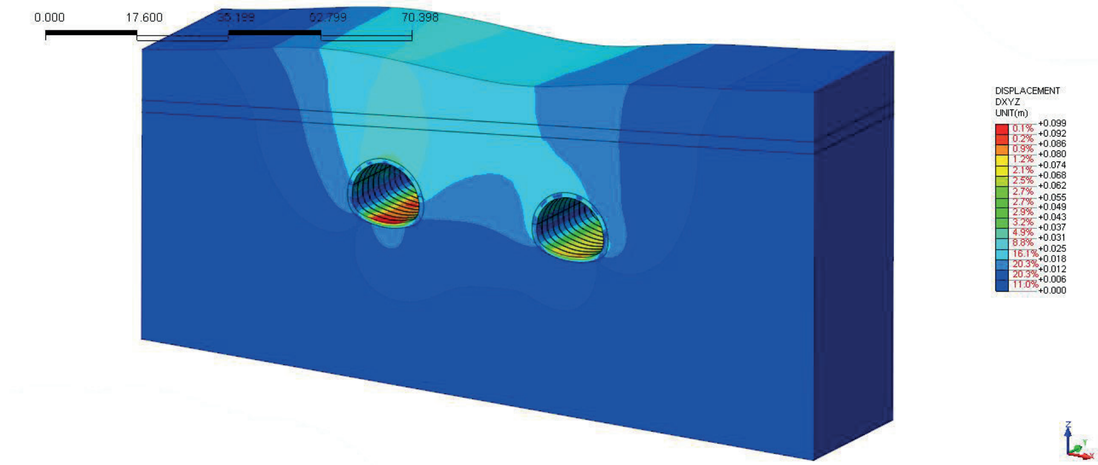


Figure 21: *Total displacements after excavation and support of the right tube (direction toward Ljubljana).*

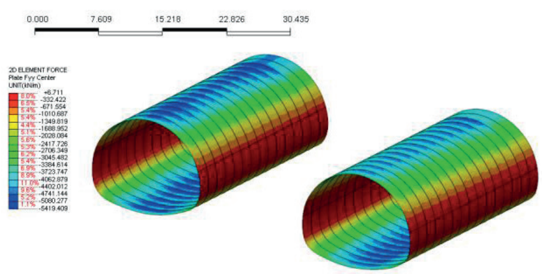


Figure 22: *Tangential axial forces in primary lining – left and right tube after excavation and support of the old one tunnel (right tube – direction toward Ljubljana).*

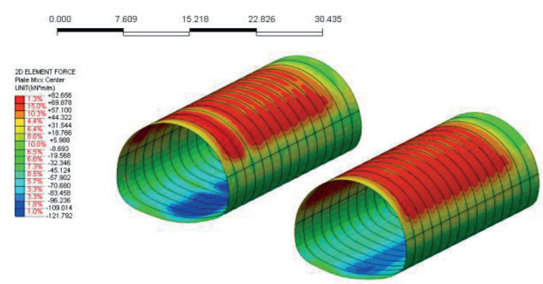


Figure 23: *Bending moments in primary lining – left and right tube after excavation and support of the right tube.*

the development of tensile cracks. Particular attention was dedicated to the construction of joint primary lining in contact between top heading and bench with invert. The results are shown in Figure 22. Analysis of the bending moments in the primary support shows that in the ceiling value regardless of the format of a positive in ground vault but negative. Their value ranges from about +82 kN m/m to about –122 kN m/m, which is not a problem for the provision of required stability (Figure 23).

Some distinctive features of the construction

A relatively short length of the tunnel was not suited for performing excavation with several phases going on simultaneously. Therefore, the excavation of the concrete invert was made first, followed by making a bench and top heading which were constructed together. Since the northern portal was not accessible, the whole excavation was made from the southern portal running towards the northern portal, where the breakthrough was made directly to the surface, meaning that no previous protective measures were necessary on the northern portal.

Figure 24 shows the breakthrough of the top heading in the inaccessible area. The last few metres of the top heading were built in the side gallery, with the cross section profile of approx. one third of the cross section of the top heading. This method of excavation reduced the risk of overbreak during the breakthrough. The breakthrough, as well as the excavation of northern portal and excavation of the remaining top heading of the tunnel, were made successively.

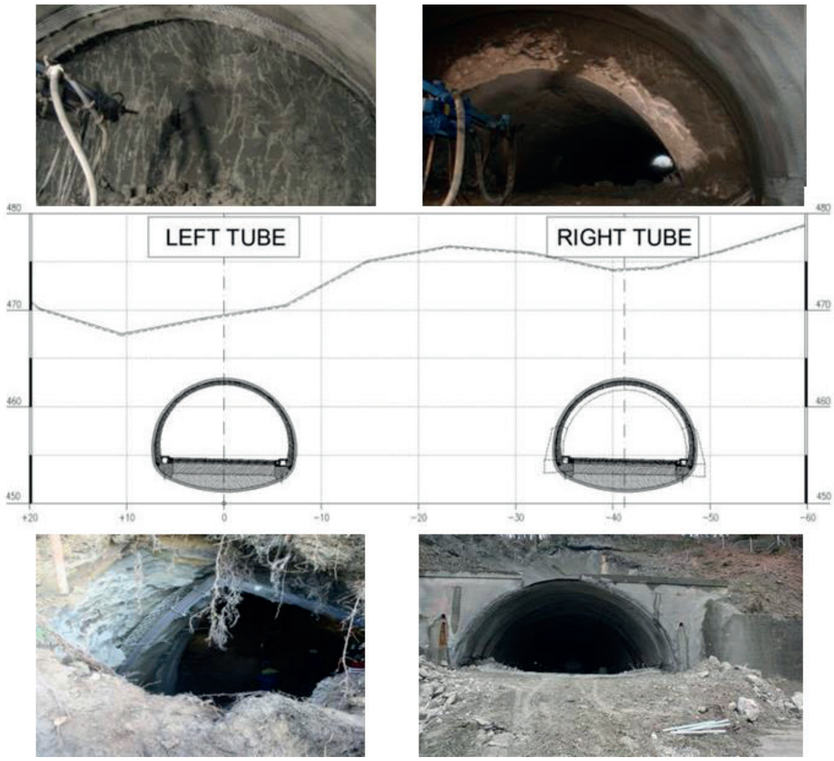
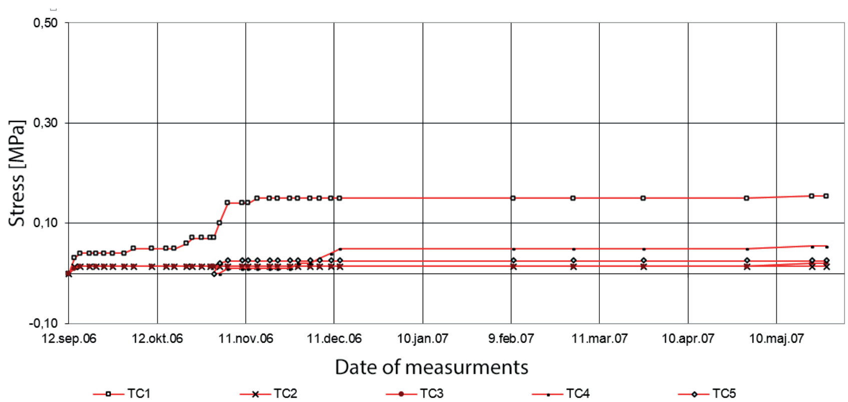
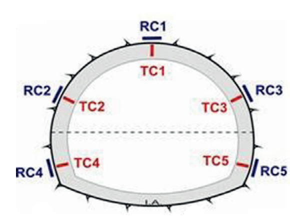


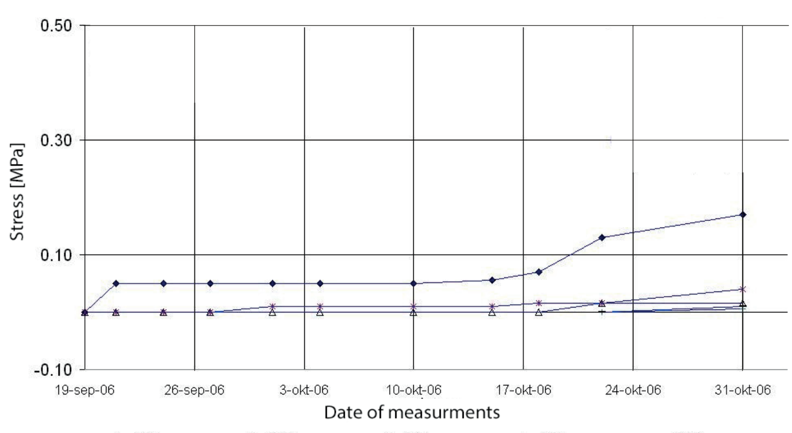
Figure 24: *Excavation face and breakthrough at the Nord portal of the new tunnel tube and reconstruction of the old tunnel.*

TANGENTIAL STRESSES IN THE PRIMARY LINING





RADIAL STRESSES IN PRIMARY LINING



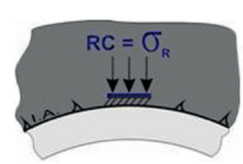


Figure 25: *Measurement results of additional circular and radial stresses in the contact between ground and primary lining in the new tunnel tube.*

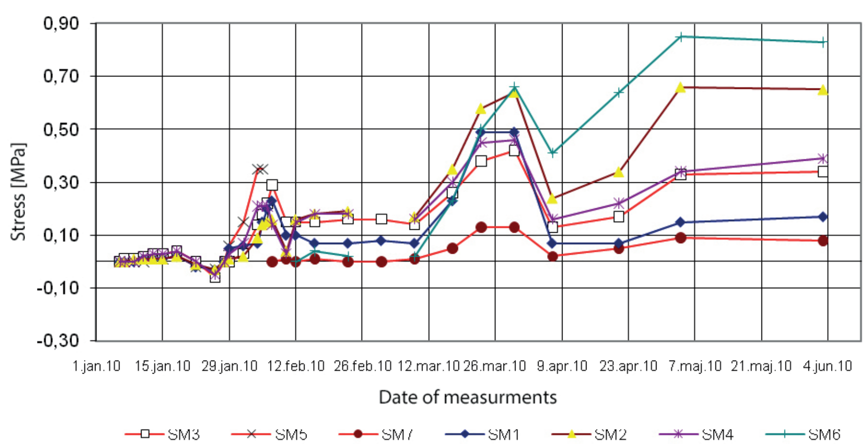
Gelogical observations and geotechnical measurement during construction and in operation

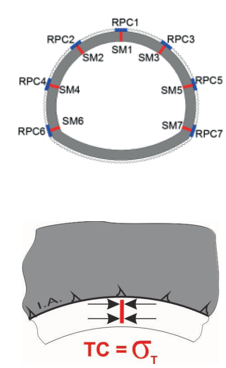
The results of monitoring measurements in pressure cells (and extensometers), which were installed during the construction of the new tunnel tube in the same geological geotechnical conditions did not show additional increases in circular (tangential) and radial stress (Figure 25 and 26) as a result of influence of the reconstruction of old tunnel.

In future the swelling potential will not produce additional stresses on the inner lining, because the primary lining has a sufficient loading capacity. The process which explains swelling pressure increase in examined ground shows that the closure of cracks depends of ground water isolation.

This is consistent with the results of extensive research of various swelling rock types [25, 26], which have rheological properties similar to hard clay (Sivica). It has been proven that naturally developed swelling pressures are much lower or minimum than those which are obtained from laboratory investigations. The determination of meaningful swelling pressure to be applied to the tunnel lining based on laboratory tests is largely depending on the scale effect, the stiffness ratio of the tunnel lining and surrounding ground and the in-situ stress and water conditions. Hence, swelling load conditions are difficult to predict in the reality. The displacements measured during the excavation were smaller than expected, i.e. less than 3 cm. This is probably due to more favourable geological and geotechnical conditions and high level of technological discipline of contractor. Measurements of displacements over time proved that the process slowed down and practically stopped one week after the excavation, with-

TANGENTIAL STRESSES IN THE PRIMARY LINING





RADIAL STRESSES IN PRIMARY LINING

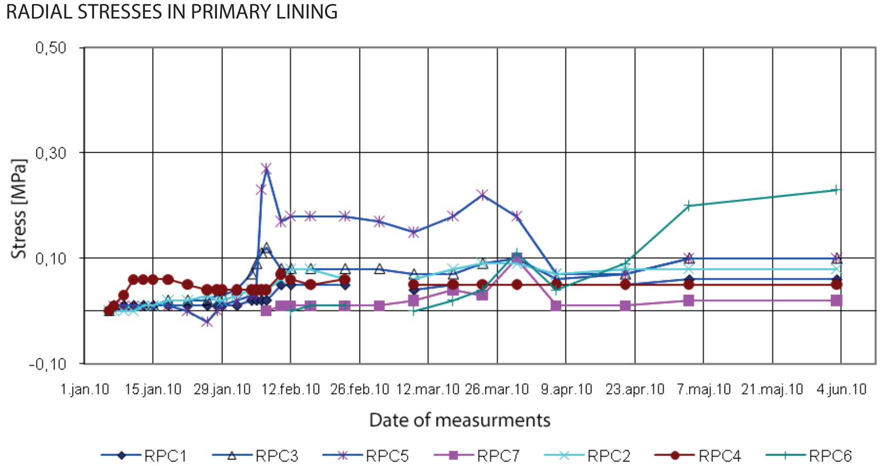


Figure 26:
Measurement results of additional circular and radial stresses in the contact between ground and primary lining in the reconstructed tunnel tube.

out any significant deviations. The excavation of the bench was followed by the excavation of the concrete invert and building of the foundations and concrete invert, at a suitable distance of about 30 m to 40 m.

During the excavation of the new tunnel tube, displacements of the lining in the adjacent existing tunnel tube have been monitored. Only minimal displacements have been noticed, mainly caused by vibrations and due to precise measurements and inaccessibility of the measuring points. The results of measurements have shown that the construction of the new tunnel tube has no impact on the existing tunnel.

Conclusions

In recent decades many studies were conducted relating to investigation of the construction of tunnels in the swelling rocks. The main goal of these investigations was determination the effects of chemical and physical processes on the swelling pressure development with time.

The essential distinction in the design of tunnels in the swelling grounds is depth i.e. primary stress state in grounds layer and other geological and geomechanical properties of geological materials.

Different approaches to research, interpretation of their results and suggesting methods of tunnel construction in these types of grounds are a good basis for the construction method decision for each case separately.

Substantial progress in understanding the development of swelling pressure is taken into account self- closure effect. This is important for the proper selection of construction technologies which should be at the time of construction imposed to strict control.

Successfully design and build a new tunnel tube and reconstruction of an old tunnel tube of the road tunnel Ljubno in the dark gray clay »Sivica« with swelling potential, is an example of good engineering practice in the construction of shallow tunnels in swelling rocks.

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