Displacement analysis of tunnel support in soft rock around a shallow highway tunnel at Golovec

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Abstract

Within the last 10 years Slovenia has been constructing its highway network. The Golovec tunnel, as a part of Slovenia’s capital ring is thus one of the most important connections of Ljubljana to the east and to the north. It is a double tube three-lane tunnel in soft rock with small to medium overburden. Its construction, following NATM, caused huge problems to all parties involved. The tunnel support was well monitored during its construction, which gave the authors a good opportunity to analyse the results.

The Golovec tunnel is constructed through one of few hills surrounding Ljubljana, of Carboniferous age, consisting of clastic rock: siltstone, claystone and sandstone. Golovec hill belongs to the first of two overthrusting zones from this area, so the rock is strongly faulted.

Tunnel monitoring consisted of daily 3-D tunnel tube displacement measurements in 97 measuring sections, and of two measuring sections within the tunnel with more complex measuring equipment, to monitor stress changes and rock deformations around both tunnel tubes. Monitoring of the surface 3-D movements gave us the opportunity to study the influence of the tunnel construction on the surface above it. The tunnel, its geology, construction procedure and monitoring results are described in the first part of the paper.

The second part consists of the interpretation of monitored results, with an emphasis on results concerning development and evolution of the excavation-damaged zone in the rock around the tunnel. Back-calculations, performed as a basis for the interpretation procedure, are also presented in this part. Calculations of the propagation of the tunnel destressed zone and stress-field around the tunnel, up to the surface, were performed by means of numerical model with the finite difference method. The evolution of tunnel displacements and their prediction was based on the use of Back Propagation Neural Networks, whose principles are presented in one chapter of this paper. Results showed that the most important, for the final settlement at the surface above the tunnel, was the time of installation and rigidity of the primary support. On the basis of the calculated final displacements, this support could easily be strengthened in a short time, when necessary.

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1. Introduction

Modern tunnelling methods include rock-support monitoring as an essential part into their construction.
program. New Austrian Tunnelling Method started 35 years ago in Austria. Germans quickly accepted this method as part of their underground construction programme (Haack, 1995). French have developed on this basis their convergence-confinement method (Panet, 2001). Swiss, though criticising the NATM, have also included monitoring into their underground construction program (Kovari, 1994).

Geotechnical monitoring during the construction time, immediate evaluation of results and quick reaction by means of support changes are basic ideas of this kind of tunnel construction (Steindorfer, 1998).

In Slovenia we have started with the first systematic geotechnical monitoring programme included in the construction process by the construction of the Karavanken tunnel (in 1986), though in some tunnels deformation measurements were sporadically taken even in the middle fifties (Šuklje and Grimsičar, 1954). Since the end of the Karavanken tunnel construction (Budkovič, 1991), all highway tunnels and even some remediation works have been done using of geotechnical monitoring programmes.

Geotechnical monitoring does not consist of tunnel tube displacement measurements only. In deformable rocks, we monitored, in some typical sections, the deformation or stress distribution in other parts of the tunnel support. In rock, we measured rock deformation. On the contact with shotcrete, we monitored stress changes; we monitored stress also within the shotcrete lining. Apart from this, the deformation within the shotcrete lining was monitored in some tunnels. In all soft rock tunnels, the anchor force distribution along the measuring anchors was monitored while in some of them; we monitored also the compression of the inner lining. Our experience is that the most instructive is deformation monitoring, because the pressure cells did not always give reasonable results, especially when they monitored radial stress components.

Since Slovenian independence, Slovene constructors excavated more than 10 highway tunnels of different lengths. Their excavation ran in different rock conditions, but many of them were constructed in highly deformable soft rock. Some of them were traced under high overburden, but Golovec tunnel is one of those, which do not have much rock above their roof.

Due to the instability of the tunnel entrance region and problems related to it, Golovec is the most expensive tunnel per constructed meter ever built in Slovenia.

Golovec tunnel is the first three-lane, two-tube highway tunnel in Slovenia. The tunnel is a part of the Slovenia’s capital (Ljubljana) ring (see Fig. 1). It is about 520 m long, with the overburden of up to 80 m. Its cross section is 148 m², with a height of 10.5 and width of 14.1 m. Details of the construction are given in Fig. 2. The two tubes were designed from 30 to 40 m apart. The designer considered the stress/strain interaction between the two pipes negligible.

Fig. 1. (a) Slovenian motorway network. (b) Ljubljana motorway ring with Golovec.
Geological and geotechnical prognosis was that rock conditions according to the Bieniawski classification (Bieniawski, 1974) were bad to medium. However, the reality showed later that they had been too optimistic. For the worst rock conditions, the designer has foreseen multistage excavation of the top heading with two head tunnels of smaller diameter. Later, when this method showed as insufficient due to roof stability problems, some major design modifications were applied.

The tunnel excavation started in June 1997, though the first preparatory cuttings for the portals, which caused a huge landslide on the southern entrance, started already in October 1995. Due to very difficult tunneling conditions, the tunnel breakthrough of the second pipe was in March 1999 and the tunnel was set to operation in August 1999.

Golovec tunnel was the first Slovenian highway tunnel using the 3-D displacement monitoring as the standard lining-deformation measuring procedure, supported with adequate software to present graphically the time evolution of measured displacements and their intermediate or final values in cross section.

2. Geology of the tunnel

Golovec is one of the few hills of the Carboniferous age raising out of Ljubljana tectonic depression in
its southeastern part. The depression is about 2400 km² large, dividing the Alpine region from the Dinara ridges. The originating movements started in the Pliocene, but the tectonic activity is still present in the Quaternary, which is today manifested by occasional earthquake.

The most influencing tectonics for the region occurred in two successive overthrusting phases, the first appearing between the Middle Eocene and Middle Oligocene and the second between the Miocene and the Pliocene (Premru, 1980). These overthrusts faulted many times later, with the main faults in the NW–SE direction (dinaric direction) and in the direction perpendicular to it (cross dinaric direction: NE–SW).

The Golovec hill belongs to the first of the two overthrusting zones. Due to the vivid tectonic history of this region, the rock is strongly faulted. General dip direction of overthrusts surfaces was to the SE.

As we already mentioned, the tunnel region is form of permocarboniferous clastic rock. This formation is extremely heterogeneous, because it consists of layers of sandstone, siltstone, claystone and tectonic clay. Siltstone and claystone are thinly folded. Usually layers of siltstone alternate with layers of claystone in thickness of few centimetres or decimetres. The share of each layer was locally changing.

Contact between siltstone and claystone is smooth which cause problems with tunnel stability if the dip direction of layers was into the tunnel. Layers of siltstone and claystone prevailed under the layers of sandstone.

Sandstone was usually fissured with three systems of joints in the distance of 10–30 cm. In some places, the layers of fissured sandstone reach the thickness of several meters. Quartz sandstone is mostly little grained with 5–10% dolomite and limestone. The contact between the sandstone and siltstone was usually highly tectonically deformed.

The fault zones and overthrust zones consist of tectonic clay with tectonically deformed sandstone and siltstone. In some places sandstone was deformed into the sand which indicate on high tectonically movements in the past.

Generally the dip direction of the layers was to the E or to the SE, but locally could vary quite a lot.

As the rock is very heterogeneous and anisotropic, the geomechanical properties from the laboratory tests were in the large range. The unit weight of rock mass $\gamma$ was 21–25 kN/m³, and wetness 8–12%, exceptionally more. The direct shear test showed the friction of angle $\varphi = 20–34^\circ$ and cohesion $c = 0–31$ kPa. The lowest parameters were measured in the slate direction. Uniaxial compressive strength for tectonically deformed rock was between 100 and 250 kPa. On the contrary siltstone and sandstone had uniaxial stress for several MPa. Claystone and siltstone had pressure-meter modulus from 150 to 800 MPa. In the highly tectonically deformed claystone and siltstone the pressuremeter modulus dropped to the 140 MPa. If layers of sandstone were present, the pressuremeter elastic modulus enlarged to 1 GPa or more.

Water permeability of the rock is low. Most of the water is thus running on the contact between soil, which is more than 10 m thick, and the unweathered bedrock. Where sandstone and conglomerate communicate with the surface, some water percolation into the tunnel occurred in the form of dripping. These water incomes into the tunnel are delayed for a week approximately.

Maximum water seepage from a spring, appearing in one place in the tunnel, was 0.2 l/s. Total water income from the tunnel was below 1 l/s.

Due to the fact that claystone looses completely cohesion in the presence of water, the water appearance induced huge stability problems, even in the tunnel pasts with dominating sandstone.

Longitudinal profile and ground plane is presented in Fig. 3. This profile was constructed after tunnel excavation, thus giving the most relevant information on geology from the constructing time. The geological interpretation is based on the geological records of the tunnel excavation and former geological investigations (Beguš and Sotlar, 2000). General structural orientation, the presence of fault zones and the thickness of weathered rock are clearly visible from the profile.

During the excavation, the prevailing rock conditions were as follows (Beguš et al., 2001):

- sandstone; fissured sandstone with thin interbeds layers of siltstone,
- siltstone; with thin layers of claystone and separate layer of sandstone, which not exceed in amount of 30%;
- fault zone; tectonic clay with tectonically deformed blocks of siltstone and sandstone.
Geology was daily mapped during the tunnel excavation, on the face and on the bench. The results were given on a geological longitudinal profile for each tube and in a geological map, all at the scale of 1/250. Where the rock was fractured and not too strongly crushed, three fault-set systems were measured. In faulted zones it was not possible to measure fracture orientation due to poor ground material. The

Fig. 3. (a) Lithological profile and settlement of the tube D. (b) Lithological profile and settlement of the tube C. (c) Lithological ground plane of the tube D and C.
rock was classified according to the RMR system. GSI classification was used in later stage (Hoek and Brown, 1997).

2.1. Details for the tube D

The southern portal was connected in the region of landslide, which occurred after the first earthenwork (Popović et al., 1998). One of the pile walls was built above the tunnel. In the southern part of the hill known as Golovec the thickness of the weathered rock is very high (Fig. 3a). The tectonically deformed siltstone lies on layers of sandstone, which were observed during tunnel excavation. In this area four collapses occurred. The estimated GSI of the rock was 15.

Along the tunnel mixed layers of siltstone are prevailing over the layers of sandstone. The GSI estimation was between 20 and 35. The tunnel was excavated in sandstone in two segments only. In the north part of the tunnel mixed layers are more tectonically deformed. Estimation of GSI was 1520.

Major faults cross the tunnel under a steep angle. Usually, the fault zone was a few meters thick.

2.2. Details for the tube C

The southern portal started in laminated sandstone (Fig. 3b). Because of the very fissured layers, smaller collapses occurred. The GSI was 35.

Tectonically heavy deformed rocks were encountered between chainage 3.140 and 3.055 km. Thick layers of sandstone lie at the contact of siltstone and between them a large overthrust zone was observed. In that zone rock was very tectonically deformed. The GSI was 15 for the overthrust zone, and 40 for the fissured layers of sandstone. In this section the largest tunnel deformations occurred.

Between chainage 3.055 and 2.890 km, layers of sandstone lie on the West side of the tunnel and on the East, layers of siltstone. Every side of the tunnel was built in different geological conditions. Estimated GSI for siltstone was 25 and for siltstone 40. Further to the north the proportion of siltstone layers grow up. For the siltstone estimated GSI was 25. On the north portal the layers of siltstone are tectonically deformed as in the tube D.

3. Construction and monitoring

3.1. Construction

Tunnel construction followed the NATM principles. The excavation started on the southern side of the hill. First excavation for the tunnel portals caused a huge landslide, 10–15 m thick in an area of 20000 m². The landslide was secured by material replacement in the lower part of the landslide (90000 m³ of
gravel), by two double-anchored pile-walls and by the construction of galleries in front of both southern portals. Similar solution with galleries was applied later also at the northern tunnel entrances, in order to prevent them from global instability.

Though the landslide movements in the south were thus stopped, the landslide had strong influence on tunneling within its first 50 m. The designer has foreseen excavation in two parallel heading tunnels in the top-heading, but due to several successive roof caving this approach was soon abandoned. Further step was grouting of the gravel above the tunnel roof, where original sliding material had been replaced for the landslide remediation reasons. This helped the contractor to proceed for the next 30 m. Finally, the tunnel entered quartz sandstone. Within this time, the contractor successfully changed technology by introducing pipe-roofing into the top heading excavation procedure. With the 23 m of unexcavated central core, reinforced by temporary bolting and shotcreting, the stability of the front has been assured.

The problem to be resolved remained the permanent squeezing of the tunnel lining behind the tunnel face. This was overcome by introducing a temporary invert on the top heading profile.

Ordinary advance cycle of the tunnel excavation consisted of (Likar et al., 1999):

- pipe-roofing: along the tunnel circumference in crown, 4’’ iron pipes, at a distance of 30 cm, were drilled ahead of the face and filled with mortar; their length was usually 1215 m, but not less than 9 m; in the worst rock conditions, the pipe roofing was doubled in an interval of 6 m (longitudinal covering of two pipe-roofs); in better rock conditions, starting from the northern side, only 6 m IBO anchors were used for this purpose;
- digging of the crushed and plastic soft rock around the central core and shortening of the central core advance to 23 m length;
- pre-shotcreting of the opened surfaces (1015 cm thick, with micro fibers);
- steel rib (TH29) erection and wire mesh installation;
- shotcreting, second mesh (which was later replaced by micro fiber shotcrete) and second layer of shotcrete (the whole thickness of shotcrete was 50 cm);
- construction of temporary invert reinforced with wire mesh;
- systematic bolting of face, roof and tunnel walls with IBO anchors of 9 and 16 m long with 1 m spacing;
- bench and invert simultaneous excavation and construction, about 5060 m behind the face.

The pipe-roof installation protects the excavation from collapses. The excavation step was between 0.8 and 1.2 m. The face of the tunnel was after the excavation protected with the shotcrete in a thickness of 1520 cm, wire mesh and anchors. Depends of the geological condition the tunnel face was supported with the support body, which was protected with the shotcrete too. After the excavation the installation of wire mesh, steel ribs, the second layer of shotcrete in a thickness of at least 20 cm was done. At the end anchors with the length 916 m were installed.

The inner lining, which was constructed when the displacement measurements showed that deformation of the primary lining had stopped, was constructed in the thickness of 30–50 cm.

Before this final technology was accepted, several face and roof instabilities occurred in the tunnel. Six times the cavings reached up to the surface; five times on the southern part and once on the northern side. Altogether we registered 53 cavings in a volume from 1 to 200 m$^3$ of rock. The majority of cavings (67%) appeared in the first 70 m of the tunnel, when the tunnel excavation took place below the landslide and when the construction method was not yet well defined.

Tunnel face instabilities were more constant along the tunnel, until the contractor started with excavation from the north. This problem was finally overcome, due to the more favourable dipping of the discontinuities (towards the south).

The tube D was broken through in August 1998 at 2.85 km and the tube C in March 1999 at 2.83 km.

3.2. Monitoring

Monitoring, which is an essential part of the NATM, was quite extensive after the designer’s program, but due to huge construction problems it has still been enlarged. Usually we monitored displacements of five observation points per each measuring
section in the primary tunnel lining. Furthermore, in five measuring sections we checked the tunnel support stability by means of stress and deformation monitoring in the lining and in the surrounding rock (which we consider the most important part of the tunnel support).

Support displacements were monitored in two different ways:

– in the primary lining and in the galleries at the tunnel entrances we used surveying method of 3-D convergence measurements; their accuracy was ±2 mm, which was considered as good enough for the measured displacements (about 300 mm in average);
– the secondary lining, which is more rigid required more accurate measuring; in this case we used measuring tape, whose accuracy is higher than 0.3 mm. The monitored distance was controlled between two points in the tunnel walls, some 1.5 m above the pavement.

3-D measurements were taken from three points in the crown and from one point from each of the tunnel walls. Measured points were fixed on the lining and measured as soon as possible, behind the tunnel face or behind the excavated bench, so that we could measure the majority of developed displacements. When the observation points were destroyed for some reason, we immediately replaced them and calculated cumulative displacements.

Results were promptly forwarded to the Contractor and to the Engineer in the form of tables. Weekly reports with time/displacement diagrams and with transversal cross section diagrams, together with comments, were made. Displacement measurements were taken daily in all measuring profiles with growing displacements. After that, occasional controlling fortnight measurements were taken.

Altogether, 97 measuring sections for deformation measurements were set in the tunnels which brought to one measuring section on 11 m of tunnel in average (without galleries). This is quite a high density, resulting from the stability problems in the tunnel.

In addition to the these measurements, one more complete measuring section was set up in the primary lining of each of the two tubes. The purpose was to monitor the influence of tunnel construction on the surrounding rock and to monitor the stress and deformation changes in some of the tunnel supporting elements. The two measuring sections consisted of the following equipment (see Fig. 4):

– 7 points to measure 3-D displacements of the tunnel lining;
– 7 simple extensometers, \( L = 12 \) m (E1–2);
– 7 simple extensometers, \( L = 9 \) m (E9);
– 7 multipoint (6 points) extensometers, \( L = 1–6 \) m; (E1–6);
– 2 measuring anchors, \( L = 6 \) m, \( F_{\text{max}} = 350 \) kN, 4 measuring points (M1,2);
– 2 measuring anchors, \( L = 9 \) m, \( F_{\text{max}} = 350 \) kN, 4 measuring points (M3,4); pairs of strain gauges (A1/1.2–A7/1.2);
– 3 pressure cells to measure tangential stress in the shotcrete lining.

Apart from this, another three measuring sections were set into the inner tunnel lining. Since the influence of the second pipe excavation was suspected in the lining of the first tube, two measuring sections were set to monitor the stress and deformation changes within the secondary lining of the first pipe, at the time the excavation of the second one. The third measuring section was intended to monitor the deformation field after the construction of the secondary lining in the second pipe, in the section where the largest surface settlement deformation had been monitored.

The readings in the complex measuring sections were taken daily in the first week, and then three times in the second week. Afterwards, the readings were

![Fig. 4. Geotechnical measuring profile.](image-url)
taken once a week until the end of the tunnel construction. The three measuring sections in the inner lining are still operative today in the tunnel exploitation phase.

Due to the large landslide generated in 1995 on the southern slope, the monitoring program was extended also to the surface. The monitoring of the surface consisted of:

- surveying of 43 observation points (3-D displacements): on the landslide, on the pile walls and on the entrance galleries of the southern entrance (24 points) and on the northern slope (19 points),
- measurements of inclination (22 boreholes): 7 on the sliding area and in its near vicinity and 15 on the northern slope above the tunnel,
- measurements of settlement of 30 observation points on the local road, close to the top of the hill and of further 14 points above the northern tunnel entrance,
- multipoint extensometer measurements from the surface down to the tunnel in two measuring points.

Surface measurements were taken once a week to once a fortnight, depending on the face advance and its position.

3.3. Monitoring results

Continuous 3-D displacement monitoring on the tunnel support helped us to evaluate the support appropriateness, mostly on the basis of the convergence velocity. In this chapter, the tunnel displacements are compared to the geology and to the displacements measured on the surface, above the tunnel.

Monitoring showed the following:

1. The largest part of deformation of the tunnel lining developed in the vertical direction as settlement of the whole tunnel, which was quite a surprise.
2. The crown settlements were the most intensive: the displacements were inclined towards the center of the tunnel, either in the roof or in the crown walls; the values were extremely high, with the maximum at 860 mm (cumulative in a remedied profile—not taking into account the part of the tunnel with instability problems due to the landsliding); in Fig. 5 it is a typical cross section with measured displacements.
3. Time/displacement diagram (see Fig. 6) shows that within the bench excavation phase, the displacements double, comparing to the top heading stabilized values.
4. Generally, displacements are higher in directions, favoured by discontinuity planes; when the rock is more shaley and when it is strongly tectonically deformed, the displacements are higher and not uniform; in these cases the deformation of the tunnel tube became asymmetric and cracks of the tunnel roof frequently appear.
5. Even more than on geological situation, the displacements depended on constructor’s working accuracy and quick reaction to the geological conditions: The sooner the invert was constructed and the tunnel ring was closed, the smaller the final deformation was; tunnel excavation heading from the north gave much smaller settlements than heading from the south in the same quality of rock.
6. The most important element for the stabilization of deformation was the final invert; when the distance between the tunnel face and invert was larger (80 m or more), huge deformation was measured even in relatively good rock conditions; in bad rock conditions, the displacements stopped in 10–14 days after the final invert construction; in better rock it took only few days.
7. The impact of excavation between the two tubes was proven: in average 45 m ahead of the second tube face, the deformation of the first one
increased, though the increase was not very high; the impact was higher in sectors with worse rock conditions.

8. Settlements of the tunnel crown were well manifested on the surface above the tunnel along its whole length, though the maximum overburden exceeded 50 m in more than one-third of the tunnel length; on the Southern slope, the first settlements on the surface appeared in average 70 m ahead of the tunnel face, while on the Northern slope, due to the more favourable dipping of discontinuities, this distance was reduced to 20–30 m.

9. The settlement above the tunnel was monitored on the southern slope in a belt, about 150 m wide; on the northern side, its width was reduced to about 100 m.

4. Interpretation of two monitored cross sections

In two tunnel sections, a more detailed study of behaviour of the rock/primary tunnel lining was made. We had the luck to set one of the two measuring profiles in the sector where the largest and symmetric displacements have later occurred (we will call it Max profile). We could consider the second as central, since the measured displacements were close to the average values from the tunnel. Besides, above this section and perpendicular to the layout of the tunnel, a monitoring profile of 30 points, measuring the surface settlements, was set up in order to compare the measured results with the ones from the tunnel.

In this chapter, we would like to give the description of all major factors for the two measuring sections which were influencing the evolution of the tunnel support deformation: position of the two profiles (overburden, geometry), geology, construction procedure, and remediation works.

4.1. Max profile

The Max profile was situated in the tube, which was excavated as the second one. This could also indicate that the rock had already been damaged to some extent due to the excavation-damaged zone from the first pipe construction. It lay beneath the southern slope of the Golovec hill, which means unfavourable dipping of most of discontinuities. Its position, about 162 m from the tunnel entrance, meant that the area was no more directly influenced by the landslide above the tunnel. The Max cross section lay around 60 m beneath the surface. The primary stress field was not measured at any point.

Geological profile for this section can be seen in Fig. 7. Geological conditions could not be estimated as the worst ever met in the tunnel. One can see that prevalent rock in the lower Western part of the section was the quartz sandstone, though strongly crushed.
Some smaller water incomes in the form of dripping have been detected. The bedding planes were dipping towards the Southeast at uniform angle of 30°. Among the sandstone bedding planes we found entirely smooth clay-schist layers. On the upper part of the section, the rock became more shaley; there was a mix of equally partitioned sandstone and siltstone. RMR gave a value of 15.

In longitudinal cross section this profile is located, 25 m behind the strongest water income into the tunnel.

Construction: the top heading advanced across the region of the profile ± 20 m in a speed of 1.3 mm/day. The bench on one side was excavated about 1 month behind the face, while the tunnel profile was closed with the invert 50 days after face excavation. Due to strong deformation successive remediation measures like additional anchoring and shotcreting had been made around the profile just before the secondary lining was put in place.

The 3-D displacement monitoring of the crown and tunnel walls are presented in two figures. Fig. 8 gives the values of final displacements. The largest displacements were measured in the Eastern wall (excavated bench), mainly in the direction of settlements. The influence of geological structure on displacements was clearly seen: the Eastern wall has suffered more than double displacements than the Western one.

The time/displacement diagrams for vertical and horizontal directions can be seen in Figs. 9 and 10. Each of the construction stages is clearly visible on the plot: top heading, bench-East, bench-West and invert construction, after which the deformation stops.
In May 1999, the remediation works are also visible on the time/displacement diagram.

**Fig. 11** shows the rock displacements, up to 14 m from the tunnel lining. The displacements are given for 10 days after installation and for the final stage—in the direction of extensometers (Fig. 12). It is clearly seen that on the Western side, in accordance with the geological structure, the excavation-damaged zone was far larger than the reach of extensometers. Thus, far before the final stage, the Excavation Damaged Zone (EDZ) reached the surface above the tunnel. **Fig. 13** gives the time development of measuring anchors deformation at different depths. At 1.5 m from the wall, the anchor reached its elastic limit already in the first week. At the distance between 3 and 4.5 m, this happened in 45 days. Similar, but smaller deformation was measured on the Central profile.

### 4.2. Central profile

The Central profile lay about 223 m from the south tunnel entrance of the first pipe. At this place, the overburden was about 70 m thick. Geological situation on this profile can be seen in **Fig. 14** and measured lining displacements can be seen in **Fig. 15**: lithological situation was very similar to the one from the Max profile. The front showed that siltstone

![Fig. 10. Horizontal displacement for Max profile.](image1)

![Fig. 11. Rock displacement after 10 days.](image2)

![Fig. 12. Rock displacement at the final stage.](image3)

![Fig. 13. Axial force in anchors.](image4)
and sandstone were equally represented on the profile in mixed layers. Tectonic zone was prevailing in the Eastern side. Water incomes were not detected.

The top heading advanced across this section ± 20 m in a speed of 1.5 m of mean daily advance. No stability problems occurred through all stages of excavation and construction of this section.

The bench and invert were excavated and constructed simultaneously on each side with delay of 1 week between both sides. The invert was constructed and the tunnel ring closed 66 days after the face excavation. It proceeded 50 m behind the face.

Evenly distributed displacements on the section are seen in Fig. 15. We appoint this to the more favourable lithology on the Western side.

One can observe the rock displacement distribution around the tunnel. It is clear that the EDZ reached the surface and that we can expect the interaction between the EDZ of the two pipes. What was also interesting was that the EDZ was not homogenous but contained some compressed and other more loosened parts, which we appoint to the geological situation.

Deformation of measuring anchors showed that they were in the plastic range 1 week after their installation.

4.3. Conclusions from both profiles

Conclusions from the two measuring profiles were the following:

- The excavation-damaged zone above the crown spread so fast around the tunnel, so fast that we could not follow its initial spreading even with the 14 m long extensometers. The anchors in the vertical direction could therefore not be expected to give much contribution to the tunnel support.
- Extensometers, measured anchor forces and tangential pressure cells in the shotcrete lining showed undoubtedly on the influence between the two excavated tubes on the Max profile, though at this profile, they were excavated ca. 40 m apart.
- Measuring anchors in bench and in the crown showed that at the length of up to 1.5 m from the tunnel wall, most of them were in plastic range of deformation already the second day after installation. Anchors should be installed immediately behind the tunnel face and should have a minimum length of 9 m. Their maximal load should be no smaller than 350 kN. They should be installed more at shorter distance than they were. Additional anchoring far behind the face has doubtful results. When they are installed at a later stage, they do not much contribute to the prevention of spreading of the EDZ; they are just preventing the EDZ from further relaxation.
- When the support measures were late or if they were insufficient at the first stage, the EDZ spread very fast from the tunnel into the rock and it soon reached the surface above the tunnel. Thus, most of deformation from the tunnel appeared on the surface in this rock type. The deformation was monitored not only above the crown, but also behind the tunnel walls and the results showed also large deformation.
Rock is the most important part of the tunnel lining, which is clearly seen from the developed displacements and geological structure in the cross section. The proposition was to support it in accordance with the measured deformation: on the side with larger displacements, thicker shotcrete and more anchors should be applied.

Closing of the tunnel support with the invert led to stabilization of the whole support system.

5. Calculation

Parallel to the tunnel construction, some computations were performed in order to predict the final tunnel displacements from the first seven measured data and to propose to the contractor the optimum support measures at the right time. On the other hand, for better understanding of the measured values, a numerical model was set up for the maximum displacement case. In this way, we reproduced by computation the stress and strain field within the soft rock around the tunnel, up to the surface.

At first, the final displacement values were predicted by the use of neural network method, while the stress/strain field was computed using the finite difference method.

5.1. Neural network displacement prediction

Neural networks are becoming more and more important tools in geotechnical engineering nowadays.

![Fig. 16. Signal flow in a back propagation network.](image)

The Back Propagation Neural Network (Neural Ware, 1998) was selected for the use in our case. Its basic principle is in learning from known examples: it is a procedure of iterative computation and comparison of computed results with the correct input data.

We have chosen this method over the others due to its ability to learn from known examples, which we possessed in sufficient quantity.

Before the prediction of final tunnel displacements, a teaching process of the neural network mesh had taken place on 43 learning cases from this tunnel, all in the same rock, and all under similar conditions of excavation. In most cases, the first seven readings, obtained within the first 10 days of monitoring were put into the base and the results from the neural mesh were compared to the final measured values.

The back-propagation network consists of an input line, one or more hidden lines, and an output line. The

![Fig. 17. Comparison between measured settlement and calculated settlement with NN.](image)

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Table 1

<table>
<thead>
<tr>
<th>Material</th>
<th>(\gamma) (kN/m(^3))</th>
<th>(E) (GPa)</th>
<th>(f) ((^\circ))</th>
<th>(f_{rez}) ((^\circ))</th>
<th>(C) (kN/m(^2))</th>
<th>(C_{rez}) (kN/m(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sanstone</td>
<td>2600</td>
<td>1</td>
<td>32</td>
<td>26</td>
<td>80</td>
<td>30</td>
</tr>
<tr>
<td>Siltstone</td>
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<td>0.4</td>
<td>28</td>
<td>22</td>
<td>50</td>
<td>10</td>
</tr>
<tr>
<td>Tectonic zone</td>
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<td>0.1</td>
<td>20</td>
<td>18</td>
<td>5</td>
<td>0</td>
</tr>
<tr>
<td>Weathered rock</td>
<td>2100</td>
<td>0.05</td>
<td>24</td>
<td>20</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>
back-propagation algorithm is repeated until the difference between the desired and the computed result converges to a minimum. The hidden neurons are linked with input and output neurons. The construction is presented in Fig. 16.

These results were obtained by computations with the Neural Back Propagation Mesh, with two hidden layers, having in the first layer 60 hidden neurons and 40 in the second one.

The correlation coefficient between the measured and calculated values was 0.7. Graphical presentation of these results is given in Fig. 17.

Thus, it became possible to predict final displacements of the tunnel tube on the base of the first few days’ results. In this way, the contractor got the possibility to react at an early stage of deformation with the support reinforcement, and thus prevent the whole support system (rock included) from uncontrolled deformation and consequently from too large radius of excavation-damaged zone.

5.2. 2-D numerical model

Numerical model was set up in two measured sections: in Max profile and in the central profile. With the two models we tried to better understand the process of development of the stress and deformation fields around the two tunnel tubes. The data from the site investigation phase during the design of the tunnel were used as input data, but soon it showed up that we needed more data about geomechanical rock characteristics. We got the missing values from backanalysis of the measured deformation in both tunnels and on the surface.
Modeling was done by the finite difference method (Itasca, 2001). A strain-softening model was used. It allows nonlinear material behaviour based on Mohr–Coulomb model properties. The model covers the space between and around the two tunnel tubes, up to the surface. An attempt was made to estimate the degree of rock deformation before the measuring profile was put in place. We tried to calculate the “invisible” rock deformation, ahead and immediately around the tunnel face, developing within the first day of excavation.

Table 1 gives the values of relevant rock parameters used in the models. Geometry and the geology of the two profiles can be seen from Figs. 18 and 20.

Fig. 20. Geology profile for the Central profile.

Fig. 21. Y displacement around the tunnel D (left) and C (right) for the Central profile.
Modeling of the Max profile run in the following steps: first, the excavation of the first tube was simulated. Rock relaxation was an input parameter; it was computed through the iteration process of input relaxation value and comparison of calculated to measured settlements of the surface. Finally, the second pipe excavation was simulated (Fig. 19).

The Central profile (Fig. 20) was modeled for the case, where, besides the tunnel deformation, also the deformation of the surface above the tunnel was measured in detail, in a cross section of 30 measuring points. The same input parameters were taken for this case, since the rock was the same rock mass characteristics as in the previous one. First, the simulation of the first tube was made. In the second step, the simulation of excavation of the second tube was done (see Fig. 21). Comparing the simulation results with the measured ones for the surface settlement gave us results, presented in Fig. 22. As one can see, there is a good agreement between calculated and measured results on the surface above the two pipes, in the middle of the model, while the difference becomes larger towards its borders, where the fixed boundary conditions influence on results. The deformations ahead the tunnel face was considered with relaxation of 30–35% of the final deformation. Those deformations could not be measured in the tunnel but was registered on the surface.

From the model, the importance of a good assessment of the first rock deformations ahead of the tunnel face is clearly seen. Modelling has also clearly shown that the tunnel displacements permitted on the tunnel face predicted the final settlements at the surface above the tunnel.

6. Conclusion

The Golovec tunnel and the problems at the beginning of its construction following NATM gave us a unique opportunity to study and analyse the tunnel/rock behaviour for the case of a large span tunnel in soft rock with small to medium overburden. From geological data, systematic wall deformation measurements and detailed measurements of rock deformation around the tunnel up to the surface, we tried to analyse the mechanisms of development of the excavation-damaged zone around the tunnel in space and time. Monitoring of rock displacements and tunnel deformation led to the following conclusions:

- good agreement between calculated and measured results were obtained when 30–35% of rock relaxation was taken into account, which meant that this range of deformation developed before the tunnel face and in its close vicinity;
- the major displacements component of the tunnel walls was in vertical direction;
- the tunnel convergence was not stopped until the pipe ring was entirely closed with the invert construction; the invert installation was thus the leading factor determining final deformation.

The space distribution of rock displacements was analyzed by numerical modeling (finite difference method), with Mohr–Coulomb criterion in strain softening constitutive model. The time development and prediction of the final displacements through the first measured data was achieved by means of Neural Networks. The results showed that the most important for the radius of the relaxed zone around the tunnel and consequently for final settlement of the surface above the tunnel was the time of installation and rigidity of the primary support. On the basis of prediction of final displacements, this support could easily be intensified in a short time, if necessary.

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