Measurements and Analysis of Tunnel Deformations

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Abstract: The paper describes the programme of the monitoring as well as selected results of the analysis of deformation processes, observed during excavation of two tunnels, each of 9m in diameter and approximately 300m long. Tunnels are located at the depth of about 55m below the terrain, in flysch rock mass in Carpathy Mountains (Poland). Systematic measurements of the roof settlements, convergence, rock mass delaminations as well as settlements of the terrain surface above the tunnels allowed for a collection of comprehensive results. The paper presents also the models for approximation and prognosis of these deformation processes.

Key Words: tunnel, rock mass, deformations, monitoring

Introduction

Realisation of underground engineering constructions (e.g. tunnels, shafts, chambers), with taking into account the principles of New Austrian Tunnelling Method, should be accompanied by monitoring of construction + rock mass system behaviour. Results of such monitoring provide important information, if the excavation procedure and support measures are appropriately designed and performed, i.e. if the system is stable. The basic data in above considerations are the results of deformation measurements, carried out in time of tunnel excavation. Analysis of these results allows to recognize the development of deformations in function of time and informs about stabilisation of the construction. Basing on these results, design assumptions could be confirmed or revised, if necessary.

The deformation measurements were systematically performed in two tunnels, in time of their realisation. The tunnels were excavated inside the slope of the river valley. These will allow for the outflow of water in the construction and exploitation stage of the dam and water reservoir. Rock mass is built of sedimentary flysch formation. It is composed of strong and hard sandstone beds, interbedded with weak and soft clay shales. The average sandstone content is equal to about 70%. Rock mass is intersected with two systems of joints, approximately perpendicular to the bedding planes. Geological picture is generally very differentiated. Rock mass is heterogeneous, anisotropic, discontinuous and tectonically disturbed. Tunnels are located at the depth up to 55m - therefore are considered as "shallow excavations". In such a case, geological features of the rock mass play an important role during the excavation works. The tunnels were performed using the procedures based on the principles of New Austrian Tunnelling Method. Construction process was divided into two phases. Preliminary supported tunnels were constructed in the first phase and permanent reinforced concrete lining was installed in the second. The support was composed of rockbolts, shotcrete and steel.
were constructed in the first phase and permanent reinforced concrete lining was installed in the second. The support was composed of rockbolts, shotcrete and steel ribs. The first phase was divided into two stages. The heading part of the tunnel (kalota) was performed at first on a whole length, and lower part (bench) was next excavated. The tunnelling procedure was accompanied by rock mass classifying. Two original classifications, KF and KFG were elaborated (Bcstyniski et al., 1990). Both of them are correlated with RMR classification (Bienawski, 1984). Support was generally designed, basing on the rock mass class.

Careful monitoring was carried out. Deformations were measured and analysed simultaneously with the excavation progress. The results allowed for direct estimation of excavation and support appropriateness. The deformation monitoring programme comprised measurements of the tunnel shape changes (convergence, vertical displacements of roof and side-walls), extensometric measurements and measurements of terrain surface settlements.

**Scope and Procedure of the Measurements**

The programme of the measurements was coincident with the excavation procedure. It comprises:

a) measurements of the tunnel shape changes,
   - convergence,
   - vertical displacements of the roof,

b) extensometer measurements of rock mass delaminations,

c) measurements of the settlements of the terrain surface above the tunnels.

The most important part of the measurements was realised in 14 cross-sections, located every 30-50m along the tunnels. Specific locations of these sections were chosen, with taking into account the geologic features of the rock mass. They were arranged in the nearest vicinity of the tunnel face and not later than 24h after new tunnel section was excavated.

The benchmarks for convergence and displacement measurements as well as the extensometers were placed in each of these sections (fig.1). Additional geodetic benchmarks were distributed along the tunnels in their roof.

Convergence measurements allowed to determine horizontal deformations, basing on the changes of L1, L2 and L3 lengths, as well as diagonal deformations, on the base of L2, L3, L5 and L7 changes.

The lengths of individual extensometers were 3m, 4.5m and 6m (see fig.1). Rod extensometers were used and their relative displacements were measured, with reference to the pipe, fixed to the shotcrete shale.

Frequency of the measurements in the initial observation stages was equal to:

- one day during the first week, directly after arrangement of the cross-section,
- one week, until the stabilisation of the deformation process,
- one month (control measurements) in long period after relative stabilisation.

The frequency was corrected, depending on deformation rate and on tunnel advance. Roof settlements were also measured, using levelling techniques. The frequency initially assumed was then corrected and finally 1-2 measurements per week for new installed benchmarks were performed in the first month. One measurement per month
was carried out in the next period, that is during the stages, in which deformation process is not intensive and came to the stabilisation. The settlements of the terrain surface were measured two times in month.

Fig.1. Schematic diagram of measuring cross-section

**Presentation and Analysis of the Results**

*Contour Deformations (Diagonal and Horizontal Convergence)*

Examples of diagonal and horizontal convergences in KG-10 cross-section are shown in the figures 2a & 2b. These results generally indicate the closure (i.e. negative convergence) of the tunnel contour.

Fig.2a. Diagonal convergences in KG-10 cross-section
Fig. 2b. Horizontal convergences in KG-10 cross-section

During the excavation of the upper part of the tunnel horizontal closure L1 in both of the tunnels was similar and equal to about 20 mm. Convergences in diagonal directions (L2 & L3) were smaller and the closure of the right side-wall (L3) was two times higher than L2 (left side-wall). The maximum changes in all the cross-sections were not greater than 10 mm in heading and 20 mm in the bench stage.

The relative stabilisation reached after few months in the heading stage was next disturbed, when the lower part of the tunnel (bench) was excavated. The dependence of the closure magnitudes on the tectonic and lithologic features of the rock mass was clearly visible. The smallest horizontal closures in the rock mass of III-rd RMR class (Bieniawski, 1984) reached few millimetres, whereas they were equal few centimetres in V-th class. The changes of the diagonal measured lengths were generally smaller in comparison to described above. The maximum change was not greater than 10 mm.

It can be generally stated, that the changes of the tunnel contour were not dangerous and the construction was stable.

Vertical Deformations

Example of the settlements in the measuring cross-section and on the terrain surface was presented in figure 3. It could be surprising, that the surface settles more than the tunnel roof (73 mm and 53 mm respectively). Calculations using FLAC (Itasca, 1995) proved, that this phenomenon is possible in the strain-softening model of the rock mass.

Final vertical displacements of the roof during the excavation of the upper part, were equal to 15-60 mm. They were generally very differentiated; the settlements in one of the tunnels are shown in fig. 4.

Vertical displacements of the points located at the side-walls did not show any general tendencies. Left-side wall in one of the tunnels settled more than the right-side, whereas opposite situation was observed in the second one.

During the excavation of the lower part of the tunnels only roof settlements were measured. These changed from few millimetres to maximum 22 mm. The sum of the settlements (i.e. these measured in upper and lower part of the tunnel) was equal to
58 mm. Despite of relatively great displacements the shotcrete shale was continuous and no cracks were visible.

Terrain surface settled relatively significantly; the curve in fig.3 shows the greatest settlement case in the surface above the tunnels.

Fig. 3. Vertical displacements in KG-10 cross-section and on the terrain surface above the tunnel

Fig. 4. Roof settlements of the roof along the tunnel

**Delaminations of the Rock Mass**

Extensometric measurements were only realised during the excavation of the heading stage. Delaminations are generally not great; in most of cases near-zero changes were observed and maximum value was equal to about 20 mm. Example charts are shown in figure 5. Much more distinct delamination was generally visible in the right side-wall, in comparison to those measured in the left side-wall and in the roof. It seems, that the reason of small (or even positive) roof delamination in not accurate shotcreting of the roof zone and presence of gap between shotcrete and rock.
Fig. 5. Delamination of the rock mass around the tunnel.

**Stabilisation of Deformation Processes**

The most important factors influencing stabilisation of the deformation processes are rock mass quality (geotechnical class), kind of the support, rate and regularity of the tunnel excavation.

The greatest part of the deformations, i.e. 70–90% $U_{\text{final}}$, takes place in the first month. Further deformation increments quickly decrease.

If the deformations in the first month were considered, with taking into account above mentioned factors, then it was evident, that the distance between measuring point and tunnel face is the most important quantity. The face has significant "supporting effect" and therefore restrains deformations. If the excavation rate was constant, then the tunnel advance in one-month interval was equal to 15–25m, depending on the rock mass quality. In the next period deformations appeared first of all as an effect of
rheologic phenomena. In time interval between first and third month of measurements, 10+30% of total 3-month’s deformations took place. The increments did not vanish after 3-month’s measurements, but their practical significance could be neglected. Vertical displacements practically stabilised after three months. If 3-month’s displacement is assumed to be 100%, then 65% of this displacement occur in the first month and 86% in the second.

It was generally observed, that during the excavation of the lower part of the tunnel, deformations stabilised in about 20 days after preliminary support installation. Invert closing caused quick stabilisation of the structure.

**Analysis of the Results**

Monitoring of the deformations in the tunnels provided very large number of the results. It was thus possible to build statistic models, which could be validated and verified. Selected results for heading stage are presented below, giving the forms of statistic relations for approximation and prediction of the deformations.

All the curves, describing deformation processes have generally similar shapes, independently on the specific quantity (i.e. convergence, settlement, delamination). Mathematical relations could be therefore considered as valid for all the processes, which were measured.

Three statistic models were tested, describing above mentioned processes. First of them has a form:

$$U = U_c \cdot \left[1 - \exp\left(-\beta \cdot t\right)\right]$$  \hspace{1cm} (1)

where $U$ - displacement (e.g. convergence, roof settlement); $t$ - time elapsed from the reference measurement; $U_c$ - parameter characterising final displacement (when $t$ is very large); $\beta$ - parameter characterising displacement rate.

Next two models take into account both the time and distance between the face and point. As a first, the model described in (Sulem et al., 1987) was analysed:

$$U = A \cdot \left\{1 - \left[\frac{B}{(B + L)}\right]^2\right\} \cdot \left\{1 + C \cdot\left\{1 - \left[\frac{D}{(D + t)}\right]^2\right\}\right\}$$  \hspace{1cm} (2)

where $L$ is a distance between tunnel face and measured point, $t$ - see (1); $A+B$ - model parameters.

This model was originally tested by Sulem in a deep circular tunnels. However, it was proved here, that it also gives satisfactory results in a relatively shallow and non-circular excavations.

Parameter $B$ means the radius of a plastic zone, and the expression $A \ast (1+C)$ gives an evaluation of final displacements. This physical meaning was not confirmed in the presented analysis.

Original model was also elaborated, in a following form:

$$U = A \cdot \left[1 - \exp\left(-\beta \cdot L\right)\right] \cdot t^C$$  \hspace{1cm} (3)

where: $L$, $t$ - see former equations; $A$, $\beta$, $C$ - model parameters.
The equation (3) seems to be the best, as it gives relatively accurate approximations and its "economy" (i.e. the number of parameters) is acceptable. The disadvantage is a lack of physical meaning of the parameters.

In figure 6 roof settlements and in figure 7 convergence in a selected cross-section are approximated.

![Graph](image1)

Fig. 6. Approximation of the tunnel roof settlement in L8 point

![Graph](image2)

Fig. 7. Approximation of the average convergence in KG-1 cross-section

As it is seen in the figures, equation (3) gives the best approximation, with taking into account the distance L. This could be also applied to the prediction of roof settlements. The example of prediction of the future settlement in point L8 is shown in figure 8. Curve A was calculated, predicting settlements after 587 days on the base of
199 days measurements, whereas curve B gives the prediction on the base of the results from 276 days measurements. The later gives much more better approximation than the first. It must be thus pointed out, that the “far” prediction (curve A) with proper accuracy is rather exceptional. It could be generally stated, that in practice prediction is sufficiently accurate for the time range equal to about 110\% to 120\% of time actually elapsed.

![Graph showing predictions A and B](image)

**Fig.8. Prediction of roof settlement in L8 point**

**Summary and Conclusions**

There were few reasons for realisation of deformation monitoring during the excavation of the tunnels, i.e.:

- control of the behaviour of the construction,
- verifying of the design assumptions and solutions,
- signalising of possible instabilities,
- confirmation of correctness of the support modifications.

The results generally confirmed appropriateness of design solutions and in some cases allowed to modify the support and excavation performance. Construction behaviour can be characterised as follows:

- closure (negative convergence) of a whole tunnel contour was observed,
- total (final) horizontal displacements in range 0\%-33\% and vertical displacements 0\%-60\% were measured,
- maximum delaminations were equal to about 20mm; it was evident, that the rock mass zone wider then 6m was also delaminated,
- surface settlements were greater than those in the roof; the rock mass strain-softening is probably responsible for this phenomenon,
- the most intensive deformation increments were measured in a first month, when the distance between tunnel face and measured point was equal to 15\%-25m,
• deformations were practically stabilised after 3 months from the reference measurement, although further deformations were also observed.

Some theoretical considerations shown the dependence between tunnel advance rate and the intensity of the deformation. Limited possibility of the prediction of deformation processes was also demonstrated.

It should be finally stated, that the monitoring should be regarded as necessary approach, when excavations are performed in rock masses, which are very heterogeneous. Cost of the measurements can be considered as very low in comparison to possible profits and savings.

References


