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ASSESSMENT OF THE BEHAVIOUR OF FLYSCH ROCK MASS DURING TUNNEL BORING IN THE PRIMARY LINING USING INDICATORS AND LIMIT VALUES OF DISPLACEMENTS AND DEFORMATIONS

The article describes the behaviour of the flysch rock massif (Carpathian flysch) during the drilling of three tunnels in the preliminary lining. These tunnels were excavated in: "Naprawa", "Laliki", and "Świnna Poręba". The distance between these tunnels in a straight line was 50 km to 90 km. The results of the displacement of the contours of these tunnels and their convergence were analysed in detail. These values were compared with the indices used to assess the behaviour of the rock mass in the tunnel environment (Zasławski index and Hoek index) and the adopted limit values of displacements and deformations. On this basis, a critical analysis of the selection of initial supports in the completed tunnels was made, showing errors at the design stage.

Keywords: Tunnels; preliminary lining of tunnels; assessment of flysch rock massifs; Hoek; Zasławski; Sakurai

1. Introduction

In Poland, tunnel construction has been developing widely in recent years. Many tunnels are excavated in the south of Poland in the Carpathian flysch, which has difficult geotechnical conditions. To properly design and then successfully build a complex building object such as an underground tunnel, correct identification of the rock mass should be made in the initial stage. Initial recognition of the rock massif is required before selecting the route and depth of

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the tunnel. This recognition is paramount for design purposes. After analysing the results, if the designer concludes that an additional diagnosis should be made, then he may order such tests during the design. Later, during the excavation, engineers confirm whether the recognition of the massif was correct. In general, the recognition of a rock massif consists of collecting data related to the geological structure, hydrogeology, geomechanical, geophysical and geochemical properties [1]. The knowledge of this data is necessary for a reliable estimation of the behaviour of the rock mass during tunnel excavation and operation. With these data, based on two indicators, it is possible to initially assess the behaviour of the rock mass during tunnel excavation. Such indicators are Zasławski's index and Hoek's index [2-4], which are based on the comparison of the strength value of the rock mass with the value of the initial stress state. An additional problem appears when the planned tunnel is to run through a highly heterogeneous rock mass such as flysch. Then, according to the research, the geological structure of the rock mass, including the ratio of the content of silt rocks, strongly affects the strength of the entire rock mass [5] and its displacement state [4,6]. One of the basic activities carried out during tunnel excavation, enabling the safety control, is the measurements of convergence, displacements, stresses, and forces in the initial and final support, as well as observations of the behaviour of the tunnel contour and the tunnel face. This monitoring is an important source of knowledge about the behaviour of rocks in the vicinity of tunnels and is predominantly used to check whether the initial and final supports have been designed adequately for mining and geological conditions [7,8]. To assess the behaviour of the initial support [9], the measurements of convergence, tunnel contour displacements, less often forces and stresses are usually used. To correctly assess the behaviour of the primary lining during excavation, limit values of displacements, and strain (stresses, forces) are determined at the design stage, which is compared with the values measured during tunnelling. If the measured values of displacements and strain are greater than the limit values, a stronger initial lining should be installed and adapted to the existing geological and geotechnical conditions. It is also possible to strengthen the tunnel face by reducing the distance between the face and the final lining.

The analysis presented in the article is used to evaluate the behaviour of the rock mass in the vicinity of excavating tunnels, taking into account the complex geotechnical conditions of the Carpathian flysch.

2. Indicators for the assessment of the behaviour of a rock mass in the vicinity of a tunnel

The first index (β_n) which allows assessing the behaviour of the rock mass was proposed by Zasławski [10,11]:

$$\beta_n = \frac{p_z}{R_c} = \frac{\gamma \cdot H}{R_c} \tag{1}$$

where:

R_c — uniaxial compression strength of rock samples,
p_z — initial vertical stress,
γ — volumetric weight,
H — tunnel foundation depth.

Based on the index value (β_n) Zasławski divided the behaviour of the rock mass in the vicinity of the underground excavation into three classes (Table 1).

TABLE 1

Classes describing the behaviour of the rock mass in the tunnel environment*

Class	Description	β_n	Tunnel contour strain, ε [%]
I	stable rock mass	≤ 0.25	$\leq 1.5\%$
II	rock mass with medium stability	$0.25 < \beta_n \leq 0.4$	$1.5\% < \varepsilon \leq 4.2\%$
III	unstable rock mass	>0.4	$>4.2\%$

* When determining the strain value, it was assumed that the excavation is without support or that it behaves passively.

On the basis of measurements made for many underground workings, Zasławski showed that after exceeding the limit value of $\beta_n = 0.3$, a significant increase in the squeezing of the underground excavation begins. The rock mass is deformed in an elastic-visco manner. The stability of the rock mass is still maintained, and only after exceeding the value of $\beta_n = 0.4$ does the rock mass become unstable, and the rock mass deforms in an elastic-visco-brittle manner. The value $\beta_n = 0.3$ corresponds to the strain $\varepsilon = 2\%$, which Hoek [4,9,12] defined as the ratio of the tunnel convergence ΔK to its diameter D:

$$\varepsilon = \frac{\Delta K}{D} \cdot 100\%$$

Hoek [9] proposed the α_n, index, which is generally the reciprocal of the Zasławski index:

$$\alpha_n = \frac{R_{cm}}{p} = \frac{1}{\beta_n}\frac{R_{cm}}{R_c} \tag{2}$$

where:

R_{cm} — uniaxial compression strength of rock mass,
$p = p_x = p_y = p_z$ — hydrostatic primary stress.

Carranza-Torres and Fairhurst [2] carried out an analysis determining the relationship between the α_n index and the value of strain of the tunnel contour:

$$\varepsilon = 0.2\left(\frac{R_{cm}}{p}\right)^{-2} = 0.2\left(\alpha_n\right)^{-2} \tag{3}$$

This equation was made assuming that the pressure from the primary lining is $p_0 = 0$ (i.e. there is no support or the support behaves passively). In the Table 2 authors present estimated values of deformation ε and the values of the coefficient α_n for rocks prone to squeezing [4].

Table 2 shows that in order to avoid problems related to tunnel squeezing and its potential failure, the tunnel contour strain should not exceed 1%, which corresponds to the value of the index $\alpha_n \geq 0.45$ or $\beta_n \leq 0.18$. This value is confirmed by the observations made by Sakurai [13]. Weak squeezing (tightening) of the tunnel contour to the deformation $\varepsilon = 2.5\%$ does not threaten its failure, while above this strain value, strong squeezing of the tunnel contour begins, and

TABLE 2

Estimated values of deformation ε and the values of the coefficient α_n and β_n for rocks prone to squeezing [4]

Type of crimping the tunnel contour	ε [%]	α_n	β_n
Extremely strong squeezing	≥10	≤0.14	≥0.62
Very strong squeezing	10 ÷ 5.0	0.14 ÷ 0.2	0.42 ÷ 0.62
Strong squeezing	2.5 ÷ 5.0	0.28 ÷ 0.2	0.32 ÷ 0.42
Poor squeezing	1.0 ÷ 2.5.	0.45 ÷ 0.28	0.18 ÷ 0.32
No squeezing, the tunnel is stable	≤1.0	≥0.45	≤0.18

stability problems begin. According to Hoek [4] the rock failure in the tunnel occurs when the indicator $\alpha_n \leq 0.3$, i.e. when the strain exceeds $\varepsilon = 2.2\%$. The decrease of α_n index corresponds to the increase of the strain.

Sakurai [13] gave 3 limit values of the tunnel risk failure depending on the value of strain of the tunnel contour:

- **acceptable value** below which no failure appears (ε_{ac}),
- **warning value**, suggesting tunnel stability problems (ε_w),
- **critical (alarm) value**, when exceeding the tunnel instability may appear (ε_{cr}).

Table 3 presents the authors' proposed limit values for strain and indices for an excavated tunnel in a flysch rock mass without support or with passive support. These values were adopted with caution due to the complicated structure of the flysch rock massif, among others: the lack of consideration of lamination, cracks, and inclination of the layers, which causes asymmetrical loading of the primary lining.

TABLE 3

Proposed limit values of ε, α_n, β_n for an excavated tunnel in flysch rock mass without support or with a passive support

Parameters	Acceptable value ε_{ac} [%]	Warning value ε_w [%]	Critical value ε_{cr} [%]
ε	1.00	1.50	2.00
α_n	0.45	0.37	0.32
β_n	0.18	0.25	0.32

In the case of using an active primary lining, the value of strain and displacement of the tunnel contour is smaller and significantly depends on the acting pressure p_0.

For example, the strain of the contour of an excavated tunnel at a depth of $H = 35$ m in a heavily weathered flysch rock mass (cohesion $c = 0.2$ MPa and friction angle $\phi = 17°$), with no support (or with passive support), was $\varepsilon = 1.3\%$, while after installing the lining with the pressure $p_0 = 0.1p$, the contour strain decreased to $\varepsilon = 0.9\%$. The same situation appears when the pressure of support installation is $p_0 = 0.25p$. It creates a drop in contour strain up to $\varepsilon = 0.7\%$.

Adopting the limit values (acceptable, warning, critical) for excavated tunnels in the Carpathian flysch, one should also take into account:

- Strength of the rock mass. The analysis conducted during tunnelling [13,14] shows that the value of the critical strain ε_{cr} is not a constant, but it decreases as the compressive strength R_c increases and increases with the decrease of this strength.

- The value of the pressure p_0 with which the support influences the rock mass. The value of pressure p_0 changes with time and is related to the excavation stages. Shotcrete lining achieves the required strength after some time. The shotcrete or steel support has the highest strength and rigidity after it is built over the entire contour of the excavated tunnel (i.e. when the support ring is closed). The lining installed only in the top heading (crown) has much lower support and stiffness, and at the distance from a few to several metres from the tunnel face behaves like a passive lining, i.e. its impact on the tunnel contour is small. For example, it was observed that the steel arch support in "Naprawa", had contact with the contour of the tunnel only at a few points at a distance of 15 m (from the tunnel face). It was clearly visible that it was "adjusting" to the irregular contour of the tunnel.

The primary lining must be designed to keep tunnel safety until the secondary lining is set up. For organisational and financial reasons, tunnel contractors aim to excavate the entire tunnel length in the primary lining and then install the secondary lining. This is a high-risk solution. For safety reasons, it is better if the secondary lining is made at a properly selected distance from the tunnel face. Tunnel excavations should be carried out so that the acceptable value for the tunnel contour strain and stresses in the primary lining is not exceeded. Sometimes only two limit values (warning, critical) are used to assess the behaviour of an excavated tunnel lining. If in a section of the excavated tunnel, deformations exceed acceptable values, the frequency of measurements should be increased, and the results should be analysed in detail. If necessary – reacted. When they reach the warning values, the lining should be adequately strengthened, e.g. by using additional anchors or increasing the thickness of shotcrete. In the event that they reach values close to the critical ones, the excavation should be stopped, and further calculations need to be done, taking into account the geotechnical conditions in this area and measurement results. The appropriately stronger support should be selected afterwards in agreement with the designer. The limit values should be determined at the tunnel design stage, accounting for the experience of other excavated tunnels under similar geotechnical conditions. It is unacceptable to change the limit values (especially the critical value) during excavation without documenting (performing appropriate calculations) and agreeing with the designer, as this may lead to tunnel failure.

3. Experiences from tunnels excavated in the polish Carpathian flysch

In Poland, tunnels were excavated in the Carpathian flysch in three different locations: two parallel tunnels (tunnels) in "Świnna Poręba" [15], a tunnel in "Laliki" [16], two parallel tunnels in "Naprawa" [15]. The experience gained from these tunnels may be valuable for determining the limit values of strain and displacements of other tunnels that will be excavated in the Carpathian flysch in the following years. In this article, the authors focused mainly on the limit values adopted for the excavation in the primary lining stage. Table 4 shows the limit values of deformations (calculated from the limit strain) for excavated tunnels in the three analys ed locations. In the case of the tunnel shape different from the circular shape, the calculations were made for the equivalent tunnel radius r_z determined from the formula:

$$r_z = \sqrt{\frac{P_t}{\pi}} \tag{4}$$

where: P_t — tunnel cross-sectional area.

TABLE 4

Limit values of deformations (calculated from the limit strain) for excavated tunnels in the three locations

Limit values		Acceptable value	Warning value	Critical value
		Strain ε [%]		
		1.0	1.5	2.0
		Calculated primary lining displacements [mm]		
"Świnna Poręba"	Top heading (r_z = 3.0 m)	30	45	60
	Tunnel (r = 4.25 m)	43	64	86
"Laliki" (r_z = 5.77 m)		58	87	116
"Naprawa" (r_z = 7.78 m)		78	117	156

The values of the limit displacements of the contour of the tunnel presented in Table 4 differ from designing for the tunnels in "Świnna Poręba", "Laliki" and "Naprawa". It is because the determination of these values did not take into account the impact of the active primary lining and previous experience with other tunnels.

The problem of selecting the limit values of strain and displacements is extremely important, especially during excavation tunnels in the flysch rock mass (which has an extremely complex structure). The flysch rock massif consists of sandstone layers interlaced with shale layers and presents strong anisotropic properties. Moreover, the Carpathian flysch is strongly fractured. There are five basic types of fractures: stratification, side cracks, schistosity, near-surface zone cracks and faults, which make the rock mass discontinuous [17]. Fractures in the stratification separate the individual layers of fissures. The side joints occur in sandstones and shales, transverse to the surface of the stratification and constitute a fairly regular network of joints, usually limited to individual layers and fissures. There are two systems of tunnel side discontinuity sets: running perpendicular to the layers and towards each other. The joint spacing in both systems is similar and with average ranges from 0.1 to 0.5 m. The thicker the sandstone layer, the greater the joint spacing. The side joints are the result of tension and have a small aperture with filling, and rough walls. In tectonically disturbed zones, the side joints are denser and less regular, and their aperture does not exceed 0.2 mm. The schistosity fracture creates a dense joint set network with a small aperture (<0.1 mm). These joints are practically parallel to the lamination. They occur at distances from a few to several millimetres , dividing the shales into plates.

The strain and strength properties of the flysch rock mass along the discontinuity planes differ significantly from the properties of the rocks in the direction perpendicular to the discontinuity. In the direction perpendicular to the stratification, the rock mass has greater compressive strength than in the direction parallel to the stratification. A rock mass has the lowest strength at a certain discontinuity dip angle to the direction of the greatest main compressive stress. In order to explain the possible mechanism of destruction and displacement in the tunnel area, it is necessary to pay attention to the values of the strength parameters of the rocks forming the flysch rock massif. The sandstone strength values determined in laboratory conditions are high. Laboratory tests carried out on sandstone samples have shown that R_c of sandstone is in a wide range of 58 ÷ 224 MPa, the mean value of cohesion c is approximately c = 5.7 [MPa], while the angle of internal friction (ϕ) is close to the value of $\phi = 60°$. Whereas shales have very low strength parameters: $c = 0.06$ MPa, $\phi = 11.50° \div 19.00°$ (in "Świnna Poręba" and "Laliki" this value was closer to 13°). Low strength parameters of shales significantly reduce the shear strength of the flysch rock massif, as shown by in-situ studies on 1m × 1m rock blocks [17,18]. These studies indicate that the estimated values of the strength parameters of the rock massif are:

- sandstone-shale structure (with a predominance of sandstone): $c = 0.55$ MPa and $\phi = 47°$,
- shale-sandstone structure (with a predominance of slate): $c = 0.33$ MPa and and $\phi = 21°$.

In the Carpathian flysch, the Quaternary overburden is usually small, on the order of a few meters. The problem is caused by the weathering of the flysch massif reaching deep from the surface, which significantly reduces its deformation and strength parameters. Below the Quaternary overburden, there is a completely weathered rock mass (W5) or heavily weathered massif (W4) reaching a depth of 15 m, and below, up to a depth of about 35 m, there is a moderately weathered massif (W3). If you compare the excavated depth of some shallow tunnels with the depth of weathered rock massif (from W5 to W3), it turns out that the tunnels for a considerable length were excavated in the weathered rock mass with low or very low strength and elastic parameters. The situation was worsened by the fact that there were also fault zones along the route of the excavated tunnels.

In a layered rock mass, with a certain inclination of the layers, it is possible to slip (layers slide). This situation may occur when the frictional resistance along the layers is lower than the sliding forces related to the inclination of the layers, a.o.

$$\tan\varphi > \tan\phi \quad \text{or} \quad \frac{\tan\varphi}{\tan\phi} > 1 \tag{5}$$

where:

ϕ — angle of internal friction along the layers,
φ — the angle of inclination of the layers (Fig. 1).

Along the joint in the lamination, the cohesion is between 0.01 ÷ 0.40 MPa, and the angle of internal friction is from 10° to 15°, on average value $\phi = 13°$. The values of cohesion and the angle of internal friction along with the fractures of the lamination increase with depth. If the rock layers' inclination φ is greater than the angle of internal friction ϕ, the layers may slip, of course, after overcoming the small cohesive forces between sandstone and shale. In formula (5), the estimated value of the horizontal stress to be applied to the tunnel side in order to prevent sliding out of the layers [19]:

$$\sigma_x = p_z \frac{\tan(\varphi - \phi)}{\tan\varphi} \tag{6}$$

In this formula, the effect of cohesion is neglected, since this value in lamination joints is insignificant. The value of the calculated horizontal stress using the formula (5) can be the basis for assessing the possibility of layer slip on the tunnel side.

During tunnel excavation, measurements are usually made of the convergence between the selected points of the tunnel contour, the displacements of the roof and sidewall, less often the invert, and the measurement of cracks. In the laminated rock mass like flysch massif, the measurements should be made not only on the roof and sidewall but also perpendicular and parallel to the lamination. Such pieces of information are crucial during the estimation of the strain and critical displacements of the preliminary lining of the tunnel. Displacements perpendicular to the lamination may indicate the possibility of buckling of rock layers (with thin sandstone layers), whilst if condition (4) is fulfilled, the displacements parallel to the lamination may indicate the possibility of layers sliding (layer by layer). Fig. 1 shows the displacements, perpendicular u_{wp} and parallel to the u_{wr} lamination, as well as the horizontal displacements of the sidewall u_{oc} and the vertical roof u_{st}, respectively.

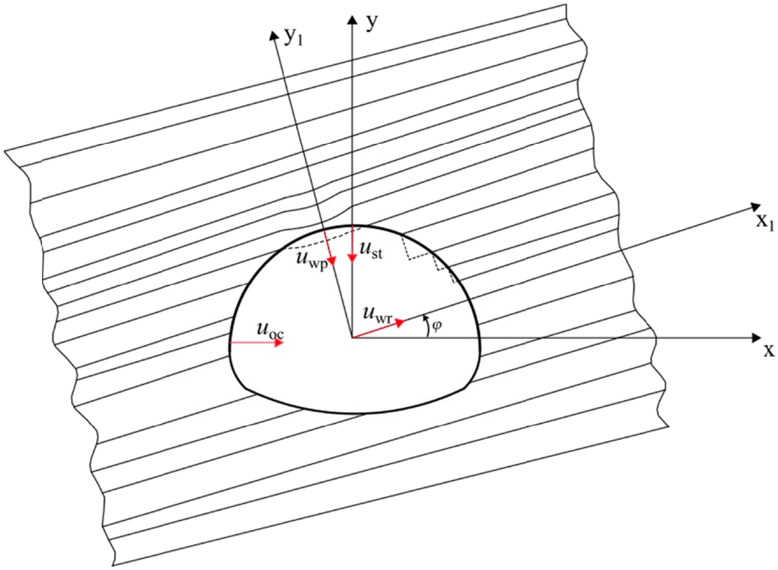

Fig. 1. Displacements of the stratified rock massif

3.1. Tunnelling experiences in Świnna Poręba

During the dam's construction in Świnna Poręba on the Skawa River, it was necessary to build two tunnels on its right abutment. Tunnels with a circular cross-section, 8.5 m in diameter in breadth and length: the discharge tunnel – 294.1 m, and the intake tunnel – 331.1 m. The tunnels were excavated from the exit of the tunnels (located below the designed water dam) towards the entrance to the tunnels (located at the level of the water reservoir). In both tunnels, along with the drilling progress, every 30 ÷ 50 m, sections called KG were built, in which measurements were made. In total, there were 13 measurements of cross-sections. In the discharge tunnel, the measurement cross-sections were as follows: KG1, KG3, KG5, KG7, KG9, and KG11, while in the intake tunnel KG2, KG4, KG6, KG8, KG10, KG12. The tunnel route consisted mainly of mixed sandstone-shale or shale-sandstone complexes. It was noticed that during the top heading excavation, the rate of sandstone in the face profile changed from 40 to 80%, and the rest was filled with slate (Table 5).

TABLE 5

Geological structure in the tunnel profile

Tunnel intake						
Measurement cross-section	KG1	KG3	KG5	KG7	KG9	KG11
Sandstone rate [%]	58.0	40.0	55.0	78.0	60.0	65.0
Slate (shale) rate [%]	42.0	60.0	45.0	22.0	40.0	35.0
Depth H [m]	27.0	34.5	52.5	53.0	40.0	35.0
Tunnel discharge						
Measurement cross-section	KG2	KG4	KG6	KG8	KG10	KG12
Sandstone rate [%]	55.0	50.0	57.0	65.0	70.0	65.0
Slate (shale) rate [%]	45.0	50.0	43.0	35.0	30.0	35.0
Depth H [m]	28.5	33.5	42.5	36.5	31.0	20.0

Very complicated tectonics in the form of longitudinal (a2, a6) and transverse (b3, b4, b5) faults and fault zones had considerable influence on the behaviour of the rock massif in the vicinity of tunnels. The dislocation a2 runs below the tunnel route (Fig. 2, according to [17]). The a6 dislocation runs in the N-S direction and crosses the routes of both tunnels running along the

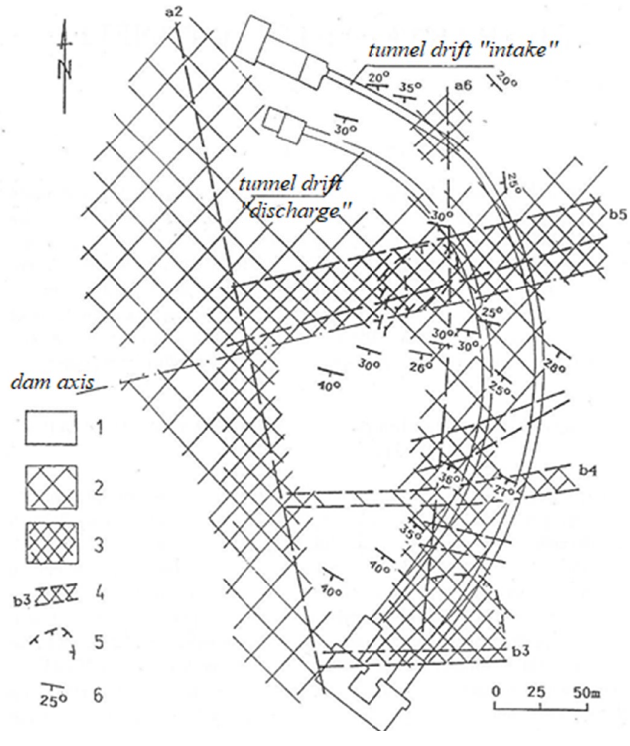

Fig. 2. Schematic tectonic map in the area of tunnels [17]. (1 – an area with weak and moderate tectonically disturbed, 2 – an area with strong tectonically disturbed, 3 – an area with very strong tectonically disturbed, 4 – fault zone, 5 – landslide area, 6 – orientation of rock layers where the direction of the graphic sign determines the course of the layer, and droop angle number)

line from the entrance to the intake tunnel to KG3. The dislocation b3 crosses the tunnel route near the tunnel entrances, and dislocation b4 is away by 100 m on the north. The entire area between dislocations b3 and b4 is strongly disturbed tectonically, and therefore the excavation of the last 100 m of tunnels was carried out in unfavourable conditions. The dislocation b5 crosses the discharge tunnel route at a distance of about 190 m ÷ 220 m from its inlet (entrance), and the intake tunnel route at a distance of 200 m ÷ 225 m creates a disturbed zone 30 m wide. Very unfavourable and complicated geotechnical conditions occur at the intersection of two dislocations.

The dislocation a6 intersects with the dislocation b5 in the middle, between the measurement sections KG4 and KG6, at a distance of approximately 200 m from the inlet to the discharge tunnel (near this point, a collapse of roof rocks occurred during drilling, described below). In the second place dislocation, a6 intersects with the highly disturbed area between dislocations b3 and b4 at a 55m distance from the inlet to the discharge tunnel. Significant problems related to excavation occurred in places of tectonic disturbances causing layers damage (e.g. KG10) and in sections where destroyed shale layers were in the roof [17]. In the area of the tunnel route, the Quaternary overburden is 2 m to 3 m thick. In Świnna Poręba, the rock mass, which was completely and heavily weathered (W5, W4), reached a depth of 9 m ÷ 13 m from the surface, and below the rock mass was moderately weathered (W3) to a depth of 24 m ÷ 33 m. If we compare the depth of excavated tunnels with the depth of weathering, we can see that both tunnels on the initial sections, up to about 80 m from the inlets, and the final sections close to 60 m from the outlet (exit) were excavated in the weathered mass from W5 to W3. Due to this fact, the rock massif in these sections had low strength parameters. The remaining length of the tunnels was cut in an unweathered or weakly weathered massif.

In the flysch massif in the area of Świnna Poręba, there are complex water conditions, which did not have a significant impact on the tunnel's excavations. Due to differences in permeability, the water flowed into the excavation through a dense set of joints rather than through solid rock. During the boring, the water inflow was minimal at about 10 l / min level. Only during the period of intense rainfall, there was an increase in water inflow as a result of rainwater infiltration through the joints.

The discharge and intake tunnels were bored with the New Austrian Tunneling Method (NATM). First, the top heading was made in the preliminary lining along their entire length, and then bench and invert. For the assessment of the flysch rock massif, a special classification of KF [20], similar to the classification of RMR Bieniawski [21] was developed. The KF classification gives slightly higher than the RMR classification point values for the medium massif, practically identical for the weak massif, and slightly lower for the very weak massif. The differences do not exceed 3 points. To switch from the KF classification to the RMR classification, the $RMR = 1.134 \cdot KF - 5.1$, conversion factor can be used [20]. The significant difference between the KF and RMR classifications is that in the KF classification, the V class of the rock mass is divided into two subclasses Va (with a point value of 20 ÷ 15) and Vb (with a point value <15). The division of class V into two subclasses is significant. Experience shows that in the case of a very weak flysch massif Vb the casing must be much stronger than in the case of a very weak flysch massif of class Va. Based on the KF value, the rock mass of the Świnna Poręba region was classified into four classes: III, IV, Va, Vb and four types of initial support were selected for these classes (Table 6).

During the tunnel top heading drilling, the measurement points were set up in sections – KG (to measure convergence, vertical displacements of the roof, and stratification of the massif).

TABLE 6

Determination of the KF classification value for the rock mass in the Świnna Poręba region

Rock mass class	Range	Support type	Rock bolt	Shotcrete	Steel arch
III	60 ÷ 41	1	Grouting steel ribbed anchors. Diam.: ϕ 32 mm, Leng.: 3 m, Spacing: 1×1 m	0.20 m	no
IV	40 ÷ 21	2		0.25 m	every 1.00 m
Va	20 ÷ 15	3		0.30 m	every 0.75 m
Vb	<15	4		0.30 m	every 0.50 m

Between the measurement cross-sections, additional points were assumed for the control of vertical displacements. In the intake tunnel, 30 such points were established, and in the discharge tunnel – 27. The arrangement of points and stations in the measuring cross-section of the KG top heading is shown in Fig. 3. It includes 3 points for measuring convergence on the sections L_1, L_2, L_3 and vertical displacements; 3 stands for measuring the stratification of the massif, each of which consisted of 3 single-point extensometers with the lengths of 3.0 m, 4.5 m and 6.0 m [22].

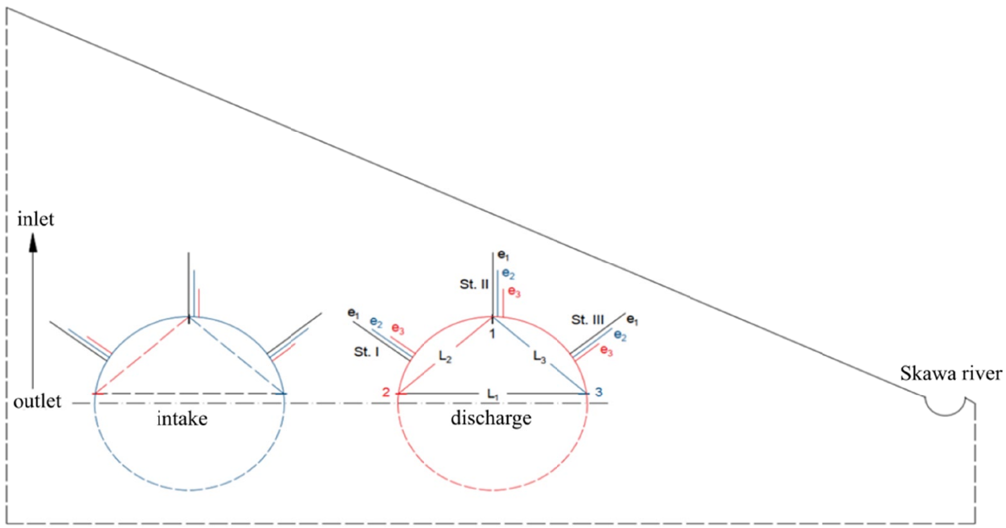

Fig. 3. Scheme of the distribution of points and stations in the measurement section [22], L_1, L_2, L_3 – convergence measurement sections; 1, 2, 3 – vertical displacement measurement points; e_1, e_2, e_3 – delamination measurement station

Assuming the acceptable value of strain of 1% for the top heading of the tunnel in Świnna Poręba, the acceptable value of displacement is u_{ac} = 30 mm (Table 4). Table 7 shows the average displacements of the tunnel lining: the sidewall, and the roof, measured in the individual measurement sections of KG. In all cross-sections, except for KG1, the values of side and roof displacements were lower than the acceptable value. In the KG1 section, the deformation value was 1.27%, and the floor displacement value was 38.2 mm. These values were greater than the acceptable values but lower than the warning values, respectively: deformation ε_w = 15% and displacement u_w = 45 mm. Table 7 also shows the perpendicular u_{wp} and parallel u_{wr} displacements and the values of maximum strains. The measured values of displacements and the calculated values of strains in individual measurement sections are much lower than the critical values and indicate that the preliminary lining of the tunnel was stable along its entire length.

TABLE 7

Displacements of the lining of the tunnel top heading measured in the individual KG sections

Tunnel	Section	RMR / KF Class	Distance from the entrance to the tunnel [m]	Av. lining displacements		Av. angle layers dip [°]	u_{wp} [mm]	u_{wr} [mm]	ε_r^{max} [%]
				Wall u_{oc} [mm]	Roof u_{st} [mm]				
Discharge	KG2	42/III	249.5	11	15.9	156	15.1	11.8	0.53
	KG4	13/V_b	230.3	3.2	13.4	162	12.4	4.2	0.47
	KG6	15/V_b	191.6	6.6	9.7	170	9.6	6.7	0.32
	KG8	38/IV	136.7	0.8	12.0	169	11.6	1.2	0.40
	KG10	22/IV	91.2	8.8	22.5	165	21.6	9.7	0.75
	KG12	21/IV	46.5	0.1	20.0	168	19.1	1.0	0.67
Intake	KG1	25/IV	291.2	7.3	38.2	155	32.7	12.8	1.27
	KG3	15/V_b	271.5	8.1	12.6	161	12.1	8.6	0.40
	KG5	16/V_a	221.1	9.9	10.7	163	10.6	10.0	0.37
	KG7	43/III	170.3	2.3	8.8	169	8.6	2.5	0.29
	KG9	15/V_b	119.6	9.6	13.1	180	13.1	9.6	0.44
	KG11	15/V_b	74.9	3.1	12.5	171	12.3	3.3	0.42

The average displacements of the roof u_{st} in the individual classes of the rock massif were as follows: class III: 30.1 mm, class IV: 12.0 mm, class Vb: 10.7 mm, class Vb – 12.2 mm, i.e. displacements in class III were nearly 2.5 times higher than in classes IV, Va, Vb. These differences in roof displacements are the result of using different types of lining for these classes (Table 6). In class III, a shotcrete lining with a thickness of 0.2 m and anchors was used, while in classes IV, Va, and Vb, the thickness of shotcrete was increased up to 0.3 m, and additional support made of steel arches (with different spacing) was used. Hoek [9,12] reported that the maximum strain of the shotcrete casing is approx. $\varepsilon_{\max} = 1.0\%$, while the maximum strain of the support was made of steel arch $\varepsilon_{\max} = 0.55\%$. The support used in classes IV, Va, and Vb is much stiffer and stronger than in class III, which translates into reduced displacements of the tunnel contour. For individual measurement cross-sections, Table 8 shows the tunnel depth H, the uniaxial rock strength R_c, the rock massif strength R_{cm}, and the following indexes calculated from these values: Zasławski $\beta_n = \frac{p_z}{R_c}$ and Hoek $\alpha_n = \frac{R_{cm}}{p_z}$.

By analysing the obtained values of the β_n index in individual measurement sections, it can be seen that these values are many times lower than the acceptable value of 0.18. This means that the index did not indicate any risk of failure along the entire length of the excavated tunnel. This indicator should not be used for tunnels at shallow depths. The analyses not included in this paper show that the β_n index can be used with the tunnel depth $H \geq 400$ m. The index α_n gives more reliable results. In two sections presented in Table 8: KG-3 and KG-5, the value of α_n exceeded the acceptable value of 0.36 and approached the warning value of 0.30. On the other hand, in

TABLE 8

Parameters for two tunnels Discharge and Intake

Tunnel	Section	H [m]	R_c [MPa]	β_n	R_{cm} [MPa]	α_n	σ_x (eq. 5) [MPa]
Discharge	KG2	28.5	44	0.014	1.75	2.46	0.31
	KG4	33.5	40	0.018	0.32	0.38	0.23
	KG6	42.5	46	0.020	0.41	0.39	—
	KG8	36.5	52	0.015	1.65	1.81	—
	KG10	31	56	0.012	0.73	0.94	0.10
	KG12	20	52	0.009	0.65	1.30	—
Intake	KG1	27	47	0.013	0.73	1.08	0.31
	KG3	34.5	32	0.024	0.28	**0.32**	0.26
	KG5	52.5	44	0.026	0.41	**0.31**	0.30
	KG7	53	62	0.019	2.61	1.97	—
	KG9	40	48	0.018	0.43	0.43	—
	KG11	35	51	0.015	0.45	0.51	—

sections KG4 and KG6, the value of α_n reaches the acceptable value. In those cross-sections, the rock mass is of classes Va and Vb, whereas, in the remaining cross-sections, the value of α_n did not show any risk. The presented analysis reveals that the risk of rock failure is well described by the indicator ε_r^{max} or u_r^{max}. The maximum displacements and strain of the tunnel contour compared to the limit values are the basis for making decisions on the choice of support during tunnelling.

The rock layers sliding is possible in the stratified rock mass when the inclination of the layers (φ) is greater than the value of the internal friction angle (ϕ) at the contact between two layers: sandstone and shale. To check whether such a phenomenon can occur, values of those two angles were compared (assuming $\phi = 13°$ and $c = 0$).

To prevent rock layers from sliding, the potential horizontal stress applied to the tunnel side has been calculated for every cross-section where $\varphi > \phi$ according to equation 5 (Table 8). These values are not too high, and if the rock cohesive between the sandstone and shale layers is taken into account, they will be even lower.

Despite the fact that none of the considered indicators showed a possible instability of the primary support, there was one case in which rocks in the tunnel face and the roof collapsed. The collapse was approximately 197 m from the tunnel entrance to the discharge tunnel. 97m from the exit to this tunnel in a highly disturbed structure, in the area of the intersection of dislocation „a6“ with dislocation “b5” (Fig. 2), there was a large sand, slate and shale mass content compared to sandstone [17]. The collapse occurred in the tunnel face after 0.5 m of the cutting distance (Fig. 4) before the installation of the initial lining. As a result of failure, the collapsed hole appears in the roof with an asymmetrical shape concerning the vertical main axis of the tunnel. The axis of this hole was practically perpendicular to the layer’s inclination, and its length was about 6.5 m. The formation of the caving was as follows: first, the layers of sandstone and shale were delaminated, and then, successively, starting from the immediate roof, the rock layers were cracked and divided into blocks near sidewall fractures (small cohesive forces occur along the sidewall joints). Those blocks collapse under their own weight till the caving zone where the layers were cut. A few metres before the place where the roof collapsed, a small value of the roof displacement (8.75 mm) was measured, even though the rock mass was classified as class Vb (RMR = 12). On the other hand, in the next point, a few metres behind the collapsed zone, the measured roof displacements were 12.78 mm with an RMR value of 14. The values of displacements and strain in this region were significantly lower than the limit of acceptable values, and the coefficients α_n and β_n did not signal potential instability. This can be explained by the fact that the roof collapse occurred before the installation of the preliminary lining. The displacement measurements at this point have not yet been made, and the coefficients α_n and β_n were determined based on the averaged parameters: R_c, R_{cm}, which didn't include local strong rock fractures.

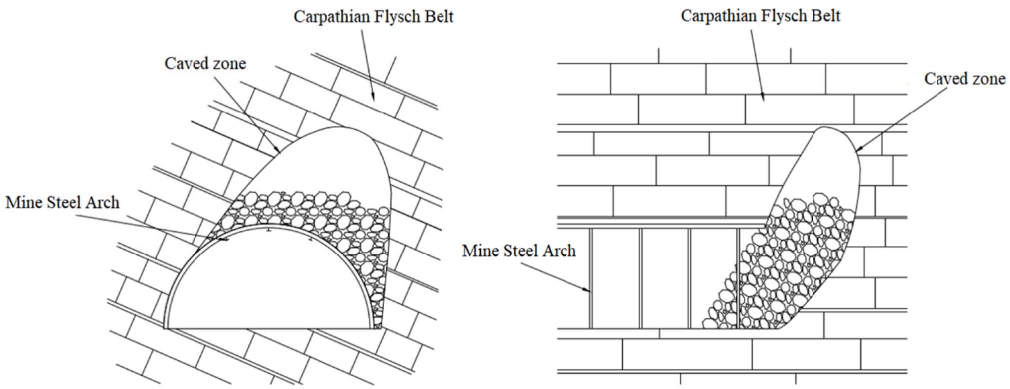

Fig. 4. Caved zone created during Świnna Poręba tunnel excavation

The discharge and intake tunnels were ultimately circular in shape, therefore, after the excavation of the top heading in the preliminary lining along their entire length, the tunnels were excavated in bench and invert. After the top heading was drilled, it turned out that the greatest squeezing of the contour occurred in the horizontal direction (convergence in the L_1 direction), i.e. there was a significant squeezing of the tunnel sidewalls (Table 9). The average strain in this direction was $\varepsilon = 0.25\%$, while the maximum (KG10 section) was $\varepsilon = 0.47\%$. Tunnel excavation proceeds in bench and invert to achieve a fully circular shape creating an increase in displacements on the measuring sections: on the horizontal section L_1 by 80% (L_1^c), and on sections L_2 (L_2^c) and L_3 (L_3^c) by nearly 50% and 40%, respectively.

By analysing the values of displacement and convergence measured in the tunnel top heading, the following conclusions are reached:

- The preliminary lining in the top heading is subjected to high vertical loads, as evidenced by much greater vertical displacements compared to the horizontal displacements. The fracture zone caused by the tunnelling almost extends to the surface. In the tunnel sidewalls, a fracture zone is also created over a distance of several metres. This pre-support load condition is well described by Therzaghi's theory. Large vertical displacements indicate that the steel support constituting a significant part of the preliminary lining in the top heading was insufficiently put on its floor (invert), and the entire support was displaced vertically.

TABLE 9

Values of measured convergence for individual sections of the tunnel in Świnna Poręba [22]

Tunnel	Section	Convergence of measuring sections ΔL_1, ΔL_2, ΔL_3 [mm] after excavation						$\frac{\Delta L_1^c}{\Delta L_1}$	$\frac{\Delta L_2^c}{\Delta L_2}$	$\frac{\Delta L_3^c}{\Delta L_3}$	Max. ε_r^c
		In top heading			For whole tunnel						
		ΔL_1	ΔL_2	ΔL_3	ΔL_1^c	ΔL_2^c	ΔL_3^c				
Discharge	KG2	–21.9	–3.1	–5.4	–33.9	–2.2	–9.8	1.55	0.71	1.81	0.40
	KG4	–6.4	+0.5	–3.9	—	—	—	—	—	—	—
	KG6	–13.2	–2.1	–6.1	–21.3	–0.9	–9.5	1.61	0.15	1.56	0.25
	KG8	–1.6	–2.9	–4.4	–4.9	5.0	–4.3	3.06	1.72	0.98	0.06
	KG10	–17.6	–8.0	–8.3	–40.0	–15.5	–10.2	2.27	1.94	1.23	0.47
	KG12	+0.1	–2.6	–2.4	–6.2	–5.8	–4.8	–	2.23	2.0	0.08
Intake	KG1	–14.6	–20.1	– 9.5	–24.7	27.2	–18.6	1.69	1.35	1.96	0.32
	KG3	–16.1	+0.8	–8.7	–25.1	–1.1	–9.8	1.56	–	1.13	0.30
	KG5	–19.7	–4.1	–9.2	–27.0	–4.8	–10.9	1.37	1.17	1.18	0.32
	KG7	–4.5	–5.0	–7.5	–8.2	–4.0	–8.7	1.82	0.8	1.16	0.10
	KG9	–19.2	–1.8	–12.6	–27.4	–1.5	–15.7	1.43	0.83	1.25	0.32
	KG11	–6.1	–5.8	–5.5	–12.4	–6.9	–7.4	2.03	1.19	1.35	0.15
Average for discharge tunnel		–10.1	–3.0	–5.1	–21.3	–5.9	–7.7	2.11	1.97	1.51	0.25
Average for intake tunnel		–13.4	–6.0	–8.8	–20.8	–7.6	–11.9	1.55	1.27	1.35	0.24
Average for whole tunnel		– 11.7	–4.5	–7.0	–21.1	–6.7	–9.8	1.8	1.49	1.4	0.25

- Significant convergence of the sides of the top heading made in the preliminary lining compared to the convergence in the direction close to the roof – the floor was due to the fact that the steel support was gradually moving towards the excavated space during its vertical movement. This displacement mechanism of the preliminary support caused several shotcrete cracks in the roof along the main axis of the tunnel. During drilling, the foundation of the steel support in the top heading floor was strengthened.

During top heading excavation in the discharge tunnel, the absolute displacements of the roof were measured (Table 7), while they were not measured during the excavation of the bench and invert. Taking into account the increase in convergence on the measuring sections L_2 and L_3, it can be estimated that the roof displacement after the excavation of the entire tunnel increased by about 30%. This increase did not cause (except for KG1) the exceeding of acceptable displacement values.

3.2. Tunnelling Experience in Laliki

In Laliki, located in the western part of the Flysch Carpathians, a tunnel was excavated with the dimensions: height w = 9.5 m, width l = 13.48 m and the total cross-section F = 104.6 m^2 [23]. The flysch massif in the vicinity of the tunnel consists of: almost 70% clay shale occurring in conglomerates and grey calcareous shales occurring between thin-bedded sandstones, 20% medium and thin-bedded sandstones, and 10% marls. Generally, the layers dip direction in the south-east direction with the angles between 38° and 86°. The strike angle varies between 50° and 90° with respect to the main axis of the tunnel. At the design stage of the tunnel, five types of preliminary support were proposed, from 1, 2, 3, 4, and 4a [24]. Meanwhile, during the drilling, it turned out that the rock mass is of lower quality than assumed at the design stage, therefore they resigned from type 1 (intended for a strong massif with RMR above 60), and changes were made to the designed types of casing 2, 3, 4, 4a. These supports were marked with the letter M (the changes concerned the number of bolts and the type of meshes), and the strongest type 5 casing was designed. Overall the preliminary tunnel support during the drilling of the top heading consists of [23]:

- shotcrete in thickness:
  - type 2 – 200 mm,
  - type 3 – 250 mm,
  - type 4, 4a, 5 – 300 mm.
- various types of anchors (for all types of support, the steel anchors with a length of 6 m were used, and additionally in support type 4a, 5 the polyester roof bolts with 4 m long were installed),
- 1 or 2 layers of steel mesh with a mesh size of 150 × 150 mm and a wire thickness of 6 mm,
- lattice girders, with different lengths and the wire thickness (depends on support type):
  - for type 2: length 114 and thickness 28 mm and 20 mm,
  - for type 5: length 180 and thickness 30 mm and 20 mm,
- in 4a and 5 support types, a micropile umbrella made of injected steel pipes was also used.

Depending on the RMR values, the following types of support were used:

- RMR < 20 – type 5, 4a, 4M,
- 21 ≤ RMR < 40 – type 4, 3M, 3,
- 41 ≤ RMR < 60 – 2M type.

When designing the tunnel, for each selected type of preliminary lining, the maximum values of displacements that can be defined as critical were assumed, and 70% of these values were taken as the warning level [24]. Table 10 shows the assumed values of warning and critical displacements, depending on the type of the pre-support and the corresponding values of critical and warning strains. Values of those parameters at the design stage are much lower than those given in Table 4 because the impact of the support has been taken into account.

During the drilling of the tunnel, measurements of the convergence and the maximum horizontal and vertical displacements of the contour of the preliminary support were performed at 45 stations spaced every few or several metres. The comparison of the measurement results with the adopted limit values answers the question of whether the limit values have been adopted correctly. The article authors divided the entire length of the excavated tunnel into sections with a similar value to the RMR index (Table 11).

TABLE 10

The estimated values of warning and critical displacements depending on the type of the preliminary support

Support type	Estimated warning displacement ($0.7u_{cr}$) for every type of the preliminary support [mm]	Estimated critical displacement (u_{cr}) for every type of the preliminary support [mm]	Calculated warning strain ε_w [%]	Calculated critical strain ε_{cr} [%]
2, 2M	28	40	0.5	0.7
3, 3M	42	60	0.7	1.0
4, 4a, 4M	42	60	0.7	1.0
5	56	80	1.0	1.4

TABLE 11

Tunnel sections for equal RMR values

No.	RMR	Distance of the section from the entrance to the tunnel [m]	Average displacements of the tunnel support		Layers angle of dip [°]	u_{wp} [mm]	u_{wr} [mm]	$\frac{u_{\max}}{u_w}$	$\frac{u_{\max}}{u_{cr}}$	$\varepsilon_r^{\max}$	$\frac{\varepsilon_r^{\max}}{\varepsilon_{cr}}$
			sidewall u_{oc} [mm]	roof u_{st} [mm]							
I	32	12 ÷ 92.4	20	43	63.5	24.6	38	0.76	0.53	0.75	0.38
II	23	92.4 ÷ 189.2	24	42	62.5	27.8	38	0.74	0.52	0.73	0.37
III	19	189.2 ÷ 232.6	64	136	62.5	79.4	121	3.19	2.23	2.36	**1.18!**
IV	35	232.6 ÷ 520.2	20	40	53.0	27.2	33	0.93	0.65	0.69	0.35
V	22	520.2 ÷ 577.7	44	130	70.0	54.6	120	2.27	1.59	2.25	**1.13!**
VI	29	577.7 ÷ 623.9	18	64	67.5	24.7	57	1.29	0.79	1.11	0.56

For these sections, table 11 presents: the average values of horizontal displacements (u_{oc}) and vertical (u_{st}), average angles of layers dip, displacements perpendicular to the stratification (u_{wp}) and parallel (u_{wr}), as well as the maximum displacements of the contour (usually for a roof). The maximum values of displacements were compared with the values of warning $\frac{u_{\max}}{u_w}$ and critical displacements $\frac{u_{\max}}{u_{cr}}$. Moreover, for the maximum values of displacements, the corresponding strain values $\varepsilon_r^{\max}$ (assuming r = 5.77 m) were calculated, which were also compared with the critical values. The analysis of the results of displacements and strain shows that sections III and V (located at the distances from 189.2 m to 232.6 m and from 520 m to 577.7 m) exceeded warning limits for displacements and strains. Additionally, the displacement assumed for warning limits were exceeded on the sections IV and VI (232.6 m ÷ 520.2 m and 577.7 m ÷ 623.9 m).

The length of the tunnel in Laliki was 678 m, of which 444 m were drilled in conditions of increased risk associated with reaching the warning displacement values, and on the total length of 101 m, displacements and deformations exceeded critical values. This leads to the conclusion that the types of preliminary supports were designed with an inadequate stiffness, influencing the rock mass with a low-pressure value of p_0, not best matched to the quality of the rock mass. During tunnel excavation, the improvement of the support was performed by additional roof bolts, thicker meshes and a new type of lining 5, which significantly increased the tunnel costs.

Table 12 shows the tunnel depth H, the uniaxial rock strength R_c, the rock massif strength R_{cm} (which was calculated using the Hoek-Brown criterion) and the following indices: Zasławski β_n and Hoek α_n. The obtained values of the β_n index in individual sections are nearly 10 times lower than the acceptable limit (0.18), which does not indicate any risk of loss of tunnel stability. The indicator α_n gives a closer look at the real ones. The acceptable limit value for this indicator is 0.36. The values of α_n in individual measurement sections are in the range 0.59 ÷ 1.79. Again, it turned out that at small depths of the excavated tunnel, these indicators are not a good measure of the risk of loss of stability. In table 12 authors also present the values of horizontal stress, which should be applied to the tunnel sidewalls to prevent the layers from sliding. These values are small (and after taking into account the cohesion, they can be smaller), so the layers sliding hazard is unlikely.

TABLE 12

Values of indicators and parameters for a tunnel Laliki

Section	H [m]	R_c [MPa]	R_{cm} [MPa]	β_n	α_n	σ_x (eq. 6) [MPa]
I	12.6	22	0.50	0.014	1.59	0.19
II	17.2	30	0.42	0.014	0.98	0.26
III	21.6	34	0.38	0.016	0.70	0.33
IV	23.5	39	1.05	0.015	1.79	0.37
V	22.2	25	0.33	0.022	0.59	0.31
VI	20.0	25	0.48	0.020	0.96	0.29

3.3. Tunnelling Experience in Naprawa

In Naprawa, on the S7 Kraków-Zakopane road, two tunnels were excavated (left and right) [25,26]. The tunnels, each 2.058 km long and with one direction of travel, are separated from each other by a 14.0 m wide pillar. The outer dimensions of these tunnels are as follows: width of 17.31 m (this is the minimum value, in parking bays, the width increases to 18.3 m), and the height of the tunnel without parts is 11.16 m. The geological structure of the rock massif in the Naprawa area is formed by Quaternary sediments, with an average thickness of several metres of weathered origin, and below flysch formations with a different proportion of sandstone and shale. In the northern part, the tunnels were excavated in the flysch massif with a significant percentage of sandstone. Whereas in the south, the percentage of shale increased, so that near the southern tunnel exit appears tectonically disturbed clay or clay shale with thin, fractured small blocks of sandstone. The tunnels were dug in rock masses of class III, IV, and V (according to the RMR classification) and depending on the class of the rock mass, the following types of preliminary support were used [26]:

- in the rock mass class III, the preliminary support consisted of double steel arches (type 2IPE 180) installed every 2.0 m, sprayed concrete 0.25 m thick, steel mesh with a wire thickness of 10 mm and, if necessary, several steel anchors with a length of 6.0 m installed in the roof,
- in class IV, the distance between the double arches of the steel lining was reduced to 1.5 m, an umbrella roof bolt support was added consisting of a dozen steel anchors with

a length of 6.0 m, and other elements of the preliminary support were adopted as for class III,
- for a class V – in the first stage, an overlapping pre-lining in the roof was used, consisting of several dozen steel pipes 15 m long, which formed a protective umbrella during tunnel drilling. Moreover, the tunnel face was bolted with several dozen 15 m long glass fibre pipes. The preliminary support consisted of double steel arches every 1.0 m, sprayed concrete 0.25 m thick with a steel mesh 10 mm thick.

Without the portals, each tunnel was 1.920 m long. The observation showed that tunnels along the length of 890 m (46% of the length) will be excavated in class III of rock mass, 740 m (38% of the length) in the rock mass of class IV, and 290 m (15% of the length) in class V. Analytical calculations carried out at the design stage showed that the radial displacements (u_r) of the tunnel contour near the face of the face will reach low values:
- class III – $u_r \leq 12$ mm,
- classes IV and V – $u_r \leq 25$ mm.

Moreover, in class V, the displacement of the tunnel face should be less than 0.3%. Estimated low displacement values were the result of a relatively good quality rock mass (RMR between 30 and 60) and the design of a stiffer and much stronger initial support than that used in the tunnels in Laliki and Świnna Poręba. On the basis of the calculated values of radial displacements of the tunnel contour, the following limit values were selected: convergence, vertical displacements and strain of the tunnel face (Table 13).

TABLE 13

Limits of strain and displacement of the contour for the Naprawa tunnels

Measurements	Acceptable limit	Warning limit	Critical limit
Pre-support convergence [mm]	30	40	50
Displacement of tunnel roof [mm]	25	30	40
Tunnel face strain* [%]	0.3	0.5	0.8

* only for class V

While drilling the left and right tunnels, it turned out that the rock massif has qualitatively better parameters than initially assumed. Almost 70% of the length of the tunnel was drilled in the rock mass of class III and 27% in the rock mass of class IV. The last 60m tunnel near the southern portal (3% of its length) was dug in a class V rock mass in a tectonically disturbed shale of low values: $RMR_b = 12$ and $GSI = 10 \div 15$.

The better quality of the rock mass and an appropriately selected preliminary support (stiffness and strength) caused the low values of tunnel contour displacements and convergence [27]. In both tunnels, practically along their entire length, the maximum values of convergence $\Delta K_{\max}$ and vertical roof displacements $v_{\max}$ were two to three times lower than the acceptable limit:
- $\Delta K_{\max} \leq -10$ mm, $v_{\max} \leq = -10.5$ mm.
- Only near the southern portals, the maximum values of displacement and convergence were higher $\Delta K_{\max} = -13$ mm, $v_{\max} = -30$ mm, i.e. it has reached the warning limit.
- Taking the above into account, it should be stated that the limit values have been adopted correctly.

4. Summary and conclusions

From the presented considerations, the following conclusions can be drawn:

- Various indicators are used to assess the behaviour of the rock mass in the vicinity of the tunnel excavation with preliminary support. The study analysed two indexes of Zasławski β_n and Hoek α_n. The analysis shows that at low depths, the Hoek α_n index can be used to estimate the potential hazard, while the Zasławski index β_n can only be used for tunnels excavated at greater depths.
- Proper selection of the type of the preliminary support adapted to the quality of the rock mass is made by measuring displacements and strain and comparing these values with the limit values (acceptable, warning, and critical). The limit values of displacements and strain should be determined at the tunnel design stage based on precise numerical calculations supported by experience gained from tunnels under similar conditions. If it turns out that the limit values have not been correctly determined during the excavation, then the values can be changed, provided that appropriate numerical calculations are made in agreement with the designer, confirming the correctness of adopting the new values.
- Various preliminary linings were used in the tunnels in Naprawa, Laliki and Świnna Poręba. This support generally consisted of shotcrete, mesh, bolts, and steel arch. The type of used steel implied that these linings differed from each other in terms of strain. Estimated calculations show that the maximum strain of the steel support is [12]:
- $\varepsilon = 0.33\%$, for double steel arch type 2IPE 180 (used in the Naprawa tunnel),
- $\varepsilon = 1.35\%$, for lattice girders (used in the Laliki tunnel),
- $\varepsilon = 0.55\%$, for a steel arch (used in Świnna Poręba tunnels).

The problem is that the maximum strain of the shotcrete lining is about $\varepsilon_{max} = 1.0\%$. In the case of the Lalika tunnel, the lattice girders were more deformable than shotcrete, which meant that less deformable concrete carried higher loads and could be damaged.

- The displacements of the tunnel contour depend on the type of preliminary support. The limit values should be adapted to the strength and deformation properties of the preliminary support so as not to destroy or damage it.
- For tunnels excavated in the Carpathian flysch (without or with passive support), the following strain limits should be assumed:
  - acceptable strain – 1.0%,
  - warning strain – 1.5%,
  - critical strain – 2.0%.

These values were carefully adopted due to the complex structure of the flysch massif like lamination, fractures, layers inclination, asymmetrical loading of the preliminary support and significant bending moments.

- From the values of the limit strain, the values of the limit displacements of the tunnel contour should be calculated, taking into account the dimensions of the tunnel cross-section. Table 4 shows the calculated values of the contour limit displacements for the tunnels in Świnna Poręba, Laliki and Naprawa. If the results of displacements measured in these tunnels are compared with the adopted limit values, it can be concluded that in Świnna Poręba and Naprawa, the limit values were adopted correctly, while in Laliki the value of warning displacements should be 70 mm, and the value of critical displacements 100 mm. These values are higher than those adopted at the design stage but lower than the maximum displacements obtained from measurements (136 mm or even more).

References

[1] A. Tajduś, M. Cała, K. Tajduś, Geomechanika w budownictwie podziemnym: projektowanie i budowa tuneli. Wydawnictwa AGH Kraków, Poland, 2012.

[2] C. Carranza-torres, C. Fairhurst, The elasto-plastic response of underground excavations in rock masses that satisfy the Hoek ± Brown failure criterion, **36**, 777-809 (1999).

[3] P. Marinos, E. Hoek, GSI: A Geologically Friendly Tool for Rock Mass Strength Estimation. In: Proc. GeoEng2000 Conf., Melbourne, pp. 1422-1442 (2000).

[4] E. Hoek, P. Marinos, Predicting tunnel squeezing problems in weak heterogeneous rock masses, Tunnels Tunn. Int. **32**, 45-51 (2000).

[5] G.P. Tziallas, H. Saroglou, G. Tsiambaos, Determination of mechanical properties of fl ysch using laboratory methods. Eng. Geol. **166**, 81-89 (2013). DOI: https://doi.org/10.1016/j.enggeo.2013.09.002

[6] W. Margielewski, Structural control and types of movements of rock mass in anisotropic rocks: Case studies in the Polish Flysch Carpathians, **77**, 47-68 (2006). DOI: https://doi.org/10.1016/j.geomorph.2006.01.003

[7] G. Lunardi, M. Gatti, Tunnel monitoring system – A contribution for the preparation of guidelines, ITA Interational Tunneling Assoc. (n.d.) 1-8.

[8] B. Prof, E. Pietro, L.C. Engineers, The design and construction of tunnels using the approach based on the analysis of controlled deformation in rocks and soils, 3-30 (2000).

[9] E. Hoek, Support for very weak rock associated with faults and shear zones. In: Rock Support Reinf. Pract. Min., Routledge, pp. 19-32 (2018).

[10] J.Z. Zasławski, Some Aspects of the Support of Deep Permanent Workings. In: National Coal Board (Ed.), Fifth Int. Strat. Control Conf., National government publication, London, 1972.

[11] J.B. Drużko, J.Z. Zasławskij, F. Pieriepiczka, Ustrojcziwost osnownych gornych wyrabotok, Donieck, Donbass, 1975.

[12] E. Hoek, Practical Rock Engineering: Evert Hoek consulting Inc, North Vancouver, BC, Canada. Retrieved 27th Febr. (2010) 237.

[13] S. Sakurai, Displacement measurements associated with the design of underground openings. In: F. Meas. Geomech. Int. Symp., pp. 1163-1178 (1984).

[14] M. Singh, B. Singh, J. Choudhari, Critical strain and squeezing of rock mass in tunnels. Tunn. Undergr. Sp. Technol. **22**, 343-350 (2007).

[15] A. Tajduś, K. Tajduś, The use of tunnels to development of transport in mountain areas, Geomatics, Landmanagement Landsc. **4**, 103-112 (2013).

[16] T. Majcherczy, Z. Niedbalski, M. Kowalski, 3D numerical modeling of road tunnel stability – the Laliki project. Arch. Min. Sci. **57**, 61-78 (2012). DOI: https://doi.org/10.2478/v10267-012-0005-6

[17] L. Zabuski, Zachowanie się fliszowego ośrodka skalnego w otoczeniu konstrukcji podziemnej na przykładzie tunelu na niedużej głębokości. Wydawnictwo IBW PAN, Gdańsk (2002).

[18] K. Thiel, Właściwości fizyko-mechaniczne i modele masywów skalnych polskich Karpat fliszowych. Wydaw. IBW PAN, Gdańsk (1995).

[19] C. Jaeger, Rock mechanics and engineering. Cambridge University Press, 1979.

[20] Z. Bestyński, K. Thiel, L. Zabuski, Zasady geotechnicznych klasyfikacji masywów fliszowych. Mater. Semin. Probl. Geotech. Przy Proj. i Budowie Tuneli Hydrotechnicznych, Świnna Poręba i Bielsk, 12-26 (1988).

[21] Z.T. Bienawski, Rock mass classifications in rock engineering. (1976).

[22] M. Dynowska, Prognozowanie deformacji górotworu fliszowego w otoczeniu tuneli. AGH University of Science and Technology, (1999).

[23] Z. Niedbalski, P. Małkowski, T. Majcherczyk, Application of the NATM method in the road tunneling works in difficult geological conditions – The Carpathian flysch. Tunn. Undergr. Sp. Technol. **74**, 41-59 (2018).

[24] T. Majcherczyk, Z. Pilecki, Z. Niedbalski, E. Pilecka, M. Blajer, J. Pszonka, Wpływ warunków geologiczno-inżynierskich i geotechnicznych na dobór parametrów obudowy wstępnej tunelu drogowego w Lalikach. Gospod. Surowcami Miner. **28**, 103-124 (2012).

[25] A. Tajduś, K. Tajduś, Construction of a tunnel between Naprawa and Skomielna Biała on the S7 road. Bud. Górnicze i Tunelowe, 1-11 (2017).

[26] F. Schiavone, U. Ryndak, M. Witczyński, Astaldi buduje nowy odcinek zakopianki, Naprawa – Skomielna Biała wraz z tunelem, Nowocz. Bud. Inżynieryjne, 36-41 (2017).

[27] M. Cała, A. Tajduś, A. Stopkowicz, D. Wałach, M. Kolano, Opinia ekspercka dotycząca deformacji mas skalnych i stabilizacji w ramach kontraktu: „Budowa drogi ekspresowej S7 Kraków – Rabka Zdrój na odcinku Naprawa – Skomielna Biała w km od ok. 721+170 do ok. 724+220 wraz z budową tunelu". Kraków (2017).