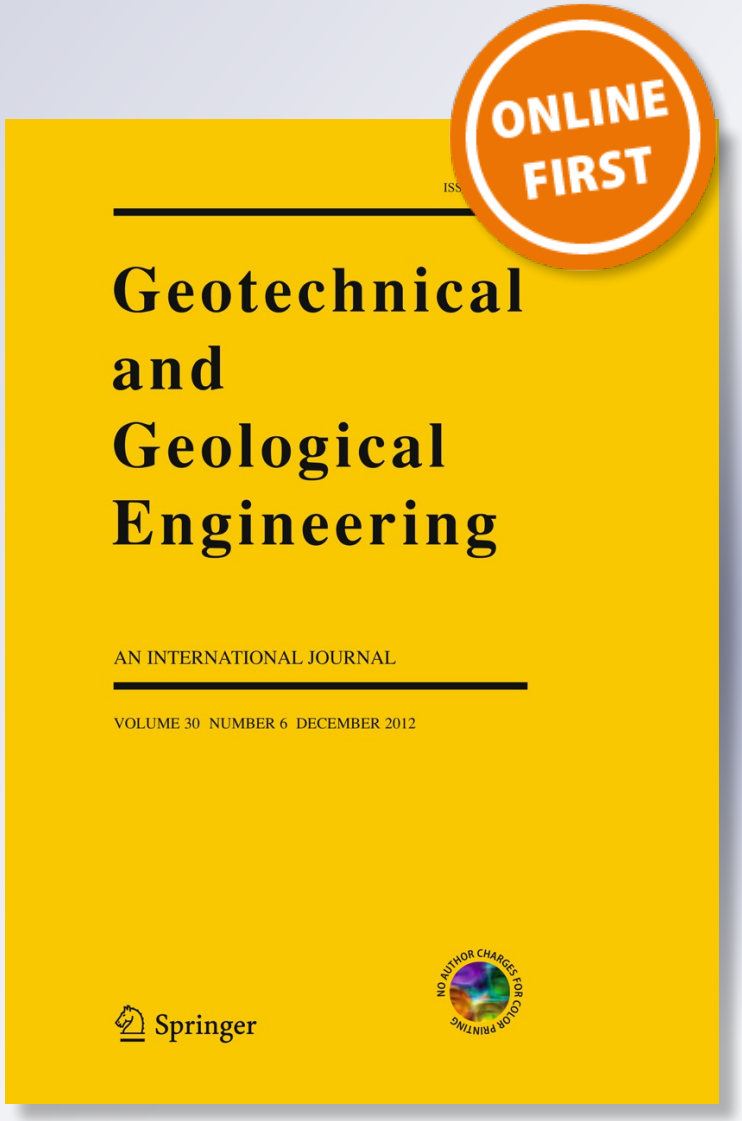
Block-in-Matrix Structure and Creeping Slope: Tunneling in Hard Soil and/or Weak Rock

Dietmar Adam, Roman Markiewicz & Markus Brunner

Geotechnical and Geological Engineering
An International Journal

ISSN 0960-3182

Geotech Geol Eng
DOI 10.1007/s10706-012-9591-5



Geotech Geol Eng
DOI 10.1007/s10706-012-9591-5

ORIGINAL PAPER

Block-in-Matrix Structure and Creeping Slope: Tunneling in Hard Soil and/or Weak Rock

Dietmar Adam · Roman Markiewicz · Markus Brunner

Received: 27 June 2012 / Accepted: 1 November 2012

Abstract The city bypass tunnel of Waidhofen an der Ybbs (Austria) is situated in an intricate geological-geotechnical complex consisting of soil, hard soil, weak rock and solid rock. Marls and marly limestone (Waidhofener formation) predominate followed by tectonic breccia of the Alpine cliff zone, which is a tectonic melange zone. Materials of this kind universally have isolated interior blocks embedded in a matrix called "block-in-matrix" (see Fig. 1) requiring a particular characterization of the mechanical rock mass properties. The complex geological and morphological situation required the application of various tunneling methods including open cuts, cut-and-cover sections and the New Austrian Tunneling Method. During construction in creeping slopes increased movements were triggered, which required the installation of a sophisticated monitoring system. On the basis of monitoring data and additional ground investigations structural measures were assessed that enabled safe tunneling in the creeping slope.

Keywords Tunneling · NATM · Creeping slope · Hard soil · Weak rock · Block-in-matrix · Bimrock

D. Adam (✉)
Institute of Geotechnics—Ground Engineering, Soil and Rock Mechanics, Vienna University of Technology, Karlsplatz 13, 1040 Vienna, Austria
e-mail: dietmar.adam@tuwien.ac.at
URL: www.igb.tuwien.ac.at

D. Adam · R. Markiewicz
Geotechnik Adam ZT GmbH, Wiener Straße 66-72/15/4, 2345 Brunn am Gebirge, Austria

M. Brunner
Authority of Lower Austrian Government, Landhausplatz 1, 3109 St. Pölten, Austria

1 Project

The city tunnel Waidhofen an der Ybbs is the core of the city by-pass of a small town of the same name in the south-west of Lower Austria. The road tunnel comprising a total length of 1,485 m is situated along the mountain ridge called Buchenberg and the maximum overburden of the tunnel is about 50 m. Tunnel construction started in November 2007 whereby different construction techniques were applied. The opening to traffic was in November 2011.

2 Geology

2.1 Overview

In the western tunnel section with high overburden marls and marly limestone (Waidhofener formation) predominate followed by tectonic breccia of the Alpine cliff zone, which is a tectonic melange zone. These are tectonically altered rock masses of the cliff zone embedded in a silty clayey matrix.

Published online: 21 November 2012

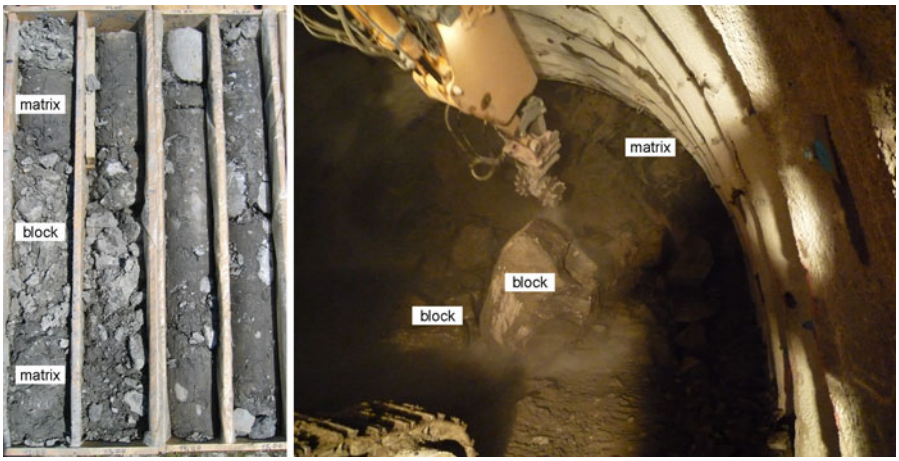


Fig. 1 *Left* Core photo of a drilling through rock mass type GA2 with a block-in-matrix structure. *Right* Tunneling through block-in-matrix structure with digged out blocks

In the portal areas and in shallow tunnel sections sediments of the lower terrace and intermediate terrace layers predominate beside soil masses of long-term creeping slopes.

The following rock mass types (GA) were classified (see Fig. 2):

- GA0: Weathered rock mass of the Waidhofener formation
- GA1: Waidhofener formation (solid rock)
- GA2: Tectonic melange (block-in-matrix structure)
- GA3: Tectonic melange (matrix)
- GA4: Areas with creeping slopes

2.2 Tectonic Melange—Block-in-Matrix

Materials of the tectonic melange universally have isolated interior blocks embedded in a matrix (see Fig. 1). Such a mélange structure was called "block-in-matrix" by Raymond (1984). Finally Medley (1994) defined the word "bimrocks" (block-in-matrix rocks) for mixtures of rocks, composed of geotechnically significant blocks within a bonded matrix of finer texture. The physical and mechanical properties of the matrix and interior blocks are not the same. In the considered case the matrix is weaker and softer than interior blocks. The interfaces between the matrix and interior blocks as well as other fractures are inherent discontinuities with limited strength. The amount of the competent blocks differs in a wide range. The blocks are irregularly shaped and have sizes from some cm^3 to more than 100 m^3. In some sections the block-in-matrix texture appears like rock throughout the entire cross section of the tunnel. In sections with a minor volumetric portion of blocks the rock mass is predominated by the matrix primarily consisting of silt and clay and decomposable rock with varying properties and a dense structure. Figure 1 shows as an example the core of a drilling through a block-in-matrix structure, where some (perforated) blocks embedded in a matrix can be seen. Furthermore Fig. 1 gives a photo of tunneling through block-in-matrix structures.

Lindquist (1994) and Lindquist and Goodman (1994) determined that the overall strength of a bimrock is related to the volumetric proportions of the blocks in the matrix. Therefore, in the considered case rock mass type GA2 is characterized by the block-in-matrix structure, predominantly consisting of rocks of the tectonic melange and in the transition zone of (limestone) marls as well, embedded in a silty-clayey matrix. Below a defined volumetric block portion resting in the matrix the rock mass properties are dominated by the properties of the matrix (GA3).

Rock mass type GA4 has the same composition as rock mass type GA3. Due to shallowness GA4 is affected by creeping movements, partially wetting and weathering.

Within the tunneling areas dry underground condition are predominant; low groundwater influxes occur only in the portal zones.

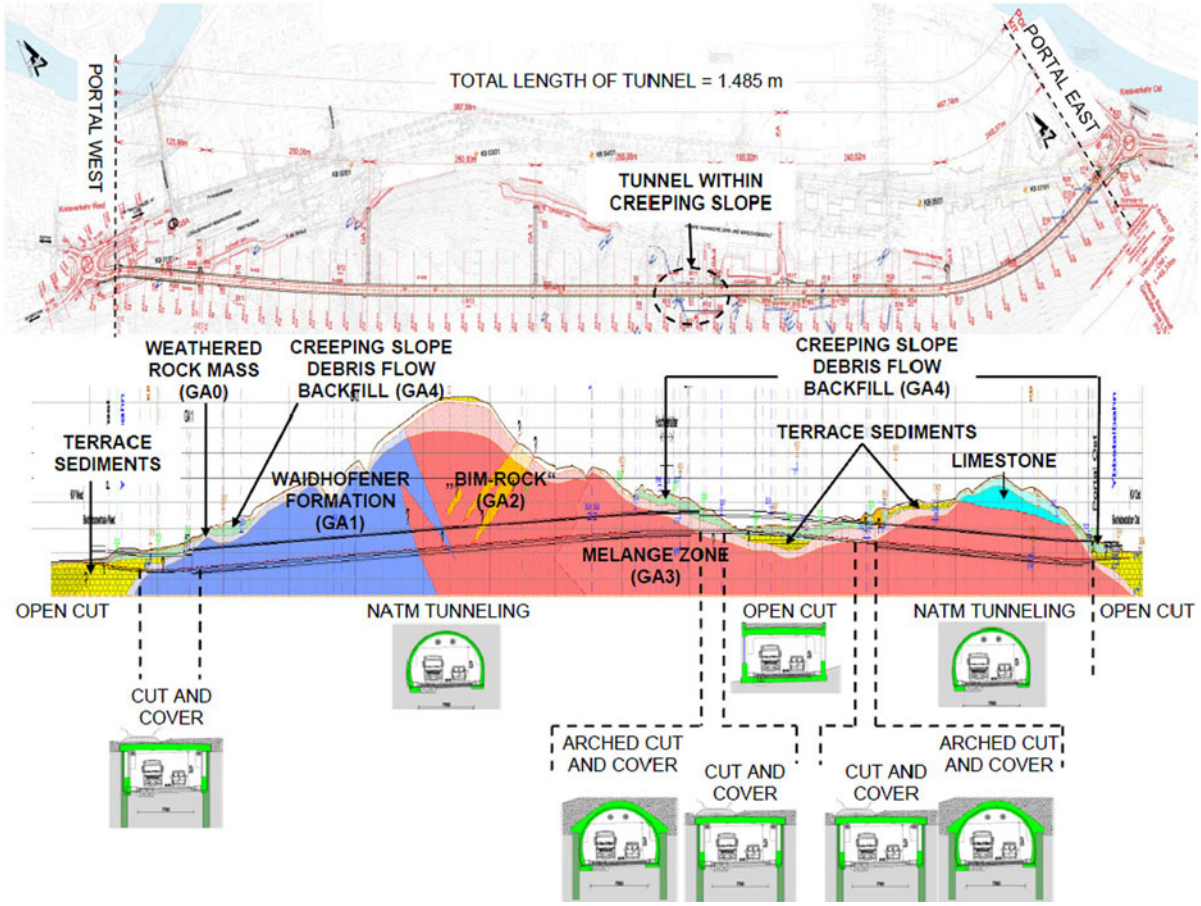


Fig. 2 Tunnel alignment in the project area and geological longitudinal section with different tunneling methods. In the ground plan the areas with creeping slopes are shown

3 Tunneling

3.1 Tunnel Construction Methods

Depending on the geological-geotechnical ground properties and the overburden the tunnel was constructed with different tunnel construction methods (see Fig. 2).

In shallow tunnel sections with an overburden of a few meters whether open cuts or (arched) cut-and-cover tunnels were designed taking into account the geological conditions.

In sections with overburdens up to about 50 m the New Austrian Tunneling Method (NATM) was applied. In the Waidhofener formation where no swelling potential was expected in the rock mass no ring closure was necessary. In all other ground conditions a closed ring was realized.

NATM was combined with blasting within the Waidhofener formation whereby the defined vibration limit values (maximum resulting velocity $v_{R,max} = 4$ resp. 8 mm/s depending on the building class) had not to be exceeded due to the vicinity of the inhabited area. In the other rock masses excavation took place with hydraulic excavation equipment with light blasting if necessary.

In the central area where the tunnel was situated above the ground surface a rectangular open tunnel was designed. Between the open cut and the NATM section the cut-and-cover method with an arched ceiling was applied. In the portal sections the tunnel was constructed in open cuts.

Tunneling through all geological sections in particular also through block-in-matrix structure (GA2) as well as matrix structure (GA3) was carried out with no significant deviations from the predicted behavior

except a tunnel section through a creeping slope that is described in the chapter "Creeping Slope".

4 Block in Matrix

4.1 Definition

As already mentioned the term "bimrock" or "block-in-matrix rocks" describes mixtures of rocks composed of geotechnical significant blocks within a bonded matrix of finer texture. The tectonic melange zone (GA2 and GA3) complies with this definition.

The words geotechnical significant in the above definition of bimrocks mean that there are criteria for scale, strength contrast, proportion and size of blocks. In bimrocks blocks must have mechanical contrast with the matrix and at the scale of engineering interest there must be enough blocks of a certain size range to contribute to the overall strength of the rock mass meeting the following criteria (Medley 1994):

- Significant mechanical contrast between the blocks and the matrix, measured as a ratio between friction angles, stiffness, or other properties.
- Significant scale factor between block size and the so called characteristic dimension, in this case the tunnel diameter. The upper limit for the block size is defined with 75 % and the lower limit with 5 % of the tunnel diameter. Beyond this value the ground is designated rock, below matrix (soil).
- If the volumetric portion of blocks is between 25 and 75 % the rock mass is classified as bimrock. It is assumed that the linearly determined block portion from boreholes complies with the volumetric block portion and thus coincides with the spatial distribution within the matrix.

Following the definition of bimrock by Medley (1994) for the city tunnel project the decisive block size d was derived from the tunnel diameter of 10 m to:

- Matrix: $d \leq 0.5$ m
- Block: 0.5 m $\leq d \leq$ 7.5 m
- Sound rock: 7.5 m $\leq d$

As already described, the differentiation between matrix and bimrock was defined by Medley (1994) with a volumetric block portion of 25 %. For the city tunnel project a volumetric block portion of 30 % was selected. With a block portion of <30 % the rock mass behavior is predominated by the matrix and the stiffness and strength parameters etc. are derived from the matrix (GA3). The rock mass classified GA2 is defined by a block portion of >30 % and is determined from the bimrock theory.

4.2 Rock Mass Behavior

The soil mechanical properties of a bimrock structure can be concluded as follows (Lindquist 1994; Lindquist and Goodmann 1994):

The overall stiffness of a tectonic melange increases with an increasing block portion. However, the stiffness modulus of the entire melange mass is assessed in accordance with the stiffness of the matrix side.

The cohesion of the tectonic melange decreases with an increasing block portion. This results primarily from the "soft" contact zones between blocks and matrix.

The angle of internal friction of the tectonic melange increases with an increasing block portion due to the curved failure surfaces. Taking into account these influencing factors the following model for the shear strength of bimrock can be defined (Lindquist 1994):

$$\tau_p = c_{\text{Matrix}} \cdot (1 - \theta) + \sigma \cdot \tan(\varphi_{\text{Matrix}} + \Delta\varphi(\theta)) \quad (1)$$

with τ_p shear strength of tectonic melange, c_{Matrix} cohesion of the matrix, σ normal stress, φ_{Matrix} angle of internal friction for the matrix, θ volumetric block proportion, $\Delta\varphi(\theta)$ angle of internal friction increase for the block proportion of interest (a 3° increase for every 10 % increase in block proportion can be considered).

The orientation, the shape and the spatial distribution of the blocks (in particular of the large blocks) influences the failure behavior. The failure surfaces pass around the blocks whereby they touch the contact zone between block and matrix due to their weak shear strength characteristics. Tables 1 and 2 give an overview about the rock and rock mass properties of the tectonic melange. The rock mass properties GA2 have been derived from the bimrock formulation. The volumetric block portion was determined on the basis of the drilling cores and it ranges from 30 to 84 % within the tunneling zone. Thereby, a significant higher cohesion and angle of friction were considered.

In addition to that it is shown in Table 2 that different soil properties due to excavation technology

Table 1 Rock and rock mass properties for the tectonic melange GA2 (block dominated)

Rock properties	Block			Matrix		
	Range	n	s	Range	n	s
UCS (MN/m^2)	28.9–62.2	4	13	0.04–0.35	9	0.11
$m_{i\ [-]}$	*7*			n.a.	–	–
φ (°)	*45–55*			14–31.5	31	4.5
$E_{S,2}$ (MN/m^2)	40,722	1		25–51.5	6	9.3
ν (–]	0.28	1		*0.25–0.35*	–	–
CAI [–]	0.32	3	0.04	–		
Rock mass properties						
GSI (–)			–			
UCS (kN/m^2)			–			
c (kN/m^2)			25–70			
φ (°)			32–38			
E (MN/m^2)			*1,000–2,500*			

m_i Hoek constant, $E_{S,2}$ deformation modulus reloading, $E_{S,E}$ deformation modulus unloading, *CAI* Cerchar Abrasiveness Index, *n* number of tests, *s* standard deviation, italic values are empirical values

(NATM resp. cut and cover) have been set. The reason for this is that different calculation methods have been used that required a different set of parameters. For NATM tunneling calculations with the Finite-Element-Method were performed and in contrast to that for the cut and cover section limit state soil mechanics was applied.

4.3 Rock Mass Classification for Tunneling

Tunneling in tectonic mélanges have been discussed by Button et al. (2002 and 2004), Moritz et al. (2004) and others. In general the rock mass behavior during tunneling strongly depends on the following factors:

- Local block portion and spatial distribution adjacent to the actual tunnel face
- Block size
- Block shape and special location
- Relative deformability between block and matrix
- Strength between block and matrix
- Groundwater

The heterogeneity of the tectonic melange produces a stress concentration in the blocks so that failure can occur within the blocks ("stiff components attract stresses"). Eventually, stress redistributions can cause additional deformation in the matrix (Button et al. 2004).

Isolated blocks within the tunnel face influence the stability of the tunnel face but only temporarily the overall stability of the tunnel (rock mass ring support). In general, deformations increase with

Table 2 Rock and rock mass properties for the tectonic melange GA3 (matrix dominated)

Rock properties	Range	n	s
UCS (MN/m^2)	0.04–0.35	9	0.11
$m_{i\ [-]}$	–	–	–
φ (°)	14–31.5	31	4.5
$E_{S,2}$, $E_{S,E}$ (MN/m^2)	25–51.5	6	9.3
ν (–)	*0.25–0.35*	–	–

Rock mass properties	Range	
	NATM tunneling	Open cut; cut and cover
GSI (–)	–	–
UCS (MN/m^2)	–	–
c_{Matrix} (kN/m^2)	50–135	0
$φ_{Matrix}$ (°)	25–31	25
E (MN/m^2)	50–100	–
$E_{S,2}$ (MN/m^2)	–	15–40

m_i Hoek constant, $E_{S,2}$ deformation modulus reloading, $E_{S,E}$ deformation modulus unloading, *CAI* Cerchar Abrasiveness Index, *n* number of tests, *s* standard deviation, italic values are empirical values

decreasing volumetric block distribution. Moreover, the position of stiff blocks within the tunnel face plays a major role for the rock mass behavior (Button et al. 2004).

Tunneling through block-in-matrix structure (GA2) was carried out with no significant deviations from the predicted behavior except a tunnel section through a creeping slope that is described in the following chapter.

5 Creeping Slope

5.1 Design Stage

Already in the design stage inclinometer measurements indicated a creep behavior in the central section of the tunnel (see Fig. 3) with a (natural) annual creeping rate of about 14 mm/a. Moreover, residual shear angles determined on samples taken from this section, swelling clay minerals and relatively high natural water content substantiated the instable slope.

Consequently, for drainage of the slope gravel piles were designed, thus, increasing the long-term slope stability.

The design of the open cut and the affected cut and cut-and-cover section of the tunnel was carried out taking into account a creep earth pressure E_{cr}. This creep earth pressure was determined according to the formulation of Brandl and Dalmatiner (1988) for the particular case that the slope inclination β equates to the friction angle φ. In Eq. (2) h represents the thickness of creep mass affecting the tunnel wall and $m(\varphi)$ is a factor depending on the stiffness of the retaining structure:

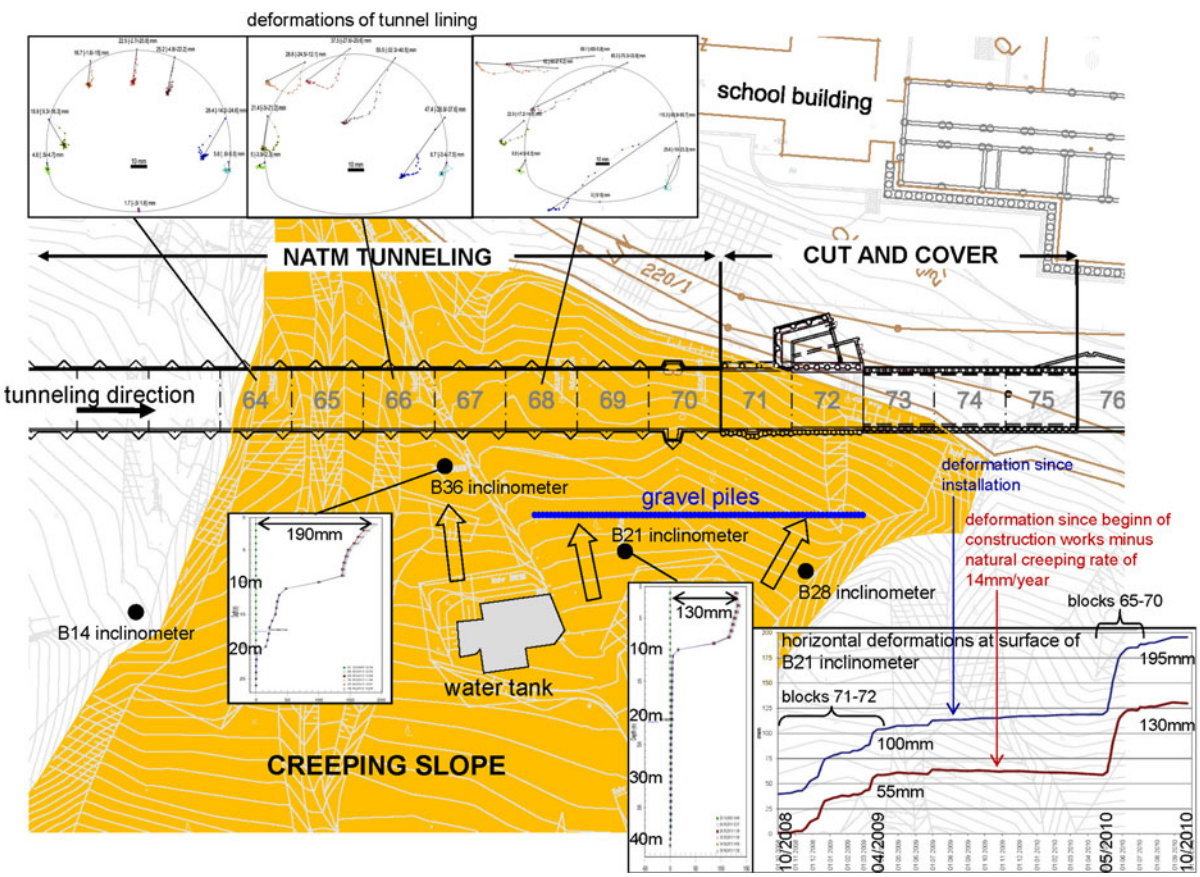


Fig. 3 Location of the tunnel within the creeping mass. The position of the gravel piles, the water supply tank and the inclinometers are shown as well as the results of inclinometer measurements, deformation measurements at the tunnel lining and deformation measurements at the surface

$$E_{cr,h} = m(\varphi)\ \gamma\ \frac{h^2}{2}\cos^2\varphi = K_{cr,h}\ \gamma\ \frac{h^2}{2} \quad (2)$$

For the city tunnel project a horizontal earth pressure coefficient of $K_{cr,h} = 1{,}1$ was assessed. The other soil parameters for GA4 were the same as for GA3 (see Table 1) with the following modifications: For NATM tunneling cohesion was reduced to a range of $c = 20$–40 kN/m^2 as well as elasticity modulus to a value of $E = 25$ MN/m^2. In addition to that a friction angle at rest of $\varphi_{rest} = 15°$ was considered for cut and cover sections.

The following measures were considered in the design:

- NATM section:
  - pipe umbrella
  - short excavation length
  - increased thickness of outer lining
  - short-term ring closing
  - increased retaining elements (self-bore anchors)
- Cut-and-cover section:
  - longer prestressed anchors

Moreover, the following additional contingency measures were foreseen, if movements would exceed a threshold value of 150 mm (based on the natural creeping rate of about 14 mm/a):

- slope drainage
- slope stabilization by shafts, piles, anchors or nails

5.2 Construction Stage of Cut-and-Cover-Section

Inclinometer measurements detected the influence of precipitation in particular after forest clearance by a significant increase of the creeping rate. Finally, in August 2008 the gravel piles were bored. The construction of these overlapping gravel columns were carried out as cased bore piles, whereby the drainage gravel material was filled in the casing without compaction. During installation significant additional slope movements were triggered and cracks were observed in a water supply tank situated above the gravel piles (see Fig. 3). The results of the near inclinometer B21 identified a sliding horizon in a depth of about 10 m below surface (see Fig. 4). By means of the morphology it was suggested that a set of local creep mass bodies with various sliding horizons exist.

Consequently, a comprehensive monitoring and data acquisition system was installed consisting of inclinometers, geodetic measurements and water quantity recordings (in the gravel piles). It has to be pointed out that the movements cannot be associated with the given creep earth pressure E_{cr}, because these noticeable movements occurred during construction of a working road and before the tunnel section was constructed.

From October 2008 additional slope deformations occurred during the construction of the tunnel sections No. 71 and No. 72 (see Fig. 3), the water supply tank was affected again. These slope movements resulted not only from the construction activities but originated from an increased wetting of the slope by intense precipitation and beginning of snow melting. After the installation of temporary anchorage of the retaining wall made of bored piles (see Fig. 4) the slope creep rate could be reduced again (surface deformation in Fig. 3).

Again in the period March to April 2009 short-term slope movements were observed during excavation works, which got under control after construction of the tunnel ceiling.

Inclinometer B21 showed total surface deformations of about 100 mm from the reference measurement in 2005–April 2009. It had to be considered that the natural creeping rate was included into the total deformations which were superimposed by the movements triggered by the construction activities. Taking into account the natural creeping rate additional deformations of about 55 mm occurred due to construction works (see Fig. 3).

Measurements of anchor forces showed that the maximum design working load was exceeded up to 25 %, whereby no additional increase could be observed after the end of the construction works.

In the period April 2009–May 2010 the monitored movements were in the range of the natural creeping rates. Gravel piles successfully drained the slope, the water flux was determined to an annual average of about 780 l/day.

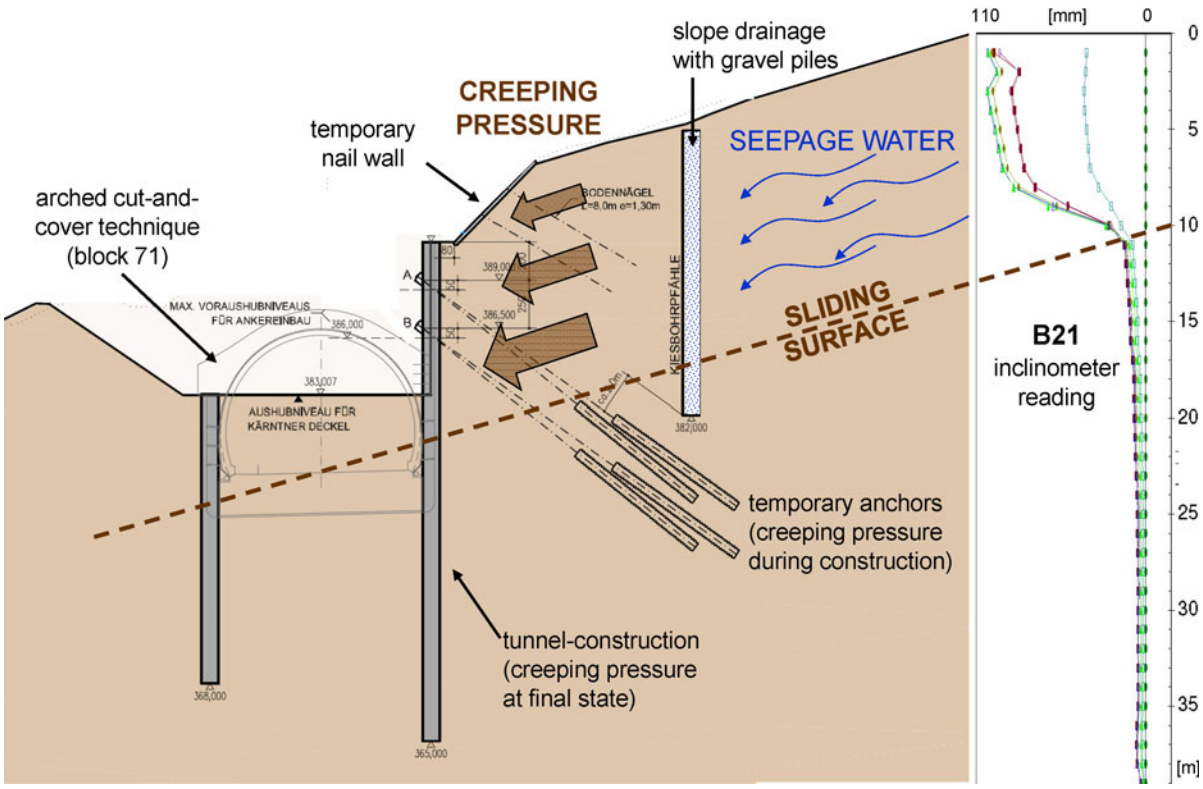


Fig. 4 Tunnel cross section in the area of the cut-and-cover tunnel with an arched ceiling (Sect. 71) with identified sliding horizon and the location of the gravel piles

5.3 Construction Stage of NATM-section

However, in May 2010 an increase of slope deformations was observed again caused by the NATM tunneling in the creep mass. In advance parametric studies were performed in order to investigate the influence of the creep behavior on the tunnel. The results of the analyses disclosed that the creep earth pressure (creep horizon close to the tunnel head) did not affect the bearing capacity but the defined serviceability limit state of the tunnel lining was exceeded so that large deformations and even cracks had to be expected. The attention had to be turned to the outer lining since the reinforced inner lining influenced the deformation only to a minor degree but served for a better distribution of the stresses and allowable fissured cracks. These findings resulted in the decision to increase the thickness of the outer and the inner lining from 30 to 40 cm and to reinforce the inner lining.

Moreover, a contingency plan was prepared to be able to carry out stabilization measures quickly if necessary. The stabilization measures included extensive slope dowelling by vertical and/or inclined piles, whereby it was not necessary to carry out these measures until now.

According to Figs. 3 and 5 the deformations in the tunnel roof amounted up to 8.5 cm and at the surface up to about 27 cm. The maximum deformations of the tunnel lining were about 11 cm, whereby asymmetric deformation of the tunnel was observed as expected due to the lateral creeping behavior of the slope.

With respect to the total deformations of the tunnel lining the static calculations yielded that the ultimate bearing capacity of the shotcrete shell will not be reached and deformations at an early stage are not critical since the tunnel lining is relatively soft and the stresses are redistributed by creep. Only deformations occurring to a later state were detected to be problematic.

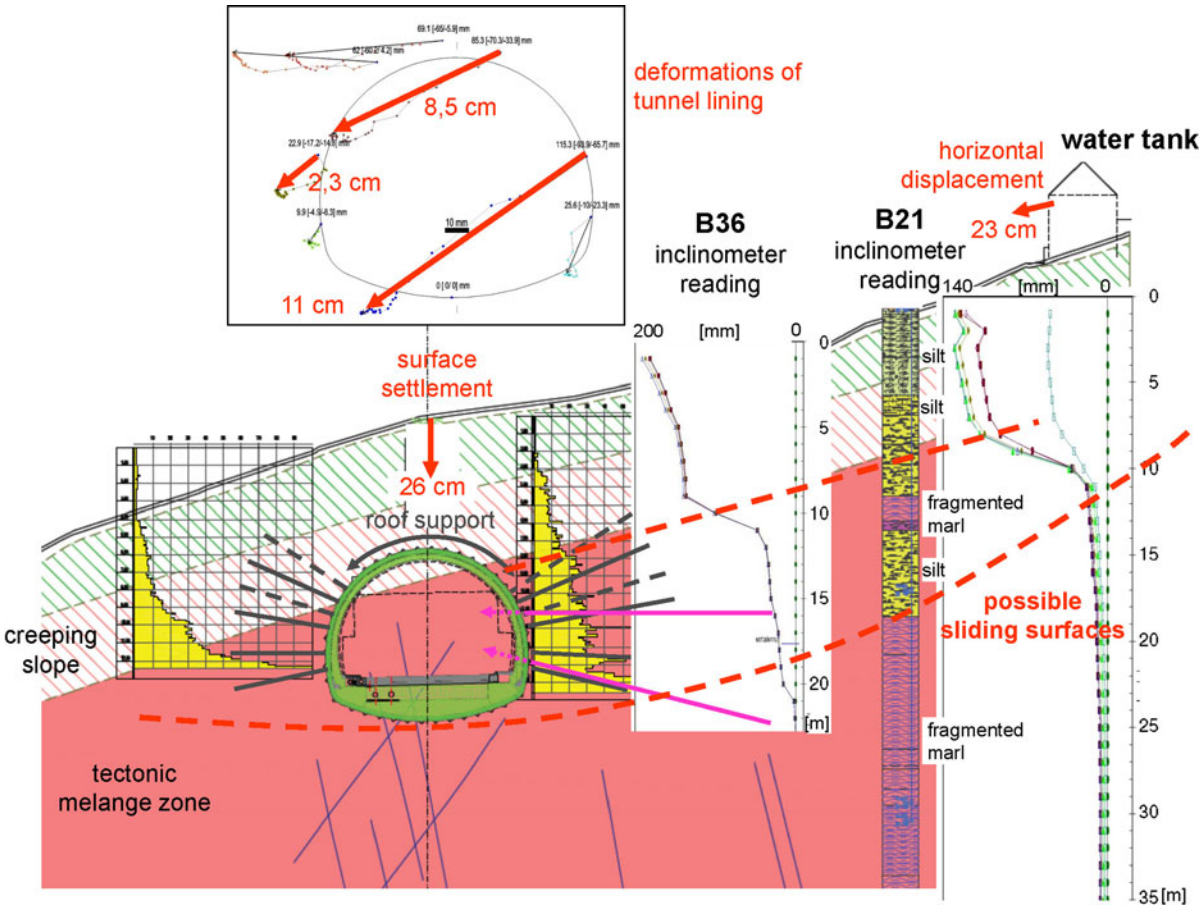


Fig. 5 Cross section in the area of the NATM tunnel (Sect. 67) with position of assumed sliding horizons and measured deformations of the tunnel lining and from inclinometers. Pipe roof, 6 m long anchors in general and 9 m (crown) and 6 m (bench) at slope side were installed. Additional anchoring in sections with 15 m long anchors (*pink*) was required at slope side

The tunnel excavation was executed in segments (top heading, bench, and invert) with short-term ring closure. A pipe roof ensured protection by means of pipes of a length up to 18 m that were driven into the ground preparative and then secured by the use of shotcrete and, if applicable, supported by steel arches. Moreover, self-bore anchors were installed (schematically in Fig. 5). All anchors oriented to the creeping slope showed a loss of the applied pre-stress and anchor heads of anchors less than 10 m long displaced in direction of the cavity punching the girder. Thus, it could be assumed that the bond length of the anchors ended in the creep mass, which was confirmed by inclinometer measurements. Inclinometer B36 (see Fig. 5) indicated a significant sliding plane. The total deformations originated from deeper regions as well, which were obviously linked with the tunneling. Presumably, in a depth of about 20 m another sliding plane was generated. However, additional contingency measures were not required because the slope deformation rate decreased within a few weeks.

Nevertheless, after completion of the tunnel additional inclinometers were installed in the creeping slope and geodetic reading points at the tunnel lining as well. Thus, the monitoring of the slope and deformation measurement of the tunnel will be continued for at least 5 years after setting the tunnel in operation.

The opening of the tunnel was in November 2011. Since then the annual creeping rate of the slope was below 10 mm/a near the surface, which is within the expected range.

6 Conclusions

Some tunnel sections of the city bypass tunnel of Waidhofen an der Ybbs (Austria) have been constructed within a tectonic mélange zone that could be characterized by the so called "block-in-matrix" structure. For the characterization of the rock mass behavior as well as for the determination of shear parameters the findings of Medley, Lindquist, Goodman and others were used. It served as a suitable design tool for the ground properties found in the exploration phase and during tunneling as well.

Furthermore, a creeping slope had to be crossed underground. During construction increased movements were triggered so that a sophisticated monitoring system had to be installed. Structural measures were assessed which enabled safe and successful tunneling in the creeping slope.

References

Brandl H, Dalmatiner J (1988) Brunnenfundierungen von Bauwerken in Hängen (insbesondere Brücken). Straßenforschung Heft 352. Bundesministerium für wirtschaftliche Angelegenheiten, Wien

Button E, Riedmüller G, Schubert W (2002) Shallow Tunneling in a Tectonic Melange: Rock Mass Characterization and Data Interpretation, Mining and Tunnelling—Innovation and Opportunity, Proceedings of the 5th North American Rock Mechanics Symposium, pp 1125–1132

Button E, Riedmüller G, Schubert W, Klima K (2004) Medley E (2003) Tunneling in tectonic melanges—accommodating the impacts of geomechanical complexities and anisotropic rock mass fabrics. Bull Eng Geol Environ 63(2):109–117. doi:10.1007/s10064-003-0220-7

Lindquist ES (1994) The strength and deformation of melange. University of California at Berkeley, Dissertation

Lindquist E S, Goodman R E (1994) Strength and deformation properties of a physical model melange. Rock Mechanics, Models and Measurements, Challenges from Industry, Proceedings of the 1st North American Rock Mechanics Symposium, University of Texas at Austin, 1–3 June 1994. Offprint. A.A.Balkema

Medley E (1994) The Engineering Characterization of Melanges and Similar Block-in-Matrix Rocks (Bimrocks). Department of Civil Engineering, University of California at Berkeley, Dissertation

Moritz B, Grossauer K, Schubert W (2004) Short term prediction of system behaviour of shallow tunnels in heterogeneous ground, Felsbau Rock and Soil Engineering-Journal for Engineering Geology, Geomechanics and Tunneling, vol 22, No. 5

Raymond LA (1984) Classification of mélanges, In: Raymond LA (ed.), Mélanges: their nature, origin, and significance: Geological Society of America, Boulder, Special Publication 228, pp 7–20