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ROCK SLOPE STABILITY DESIGN

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ABSTRACT

This paper explores the numerical analysis used to determine the global stability Factor of Safety (FS) of a rock slope, specifically applied during the design of a railway cut in Central Bohemia. Slope stability is a critical aspect of geotechnical engineering, ensuring the safety and reliability of infrastructure projects. Both traditional and numerical methods were employed in designing protective stabilization measures for the new rock slope. The analysis supports results obtained from the conventional geotechnical approach, specifically using the “limit equilibrium method” along the primary planar shear surface. This method considers the anisotropic behaviour influenced by the strikes and dips of discontinuity surfaces, such as bedding planes and schistosity. The Anisotropic Jointed-Rock Model (AJRM) used in the numerical analysis offers a significant advantage by accounting for the anisotropic behaviour of the rock mass—an aspect often overlooked in standard slope stability methods or when isotropic constitutive models are used in numerical simulations. Mathematical modelling proves beneficial in cases where different constitutive models are required for various geotypes. The combined use of the Jointed-Rock model for the rock mass and the Hardening-Soil model for the quaternary part of the slope within a unified calculation environment provides additional advantages. The primary objective of the static analysis was to design the rock cut within the geological context of metamorphosed pre-Cambrian rocks, specifically gneiss, located in the southern part of Central Bohemia. This comprehensive approach ensures a robust and reliable design, enhancing the safety and stability of the railway cut.

KEYWORDS

Anisotropy, Limit equilibrium method, Rock slope, Slope stability, Anisotropic jointed rock model, Numerical method, Joint, Bedding, Schistosity, Geotechnical approach

INTRODUCTION

This article aims to outline the techniques used for assessing the stability of rock-cut slopes and notches in a specific region of Czechia, often characterized by limited engineering-geological data. It also explores the application of various calculation methodologies to achieve practical solutions. In local engineering practice, the approach to addressing such geotechnical challenges appears to be somewhat "old-fashioned". Insufficient experience and a limited knowledge base frequently result in semi-empirical design methods tailored to the local geological conditions. These methods, rooted in the traditional approaches of geotechnical engineers, can sometimes unnecessarily inflate the costs of implementing new structures, particularly in infrastructure development.

The Czech Republic (and formerly Czechoslovakia) possesses a solid foundation for addressing such challenges, with notable contributions that warrant recognition. Among these, K. Terzaghi's pioneering work stands out, particularly his differentiation of slope and rock deformations based on the mechanical properties of the analysed rock samples, as well as the orientation and inclination of their discontinuities [1]. It is worth emphasizing, though not widely acknowledged within the geotechnical community, that Karl von Terzaghi, often regarded as the "father of soil mechanics", was a native of Prague. This topic was further explored by Q. Záruba, who, in collaboration with V. Mencl, significantly transformed the understanding of the issue [2]. Their work led to the emergence of rock mechanics as a distinct field, diverging from the traditional perspective that viewed geomechanics as a unified subject for both soil and rock mechanics, commonly taught at technical and natural science universities in our country. Quido Záruba was the professor of geotechnics at the Czech(oslovak) Technical University. His work and live between both university and the construction sites are legendary until today. Another publication "*Anchoring in rocks*" by Hobst L. and Zajíc J., represents significant contribution to the field [3]. Another landmark work, particularly influential in our region, is "*Geotechnical Methods for Determining the Stability of Rock Slopes*" by Pavlík J. [4]. This highly regarded and widely utilized text provides an in-depth examination of the limit equilibrium method for analysing of the critical issues such as rockslides and rockfalls. In addition to synthesizing existing knowledge from Western countries, Pavlík delivered a comprehensive guide tailored to practical engineers and introduced pivotal concepts, including the Hoek-Brown rock strength criterion [5] and provided insights into the anisotropic behaviour of rocks [4].

During the 1970s and 1980s, progress in this area of rock mechanics experienced a slight slowdown. The primary focus during this period shifted toward projects associated with the expansion of the Prague underground (metro) system and the initial construction phases of the Prague inner ring road, which began around 1983. Despite this shift, notable advancements were made. New methodologies, including rock classifications such as RQD, RSR, and RMR, as well as triaxial testing of rock samples, were introduced and adapted for construction purposes. Additionally, a specialized local index rock classification, QTS, was developed by Tesař, representing a significant achievement within the Czech geotechnical community. This innovation underscored the region's contributions to advancing geotechnical practices [6].

The period after the so-called Velvet Revolution was progressive in terms of numerical analysis, which became part of the design of major geotechnical works. The system of sequential type of excavation (NATM/NMT/Drill and Blast) was adopted in 1990s. The principles of characterization of the rock mass were rather traditional not taking discontinuities into direct account in numerical modelling. Only in exceptional cases has the use of the Generalized Hoek-Brown rock strength condition [5] been considered in the calculations, and these cases were truly unique in our country in 2000s and 2010s.Foreign developments in the field of rock slope stability have not been fully adopted in mainstream geotechnical practice and this paper focuses on the (apparently) first practical use of the numerical slope stability modelling approach using ARJM in the Czech Republic and its use for the implementation of stability reinforcement works in a rock cut of those days' new section of railway line under construction. Problematics of the anisotropic behaviour of disturbed rock mass is evolving at research and educational institutions (such as The Faculty of Science of the Charles University, Czech Technical University, Brno University of Technology or Technical University of Ostrava). For instance, the work of some students should be acknowledged, such as that of Vilimová [7].

Nowadays, no rock triaxial tests are available in our country. These types of tests are significantly very expensive and no such equipment for common purposes is in use in the Czech Republic nowadays. This poses a challenge for designers, as geological survey works typically only include uniaxial compressive strength (UCS) tests.

METHODS

An overview of the methods used to assess the overall stability of slopes is presented in the following subchapters. The limit equilibrium method is very common and widely applied in the Czech Republic. In contrast, numerical modelling of rock slope global stability using the finite element method combined with the Anisotropic Jointed Rock model is considered exceptional.

Anisotropic Jointed-Rock model

The study material primarily revolves around the anisotropic jointed rock model, which is designed to capture the intricate behaviour of rock masses containing joints with varying orientations and properties. Such models are indispensable for simulating of the mechanical response of jointed rock masses under different loading conditions [8]. Subsequently, numerical simulations are conducted using the FEM approach implemented in Plaxis software [9]. This study implements advanced numerical algorithms to accurately model the interaction between joints and intact rock, as well as the influence of external factors such as stress distributions and boundary conditions. By incorporating anisotropic material properties based on empirical data, the simulations provide valuable insights into the mechanical behaviour of jointed rock masses under different loading scenarios [10].

Plaxis' implementation of the Anisotropic Jointed-Rock Model (AJRM) is not complex as it is in specialized softwares (UDEC, Roscience etc.) [11], but offers fast work of numerical model compiling. Combination of ARJM constitutive model with another constitutive models (such as Hardening-Soil [12]) is very advantageous for design in desired geological conditions. The slope stability was assessed using the φ - c reduction method, which determines the resulting degree of stability based on the ratio of stress-strain state to the ultimate collapse of the structure upon reaching the strength envelope [10]. Part of the design work included studying the state-of-art of the foreign case studies [13; 14; 15; 16].

Estimating of the joints' characteristics were implemented based on Mohr-Coulomb linearization of the Barton - Bandis approach but the geological survey lacked all joint roughness coefficients. Author tried to solve the problem with open-source data from many basic works [17; 18] and from newer sources, such as [19; 20].

The Jointed-Rock material model is essentially an anisotropic variant of the Mohr-Coulomb material model. Its advantage is the ability to describe different material behaviour in different directions. It is therefore a suitable constitutive model for a fractured rock mass where the predominant directions of bedding and schistosity can be defined.

The elastic characteristics of the rock mass can be described by a set of elastic moduli or stiffness constants along each principal axis. For an orthotropic material, this results in a 6x6 stiffness matrix, where the diagonal terms represent the elastic moduli along each principal axis, and the off-diagonal terms represent the coupling effects between different directions.

In the case of Plaxis software, the intact rock is modelled only as an elastic material without including the strength parameters of the continuum. The criterion of the strength behaviour of the jointed materials in this case is given by the strength characteristics of the discontinuities. The strength condition of the discontinuity (according to the local system coordinates) is given by the classical Mohr-Coulomb equation [10]:

$$f_i^c = |\tau_s| + \sigma_n \tan \varphi_i - c \tag{1}$$

The criterion also includes a tension cut-off [10]:

$$f_i^t = \sigma_n - \sigma_{t,i} \left(\sigma_{t,i} \leq c_i \cot\, \varphi_i \right) \tag{2}$$

The general condition for the determination of the functional stiffness matrix is to assume a low value of v_2 so that it does not limitingly approach the singular stiffness matrix. The condition takes the following form [10]:

$$-\sqrt{\frac{(1-v_1)\cdot E_2}{2E_1}} < v_2 < \sqrt{\frac{(1-v_1)\cdot E_2}{2E_1}} \tag{3}$$

Transversal anisotropy is determined by anisotropic stiffness matrix. The elastic behaviour of the ARJM is characterized by 5 independent variables compared to the standard isotropic model (according to the global system coordinates) [21]:

$$\{\sigma'\} = [\mathbf{D}'] \cdot \{\varepsilon'\} \tag{4}$$

In full version, the stiffness matrix leads to following form [21]:

$$[\boldsymbol{D}'] = \begin{bmatrix} E_1 \cdot \frac{1 - n \cdot v_2^2}{(1+v_1)\cdot m} & E_1 \cdot \frac{1 + n \cdot v_2^2}{(1+v_1)\cdot m} & E_1 \cdot \frac{v_2}{m} & 0 & 0 & 0 \\ E_1 \cdot \frac{1 + n \cdot v_2^2}{(1+v_1)\cdot m} & E_1 \cdot \frac{1 - n \cdot v_2^2}{(1+v_1)\cdot m} & E_1 \cdot \frac{v_2}{m} & 0 & 0 & 0 \\ E_1 \cdot \frac{v_2}{m} & E_1 \cdot \frac{v_2}{m} & E_1 \cdot \frac{1 - v_1}{m} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{E_1}{2\cdot(1+v_1)} & 0 & 0 \\ 0 & 0 & 0 & 0 & G_2 & 0 \\ 0 & 0 & 0 & 0 & 0 & G_2 \end{bmatrix} \tag{5}$$

where [21]:

$$\mathrm{n} = \mathrm{E}_1/\mathrm{E}_2$$
$$\mathrm{m} = 1 - \mathrm{v}_1 - 2 \cdot n \cdot v_2^2$$

Transformation to the local coordinate system of the discontinuity is realized by the transformation matrix T [21]:

$$\{\sigma'\} = [\mathbf{T}] \cdot \{\sigma\} \tag{6}$$

In full version, the transformation matrix leads to following form [21]:

$$[T] = \begin{bmatrix} \ell_1{}^2 & m_1{}^2 & \mathrm{n}_1{}^2 & 2\ell_1 m_1 & 2\mathrm{m}_1\mathrm{n}_1 & 2\mathrm{n}_1\ell_1 \\ \ell_2{}^2 & \mathrm{m}_2{}^2 & \mathrm{n}_2{}^2 & 2\ell_2\mathrm{m}_2 & 2\mathrm{m}_2\mathrm{n}_2 & 2\mathrm{n}_2\ell_2 \\ \ell_3{}^2 & \mathrm{m}_3{}^2 & \mathrm{n}_3{}^2 & 2\ell_3\mathrm{m}_3 & 2\mathrm{m}_3\mathrm{n}_3 & 2\mathrm{n}_3\ell_3 \\ \ell_1\ell_2 & m_1 m_2 & \mathrm{n}_1\mathrm{n}_2 & \ell_1\mathrm{m}_2 + \ell_2\mathrm{m}_1 & m_1\mathrm{n}_2 & \mathrm{n}_2\ell_1 \\ \ell_2\ell_3 & m_2 m_3 & \mathrm{n}_2\mathrm{n}_3 & \ell_2\mathrm{m}_3 + \ell_3\mathrm{m}_2 & m_2\mathrm{n}_3 + m_3\mathrm{n}_2 & \mathrm{n}_2\ell_3 + \mathrm{n}_3\ell_2 \\ \ell_3\ell_1 & m_3 m_1 & \mathrm{n}_3\mathrm{n}_1 & \ell_1 m_3 + \ell_3 m_1 & m_1\mathrm{n}_3 & n_3\ell_1 \end{bmatrix} \tag{7}$$

where [21]:

$$\ell_1 = \sin\ \alpha, \qquad m_1 = \cos\ \alpha, \qquad n_1 = 0,$$
$$\ell_2 = \cos\ \beta\cos\ \alpha, \qquad m_2 = -\cos\ \beta\sin\ \alpha, \qquad n_2 = -\sin\ \beta,$$
$$\ell_3 = -\sin\ \beta\cos\ \alpha, \qquad m_3 = \sin\ \beta\sin\ \alpha, \qquad n_3 = -\cos\ \beta.$$

The joint surface is idealized as a saw-tooth-like shape with an angle of roughness α. The plasticity condition function and the plastic potential function are derived based on the idealized joint surface governed by Coulomb's law. The direction of displacement is always considered parallel to the direction of discontinuity and the initial plasticity condition function f is obtained [22]:

$$f\ = \sqrt{(\tau\cos\alpha + \sigma\sin\alpha)^2} + \mu(\sigma\cos\alpha - \tau\sin\alpha) \tag{8}$$

$$g\ = \sqrt{(\tau\cos\alpha + \sigma\sin\alpha)^2} \tag{9}$$

Where [23; 22]:

f plasticity condition function and

g plastic potential function

Planar limit equilibrium method

The initial rough results of the FEM calculations were compared with the simple but effective limit equilibrium method on a planar surface for a quick analysis to obtain indicative results.

The theoretical framework for calculating the global stability of the rock mass, conceptualized in this scenario as a 2D rock wedge with a planar shear surface aligned with the dip of the most critical discontinuity, is relatively straightforward. Many often the more complex shapes of polygonal shear surfaces are simplified to this problem. If we proceed from the classical Coulomb's failure criterion [24] (on the discontinuity), the stability degree of such a block has the following prescription [24]:

$$\tau = c + \sigma \cdot \tan\,\varphi \tag{10}$$

$$\sigma = \frac{N}{A} = \frac{W \cdot \cos\,\psi_p}{A} \tag{11}$$

$$\tau = c + \frac{W \cdot \cos\,\beta}{A} \cdot \tan\,\varphi \tag{12}$$

If we also consider the presence of groundwater at discontinuities, its presence must be considered in the adjustment of the formula [25] (see Fig.1).

$$\mathrm{FS} = \frac{cA + \left(W\cos\,\psi_\mathrm{p} - U - V\sin\,\psi_\mathrm{p}\right) \cdot \tan\,\varphi}{W\sin\,\psi_\mathrm{p} + V\cos\,\psi_\mathrm{p}} \tag{13}$$

where:

L Length of the shear surface per 1 m depth

c Cohesion on the shear surface

W Weight of the analyzed block

U Resultant hydrostatic pressure perpendicular to the shear surface

V Horizontal resultant pressure on the vertical tensile crack (if present)

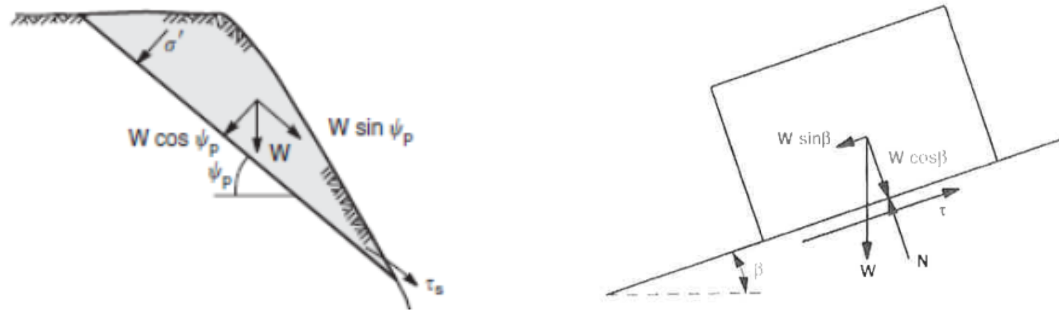

Fig. 1 - Simple model of the planar equilibrium method [25]

The use of this method is limited. If the calculation is used to determine the global stability analysis of a rock slope, it considers very unfavourable boundary conditions, which rarely occur in nature, such as continuous fracture defining a rock block from the base of the slope through the entire rock mass. These circumstances can be influenced by specifying the roughness of the shear surface, defining the angle of asperities and other adjustments of the shear strength criterion (dilatancy angle and so on). Planar limit equilibrium method provides an efficient tool of determining the local FS for a well-defined and mapped risk block. In terms of global stability, the method gives only indicative values of global FS.

Wedge limit equilibrium method

Another failure mode used in the kinematic method is block failure. This type of failure occurs in slopes that incline diagonally away from the slope, with the loss of stability of the wedge occurring along the intersection of two planes of dip and discontinuity directions (see: Fig.2). This method was used only marginally in the presented case, as a supplement to the other calculations.

For example, the wedge is formed by two continuous, planar discontinuities and the line of intersection of these two planes, which protrude at the base of the rock face. The direction of the line of intersection and the direction of slip of the face are approximately the same. If we compare the slope of the line of intersection of about 50-55° with the angle of φi, (and the friction angle of the discontinuities is lower than 50° [18]), it follows that the line of intersection falls more steeply than the friction angle. These conditions satisfy the kinematic requirements for wedge failure.

The point of penetration is represented by the point where the two principal circles of the planes intersect, and the orientation of the line is defined by its direction α_1 and slope ψ_1 [25].

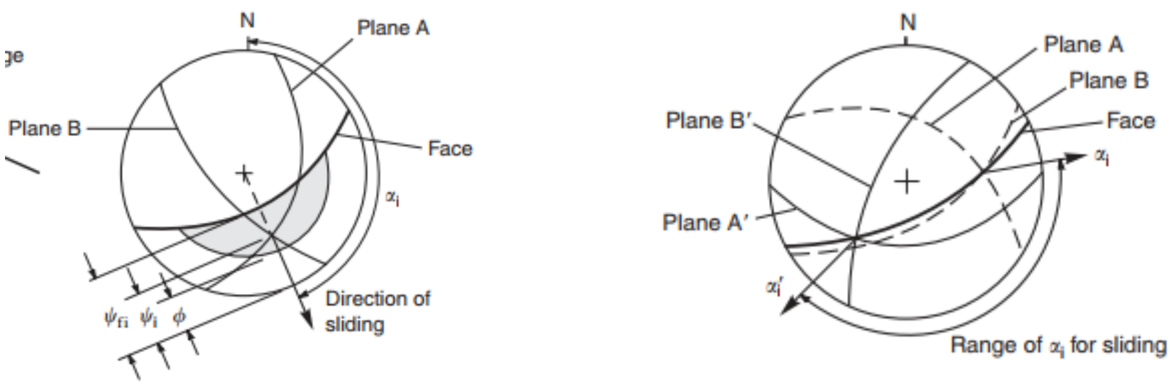

Fig. 2 - Direction of intersection of sliding planes (left side) and range of sliding planes (right side) [25]

The degree of stability of such a wedge is given by the assumption that the resisting forces are shaped only by the angle of internal friction at the discontinuity [25]:

$$FS = \frac{(R_A + R_B)\tan\varphi}{W sin\ \psi_i} \tag{14}$$

where [25]:

Ra, Rb Normal forces (reactions) provided by planes A and B

W The component of weight acting downward along the line of intersection is determined by W * sin ψ

The effects of the two resistance forces add up [25]:

$$R_{\mathrm{A}} + R_{\mathrm{B}} = \frac{W\cos\ \psi_{\mathrm{i}}\sin\ \beta}{\sin\ (\xi/2)} \tag{15}$$

FS obtained by the wedge equilibrium method has the following prescription [25]:

$$FS = \frac{sin\,\beta}{sin\ (\xi/2)} \cdot \frac{tan\,\varphi}{tan\,\psi_i} \tag{16}$$

CASE STUDY

Historical Background and Area Description

The area of interest is situated in the southern part of Central Bohemia near city of Benešov in the Czech Republic (see Fig.3).

The original railway cut was constructed between 1869 and 1871. Unfortunately, extensive invasive vegetation has since covered the slopes, making detailed surveying difficult. Despite this, observations of the slope's condition indicate that the excavation during construction was primarily done manually, rather than through blasting. The overall slope gradient ranges from approximately 40° to 60° from the horizontal.

Fig. 3 - Location of the railway cut on map of the Czech Republic (49.6258847N, 14.6044422E WGS84

The first 150 meters were already reconstructed during the works in 2011. However, during the earthworks in this section, there were incidents of rock blocks spontaneously sliding out.

The design for stabilizing the subsequent sections was developed based upon the experience gained during the construction of the initial section. The solution of the design was also clarified after discussions with the investor's geotechnical supervisor.

This design incorporated rock cut stability assessments using various procedures, as detailed in the following sections of the article. The widening of the notch is directly influenced by the need to accommodate the width of the railway line. A key constraint in the design process was to adhere strictly to the boundaries of the railway land property. The original design was conceived optimistically, without the benefit of hindsight gained during the reconstruction of the initial stage of works.

The structure falls under geotechnical category 3, featuring a height difference of up to 23.5 meters. Additionally, the general angle of dip of the discontinuities poses significant challenges to the slope of the cut (see Fig.4 and Fig.5).

The solution has been revised from securing the slope face with concrete blocks to implementing an anchored slope system. This system includes a rock net combined with an anti-erosion geogrid made of polypropylene fibres, reinforced with a grid of rock bolt anchors.

Geological conditions

Rock mass is contained of irregular granite insertions of several decimetres to metres in thickness, and there have also been sightings of mica schists. In terms of strength, the rock mass is primarily composed of rocks with a medium strength classification of R3. However, in the upper sections of the notch, rocks with lower strength classifications of R4 and occasionally very low strength classifications of R5 can be anticipated [26]. Classification of rock strength is given by the Czech national standard no. ČSN 73 6133 [27].

Quaternary soils

The Quaternary cover primarily comprises deluvial, fluvial, and anthropogenic deposits. Along the route of the proposed extension of the notch, the total thickness of the Quaternary cover varies significantly. It is typically around 1 meter thick, but in areas at the base of slopes or near small intermittent watercourses, it can reach thicknesses of 2-4 meters.[26]

Fig. 4 - Excavation works on the rock cut slope (Author)

Bedrock

The pre-Quaternary bedrock in the area of interest comprises metamorphosed rocks of Precambrian age, specifically paragneiss. These rocks are predominantly medium-grained, occasionally exhibiting clear directional features, and are characterized by biotitic and sillimanite-biotitic composition. In some areas, they may be partially strongly migmatitized, with quartz veins often aligned along predisposed tectonic structures [26].

Fig. 5 - View on the refurbished first part of the from the year 2011 (Author)

The rocks exhibit evidence of repeated tectonic movements along the fault structure. Recent exploration works and archival drillings have revealed manifestations of tectonics. Along the fault zones, fluid circulation and mineral deposition occurred during tectonic and post-tectonic periods, respectively. Due to the inclination of the foliation, rock falls and outcrops are expected to predominantly occur on the left (eastern) side of the notch.

In addition to the direct inspection of the terrain and the problematic site, two tectonograms were accessible during the preliminary survey of two rock outcrops (see Fig.6). These tectonograms provided at least an indicative understanding of the main strikes and dip angles of the discontinuity surfaces. The following geotechnical parameters were evident from the point-by-point survey:

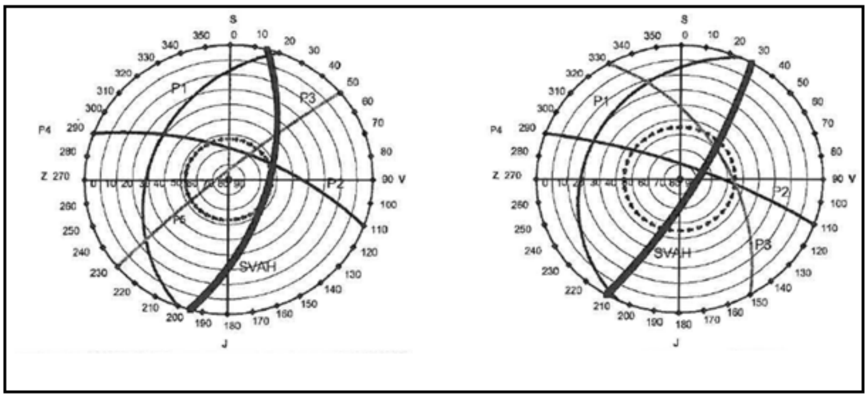

Fig. 6 - Tectonograms of the area DB1 and DB2 [26]

Discontinuity systems

In addition to assessing the stability of the local spatial wedge (3D rock wedge), two-dimensional cross-sections perpendicular to the axis of the line were also examined, corresponding to the tectonogram with a strike and dip of the slope: 45° (225°)/90°. The P2 discontinuity roughly parallels the geometry of the cross-section, with an almost vertical inclination (70°-80°). Due to its strike and dip, it was not feasible to incorporate it into the 2D sections. The P3 discontinuity area exhibits different strike and dip properties in each case, making it challenging to determine its predominant geometric characteristics. Among the discontinuity systems, the P1 discontinuity seemed to be the most unfavourable, as it closely aligns with the slope of the notch and slopes outward in a disadvantageous manner.

PRACTICAL PART

Having sorted out the necessary and unnecessary data supplied by the engineering-geological survey developer and supplemented them with our own study to obtain input parameters for all types of calculations, the characteristic values of the geotype of interest were determined (see Tab. 1) and a variety of assessment methods were employed to evaluate both the global and local stability of the slope (see Tab. 2).

Tab. 1 - Gneiss characteristics given by the geological survey

Parameter	Value Range
Strike and dip of the slope geometry	315°/35°-52,5° (dip varies)
Strike and dip of discontinuity system P1	290°-295°/30°-40° (DB-1 and DB-2)
Strike and dip of discontinuity system P2	20°/70°-80° (DB-1 and DB-2 tectonograms)
Strike and dip of discontinuity system P3	320°/85° (according to DB-1 tectonogram); 60°/60° (according to DB-2 tectonogram)
Characteristic fracture interval	150 mm
Character of dominant surfaces	Undulating rough
Peak friction angle at discontinuity	φp = 48°- 65° (p: peak)
Basic friction angle at discontinuity	φb = 27° (b: basic)
Initial shear strength of intact rock	ci = 46-47 kPa (c: cohesion)
Friction angle of intact rock	φi = 71°-72°
Jv	19,5-26,5/m³
RQD	28%-50%
RMR	43-44
SMR	34-38
Parameter m DB1	0.4778 (Hoek-Brown)
Parameter m DB2	0.4344 (Hoek-Brown)
Parameter s DB1	0.000086 (Hoek-Brown)
Parameter s DB2	0.000082 (Hoek-Brown)
Parameter A DB1	0.3405 (Hoek-Brown)
Parameter A DB2	0.3314 (Hoek-Brown)
Parameter B DB1	0.7066 (Hoek-Brown)
Parameter B DB2	0.7063 (Hoek-Brown)
Parameter T DB1	-0.00019 (Hoek-Brown)
Parameter T DB2	-0.00019 (Hoek-Brown)

Tab. 2 - Overview of the types of calculations used during design works

Number	Method	Desciprion
1	Limit equilibrium method of the rock wedge along the main planar shear surface – global stability	Analyzes the global stability of the slope by considering the main planar shear surface using the limit equilibrium method
2	Limit equilibrium method of the 3D spatial rock wedge – local stability	Assesses the local stability of the slope by analyzing the 3D spatial rock wedge using the limit equilibrium method
3	Verification of the global slope stability by the ϕ–c reduction method in Plaxis software	Validates global slope stability using the ϕ–c reduction method in Plaxis geotechnical software. This method utilizes Hardening-Soil and Jointed-Rock models.

Stability calculations using the planar limit equilibrium method

The global FS (Factor of Safety) was validated by analysing the distribution of forces along the planar shear surface in a 2D cross-section. The dip angle of the shear surface was determined based on the P1 system, specifically at 40°.

In some areas, optimal design wasn't feasible using this method. The extent of the cut was limited by land ownership at the top of the slope. Consequently, the steepest slope in certain sections was set at an inclination of 52.5° (see Fig.7).

The inclination of the original slope from the year 1871 (see Fig.7): α = 42°. This corresponds exactly to the discontinuity system P1.

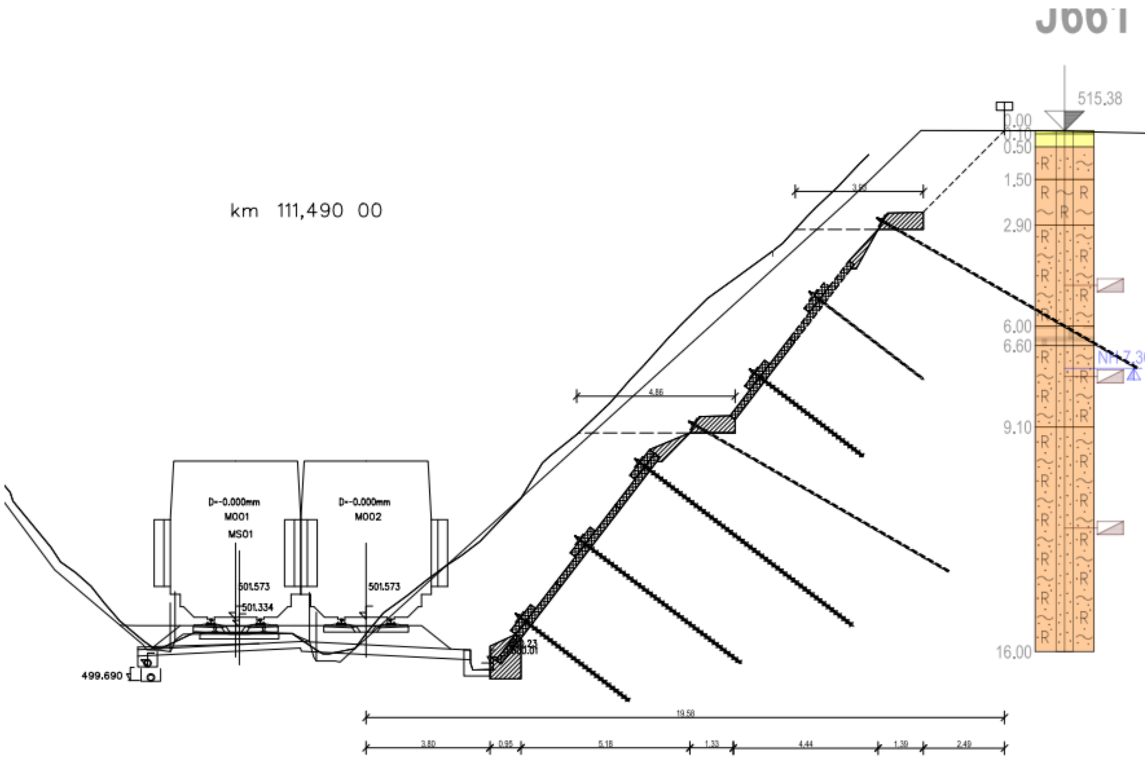

Fig. 7 - Rock slope stabilization protection at km 111,490 alignment

Geotechnical inputs

The following geotechnical parameters were selected based on the geological survey evaluation (see Tab.3):

Tab. 3 - Geotechnical characteristics of the MC criterion used in calculation

Material	γ [kN/m^3]	c_{joint} [kPa]	ϕ_{joint} [°]
R2-3/M4 (weathered gneiss)	25.0	1	27

Given the limited availability of detailed data, conservative parameters were selected for the shear strength of the material along the discontinuity surface. Additionally, the potential presence of a vertical tensile crack in the slope crown was taken into account, considering its location within the Quaternary cover.

3D spatial rock wedge – local stability of the wedge

Rock wedge geometry

For the aim to determine the local stability of the rock wedge (3D spatial rock wedge), a combination of P1 and P2 systems; and a combination of P1 and P3 systems were considered.

The rock wedge geometry was considered along the planar surfaces of the above-mentioned discontinuity systems. The program addresses the stability of a rock wedge between two surfaces (planes) and moving along the intersection of these planes. The slope of this intersection must be significantly greater than the angle of internal friction along the dividing planes, with the dips of both dividing surfaces oriented towards the intersection.

Geotechnical inputs

The following geotechnical parameters were selected based on the geological survey evaluation (see Tab.4):

Tab. 4 - Geotechnical characteristics of the MC criterion used in calculation

Material	γ [kN/m^3]	$c_{joint1;2;3}$ [kPa]	$\phi_{joint1;2;3}$ [°]
R2-3/M4 (weathered gneiss)	25.0	0	27

φ - c reduction using AJRM in the FEM approach

To validate the conventional computational methods, modelling of the same cross-section was conducted using the finite element method. The slope stability was assessed using the ϕ - c reduction method, which determines the resulting degree of stability based on the ratio of stress-strain state to the ultimate collapse of the structure upon reaching the strength envelope. The static calculation of the slope was carried out by PLAXIS 2D software. The AJRM (Anisotropic Jointed-Rock Model) constitutive model was primarily selected to align with the dips of discontinuities. Additionally, the hyperbolic constitutive model Hardening-Soil was employed to characterize the behaviour of the Quaternary surface and the heavily weathered gneiss geotype [12].

Constitutive models used in FEM calculations

The constitutive Hardening-Soil model [12] is a modified isotropic elastic-plastic model describing the elastic deformation behaviour of materials with the introduction of a nonlinear (hyperbolic) dependence of the deviatoric stress $q = |\sigma 1 - \sigma 3|$ on the axial strain $\varepsilon 1$ [12]. This hyperbolic dependence was already described in the 1960s [28] and is part of the relatively widely used Duncan-Chang model [29].

In general, it is a nonlinear elastic-plastic model with 2 types of isotropic stiffening described by plasticity theory (see Tab. 5):

Tab. 5 - Two types of isotropic stiffening [12]

Number	Description
1	Shear hardening in non-associated plasticity
2	Compressive strength in associated plasticity

The Jointed-Rock material model is essentially an anisotropic variant of the Mohr-Coulomb material model. Its advantage is its ability to describe different material behaviour in different directions. It is therefore a suitable constitutive model for a fractured rock mass where the predominant directions of bedding and schistosity can be defined [10; 21; 30].

Geotechnical inputs of the hardening-soil model

The recommended approach for Hardening – Soil model input parameters were considered (see Tab.5):

Tab. 6 - HS model approach [12]

Description	Value
E_{oed} can be approximately defined from E_{def} using the relation over the coefficient β	β (function of Poisson's ratio)
E50 is the stiffness modulus at 50% shear strength depletion in semi-solid materials	E50 (1.0 - 1.1) * E_{oed} can be considered
Eur triaxial unloading stiffness modulus	(3 - 5) * E50 depending on material type

The suitability of employing the Hardening-Soil constitutive relation, particularly for unloading tasks, stems from the nonlinearity of the deviatoric stress-strain behaviour [12]. Therefore, despite the approximation of these parameters, a more suitable deformation behaviour of semi-rock materials can be anticipated compared to the basic linear elasto-plastic Mohr-Coulomb model. While it's advisable to correlate the parameters based on laboratory test results to fully utilize this advanced constitutive relationship, precise deformation characteristics are not essential for determining slope stability (see Tab.6).

Tab. 7 - HS model inputs of upper layers of the FEM model

Material	γ [kN/m^3]	E_{oed} [MPa]	E_{50} [MPa]	E_{ur} [MPa]	c' [kPa]	ϕ' [°]	ν [-]	$ν_{ur}$ [-]	ψ [°]	m [-]
Quarternary layers	18.2	12.8	14.1	42.3	18	27	0.35	0.20	0	0.5
Heavily weathered gneiss	20.7	16.0	17.6	52.9	15	27	0.35	0.20	2	0.5

Geotechnical inputs of the AJR model

In the implemented version of the Jointed-Rock model in Plaxis, the environment operates as a "laminar model." This means that it decreases the strength properties of the environment at a specified inclination of the main discontinuity surface. It distinguishes between two types of elastic deformation moduli (see Tab.8) [10] and the input characteristics are shown (see Tab.9):

Tab. 8 - Description of the input values of the ARJM

Symbol	Description
E_1	Modulus of elasticity of the intact rock (parallel to the dip of discontinuity)
E_2	Modulus of elasticity perpendicular to the dip of discontinuity
$ν_1$	Intact rock Poisson's ratio
$ν_2$	Poisson's ratio perpendicular to the dip of discontinuity
ϕ'	Friction angle of the discontinuities
c'	Cohesion of the discontinuities

Tab. 9 - Geotechnical characteristics of the AJRM used in calculation

Material	γ [kN/m^3]	E_{def1} [MPa]	$ν_1$ [-]	E_{def2} [MPa]	$ν_2$ [-]	c'_{joint} [kPa]	$ϕ'_{joint}$ [°]	dip [°]
Semi wheathered gneiss	21.5	35.0	0.32	25.0	0.10	1.0	27.0	40.0
Gneiss	25.0	400.0	0.22	100.0	0.10	1.0	27.0	40.0

The rock bolts were simulated using a node-to-node anchor element, a 1D component that transfers the necessary axial force between two nodes of the FEM network. This element is elastic and has a single input parameter, the normal stiffness EA. Standard methods were used to assess the anchors, checking their normal and shear capacity of the steel cross section, as well as verifying the pullout of the grouted root from the solid rock material.

RESULTS AND DISCUSSION

Particular results of the Limit Equilibrium Method

However, practically defining shear parameters on a fracture presents a significant engineering challenge, particularly when laboratory tests fail to establish the strength envelope along the predisposed shear surface.

The initial shear strength (cohesion) of a schistosity system is primarily influenced by the filling of discontinuities. Nonetheless, its magnitude significantly impacts the calculations, thereby influencing the final value of the Factor of Safety (FS). Tab. 10 presents the results of the calculation based on the specified shear properties.

Tab. 10 - Improvement of the slope stability due to proposed refurbishment using 2D equilibrium method

Alignment	Method	Shear Surface	FS Initial	FS after reconstruction	Percetage Improvement
111.49	LEM 2D	Planar Surface	0.86	1.41	64%

The wedge method calculation, utilized to ascertain the global Factor of Safety (FS), provides only indicative results and should be approached with caution. In practical design applications, this methodology exhibits several shortcomings in the current scenario:

The calculation presumes a singular geotype, meaning that different geological interfaces cannot be adequately accounted for. In our case, the Quaternary layer extends to a depth of approximately 1.50 meters. Furthermore, the degree of weathering varies with greater depth of the cut. This phenomenon cannot be accurately captured by this method.

Despite utilizing the geotechnical parameters provided by the survey geologist and incorporating additional assumptions into the calculation, the existing slope was determined to be unstable (FS = 0.86). However, over the course of the 147 years since its construction, the cut has not exhibited signs of global instability.

The improvement of FS before and after the proposed construction can be expressed as a percentage change in condition, and in this case, it was 64 % (FS, hypothetical as built = 1.64).

In order to meet the requirements of the Czech standard ČSN 73 6301 'Design of railway lines' [31], the stability is sufficient even without adjustments of the input parameters (FS, post-construction = 1.41 > FS, required = 1.15).

Particular results of the limit equilibrium method of the 3D rock wedge

Tab. 11 presents the results of the spatial rock wedges' stabilities calculated based on the specified shear properties. The geometry of each type of wedges can be seen in Fig.8.

Tab. 11 - Improvement of the slope stability due to proposed refurbishment using 3D equilibrium method of the rock wedge

Rock Wedge	Method	Shear Surface	FS Initial	FS after reconstruction	Percetage Improvement
DB 1	LEM 3D	Planar Surface	0.68	2.89	325%
DB 2	LEM 3D	Planar Surface	1.33	2.41	81%

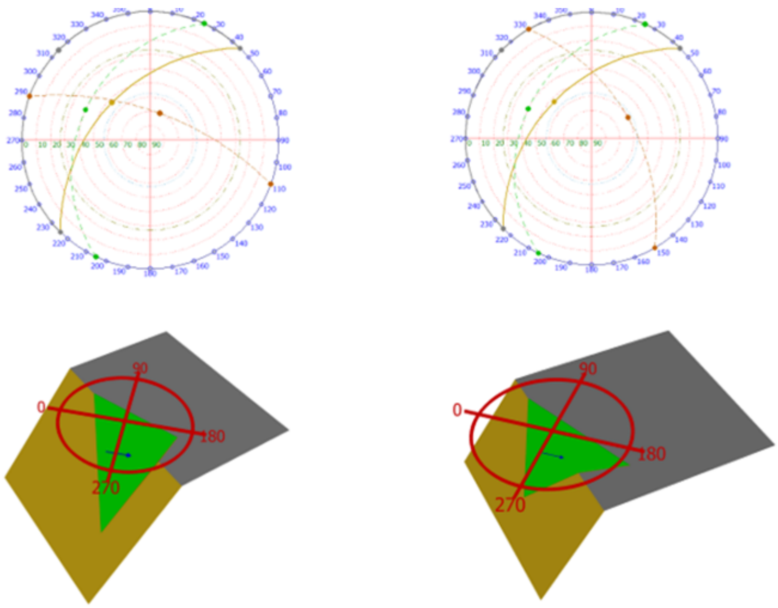

Fig. 8 - Two types of the rock block wedge according to in situ conditions (Author)

The calculation of the local stability of a rock block is effective for determining the size and spacing of the required anchor force. For the rock cut, a procedure was selected to verify the capacity of one anchor element for a specific rock block.

This method is highly suitable and straightforward for standard geotechnical design tasks involving the determination of anchor spacing and verification of the necessary anchor force.

Particular results of the FEM φ - c reduction method

Modern geotechnical software enables rapid construction of verification models for the environment. However, selecting the right parameters and sequencing the calculation process are both crucial. Unfortunately, employing an Anisotropic Jointed Rock Model in Finite Element Method (FEM) modelling frequently results in numerical instabilities compared to the conventional use of an isotropic model.

The calculations yielded a relatively close agreement (8.5% difference) with the classical limit equilibrium method, as shown in Tab. 10 and Tab. 12, confirming their accuracy.

Tab. 12 - Improvement of the slope stability due to proposed refurbishment using 2D FEM φ-c reduction method

Alignment	Method	Shear Surface	FS Initial	FS after reconstruction	Percetage Improvement
111.49	FEM	Planar Surface	0.85	1.53	68%

The stability assessment of the original unsecured slope yielded an intriguing result. While there was agreement, it was determined that the slope is unstable with a factor of safety (FS) of 0.85 (see Tab. 12). This finding, however, does not align with reality, as discussed in Chapter "Particular results of the Limit Equilibrium Method".

The global improvement of FS before and after the proposed construction can be expressed as a percentage change in condition, and in this case, it was 68 % (FS, hypothetical as built = 1.68).

In order to meet the requirements of the Czech standard CSN 73 6301 'Design of railway lines' [31], the stability is sufficient even without adjustments of the input parameters (FS, post-construction = 1.53 > FS, required = 1.15).

The behaviour of the rock mass model corresponds with fundamental conclusions according to [10] and [32]. In case of the isotropic Mohr – Coulomb variant of this problem, which had been modelled as well, deformations of the rock mass occurred in different values and shapes.

The numerical model results, presented in Figures 9-11, illustrate the behaviour of the calculations based on the characteristics of the slope's discontinuity system P1. Unlike the isotropic model, the primary shear surface of the slope aligns with the dip direction of the specified discontinuities and adopts a predominantly planar form. Furthermore, due to anisotropy, the model reveals plastic zones surrounding the thrust elements (anchors) as well as tension-induced plastic zones (plastic points) perpendicular to the main discontinuity system's slope (See Fig. 10). Notably, the plastic behaviour is observed in steeper portions of the slope compared to the specified dip angle of 40 degrees. The final verification of the overall expected behaviour of the numerical model is the depiction of the normal stress in the anchoring elements of the slope stabilization system, which decreases along the length of the anchors towards the rock mass. This decrease is caused by the friction between the anchors and the surrounding rock (See Fig. 11).

It is important to mention that the load-distributing components (plates) of the anchors were designed as prefabricated concrete elements. However, these components were ultimately not been installed during the construction phase (See Fig. 9).

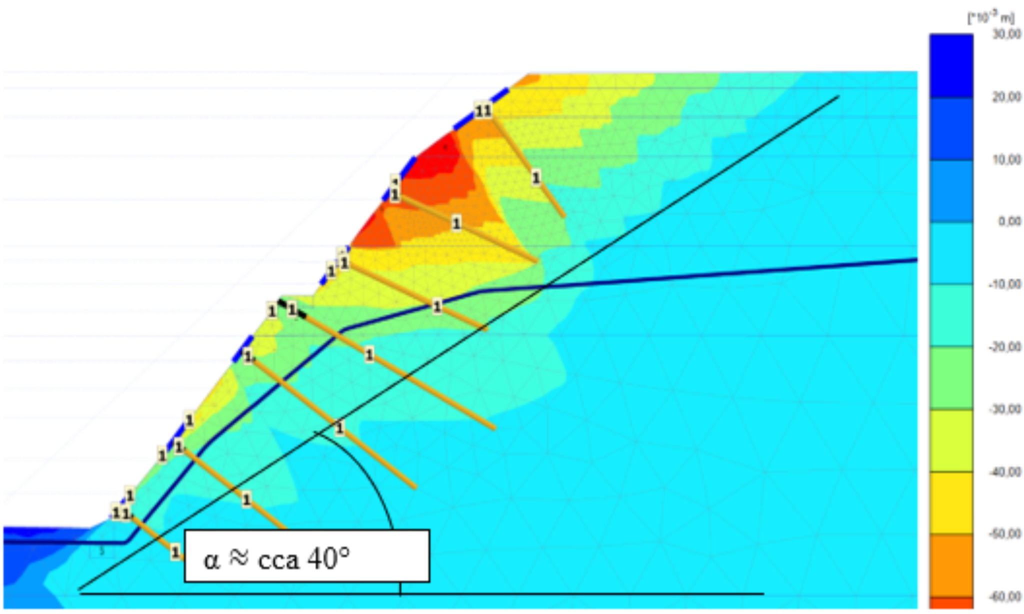

Fig. 9 - horizontal displacements in rock mass

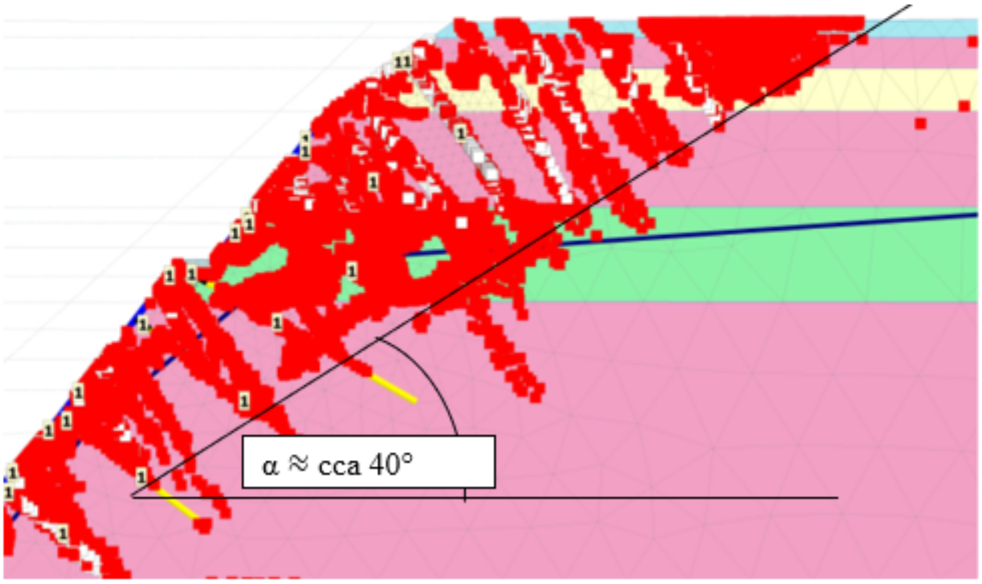

Fig. 10 - Plastic points

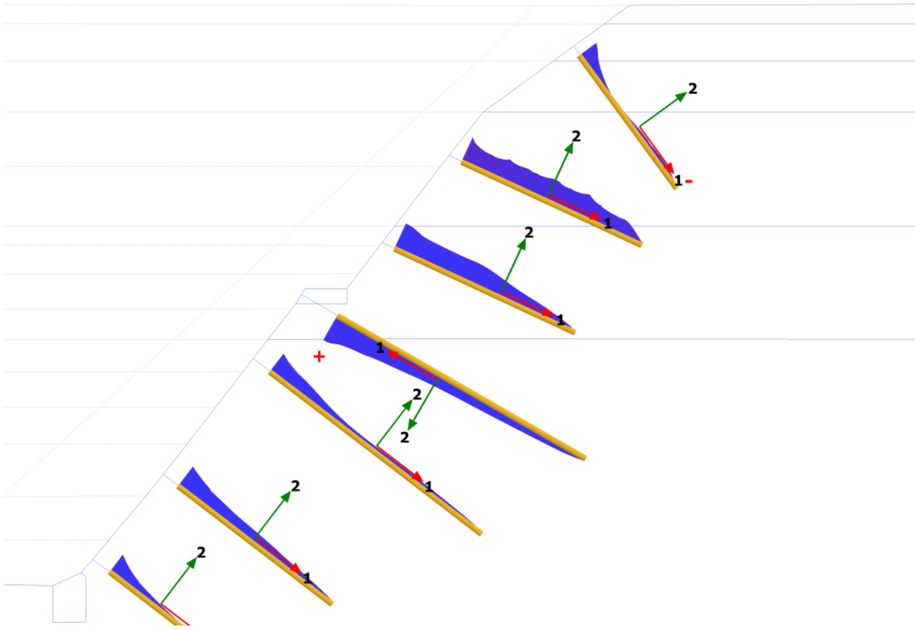
Fig. 11 - Trends of the normal forces along the designed rock bolts

On the other hand, the numerical model employing the Anisotropic Jointed-Rock Model (ARJM) had certain limitations. One major issue was the convergence of the strength reduction analysis, which did not yield satisfactory results when using arc-length control in the calculations. Arc-length control is a solution technique for nonlinear finite element method calculations. Instead of solving for a specific load increment, the method computes the solution along a defined arc length. However, in this case, arc-length control often identified a local plastic point within the model rather than capturing the global failure mechanism of the entire slope. Consequently, when utilizing the ARJM approach in numerical modeling, it is advisable to avoid using arc-length control in such scenarios. Minor issue is a need to generate very fine element mesh. When considering such complicated constitutive model, the rough mesh is inadequate to reach to results. Calculation is somewhat time consuming.

FINAL CONCLUSIONS

The use of the Anisotropic Jointed-Rock Model (ARJM) in numerical analysis for standard design of rock slope protection and stability assessment is not commonly practiced in our country in comparison with the approach of China and Western countries. Model has validated assumptions made by the classical wedge method, that is still the main approach to solve such geotechnical problems in our country. The combination of defining individual geotypes with different constitutive models has proven advantageous in numerical rock slope stability analysis.

When evaluating the global stability factor of the analysed slope using an isotropic Mohr-Coulomb model or applying the Hoek-Brown strength criterion to represent rock mass as a continuum, the resulting global stability factors tend to yield unrealistically high values (FS ≈ 4.00). The proposed methodology (using ARJM) offers a more accurate approximation, bringing the global stability factor closer to realistic and representative values for engineering and geotechnical analyses.

As mentioned in the results section, the value of the global stability factor for the rock slope, considering the predisposed discontinuity planes, aligns with the results obtained using the limit equilibrium method on a planar shear surface (FS = 1.53 vs. FS = 1.41).

The presented paper likely describes the first application of this approach in the Czech Republic. The author is not aware of any other practical case where this methodology was employed for a real geotechnical design that was subsequently implemented. Successful construction implementation of the calculated design (year 2013) resulted into ceremonial opening of the new railway line in 2020, and the entire project was officially completed in June 2023. The Nazdice rock slope excavation construction implemented the results of presented method published in this article. The backbone railway line to Austria is now full operational.

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