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Design of underground structures and analysis of self-support capacity

Projektiranje podzemnih objektov in analiza samonosilnosti hribine

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Abstract

The complicated rock structures and the stability of surrounding rocks of the underground powerhouse are key ground mechanical challenges for hydropower projects.

In this paper, an example of contributing self-support capacity of rock mass to evaluate optimised support for long-term usage of structure is given. It describes importance of investigations in the initial in situ stress distribution, rock mechanical and geological properties, engineering rock mass classifications by different methods, numerical modelling, comparison of tools for stability and support analysis and proper stability control for rock excavation and support.

The results show that after underground excavations in hard rock, detailed analysis of measures to investigate deformation and self-supporting capacity creation is useful and a cost-saving procedure.

Key words: Hydropower tunnel, Power house cavern, Self-support capacity, Underground Excavation, Support installation

Povzetek

Kompleksna struktura tal in stabilnost hribine pri gradnji velike podzemne strojnice so ključni inženirski izzivi pri projektiranju hidroelektrarn. V tem prispevku je prikazan primer, ki prispeva k razumenvanju karakteristik samonosilnosti hribine, ocenjevanje optimalnih podpornih ukrepov za dolgoročno obratovanje objekta. Prikazan je pomen preliminarnih raziskav o porazdelitvi napetosti v izkopu, mehanskih in geoloških lastnostih kamnin, geomehanske klasifikacije po različnih inženirskih metodah, numeričnem modeliranju in primerjavi orodij za analizo stabilnosti in podporanja, ustrezen monitoring stabilnosti pri gradnji.

Rezultati kažejo, da je pri podzemnih gradnjah v trdnih kamninah, podrobna analiza ukrepov za spremljanje in preiskovanje deformacij ter lastnosti samonosilnosti hribine, koristen proces, ki predstavlja ekonomičnost uporabe podpornih ukrepov.

Ključne besede: Dovodni tunel, podzemna strojnica, samonosilnost hribine, podzemne gradnje, vgradnja podporja

Introduction

The demand of hydropower projects has been increasing significantly since last few years in countries with hydropower potential. At the same time, standards for environmentally and economically favourable design of power plants have been set at higher levels. As a consequence, power plant underground structures with cross-sectional area more than 1500 m^2 are under construction with an increasing number. In this paper, some reliance concerning important decisive technical solutions for the design of power caverns and intake tunnels is introduced. Images (Figure 1) may provide a first idea concerning required size of underground excavations in hard rock for required power capacity and support systems, which depend on understanding of right rock mass behaviour [1,2].

In the design of underground excavation projects, the main approach is to adjust and minimise rock support to achieve limit state of actual rock mass strength and acceptable displacements.

With technological development in observing deformations, material analysing, and modelling during the design stage, understanding of self-support capacity to achieve acceptable strength has become much easier, which is required to investigate deeper in material behaviour. This method and a more precise interpretation provide a set of parameters for geotechnical design of underground structure and define conditions for precise support installation, which means economical optimisation of used supporting elements. When excavating large underground structures, the cost control is even more important. In large hydropower projects, there are usually excavated complex massive underground structures such as powerhouses, waterway tunnels and shafts. These structures are designed to transport water under a high variable pressure of up to 100 bars, and potential of erosion and dynamic loads are much higher than known from transportation infrastructure (Figure 2a and 2b). Leakage of water has to be controlled with detailed construction and grouting works; hard rock strength could be significantly improved with grout injection in empty spaces.

Main geotechnical and geological challenges allied to the construction of underground

Caverns are stability problems for long-term uninterrupted energy generation. In addition, all technical and economic conditions have to be considered when evaluating suitable

Figure 1: *A view on existing cofferdam (50 m high), valley to be filled and underground structures in the hydropower project of a future main dam (330 m high).*

Figure 2 *(a and b): Waterway tunnel portals and foundation construction of connection tunnels.*

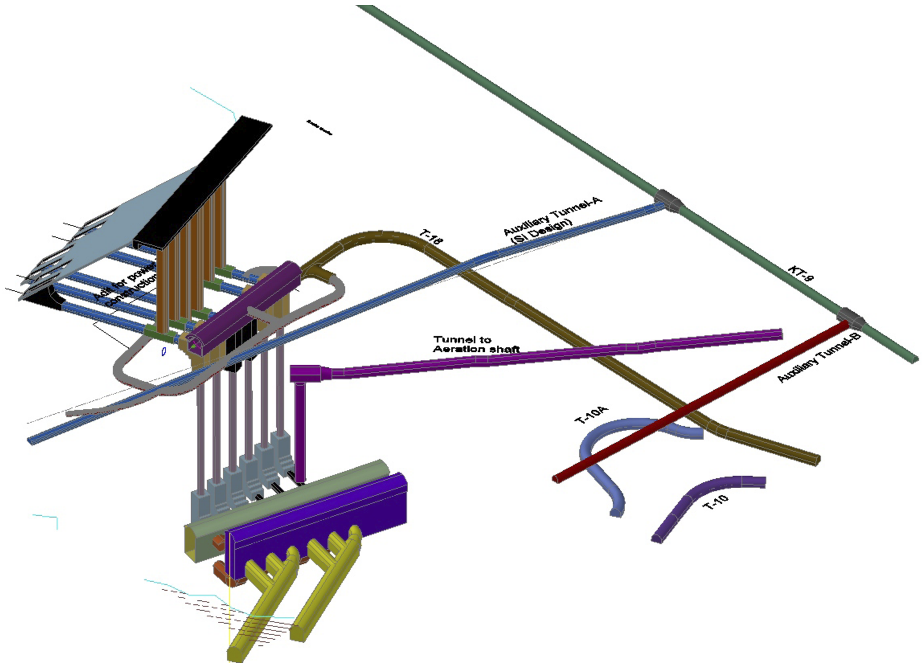

Figure 3: *3D model of underground powerhouse cavern and connection tunnels.*

location and orientation of the underground power cavern (Figure 3).

With correct selection of excavation technology and planning sequences, there are many projects where self-supporting capacity of the rock mass is fully used. Practice shows some cases without concrete lining or some cases used only in distressed sections where a tunnel crosses highly fractured or fault zones.

The underground power house with a cross-section of 22 m × 50 m is taken as an example in this paper.

Positioning of the power house cavern is selected taking into examination the geometry of structural discontinuities in the rock mass. For improvement of self-stability of the crown and side walls, also to minimise issues with blasting over breaks, the excavation direction of underground cavern and waterway tunnels should be intersecting strike orientation of principal discontinuities. Usually some compromises have to be made in the structural design when it is not possible to align it in the perfect direction to achieve minimal influence by discontinuities. According to the project, most suitable positioning of power house cavern has to take consideration of in situ stress measurement and stress directions (Figure 4). It is important to analyse range of *s*H (maximum horizontal principle stress), which can cause tensional cracks on the upper part of excavation along the length.

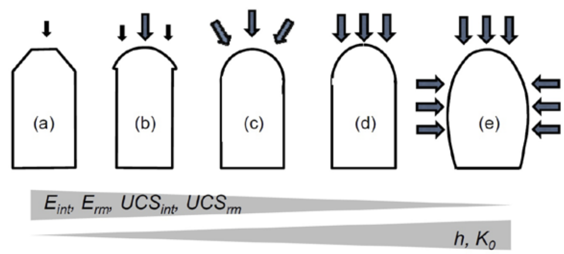

Figure 4: *Different cavern shapes and their applicability according to rock mass properties and stress conditions [3].*

Stability control

In hard rock underground excavation, failures occur on roof and side walls in heavily jointed rock or when the material is exposed to high in situ stress. Wedge or rock blocks failure is the most common type of failure in hard rock excavation where three or more structural intersecting planes form a block with excavation boundary as the fourth plane (Hammett and Hoek, 1981). When wedges are free to fall, the whole stability of cavern will decrease rapidly, causing further progressive rock fall, which leads to reduced rock mass strength. This effect will cause the other wedges to be destabilised, and the failure process will continue until natural shape stage is reached. Orientation of discontinuities, the shape of the cavern and condition of the structural features, i.e. friction and weathering, will influence the structurally controlled instability.

For unfavourable geological conditions, engineering measures have been proven to be successful:

- concrete layer for water tunnels combined with anchors;
- concrete replacement in faults combined with anchor cables;
- systematic installation of anchor cables in the roof and sidewalls of main underground powerhouse and
- careful treatment of open joints with grouting works.

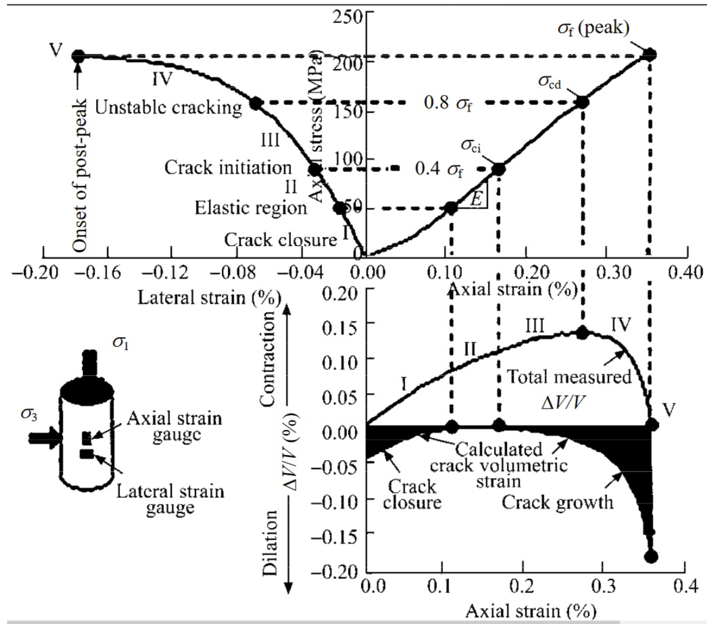

Figure 5: *Stress–strain curves obtained from a single uniaxial test [4].*

It is also important to investigate the failure mechanism process of brittle rock using the laboratory scale. Detailed monitoring of rock sample using compression tests shows crack development behaviour. Stress–strain curve of a brittle rock sample could be divided into four stages:

- crack closure phase;
- linear elasticity phase;
- stable crack growth phase and
- unstable crack growth phase

Figure 5 shows the definitions of crack initiation stress (σ_{ci}), crack damage stress (σ_{cd}) and peak strength (σ_f). Initiation stress of cracking σci is defined as the stress level marking onset of dilation and the beginning of phase III where the stress–strain curve is deviated from linear-elastic behaviour, indicating the development and growth of stable cracks.

Cracks in this stage are referred as stable cracks until an increase in load is required to raise further cracking, and time-dependent crack growth does not occur under a constant load. The crack damage stress, σ_{cd}, is defined as the stress level marking the beginning of phase IV, where the reversal of volumetric strain curve occurs, indicating that the dilation due to the formation and growth of cracks exceeds the elastic compression of the rock resulting from increasing load. Loading a sample with stress more than σ_{cd} results in time-dependent increases in damage to the material, leading to an ultimate sample failure under a sustained constant load. The crack damage stress, therefore, is believed to be indicative of the long-term strength of the rock [4].

The presence of macrocracks in rock mass leads to discontinuous material behaviour, and then under the effect of high pressure, free fragments start to flow, pores form and swell causes slips and rotating particles between blocks (Figure 6). In this failure mechanism, the theory for continuum calculation does not match and the granular medium theory could be used to calculate the dilatancy of the large-scale rock mass geometric model, which means that it is

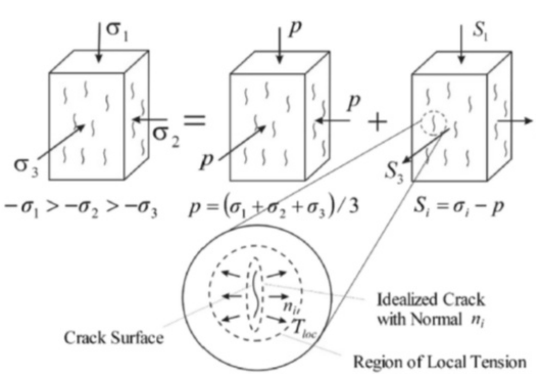

Figure 6: *Relationship between local stress and applied stress [5].*

necessary to use particle flow code (PFC) to demonstrate deformations in the design stage. These phenomena of hard rocks have to be considered in the selection of correct design methodology, especially in projects with large-span underground excavations [5–7].

Self-support capacity

Hard rock mass itself around underground openings has a certain self-support capacity (Figure 7). After tunnelling or larger underground excavation, surrounding rock goes in deformation along the unloading direction, and in the tangential direction, there occurs squeezing of material under the load forces. The inner surrounding rock mass starts to interconnect, and rock mass structure begins to degenerate after an unloading zone is created in the surrounding rock. In the unloaded zone, a self-supporting zone is formed because rock blocks occlude. After displacements, it takes all the load from itself and the above rock mass is integrated with structure [8]. The created zone with new mechanical properties allows the surrounding rock to stabilise in short time. The creation of self-supporting zone is a phenomenon of self-regulation of stress that keeps resistance of deformation in rock mass (Figure 8). Before installation of support systems, it is useful and considerable to control the estimated decompression period correlated with the pre-deformation. The stiffness of the inner zone determines where both curves on the graph intersect. At this stage, equilibrium and compatibility are also completed.

The boundary of self-supporting zone can be determined according to the stress path analysis procedure [9,10].

Figure 7: *Powerhouse cavern under excavation of lower part.*

Design procedure

Complex ground conditions and limit state may affect the stability of caverns and tunnels by the geometry of joints and density of fractures in the surrounded rock mass. With the development of technology and research in numerical analysis of material deformability, comparisons of the calculation obtained by different numerical methods such as finite element method, discrete element method and indirect boundary element method and in case of fractured rock mass also by PFC for better understanding of stress distribution and deformation effects on joins around excavations are numerically studied [5]. In practice, comparisons that indicate the validity of the stress analyses around excavation openings have already been performed.

The influence of model geometry (Figure 9) on each numerical method has to be analysed. Groundwater is one of main issues in underground excavations, where numerical simulation is necessary to estimate the amount of water inflow. A proper scale has high importance for correct analysis in such models. With comparison of analysis, it was found that if the distance between excavation boundary and outer model boundary is too small, then the simulat-

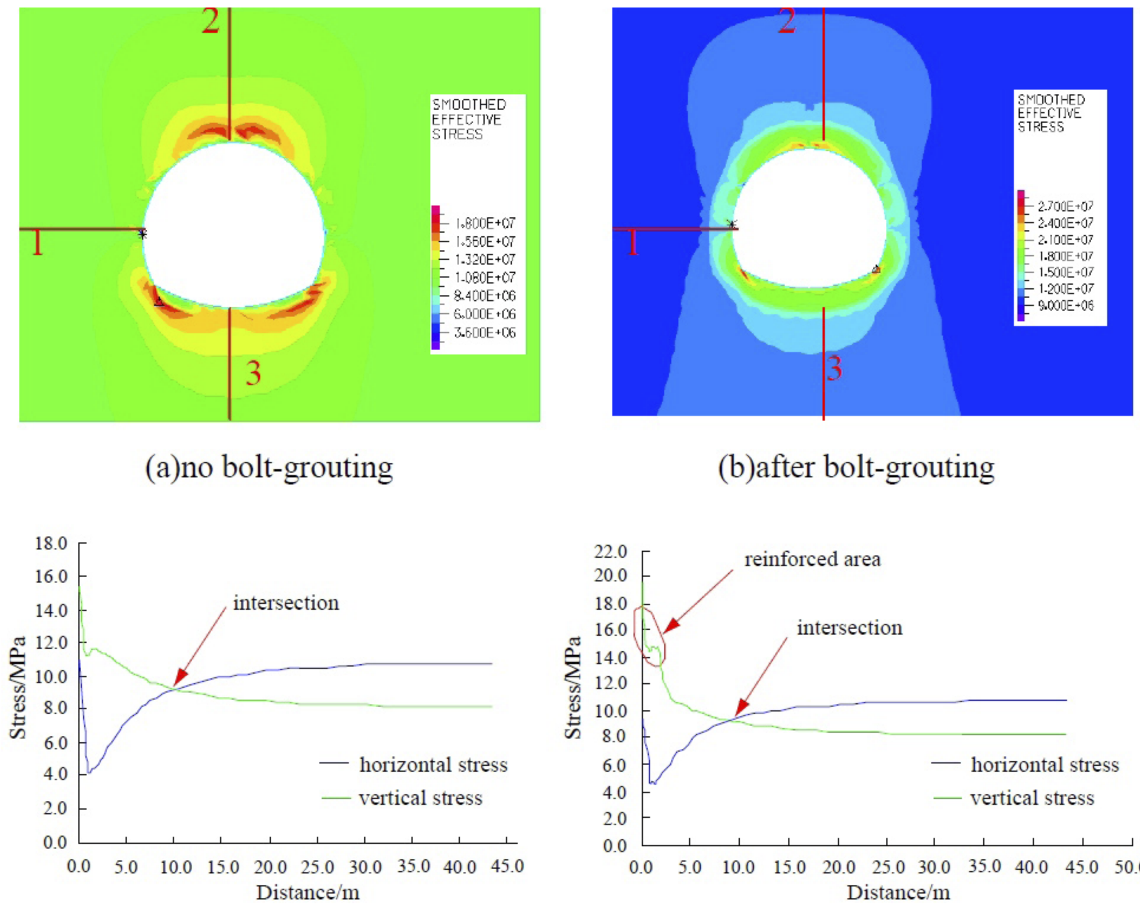

Figure 8: *Stress and distance graph (radial excentre). Example of stress distribution and curves of horizontal stress and vertical stress in self-support arch (before and after rock bolt installation and grouting); horizontal stress is higher than vertical stress, which is applied on side walls of tunnel.*

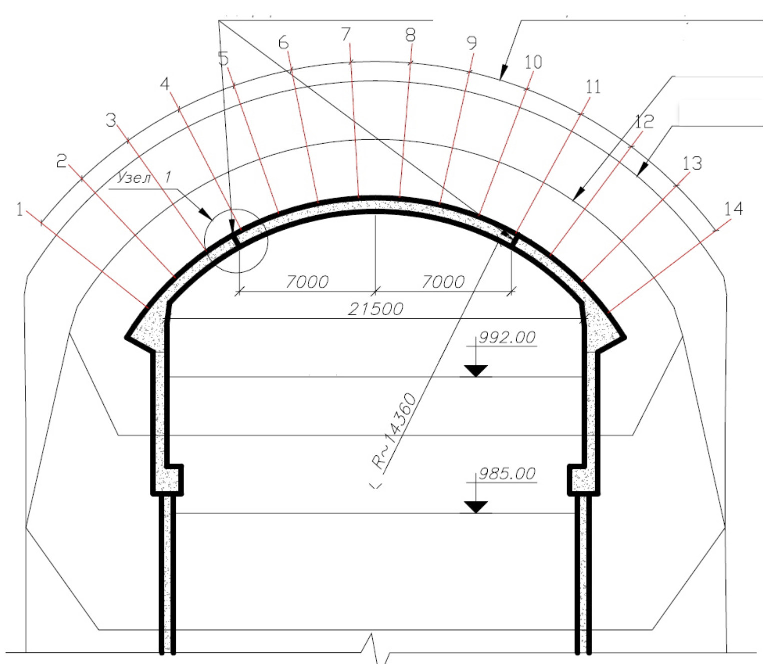

Figure 9: *Cross-section of cavern.*

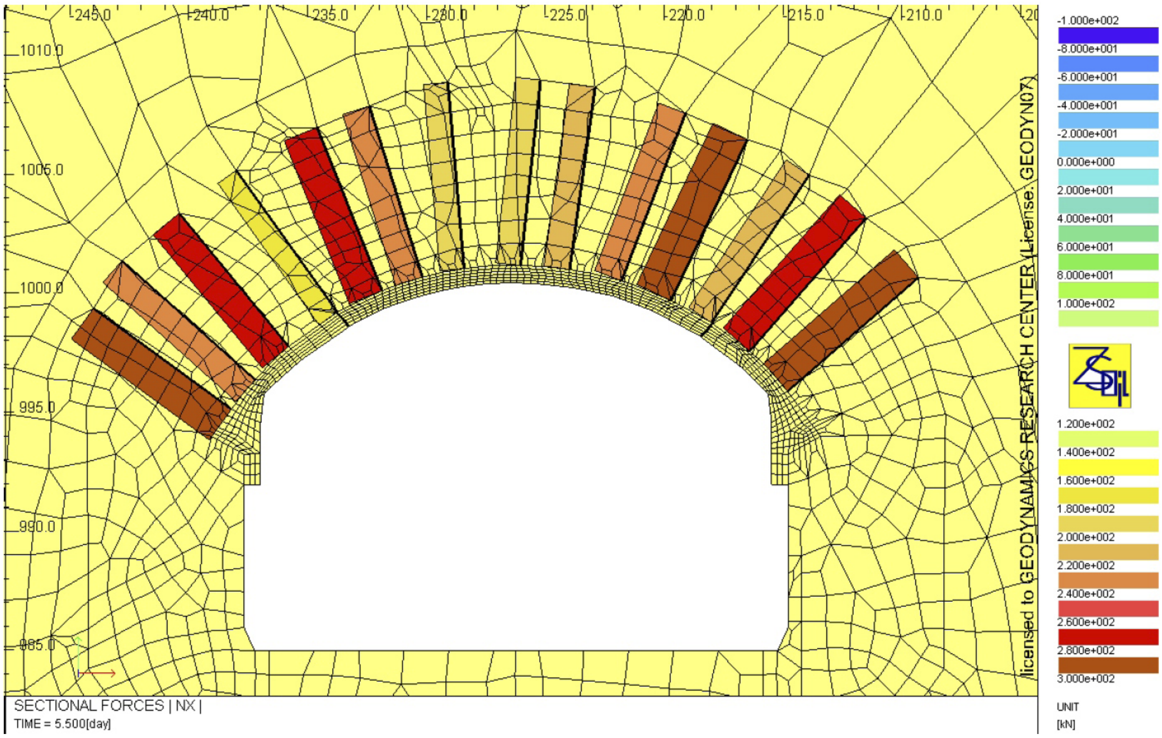

Figure 10: *Distribution of loads (kN) in anchors around arch zone.*

ed water flow rate into the excavation is overestimated.

Similar incorrect results of displacements could be calculated in case of not optimised number of elements and also line assembling boundary or geometry. In case of very large models, it may be difficult to process because of huge operation that has to be done using computer hardware that an average user does not have [11–13].

Estimating stress distribution in self-support zone

Long-term prediction of behaviour of underground structures is a complicated procedure; therefore, a reliable constitutive model is needed, which can interpret the measurement of viscous phenomenon. Because of scale effect, the rock rheological property measured on samples in the laboratory cannot be extrapolated directly to field scale. It is necessary to correlate numerical results with in situ measurement over a long period of time.

In this example, the need for additional anchorage of the arch in large powerhouse cavern, after primary support was completed for temporary protection, is analysed.

For this purpose, a series of numerical calculations of the stress-deformed state of the array distributed around cavern was performed, with the determination of the forces in anchors of the arch. Calculations were performed for siltstone and sandstone rock with GSI = 45, with different capacities of the unloading zone of the massif above the arch.

In addition, it was assumed that the excavation development of cavern is conducted by a mining excavation method of drill and blast in particular sequences.

Analysis of changes in the stress state of concrete lining in the zone of arch was done on the basis of a comparison of the stressed state of the concrete and the loads in the anchors, for example while leaving the existing primary support anchors and when they are completely replaced with new anchors.

It was found that the types of anchors and the distance between them generally correspond to the project requirements. According to the example draft, primary support contains passive rod anchor AGG 8 m long on a grid 2.0 m × 2.0 m. The diameter of the rods is 36 mm, and design-bearing capacity of such type of anchors is 30 tons.

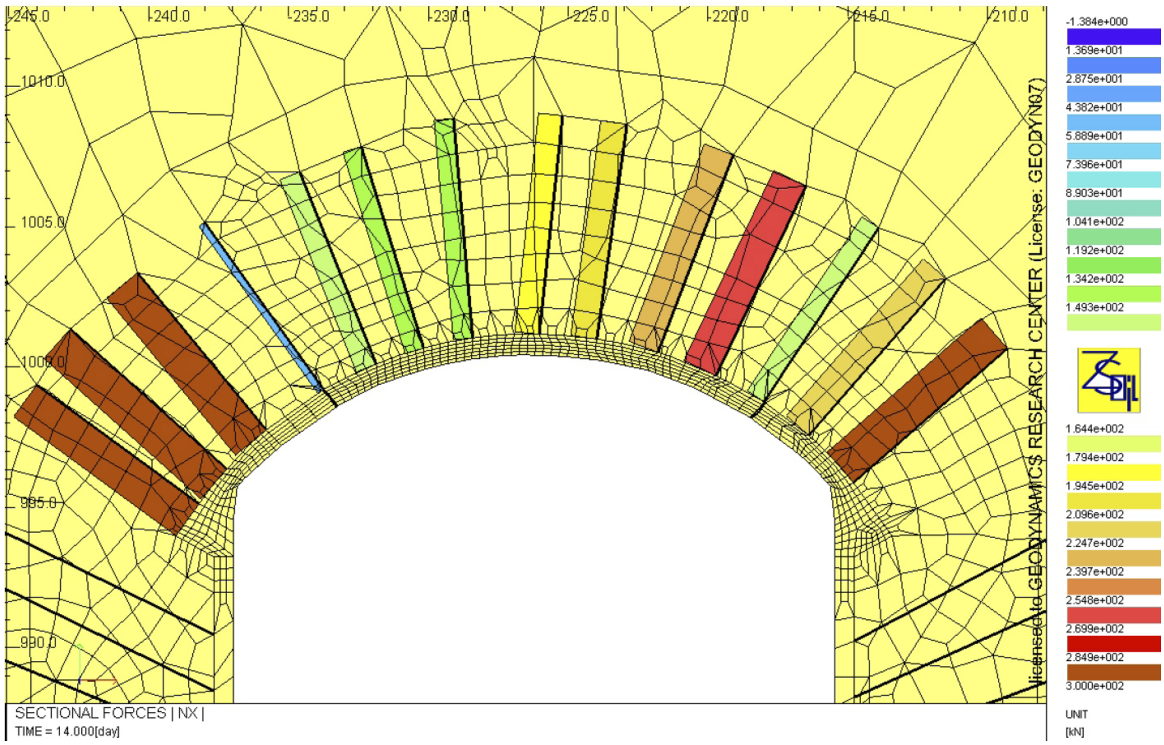

Figure 11: *Anchors' load (kN) decreases after initial displacements.*

The rigid arch is made up of monolithic reinforced concrete with a thickness of 0.7 m, having two deformation seams. Concrete design strength for compression is 13 MPa and for tensile is -0.98 MPa.

The structure was modelled by thin interlayers of a weak material, with a strain modulus E = 2000 MPa. The design of the expansion joint according to the project is shown in Figure 9.

For long-term usage of the structure, the visual inspection shows no significant damage of support concrete layer, which leads to reasonable doubts about the future stability of the structure. During the blasting works and excavation sequences for deepening the power house, some passive anchors in the upper part of side wall were also damaged. Main defects of the structure were visible at joints of side wall and lower part of the arch, practically at the areas where it joins the walls of the turbine hall.

Consideration for two optional support solutions was studied for reinforcement propagation of the zone under higher stress to a depth of 3 m and 6 m, paying attention to calculations in the zone of siltstones.

With design revision noted, the anchor of the arch in the zone of siltstones began to yield already at the initial stage of the cavern deepening excavation works (985.0 m).

This behaviour was observed for both a 3-metre and a 6-metre deep zone. With further excavation in depth of the power house, the change in stresses of the anchors is multidirectional: in one part of the anchors, there is an increase in stress, and in the other part of the anchors, there is a decrease in stress. In some stages of construction distribution of forces in the anchors in the two parts of the zone considered unloading is shown in Figure 10.

The distribution of forces in the anchors is explained by the distribution of radial movements of the arch directed to its geometric centre; in the 3-m zone of relaxation in siltstones and sandstones, it can be seen that largest displacements of 8 and 2 cm occur near its abutment to side walls where the anchors experience maximum loads and failures.

Displacements in arch that occurred during the development of the maximum compressive stresses in the arch were at the junction of the arch with the side wall and, with its external side, near the expansion joints, which were designed for the purpose of achievement of self-support capacity of rock mass (Figure 11).

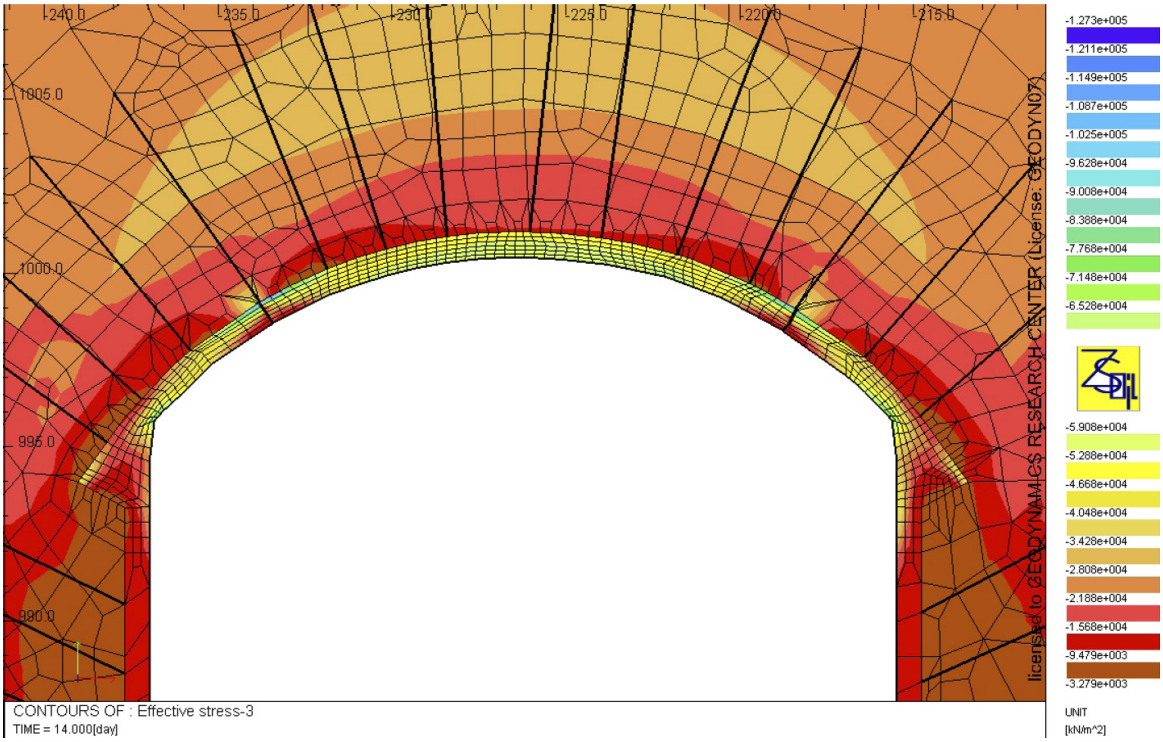

Figure 12: *Distribution of compressive stress (kN/m^2) in the arch.*

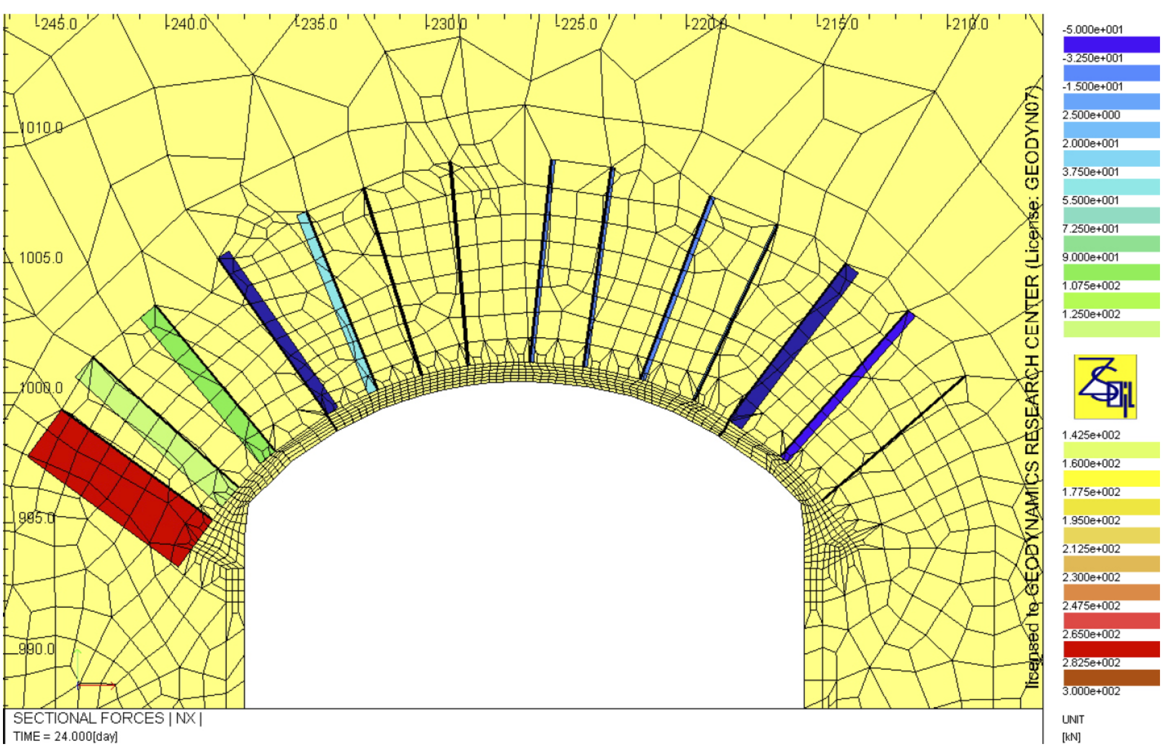

Figure 13: *Load distribution (kN) in case of installation of additional anchors.*

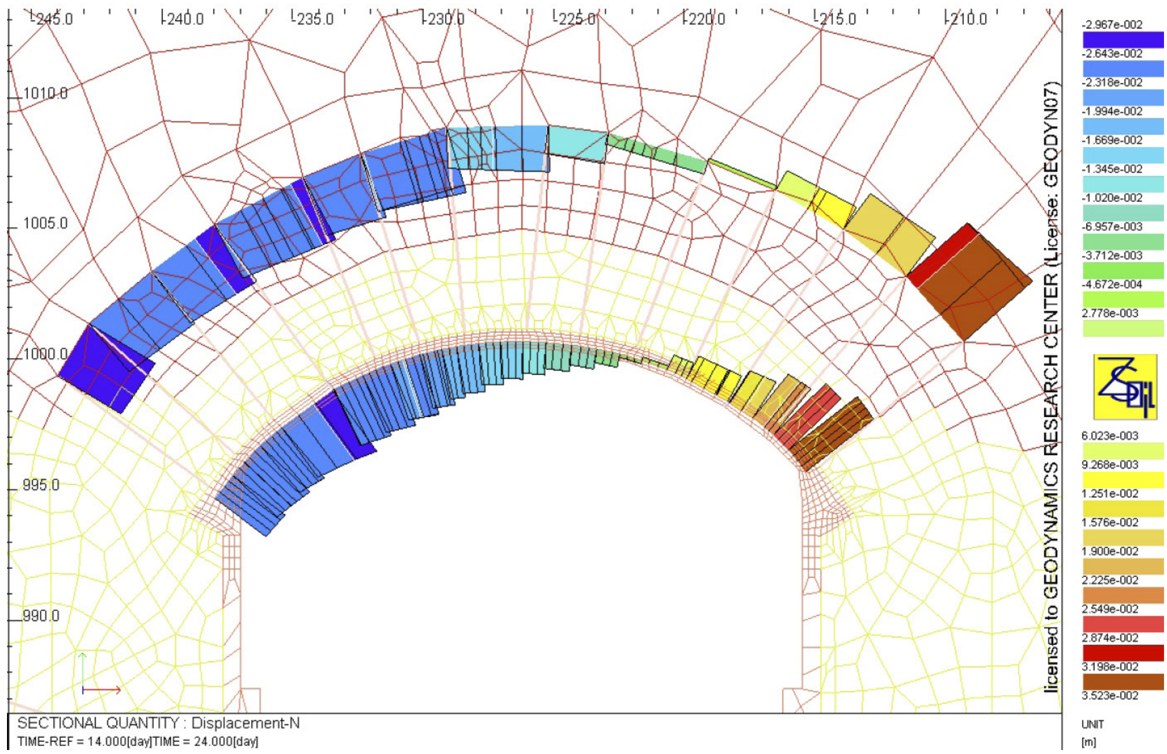

Figure 14: *Radial displacement (m) of the arch array.*

Incensement of stress is a consequence of deformability mechanism of the arch while closing of two expansion joints.

The maximum stresses on the inner surface of the arch develop exactly where the support side wall is acting like a pillar and causing destruction of the protective layer of crown excavation (Figure 12).

To examine the need for additional anchorage of the arch, it was necessary to reanalyse the old design and all changes in the anchors and stresses in the structure for optimised completion of underground structure. These changes consequently considered two options – upgrade of anchorage system or its full replacement before completion of structure.

Calculations show that for the final stage of excavation, the loads on anchors installed in the arch zone do not increase (Figure 13).

Reduction of forces in anchors is caused by a relative convergence of their ends, which leads to a reduction in the previously existing tension. This phenomenon is observed in some anchors of the arched part of the power house. Illustration of this is represented in Figure 14, which shows the radial displacement of the array near both ends of the anchors that occurred during the completion.

Calculations show that maximum tensile forces in additionally installed anchors in the final stage of work would not exceed 60 kN, which is significantly less than the maximal load of 300 kN.

Deformations in the period of completion of excavation at the bottom of cavern differ both qualitatively and quantitatively from those obtained with the preservation of the anchorage.

Analysis of the data presented shows that the additional installations due to displacements of surrounding rock mass do not significantly affect the stability of the arch.

Comparison of stresses in the arch for the final stage of deepening of powerhouse shows that no additional support will be needed because of self-supporting behaviour of rock mass after initial deformation have occurred and primary support distributed all load on side wall part of excavation.

Conclusions

Structural geology and underground infrastructure projects are complex and behave three dimensionally in nature. Rock support design procedure cannot be performed in a systematic manner without taking into account geometric and geological/geomechanical complexities. When performing designs, the selection of a suitable analysis tool, methodology and data judgement should be done and the design should be well understood; otherwise, costly mistakes regularly occur during construction, because of various influence factors outlined in this paper.
Ideal surrounding rock self-supporting arch should be of curvilinear shape, consisting of compound curves in arch and, when necessary, also in invert. This will allow the smooth distribution of stress around caverns or tunnels.
The pressure located at the vault zone and bottom cannot be acceptable as an effective connection.
This kind of distribution often leads to a decrease in surrounding rock self-supporting capacity. Correct selection of support systems, which have to be compared with few alternative solutions to achieve economic and technical best solution, is the right way to improve the stress concentration in exposed zones, and consequently, the capacity of surrounding rock self-supporting arch increases significantly.

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