*Proceedings of the 21st ICSMGE, Vienna, Austria, 14 – 19 June 2026. Pistrol, Adam & Schweiger (eds.)*
*Published by: ÖGG, Austrian Society for Geomechanics, Salzburg, Austria, ISBN 978-3-9503898-4-5*
*https://doi.org/10.53243/ICSMGE2026-1875*

# Rock slope stabilization of highly weathered granite on Vydrica site in Bratislava

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ABSTRACT: A large development project is being constructed below a hill where Bratislava castle is located. Major rock slope stabilization measures were designed and executed to allow for construction of planned buildings, embedded in the terrain with a significantly rising character. The total slope is around 60 m high and 450 m long. Stabilizing measures using a combination of permanent strand anchors (up to 19 strands) and permanent tension micropiles were executed on lower 25 – 35 m of the slope. Geologically, the castle hill is covered by surface layers of silts and debris, under which a highly weathered granite rock can be found. During site investigations works multiple joint systems were identified in the rock mass as well as highly altered zones consisting of clayey material of soft consistency. Additionally, a major tectonic fault is crossing through the site. All this constitutes complex geotechnical conditions for design and made a selection of characteristic parameters for rock slope characterization challenging. Observation method during the execution was adopted due to a wide range of possible strength parameters which were identified during the site investigation. Designed permanent anchors holding back reinforced concrete beams were proof loaded up to 4200 kN. Monitoring system as an integral part of observational method consists of automated geodetical survey of surface points, built-in inclinometers and extensometers as well as anchor load cells. Additionally, selected anchors were equipped with optical fibers to monitor a long-term behavior. All these measures have helped to successfully finish the first project phase.

KEYWORDS: Rock slope stabilization, ground anchors, monitoring, endoscopy, observational method.

## 1 INTRODUCTION

The Vydrica project is located in the center of the capital city of Slovakia, under the Bratislava castle. In the area from the waterfront towards the castle, three buildings in the first phase and seven buildings in the second phase, which will be linked with the street of the former Oeser's Row, were built into the terrain, with a significantly rising character.

In the first phase of the project, for a length of approximately 170 m, a cut of approximately 20-35.5 m was created in the terrain.

In the second phase of the project, for a length of approximately 280 m, a cut of 17-30 m was created in the terrain.

According to Eurocode 7, the cut into a rock hill of this extent falls into the 3rd geotechnical category.

Permanent stabilization measures were developed to meet structural, durability, and aesthetic requirements. A geotechnical structure employing small-diameter drilling and shotcrete was suggested. For stability, a system of anchor elements combined with reinforced concrete structures was designed to maintain the slope's integrity.

## 2 GEOLOGY

Geologically, the castle hill is formed by surface layers of sediments and debris (about 1-2 m), under which the bedrock is formed by a highly weathered granite rock that gradually falls towards the Danube River. The lower levels close to the river are formed by layers of gravel. From the geological survey, after the evaluation of core drilling from the upper level of the cut, the rock parameters were determined for the calculations.

### 2.1 *Parameters for the first phase*

Initial parameters of the rock with friction angle of 35.5° and cohesion of 363 kPa were determined by Geologists. Rock quality designation was 25-50 %, which means poor quality. During further site investigations works multiple joint systems were identified in the rock mass as well as highly altered zones consisting of clayey material of soft consistency. Additionally, a major tectonic fault is crossing through the site. All this constitutes complex geotechnical conditions for design and required an update of rock strength parameters. In the first approach, the modelling of the rock mass with homogeneous approach was abandoned and rock joints with cohesion of 15 kPa were introduced to the model.

### 2.2 *Endoscopy of boreholes*

An additional geological survey was conducted during the execution of stabilization works. Consequently, borehole endoscopy could have been performed not only on designated core drillings, but also within anchor boreholes prior to anchor installation.

Within the implemented inclinometers at the highest point of the cut were scanned (not only vertical, but also horizontal inclinometers).

Endoscopy of the vertical borehole (V-1) verified the presence of an open fracture at a depth of 16.5 m as well as a high density of fractures in the interval 14.0-17.0 m below the borehole collar.

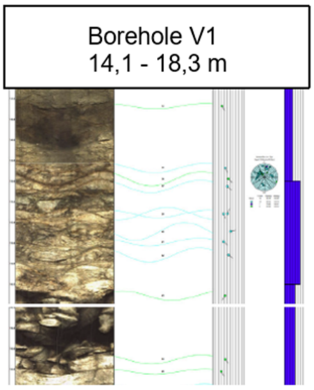

Figure 1. Endoscopy of borehole V1.

The endoscopic survey confirmed not only the presence of joints in the bedrock, but also their omnidirectionality. In addition to the joint system, the endoscopy revealed relatively large void spaces deep within the massif, at depths of approximately 45 to 60 m, primarily in subhorizontal boreholes.

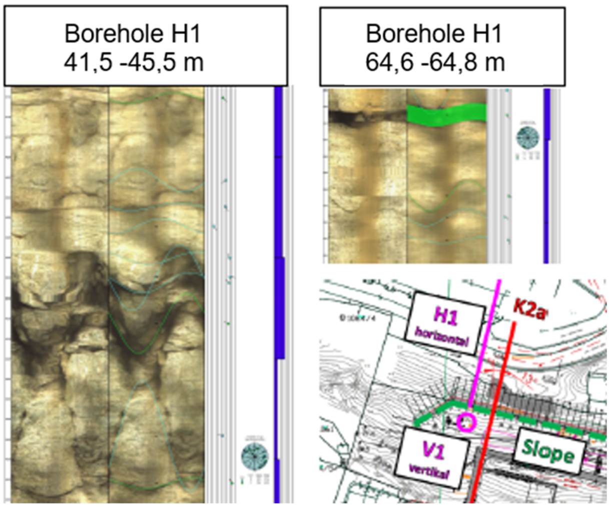

Figure 2. Endoscopy of horizontal borehole H1.

Due to this fact new parameters had to be set in the stability calculations. Previous analyses indicated that the critical factor for the stability of castle hill is the joint system. The system of fractures in the bedrock creates blocks of varying sizes that must be secured by soil nails and anchors to prevent sliding.

## 3 ROCK SLOPE STABILIZATION OF THE 1ST PHASE

### 3.1 *First design of stabilization in the 1st phase*

The planned Oeser's Street (Figure 3) divides the excavation into two parts. The upper part of the cut reaches a height of approximately 4 to 17 m, while the lower part reaches approximately 16 to 18 m. The upper section (above street level) is secured by permanent passive bars combined with temporary shotcrete and a permanent reinforced concrete wall. At designed levels, the cut is stabilized by permanent strand anchors supported by reinforced concrete beams.

In the lower part, permanent soil nails with permanent shotcrete and permanent strand anchors with reinforced concrete headers are proposed. Below street level, the bottom of the cut is concealed behind the buildings, however, the space between the buildings and the cut will not be backfilled but will remain empty and permanently accessible for monitoring and permanent anchor maintenance.

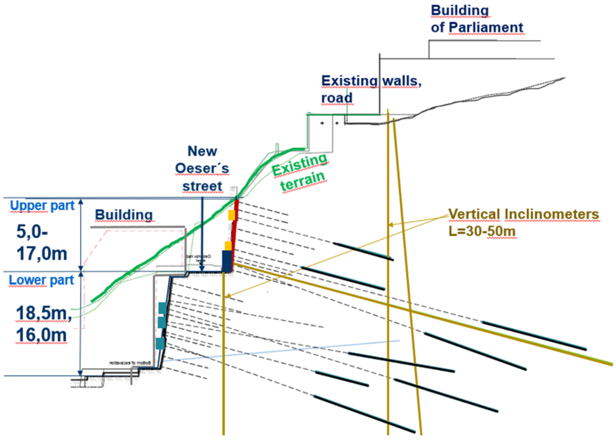

Figure 3. The highest cross section.

During the excavation works higher than predicted movements of the slope were identified by the monitoring. Some displacements were detected approximately 65 m behind the slope, where cracks in the pavement were observed. Displacement pattern unveiled the existence of a fault zone system which could possibly lead to the development of a sliding surface.

Based on these findings, the whole project was reconsidered. Due to the highly complex geotechnical environment, it was decided to proceed with the stabilization works using the principles of the observation method. Most probable geometry of the sliding mass was determined, and anchors were prolonged behind the sliding surface. By means of back calculations the actual strength properties of the rock mass were determined which was a basis for dimensioning of necessary tension elements.

### 3.2 *Analysis*

To increase the reliability of the new design an external company SKAVA Consulting ZT-GmbH (Salzburg), which specializes in computational rock mechanics, was commissioned to perform an independent parallel design calculation for verification.

The calculation of the highest cross section (35 m) was recalculated using cohesion c=0 kPa for the general joints.

Subsoil parameters were further refined based on back analyses of slope stability, using the original terrain configuration (before commencement of excavation works) and the current state of the excavation assuming that a safety factor (SF) equals unity (1.0).

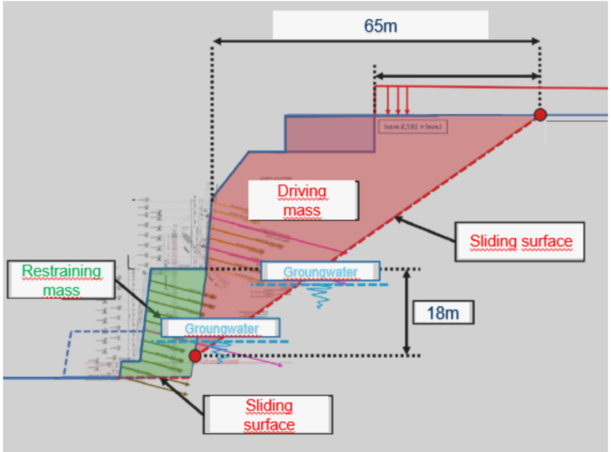

Figure 4. Wedge method stability design (Skava consulting).

Calculations were carried out using multiple software tools to best represent actual site conditions. For the highest part of the cut, a control calculation was performed in Plaxis by Keller. The SKAVA Consulting ZT-GmbH modeled the slope using the Wedge method and integrated the joint system within the UDEC software, which allowed the use of directly evaluated stereograms from endoscopy and the results of laboratory tests of rock mechanics in the form of an environmental model of discrete rock blocks with discontinuities.

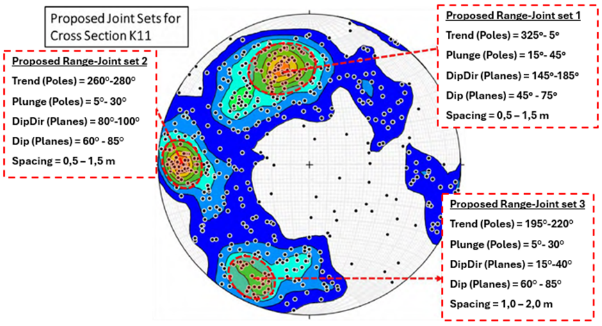

Figure 5. Example of joint sets for cross section.

Comparison of different modeling approaches indicated that the wedge method calculation was the most realistic, partly because it utilizes a predefined shear/slip surface. In Plaxis, a Mohr-Coulomb model (MC model) was used with parameters φ=35.5° (according to the original geological survey) and calculated cohesion c=38.5 kPa, and a variant for φ=33° with calculated cohesion c=55 kPa, were modeled for the highest section. Skava calculated their model using the Wedge method with cohesion c=0kPa and calculated friction angle φ=34.5° for SF=1.

Following the analysis, updated parameters were incorporated into the calculations, selecting program-specific methods that best represent the real conditions and structure. Keller in the GGU-Stability program considered a friction angle of 35.5° and a cohesion of 38.5 kPa and 43 kPa, respectively, according to the respective section.

In all cases, using the calculated parameters (φ, c), the required additional anchorage forces were determined to be added to the system to achieve a degree of safety of SF=1.25 for the permanent condition of the cut.

### 3.3 *New design after new parameters set*

For each section of the excavation, the total required stabilization resistance Rd per linear meter of the cut that needs to be incorporated to achieve SF≥1.25 has been calculated. For each section, the required forces, from which already installed anchor resistances are to be subtracted.

Finite element calculation in Plaxis showed lower safety compared to wedge method or limit equilibrium calculations with GGU software. It was decided that safety factor of 1.20 is acceptable for finite element analyses.

Due to the different calculation results of different approaches, it was considered for further design as follows:

- The amount of the additional required anchor forces will be determined using the Wedge model (Skava).
- The anchor lengths will be designed according to the slip surface position calculated by Plaxis (Keller). To enhance reliability, the anchor bond length will start at least 5m behind the Plaxis slip surface.
- The calculations were verified in GGU Stability Software, all necessary cross sections of the cut, as a confirmatory method.

#### 3.3.1 *Built-in forces*

With the calculated parameters (φ, c), we then calculate the necessary additional anchorage forces to be added to the system to achieve a degree of safety of SF=1.25 for a permanent cut state.

Table 1. Additional anchoring forces-1st calculations.

| Current stage | | Reduced/ **Final** excavation | Lowered ground water |
|---|---|---|---|
| Safety factor | 1,0 | 1,25 | 1,25 |
| Wedge model (Skava) | *34,5° + 0 kPa* | 4039 kN/m | 4061 kN/m |
| | | **5227 kN/m** | 4061 kN/m |
| Plaxis -MC model (Keller) | *35,5° + 38,5 kPa* | 4872 kN/m | 3963 kN/m |
| | *33° +55 kPa* | 5034 kN/m | 3963 kN/m |

Therefore, it was considered to add an additional anchor force of approximately 4000-5000 kN/m for the slope anchorage. To transfer these forces due to the limited area, it was found necessary to use up to 19-strand anchors with a load capacity of 4064 kN, which, at a minimum head spacing of 2.0 m, represents the deployment of these permanent anchors to 4-5 anchor levels. The anchors needed to be placed not only below Oeser's Row, but also at Oeser's Row level, considering the existing beams.

The total stabilization force for the final excavation at the highest cross section (35.0m) was calculated 5277 kN/m, while the built-in force was 1172 kN/m.

Table 2. Additional anchoring forces for the highest cross section

| Total required stabilization forces/resistance: | $R_d$ = 5227 kN/m |
|---|---|
| Already installed resistance | $R_d$ = 1172 kN/m |
| Additional stabilization measures | $R_d$ = 4055 kN/m |

The design strategy focused on optimizing the distribution of anchoring forces along the height of the slope. By carefully coordinating the placement of anchors in both elevation and plan, the design aimed to maximize the load-carrying efficiency of each anchor level while minimizing interference with existing structural elements. This approach ensured that the transfer of forces into the rock mass was both effective and compatible with construction limitations, providing robust and long-term stability for the slope.

Designed anchors included permanent 19-strands anchors ANP Y1860 S7 - DN15.7mm (19 x 150 mm²), permanent bars SAS950 ($f_u$= 950 MPa) ϕ47 mm, ϕ75 mm, further SAS 550, SAS670 ϕ25 to ϕ40 mm.

Permanent bars were reinforcing the rock mass to create a gravity wall of considerable thickness and provide stability to the cut.

The reinforced concrete wall securing the upper part of the excavation was designed to withstand the applied earth pressure. Structural design of the wall was done to accommodate forces which equal to design resistances of the elements. All elements are designed in double corrosion protection to ensure permanent function. The backfilling between reinforced concrete structures and the rock surface was done using concrete of minimum class C8/10.

In addition to the passive elements, the permanent active anchoring elements form a substantial part of the stabilization. Permanent strand anchors have been designed for each section of the wall, with the number of stands ranging from 5 to 19 of different total anchor lengths, depending on the design of the section. The characteristic load capacity of the 19-strand anchor is Rtd =4064 kN and the maximum test force of the anchor is Ppmax = 4241 kN. At each anchor level, the reinforced continuous concrete beams or concrete plinth were designed and the permanent anchors pre-tensioned to the required forces.

## 4 DESIGN OF ANCHORS

The resulting anchor force per linear meter of design section must be anchored beyond the sliding surface and open joints identified by endoscopy, which were located up to 48m distance. The anchor bond length of 20 m was designed resulting in total anchor length of 70m. Re-tensionable anchor heads were used, allowing later additional stressing or releasing of anchors if necessary.

### 4.1 *Design of 19-strands anchors*

Based on the parameters of the rock hill and the specified geometry of the cut, calculations were carried out, and for each cross section the anchoring elements and their associated anchoring forces were designed. Due to the wide range of elements designed (passive/active, different steel type and reinforcement diameters, number of ropes for strand anchors), it was necessary to design an anchor detail for each element to perform a permanent cut stability function.

### 4.2 *Deformations*

A deformation allowance of approximately 6 cm should be provided to enable the determination of prestressing forces for each anchor type. This allowance allows for the measurement of anchor forces in response to massif pressure and provides time to react before the 19-strand anchor reaches its load-carrying capacity.

In order to consider the 6 cm deformation allowance, the slack length must be included during prestressing, as it can differ between sections. The 6 cm elongation is associated with varying values of prestressing force (**P₀**), depending on the free length of the anchor.

Table 3. Extensions of anchors with deformations.

| Free Length (m) | Anchor elongation of xy% $R_{td}$ (mm) | | | | Deformation difference (mm) | | |
|---|---|---|---|---|---|---|---|
| / | 75% | 80% | 85% | 100 % | (100-75) | (100 -80) | (100 -85) |
| 50m | 274 | 293 | 311 | 366 | 91,4 | 73,1 | 54,8 |
| 40m | 219 | 324 | 248 | 293 | 73,1 | 58,5 | 43,9 |
| 30m | 164 | 176 | 187 | 219 | 54,8 | 43,9 | 32,9 |

## 5 DESIGN OF CONCRETE BEAMS

Structural design of all types of reinforced concrete components – plinths, beams and walls – was done.

Structures with **passive anchoring elements** were modelled as supported by point supports, defined either as rigid (infinite stiffness) or elastic, at the locations of the micropiles and were subjected to the prescribed earth pressure.

In contrast, for structures with **active (prestressed) anchoring elements**, the reinforced concrete components were modeled on an elastic half-space and the prestressing force was applied as action in the model.

### 5.1 *Concrete beams with anchors*

Beams for 5 - 10-strand anchors, with cross-sectional dimensions of approx. 0.7 x1.0 m, of max. 30 m length, with the anchors in a single with spacings of 2 m were used. The prestressing forces for these beams are a maximum of 1605 kN (design value for a single 10-strand anchor).

The beams with 19-strand anchors have cross-sectional dimensions of 1.0 x 2.0 m for the single anchor row variant and 1.0 x 3.0 m for a double anchor row. Their length ranges from 10 to 30 m.

Figure 6. Concrete beams with wall in the upper part of slope.

### 5.2 *Concrete plinths with anchors*

Below Oeser's Row, a large portion of the slope in the lower section was stabilized using 19-strand anchors with reinforced concrete plinths. For each 19-strand anchor, an individual reinforced concrete plinth measuring 2.0 x 2.0 x 1.0 m was designed.

The maximum contact pressure beneath each plinth was 1.5 MPa, calculated based on a total design load of 6000 kN acting on a 4 m² contact area.

Figure 7. Concrete plinths in the lower part of the stabilization.

## 6 OBSERVATION METHOD

Observation method during the execution was adopted due to uncertainties regarding possible strength parameters which were identified during the site investigation.

In view of the actual situation on the site and the results of the measurements, it was necessary to proceed with the observational method to ensure the stability of the cut. The slope was monitored at weekly intervals and at each induced change (excavation). On the basis of the observation results and their evaluation, the necessary measures were determined. In the first step, permanent strand anchors were made below the level of Oeser's row and all measurements (geodetic and inclinometric) were done. The next level could be excavated only after their evaluation was approved by the responsible designer.

After applying the permanent strand anchors in the lower part, the anchors were added to the upper part into the made concrete beams. In this part of the cut anchors were installed in axial distance of 4 m, but the beams are prepared for installing the anchors in distance of 2 m. Every second hole in concrete beams in this part is empty to be prepared in case of the future.

In addition to the construction of permanent strand anchors, it is designed to construct horizontal drainage boreholes from the level of approx. 140.0 m above sea level, length approx. 30.0 to 45.0 m, at a slope of 5° from the horizontal upwards, with the release info future drainage system at the foot of the cut. The objective of the horizontal drainage boreholes is to divert groundwater and reduce the water column pressure in the castle hill. The lengths are based on the levels of the groundwater table encountered during drilling. Boreholes were drilled with an overlap of approximately 15.0 m, from the front of the current excavation. Once excavated to the final level, the dewatering pipes were cut with ending into the future drainage system.

## 7 MONITORING

The monitoring system, as an integral part of the observational method, comprises automated geodetic surveys of surface control points, embedded inclinometers and extensometers, as well as anchor load cells. Additionally, selected anchors were equipped with fiber optic sensors to monitor long-term behavior. These combined measures contributed significantly to the successful completion of the first project phase.

### 7.1 *Monitoring of all installations*

During the execution of the works, continuous monitoring of the individual components of the permanent slope stabilization was essential. This included geodetic monitoring of completed reinforced concrete elements (such as beams and walls) and verification of the permanent anchors' performance by measuring the applied anchoring forces using hydraulic dynamometers. The locations of measuring points, dynamometers, and the measurement frequency were established based on construction progress and the expected service life of the elements.

Monitoring activities are executed in accordance with the established design plan, including force readings from load cells installed on permanent strand anchors, geodetic evaluation of control points—covering spatial orientation measurements of anchor beams, shotcrete layers, and exposed rock surfaces—as well as high-precision levelling at designated points on existing structures. Data from embedded instrumentation, such as inclinometers and extensometers, are regularly collected and documented.

Monitoring is required during construction until all permanent load-bearing structures are complete. After assessment, measurement intervals will be set and continue through the project's operation or until risks to life and property are resolved.

### 7.2 *Monitoring of anchor forces*

The tensile force in permanent strand anchors must be continuously monitored and re-tensioned if any force drop is detected. For this purpose, anchors are designed with re-tensionable anchor heads.

Each anchor is equipped with a removable 3 cm thick circular plate to allow controlled reduction of force in the anchor prior to reaching its ultimate capacity.

Load cells are installed on approximately 20% of the anchors for ongoing force measurement and monitoring. These instruments are distributed uniformly both above and below the Oeser's Street row.

Figure 8. Anchor with dynamometer (photo before final coverage).

All installed dynamometers are equipped with wireless transmitters that record the current anchorage forces and temperature at designated intervals. The collected data are transmitted in real time and published on a dedicated monitoring platform overseen by the Monitoring Board. For each anchor type, classified according to strand number, specific alert thresholds are defined and regularly monitored to enable early identification of potential deviations from design assumptions.

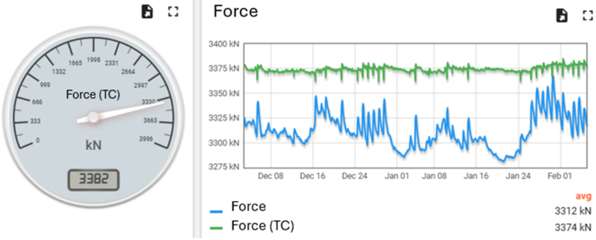

Figure 9. Transmitted data from anchors in real time.

### 7.3 *Anchors with optical fibers*

To determine the performance of the anchors during the stressing progress particularly regarding activated length at various load steps, two ground anchors were equipped with fiber-optic strain sensing cables. The cables were attached to the tendons (inside the anchor) and embedded into the grout (outside the anchor).

The main task of this monitoring anchors was to collect data during testing and prestressing of the anchors.

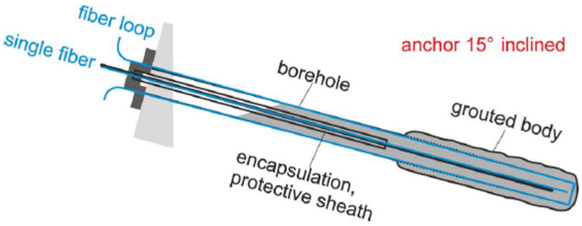

Figure 10. Scheme of the attached fibers of the anchor.

Two different tasks were considered for the selection of the sensing fibers and their set-up at the anchor:

- measuring the activation length of the anchor at different load steps during anchor testing and stressing at the final load step,
- possible additional measurements, i.e. long-term measurements, during/after the excavation process, depending on the requirement.

The latter might be needed in the case of unpredicted movements and might give insights in the sliding behavior by e.g. revealing the depth of the sliding surface inside the rocky material. There were different fiber-optic sensing cables attached inside and outside the corrugated pipe.

The tested anchors were 50 m long of which 30 m was free length and 20 m of fixed length. Anchors were located in lower part of the slope.

Compared to the standard procedure, some additional intermediate steps were performed to avoid decorrelation of the fiber-optic data. The settling time at the intermediate steps was chosen a little longer as usual, in order to get redundant, unaffected data.

Both anchors behave in a similar way. The strain profiles on the tendons as well as the strain profiles of the "inside" and the "outside" showed almost exactly the transition zone between the free length and the fixed length.

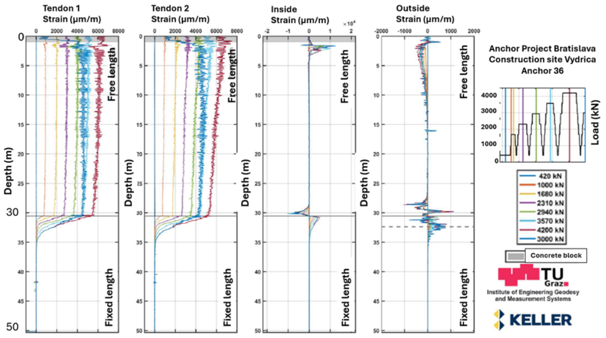

Figure 11. Strainprofiles for inside and outside tendons along 1 anchor.

Further down at the fixed length, where the anchor is not activated yet, the strain is about zero. But the higher the applied force gets, the more of the anchor gets activated and the strained region moves down, which can be clearly seen. Finally, the activated fixed length is about 4.1 m - 4.7 m for anchors.

The activated length of the anchor can also be seen clearly in these data sets.

In the transition zone between the free and fixed length, there are also strained (below the grey line - Figure 11) and compressed (above) regions, which are overlaid by “spikes” indicating cracks in the grout.

Additionally, the “V9 outside” of one anchor shows a strained and compressed region in a depth of about 17 m to 20 m, which indicates a softer layer in this depth. At the upper end, down to about 10 m to 15 m below the abutment, negative strain is visible, which shows the compression of the rocky material due to the applied force. This can be seen more clearly for the anchor, where geology was reported to be worse. For the other anchor, this loss of pressure with increasing depth cannot be seen that clear, as there are obviously some layers of different stiffness close to the surface.

## 8 ROCK SLOPE STABILIZATION OF THE 2$^{ND}$ PHASE

Project Vydrica continues to 2$^{nd}$ phase, which linked this new area to the old city center. During the implementation of the first phase, preparations for the second phase were already underway, including project development, geotechnical investigations, and monitoring activities. For the second phase, the design calculations built upon the experience gained from the first phase and followed the observational method. This approach was adopted because the endoscopic inspection and monitoring of the rock mass revealed different geotechnical parameters than initially assumed.

The project of the second phase is already prepared, and the upper part of slope is stabilized with soil nails and anchors. The implementation works is ongoing.

## 9 CONCLUSIONS

The stabilization of Castle Hill required a comprehensive, multi-phase geotechnical approach, combining detailed site investigation, continuous monitoring, and iterative modeling. The design evolved through the application of the observational method, allowing refinement of input parameters based on the ongoing assessment of rock mass behavior.

The use of different modeling approaches allowed cross-verification of results and increased confidence in the proposed stabilization strategy. The implementation of stabilization measures on the Vydrica construction site is an example of the combination of modern geotechnical and construction solutions for securing slopes in challenging conditions.

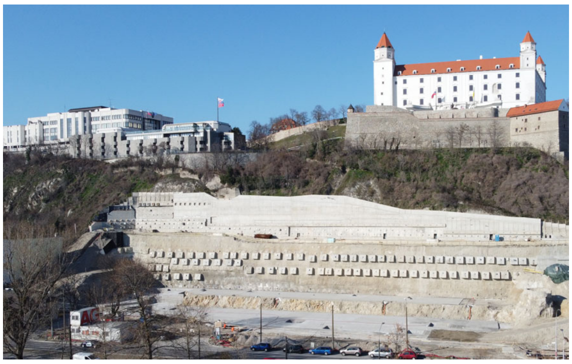

Figure 12. Final stabilization of castle hill.

Stability was achieved through the use of permanent strand anchors, passive reinforcement bars, and reinforced concrete structures, such as beams, plinths, and retaining walls, designed to withstand significant slope gradients. All stabilization measures were conceived with a focus on long-term performance and resistance to geological and environmental influences. The use of modern technologies, such as permanent strand anchors with re-tensionable anchor heads, allows for ongoing monitoring and future adjustments when necessary.

The applied methodology and results provide a valuable reference for the stabilization of similarly complex rock slopes, particularly in urban areas with historical and spatial constraints.

In addition to technical efficiency, the project also respected architectural and heritage preservation requirements. The final design including the use of a front-facing stone wall—successfully integrates safety, durability, and aesthetic value, making it suitable for the conservation zone in which the structure is located.

This solution demonstrates a methodical and well-balanced engineering approach to addressing geotechnical challenges in historically sensitive environments and may serve as a reference for future projects of similar complexity and technical requirements.

Figure 13. Completed wall stabilization.

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