Change to bedrock modifications as a result of additional geological survey results related to the renovation of sports premises

Stanislav Smugala¹, Darja Kubečková¹, František Indra²

¹Department of Construction, Faculty of Civil Engineering, VŠB - Technical University Ostrava, Ludvíka Poděště 1875/17, 708 33 Ostrava - Poruba, Czech Republic
²SAFETY PRO s r.o., Rudná 1117/30a, 703 00 Ostrava – Vitkovice, Czech Republic
darja.kubeckova@vsb.cz

Abstract. When foundations are uncovered during a construction project, the foundation load capacity is often found not to correspond with the prescribed limit values in the project’s documentation after being compared with a given deformation module. This is often the result of a corresponding geological survey not having been conducted satisfactorily according to the given construction project’s surface area. The most common method of stabilizing bedrock, i.e. increasing the deformation module value, is lime stabilization. This method represents a suitable technical solution for modifying foundation layers, especially from an economic perspective. However, this modification method may not necessarily represent a suitable technical solution because of either the heterogeneity of underlying layers or the existence of fluctuating groundwater levels at the construction site, which can, when the final work is used, affect the mechanical and physical characteristics of the already stabilized bedrock layer.

1. Introduction

The most commonly used additives for achieving the specified strength characteristics of bedrock are cement and lime. This stabilization method is used when the structure of the bedrock layers is finely granular [1]. The stabilization process involves excavating the low-bearing capacity bedrock layer and mixing additives and water. Besides traditionally applied additives, new bitumen and bentonite-based additives have recently been developed. Their application results in strength characteristics comparable to the given road’s bedrock layers [2]. A principally different bedrock stabilization method is its complete replacement with a gravel layer. This method is used when individual bedrock layers are heterogenous and stabilization using additives would not guarantee achieving the required results, i.e. improving strength characteristics. When bedrock layers are heterogenous, the characteristics of the strengthened bedrock may change during works because of the degree of water saturation. Swelling of the bedrock layers particularly can result in the final construction to become dysfunctional as a whole. This problem is addressed by much of the international literature [3][4]. Exploration of the volumetric changes in individual bedrock layers according to their respective saturation is the subject of technical literature [5][6]. The problem of swelling clay bedrock layers and the tension occurring in relation to volumetric changes is also addressed, for example, in publications [7][8]. The initial bedrock moisture during the compacting process plays a definitive role when bedrock volume increases once the final construction is used. This can lead to the deformation of subsequent construction layers. It has been demonstrated that when bedrock moisture increases during
the compacting stage, no deformation tension forms in later stages if the bedrock is compacted under low pressure. Underestimating survey works in construction practices is an adverse factor that contributes to the poor final quality of works and the defects that already occur during the construction process or immediately after the final construction commences operation. Defects and faults incurred in this manner prolong construction time and can lead to warranty claims immediately after the final construction commences operation. The investment cost of the given construction as a whole therefore also increases.

Apart from the accompanying construction objects of the SO 15 entry road, SO 16 road at the premises, SO 17 road and parking lot, SO 18 road and bike path and SO 21 supporting walls, the soccer stadium reconstruction project also addressed the soccer field itself (SO 12 construction object). This part of the project included replacing the existing grass on the soccer field and installing an irrigation system. After receiving the corresponding project documentation and geological survey, the construction contractor’s examination focused on the composition and characteristics of the individual geological bedrock layers. Changing the technical specification of the irrigation system, which was proposed by the contractor as more economical in terms of investment costs and operating expenses, involved improving the stability and volumetric constancy of the bedrock layers. If this greater stability and volumetric constancy had not been achieved, the bedrock instability when the final construction was used could have resulted in irrigation system leaks and the potential dysfunction of the whole project. Considering the size of the reconstruction surface, repairing this type of defect would be very expensive, and more significantly, contribute to additional investor losses since the final construction would not be usable. The bedrock layers of the above-mentioned accompanying road projects and parking lots were also examined; however, they did not represent a fundamental problem because of their size and type.

2. Area geology and survey

The survey’s purpose was to verify the composition of individual geological bedrock layers and their load bearing capacities stated in the project engineer’s geological survey. The survey was insufficient since a satisfactory number of geological boreholes on the soccer field could not be made while the field was being used by the investor. For this reason, several additional excavated bores were implemented at the construction site in order to examine the geological conditions.

2.1. Initial tests and documentation

In the initial stage, the following documentation was made available by the investor to the construction contractor and geological technician: project documentation prepared by Coplan Projekt, s r.o. for the necessary building permit, hydrogeological assessment of the construction site conducted by GHE, a.s. in January 2015, layouts and profiles of the IG boreholes from the IG survey conducted by G-Consult, s.r.o. in November 2009, and profiles of two archive boreholes from 1987 and 1997 conducted by the Geofund of the Czech Geological Survey. The contractor conducted two static load-bearing tests to verify the bedrock loadbearing capacity at the structure SO 11 ground level, according to which the corresponding laboratory protocols were prepared.

2.2. Survey works

As mentioned above, the composition of the geological layers and bedrock loadbearing capacity based on the documents submitted by the investor for individual construction sections was verified with ten excavation probes designated KS 1 to KS 10. The probes were positioned at the construction site to allow, where possible, geotechnical information about all the important building structures to be obtained in a balanced manner. An indicative layout of the excavation probes is shown in Figure 1.
Besides the information from the excavation survey probes, information about the geotechnical profile affected by the excavation on a multipurpose cable in the south-western section of the field and near the excavated slope in the eastern section of the construction site was also included. A list of all excavation survey probes, including their depths, elevations and locations, is included in Table 1. Individual profiles of the excavation probes, including the profile of the excavation for the multipurpose cable, are included in Table 2. For photo documentation of examples of the excavation probes, see Figures 2 and 3.

Table 1. List of all excavation probes

| Excavation probe (number) | Depth (m) | Elevation (metres above sea) | Location (number of construction object) |
|---------------------------|-----------|------------------------------|------------------------------------------|
| KS 1                      | 1.5       | 242.51                       | SO 12, approximately the centre of the field |
| KS 2                      | 2         | 242.54                       | SO 12, N section of the field            |
| KS 3                      | 2.5       | 242.52                       | SO 12, NE section of the field           |
| KS 4                      | 2         | 242.41                       | SO 12, SE section of the field           |
| Excavation for the multipurpose cable | 1.5 | 242.32 | SO 12, SW section of the field |
| KS 5                      | 1.5       | 242.1                        | SO 21, Supporting wall, SW section of the field |
2.3. Geological conditions at the construction site

The geological conditions at the construction site were described in detail in the hydrogeological assessment prepared by GHE, a.s. in January 2015 and the IG survey conducted by G-consult, s.r.o. in November 2009. It is clear from these sources that the geological order of individual layers with classification estimate following ČSN 736133 standards of the foundation soil of the building structures at the given location comprises (from the top):

- Anthropogenic landfills – Y of variable thickness and character
- Diluvial and deluvio-fluvial sediments of variable thickness (up to 2 m) and soil character class F4 CS – F6 Cl (Quaternary)
- Eolithic sediments (loess loam) of class F6 Cl – F6 Cl (Quaternary)
- Glacial sediments – gravel and sandy gravel of S5 SC class (Quaternary)
- Fluvial sediments – deposited gravels of the valley and main terrace of the Olše river with a thickness of 3–5 m and soil character class G3 G – F – G5 – GC (Quaternary)
- Neogene clays of class F8 CH (Neogene – pre-Quaternary mountains)

Implemented by the contractor with the assistance of the geological technician, the excavation probes confirmed the results of the previous survey operation stages.

Table 2. Profiles of individual excavation survey probes

| Excavation probe KS 1 | Date 21.08.2015 | Profiled by ing. F. Indra |
|-----------------------|-----------------|--------------------------|
| Location:             | SO 12, approximately the centre of the field |                           |
| Depth (m)             | Soil description | classification estimate following ČSN 736133 standards |
| 0.0–1.0               | Landfill – drainage landfill, anthropogenic | Y (character G2 GP)         |
| 1.0–1.5               | Sandy clay, grey and brown with rusty streaks, firm to hard consistency, Quaternary | F4 CS                       |

Under the anthropogenic layers of the gravel class G3 G – F – G5 GC Y landfills and layers of occasionally stony character class G2 GP Y, single-grained Eolithic sediments of soil character class F6 Cl (KS 5,7) could be found at the western, southern and eastern sides of the field. In some spots, these sediments were of sandy soil character class F4 CS (KS 1, 2, excavation for the multipurpose cable). In order to assess the project’s modifications of soils in the active zone (0.5 m below the projected earth surface level of the accompanying structures, i.e. SO 15, 16, 17, 18 and the soccer field structure SO 12), it was useful to process Table 4 and the indicative values of the projected earth surface levels of the foundations and soils located in the active zones. On the eastern side of the construction site near the parking lot (KS 6), fine-grained, probably diluvial sediments of light brown colour with rusty streaks, firm consistency and soil character class F6 Cl were identified. These soils contained deposits of grey clays with a humus-like odour and organic class F6 Cl O remains. Glacial, fine-grained to sandy sediments of soil character class F4 CS – S1 SW (KS 3, 4, 8 and KS 9) were found at the eastern and especially northern side of the site. Grey deposited clays of soft consistency with a humus-like odour and organic remains of class F6 Cl – F8 CH O (KS 2, 4, 7 and KS 10) were found in the bedrock of the Eolithic sediments at the western, southern and eastern sections of the site.
field. Saturated, grey fluvial gravels of soil class G3 G – F (KS 3) were located in the bedrock of the glacial sandy sediments in the north-eastern and northern sections of the construction site. For more details about the geological profiles of individual excavation probes, see Table 2 (excavation probe KS 1). All geological soil descriptions, including indicative classification according to ČSN 73 6133 standards [9], were done by the geological technician exclusively from macroscopic assessments. No laboratory analyses were conducted to characterize the individual soils.

2.4. Hydrogeological conditions at the construction site
From the hydrology perspective, the glacial and fluvial gravels form a saturated collector restricted from the bottom and top by slightly permeable to impermeable fine-grained sediments: from the top, Eolithic loess loams or diluvial and deluvio-fluvial sandy to plastic clays, while from the bottom, Neogene clays of high plasticity. The stated sequence of individual layers forms an aquifer mode with a tight groundwater level. This is clear from the submitted hydrogeological assessment conducted by GHE, a.s., the survey boreholes executed by G-Consult, s.r.o. in November 2009 and the two archive boreholes from 1987 and 1997 executed by the Geofund of the Czech Geological Survey. A comparison of groundwater levels at individual survey boreholes demonstrating a hydrological aquifer mode with a tight groundwater level is shown in Table 3. The excavated survey boreholes found groundwater in KS 2, probably from deposited clays of class F6 Cl – F8 CH, at a depth of 2 m below the terrain, i.e. approximately 240.5 m above sea level. The excavation probe KS 3 discovered groundwater at the ceiling level of fluvial gravels class G3 G – F at a depth of approximately 2 m below the terrain, i.e. also at 240.5 m above sea level. Groundwater in the excavation probe KS 8 was located in the layer of glacial sands mixed with gravel class S1 SW – S5 SC at a depth of approximately 0.7 m below the terrain, i.e. approximately 241.8 m above sea level. A stabilized groundwater level in the excavation probe KS 2 was found by the construction contractor at 240.93 m above sea level, while the level in the excavation probe KS 3 was at 240.58 m above sea level. Determining the stabilized groundwater level in the excavation probe KS 8 was not possible because the excavated probe was partially buried. Stabilized water levels were apparent in all three of the above-mentioned excavation probes.

| Survey probe | Level of the encountered groundwater level (metres above sea level) | Level of the stabilized groundwater level (metres above sea level) | Difference between the two levels (m) |
|--------------|---------------------------------------------------------------|---------------------------------------------------------------|--------------------------------------|
| KS 2         | approximately 240.5                                          | 240.93                                                       | approximately 0.43                   |
| KS 3         | approximately 240.5                                          | 240.58                                                       | approximately 0.08                   |
| KS 8         | approximately 241.8                                          | 0                                                            |                                      |
| J – 1        | 240.26                                                       | 241.46                                                       | 1.2                                  |
| J – 2        | 239.63                                                       | 240.83                                                       | 1.2                                  |
| J – 3        | 237.28                                                       | 238.98                                                       | 1.7                                  |
| PN – 1       | 241.3                                                        | 241.3                                                        | 0                                    |
| PN – 2       | 237                                                          | 239.5                                                        | 2.5                                  |
| Archive borehole S – 1 (1987) | 0                                                      | 246                                                          | 0                                    |
| Archive borehole KPV – 428 (1997) | 0                                                      | 246                                                          | 0                                    |
2.5. Soils at the earth surface and the foundation levels of individual construction objects

In order to assess modifications to the soils in the active zone (0.5 m below the projected earth surface levels of construction objects SO 12, 15, 16, 17 and 18 and the foundation at SO 21 of individual construction objects), it was useful to process Table 4 and the indicative values of the projected earth surface levels of the foundations and soils located at the active zones. The project’s earth level values were obtained by deducting the thickness of the structural levels of the field, roads, parking lot and bike path from the projected surface levels of individual construction objects (readings from the DPS drawing documentation). The surface levels in the area observed vary. The values in the table should be understood as indicative values only. According to the data in Table 4, it is clear that landfills Y of a heterogenous character and fine-grained soils (diluvial, deluvio-fluvial, Eolithic and glacial sediments) of class F 6 Cl and F 4 CS character are located in the active zone levels of individual construction objects. According to the information obtained from the excavation survey probes and the data stated in Table 5, it is clear that the active zone levels of individual construction objects include landfills Y – of gravel character, soils of class G2, GP and G3 G – F, fine-grained soils (diluvial, deluvio-fluvial, Eolithic, sedimentary and glacial sediments) of class F 4 CS, F 6 Cl and F 6 Cl O – F 8 CH O character, and glacial, sandy sediments with gravel of the character of soil class S1 SW – S5 SC (according to classification estimate in accordance with ČSN 736133 standards).

### Table 4. Indicative levels of the projected earth surface/foundations of the active zone

| Construction object | Projected level of the earth surface/foundation (metres above sea level) | Classification of soil following ČSN 736133 standard | Thickness of the layers above the level of the earth surface/foundation (m) |
|---------------------|---------------------------------------------------------------------|-----------------------------------------------------|--------------------------------------------------------------------------------|
| SO 12, Soccer field | 241.61–241.27                                                      | Y, F6 Cl, F4 CS                                      | 0.5                                                                               |
| SO 15, Entrance road| 241.55–242.14                                                      | Y, F6 Cl, F4 CS                                      | 0.5                                                                               |
| SO 16, Road on the premises | 243.39–245.45                                    | Y (landfills)                                        | 0.4                                                                               |
| SO 17, Road and parking lot | 244.35–245.80                         | Y (landfills)                                        | 0.52                                                                              |
| SO 18, Road and bike path | 241.43–245.64                  | Y, F6 Cl, F4 CS                                      | 0.24–0.57                                                                         |
| SO 21, Supporting wall | 240.70–241.50                                      | Y, F6 Cl,                                              | 0                                                                                 |

The character of the soils in the active zones of individual construction objects at the site is predominantly heterogeneous and inhomogeneous, which is not a favourable finding from the perspective of the proposed soil modification procedures. It was determined that most of the above-mentioned soils were not suitable for lime treatment, as originally foreseen in the project’s documentation.

### Table 5. Soils in the active zone levels of individual construction objects

| Construction object | Projected level of earth surface/foundation (metres above sea level) | Classification of soil in the active zone following with ČSN 736133 standard | Survey probes KS |
|---------------------|---------------------------------------------------------------------|-----------------------------------------------------------------------------|------------------|
| SO 12, Soccer field | 241.61–241.27                                                      | G2 GP Y, F4 CS                                                            | KS 1             |
|                     |                                                                    | F4 CS F6 Cl – F8 CH                                                       | KS 2             |
|                     |                                                                    | F4 CS – S5 SC                                                             | KS 3             |
|                     |                                                                    | F4 CS – F4 CS O                                                           | KS 4             |
|                     |                                                                    | F4 CS                                                                     | multipurpose cable |
| SO 17, Road and parking lot | 244.35–245.80                         | F6 Cl + F6 Cl O                                                           | KS 6             |
|                     |                                                                    | G3 G – F Y, F6 Cl                                                         |                  |
Sandy, gravel, chemically problematic and other soils with a significant proportion of organic substances are generally not suitable for chemical treatment using lime. Many of these soil types are located in the active zone area at the construction site. Another significant adverse factor related to treating soil with lime is the indisputable groundwater impact, which can change the mechanical and physical characteristics of the bedrock material. This problem is discussed in more detail in the next chapter. This section of bedrock also comprises Chlebovice layers, mostly formed by mudrocks with local benches of Štramberk limestone. Furthermore, the location also contains grey, marine calcic mudrocks with occasional sandstone and pudding stone of the Ždanice and Under-Silesian unit, formed by Frýdek formation. The ČGS information indicates that the covering quaternary formations at the location of the given object comprise Eolithic loess loams.

The project requires a load bearing capacity of at least 45 MPa (value of the deformation module $E_{\text{def}}$ from the second loading branch during the static loading test using a plate) on the earth surface at field of construction object SO 12, local roads (construction object SO 15, 16, 17 and 18), parking lot SO 17 and bike path SO 18, and a minimum load bearing capacity of 80 MPa at the foundation under the supporting walls at SO 21 construction object. Since it was not possible to achieve the required load bearing capacity parameters in the soils at the construction site at the earth surface/foundations (in their natural condition) (the results of 2 SZZ conducted on these soils at construction site 2 returned realistic values $E_{\text{def}}$ of 1.62 and 4.82 MPa, with a maximum value under optimum conditions of $E_{\text{def}} \leq 10$ MPa), the project envisioned treating the soil with lime. According to the original PD, the fine-grained soils at the construction site were meant to be treated with aired calcium oxide (CaO), which would be mixed with the soil utilizing a ground milling cutter. Soil treated with a bonding agent immediately changes its characteristics by reducing its natural moisture content, increasing its load bearing capacity and modifying its plasticity as a result of the chemical reactions. “When clay minerals react with lime, a cation exchange occurs and free Ca$^{2+}$ ions in the crystal clay structure replace the sodium (Na$^+$) and potassium (K$^+$) ions. In this process, the structure of the resulting material changes from a stratified structure, which is typical for clay materials, into a grainy structure, which allows flocculation and agglomeration. The resulting product is therefore no longer a plastic clay. It becomes a grainy (lumpy) material instead [1].

From a long-term perspective, the gel as a result of the reaction between the lime and clay materials gradually crystallizes until a firm and compacted material is formed. Hydroxyl ions (OH$^-$) released because of the lime create an environment with a pH that allows SiO$_2$ and Al$_2$O$_3$ to be dissolved from the clay material and a pozzolanic reaction to occur. SiO$_2$ and Al$_2$O$_3$ in the clay minerals react with water and lime. In this reaction, calcium, silicone and aluminium hydrate gels are formed. These gels gradually crystallize and mutually bind the given structures. The result of this reaction is a compact material that is stronger than the original soil [1]. Some of the immediate effects of the soil lime treatment include drying of the soil, increased plasticity limits, greater strength, IBI and CBR, and reduced vulnerability to frost. [2] When the soil is drying, burnt calcium oxide hydrates, (CaO+H$_2$O $\rightarrow$ Ca(OH)$_2$ + 65 kJ.mol$^{-1}$) and some of the water evaporates because of the heat released during the exogenic reaction. By adding the dry material (lime), the ratio between the water mass and mass of the solid particles decreases. Some of the long-term effects of lime treatment are also a pozzolanic reaction (a long-term process during which the gels crystallize and the soil and lime structures bond together). As a result of the modification, the permeability of the treated soil increases by 1 to 2 degrees. It was expected that after treatment in the

|         |          |                     |     |
|---------|----------|---------------------|-----|
| SO 18, |          | F6 CI – F8, CH O    | KS 10|
| Road and bike path | 241.43–245.64 | F6 CI, F6 CI – F8 CH O | KS 7 |
|         | G3 G – F Y, F4 CS, S1 | SW – S5 SC | KS 8 |
| SO 21, |          | F6 CI,              | KS 5 |
| Supporting wall | 240.70–241.50 | F4 CS               | KS 9 |
active zone, the soil IBI values along the roads at the construction site would comply with ČSN 73 6133 standard to a degree of at least 10% or more [9].

2.6. Results and comparison of the survey and research works

It is clear from the data comparison in Tables 3 and 4 that the earth level of the construction object SO 15, 16, 17 and 18 and the foundations at SO 21 structure at the construction site in many cases penetrate the stabilized groundwater level, i.e. the level to which the tight groundwater level rises when the HG insulator in the top layer of the saturated collector’s bedrock is breached. The insulator can be breached in many ways, for example:

- a drilled borehole
- an excavation for any building structure
- a cut or removed portion of the earth body
- milling conducted in a soil treatment process with close contact between the collector’s ceiling and the base of the above-the-bedrock insulator
- a previous intervention into the given geological conditions.

It can therefore be assumed that groundwater will, with great probability, affect the mechanical and physical characteristics of the soils at the earth surface and foundation levels of individual construction objects. Since soil treatment with lime was proposed to a depth of 0.4 m below the projected earth surface and foundation levels, the risk of the treated soil being affected increases due to this fact. The groundwater level represents a direct risk for the soil layer. After being modified with a bonding agent, the volumetric weight of the given fine-grained soil will decrease, its permeability will increase, and a suction process will be initiated. In this case, a water source near the treated soil is in the form of Rajec Creek, whose level can fluctuate. The treated soil will be able to absorb this water and thereby increase its natural moisture content by tens of percent. Treated soil saturated in this manner subsequently loses its mechanical and strength characteristics gained through treatment as a result of (highly probable) recurring saturating and drying processes.

This process represents a risk over time of partially losing the stability gained immediately after treatment during the 24-hour maturing period of the soil and bonding agent mixture. When affected by groundwater, the soil’s chemical treatment process also represents a risk to the long-term usability of the construction objects whose foundations are located on the bedrock affected in this manner, particularly since the premises concerned are located along the meadow of the Olše River in its inundation area. Understanding that any increase in the river’s water level will proportionately increase the groundwater level in its surroundings is important. Moisture content fluctuations in the bedrock layers, including the 0.4 m treated layer proposed by the project, will affect volume changes. The contractor ordered laboratory testing of collected samples from excavated probes KS 1 to KS 10 in accordance with ČSN 66 6635 for this reason, in order to determine swelling capacity and absorbing power [10].

3. Swelling capacity laboratory tests

Ten samples were collected and subjected to swelling capacity laboratory tests. Their results were provided as volumetric and linear swelling coefficients B (%). The results were assessed in accordance with the methodology “Proposal of unified methodological procedures and instrumental equipment for geotechnical tests, swelling capacity, 1984” [10]. The soil swelling capacity test consisted in measuring the increase in single or multiple dimensions of the tested object during a short-term rise in moisture content in relation to time until the volumetric changes caused by swelling in the tested material stabilized. No external loads were applied to the tested object during the analysis. In this test, the object was fixed to the given instrument (in accordance with ON 44 0051 or odometer) and water was applied. The increase in height of the object was then monitored using an appropriate indicator.
The increases were monitored after 1 minute, 10 minutes, 30 minutes, 1 hour, 3 hours, 8 hours, 24 hours, and if needed, on a subsequent daily basis.

The test was considered complete when the difference between the two subsequent measurements became less than 0.005 mm/day. The test simulated the behaviour of clays (weathered mudrocks) coming into contact with water. The stated soils generally have a low moisture content. After coming into contact with water, they start absorbing moisture enormously, which creates suitable conditions for the development of swelling tensions. The test provided indicative information about the composition and character of the clay materials contained in the tested soil. “Montmorillonite minerals absorb water not only on the surface of individual particles but also inside their crystals.” For this reason, the highest swelling values were recorded in materials with a proportion of the stated clay materials [10]. From the perspective of their swelling capacity rate (volumetric swelling coefficient B expressed in %), the conducted laboratory analyses confirmed that the soils represent a marginal problem—a swelling capacity coefficient of 1.5% was only locally demonstrated at the KS 2 probe’s location, which did not represent any problem considering the thickness of the layer.

4. Theoretical principles and assumptions

The characteristics of the strengthened bedrock could change during the use of the final construction due to the degree of water saturation—for example, plastic deformation. Swelling of the bedrock layers especially causes the subsequent compression and deformation of the top construction layers. This problem has also been addressed by several international authors, such as Gomes [5] and Tatsuoka [3]. The exploration of volumetric changes in bedrock layers in relation to their saturation was the subject of publications written by Suriola, Gense and Alonsa [4] and Goulda, Kodikary, Rajeeva, Zhaoa and Burna [6]. The authors Hogentogler [11] and Holtz and Gibbs [12] explored theories related to bedrock characteristics during the strengthening process. Their works include descriptions of the individual development stages of individual bedrock layer characteristics. The definition of the mutual relationship between the dry density of the bedrock yd (or empty ratio e) and the moisture content of the bedrock (w) has been used since the 1930’s. The Proctor compaction curve, whose basic principles were published by Proctor [13], and the CBR test (California Bearing Ratio) form a part of the Australian AS 1289.6.1.1 standard [14] and the American standard ASTM [15]. The mutual relationship is given by the typical compaction curve of the reverse parabolic shape along a plane, which defines the maximum density in dry conditions and corresponding optimum moisture content in accordance with, for example, the Australian AS 1289.5.1.1 standard [16].

The problem of swelling in clay-based bedrock and the tension that occurs in relation to volumetric changes is addressed in several publications by Alonso [17] and Kodikara [18]. The initial bedrock moisture content during the compaction process plays a decisive role when bedrock volume grows as the final construction is used, which can lead to deformation of the top construction layers. When the bedrock moisture content during the compacting stage increases, it has been observed that no deformation tension forms during the later stages if the bedrock has been compacted with low pressure. This phenomenon has been elaborated on by Lawton, Fragaszy and Hardcastle [19]. By contrast to saturated bedrock, when compacted clay bedrock with a low moisture content increases or decreases its moisture content as the final construction is used, the result is a change in the volumetric behaviour of individual bedrock layers, which can deform the given structures. Besides this, bedrock can also be exposed to plastic deformation as a result of the wet/dry cycle (Figure 3), which results in a stabilized state. This stabilized state can be disturbed by flooding, long-term drought, or in this case, changes in the geological conditions.
In order to model these bedrock behaviour characteristics, several numeric models have been created, for example, the BBM (Barcelona Basic Model). These are discussed in publications by Alonso, Gens and Josa [20] and others. The contemporary constitutive modelling of external and environmental loading for unsaturated bedrocks is expressed in technical publications by Houlsby [21] through two equations:

\[ dW = -\frac{\sigma_{ij}}{s/(1+e)} \right] e_{w}, \]  \hspace{1cm} (1)  

\[ dW = -\left(\sigma_{ij} + s\delta_{ij}\right) \right] d\epsilon_{ij} - n s dS_{r} \right] \]  \hspace{1cm} (2)

where \(\sigma_{ij}\) is the pure standard pressure, \(e_{w}\) is the tension tensor, \(s\) is the saturation capacity, \(e\) is the ratio of cavities, and \(e_{w}\) is the ratio of moisture content. Based on equation (1), volumetric behaviour is expressed as pure pressure (with volumetric tension \(d\epsilon/(1 + e)\) as a conjugation), with an absorption capacity (more precisely \(s/(1 + e)\) with \(e_{w}\) as a conjugation). In reality, two situations can occur. In the first, partial saturation and loading is followed by the swelling process. In the second, the bedrock layers become fully saturated from flooding or changes in the geological conditions, which results in deforming the bedrock layers. This situation can then disturb technological structures, provided that the bedrock layers comprise reactive materials such as mudrocks. By implementing the lime stabilization procedure proposed in the project, recurring bedrock saturation results in decreased strength and rigidity values [22]. According to numerous publications, a balanced state occurs only after the third bedrock saturation and drying cycle, during which the volume of the bedrock layers increases and decreases. A balanced state is achieved when the bedrock volumetric increase in the saturation stage equals the bedrock volumetric decrease in the drying stage. The maximum bedrock volumetric increase occurs during the second saturation cycle [23]. However, this state is only true under the natural conditions of the construction site. When the sporting facilities and soccer field were built, the groundwater level was affected by the nearby river’s water level. This means that the bedrock saturation percentage could become repeatedly elevated compared to its previous, natural state. It is thus questionable whether a balanced state between bedrock volumetric increase and decrease in the drying stage can be achieved with repeated saturations of the bedrock due to increased groundwater level related to increases in the river’s water level. The reduced strength and rigidity values can be expressed using the flexibility module \(M_{r}\) (resilient module), which can be expressed according to the following equation [23]:

![Figure 3. Moisture values during the use of the final construction equilibrium m.c.](image-url)
where $\sigma_d$ is the stress deviator and $\varepsilon_R$ is the recoverable axial strain.

Figure 4 shows the consequences of saturated bedrock based on the values $\varepsilon_R$ and $M_r$ after the bedrock’s treatment with lime (6%).

![Figure 4a. Flexibility module values $M_r$](image1)

![Figure 4b. Permanent strain values $\varepsilon_p$](image2)

The publications by Mavrolidou [24] confirm a reduction of the $G_{\text{max}}$ resilient module values for repeated saturation and drying cycles. Nevertheless, when the bedrock was stabilized with lime, the reduction in this value was somewhat limited. Bedrock treated with lime and repeatedly exposed to saturation and drying processes generally loses its adhesive ability between the solid bedrock particles and the lime, which leads to an increase in its volume [25]. Lime treatment is therefore suitable only at locations where repeated saturation and drying processes do not occur.

Another alternative bedrock treatment method is stabilization with cement. By contrast to bedrock treatment with lime bonding agents, cement bonding agents do not penetrate the structure of solid bedrock particles. Instead, they coat the particles, which significantly reduces bedrock volumetric changes. However, this bedrock treatment is not suitable for soils with a high degree of plasticity [26]. Over time, this process also improves the strength characteristics, elastic module and resilience against the consequences of moisture, frost and thawing. When seeking the best bedrock treatment, other treatment options may also be considered. These include treatment with fibre, ash and bitumen or bentonite-based additives.

5. Proposed change in the method of renovation measures

Instead of soil treatment as proposed in the project, the construction contractor suggested mechanically replacing the bedrock across the entire area down to the necessary depth, which was expected to be 0.5 m below the projected earth surface level at SO 12, 15, 16, 17 and 18, and 0.8–1.0 m below the projected foundation level of the supporting walls at SO 21 of the construction projects. The area requires unsuitable soils to be removed from the above-mentioned construction objects and taken either to landfill or retained for further processing and use at the construction site. The contractor and geological technician recommended laying a separating sheet made of geotextile with a weight of 200 g/m² over the entire area in question. The geotextile sheet should be applied in a manner that ensures compliance with the filtration criterion in ČSN 73 6133 standard[9], making sure that the grains of the gravel materials in the rehabilitation layer can still be pushed into the bedrock’s fine-grained sediments, thereby preventing uneven settling of the construction objects. It was recommended that the deficient material be replaced in its entire thickness with a non-freezing, well compactable, frost-resistant material with a fluent granularity curve and proportion of up to 15% by mass of fine particles of soil class G1 GW, G3 G – F and S1 SW according to classification given by ČSN 73 6133 standard [9].
Applying and compacting the material layer by layer, each with a maximum thickness of 0.3 m, was also recommended. Some of the materials that may be used include sandy gravel with a fraction of 0–32 mm or 0–63 mm, crushed stone, recycled concrete products or volume-stable artificial gravel of similar fractions. The suitability of the proposed solution, especially the thickness of the rehabilitated layer, must be verified with a compaction test in accordance with ČSN 72 1006 standard [27]. When excavation works are conducted, the requirements relating to demonstrative and control tests must be fulfilled in accordance with ČSN 73 6133 [9], ČSN 72 1006 [27] standards, TP 4 technical condition [28] and other applicable regulations. [29] This solution represents the best possible technical solution for the bedrock’s volume stability in consideration of the fluctuating groundwater level, even though the solution is relatively expensive.

6. Conclusion
Under the survey, the exploration and subsequent assessment of excavated probes KS 1 to KS 10 at the site of a reconstruction of the sports premises and soccer field demonstrated that changing groundwater levels occurred because of the character of soils in the active zones of the projected construction objects. The high potential risk of the soils being affected by lime treatment and subsequently saturated could, with a high probability, result in recurring saturation/drying cycles that lead to a loss in the strength characteristics of the already stabilized bedrock and an increase in its volume. All of this could result in the irrigation system becoming dysfunctional after constructing the SO 12 construction object. According to the assessment of risks arising from the submitted theoretical and practical knowledge, which were based on surveys conducted to date, the investor decided to change the manner of renovation by applying measures that would stabilize the bedrock and ensure the irrigation system installed at the soccer field would be fully functional. It was decided that cement stabilization using type I to V Portland cement as specified in the standard ASTM C150 would be applied. The corresponding mixing ratios were communicated to the contractor only after conclusion of the necessary tests.

References
[1] I. Akinje, Comparison Characterization of A-6(10) laterite soil stabilized with powermax cement and hydrated Lime separately. International journal of Engineering and technology, 5 (7): pp. 392 – 401, 2015.
[2] AS 1289.6.1.1. Methods of testing soils for engineering purposes – Soil strength and consolidation tests – Determination of the California Bearing Ratio of a soil – Standard laboratory method for a remoulded specimen. Australian Standards, Sydney, Australia; 2014.
[3] F. Tatsuoka, Compaction characteristics and physical properties of compacted soil controlled by the degree of saturation. In: Keynote lecture, deformation characteristics of geomaterials, Proc. of the 6th international conference on deformation characteristics of geomaterials, Buenos Aires; pp. 40–78., 2015.
[4] J. Suriol, A. Gens, and E.E. Alonso, Volumetric behaviour of a compacted soil upon wetting. In: Proc. 3rd Int. Conf. Unsat. Soils. Recife. Brasil, vol. 2; pp. 619–623., 2002.
[5] F. Tatsuoka, and A. Gomes Correia, Importance of controlling the degree of saturation in soil compaction linked to soil structure design. Transp Geotech https://doi.org/10.1016 /j. trgeo.2018.06.004, 2018.
[6] S.J.F. Gould, J. Kodikara, P. Rajeev, X-L Zhao, and S. Burn, A void ratio – water content – net stress model for environmentally stabilized expansive soils. Can Geotech J;48(6): pp. 867–77. https://doi.org/10.1139/t10-108, 2011.
[7] E.E. Alonso, J. Vaunat, and A. Gens, Modelling the mechanical behaviour of expansive clays. Eng Geol;54: pp. 173–83, 1999.
[8] J.K. Kodikara. New framework for volumetric constitutive behaviour of compacted unsaturated soils. Can Geotech J 49; pp. 1227–43, 2012.
[9] V. Kuchta, Standard ČSN 73 6133 Proposing and implementing ground objects, katal. number
84654, EAN kód 8590963846545, Český normalizační institut, 2010.

[10] J. Hrazdíl, ČSN 66 6635, Determining swelling capacity and absorption capacity. Proposal of unified methodological procedures and instrumental equipment for geotechnical tests, Český normalizační institut, kat. number 30342, EAN kód 8590963303420, 1984.

[11] C. Hogentogler, Essentials of soil compaction. Proc Highway Res Board: pp. 309–16, 1936.

[12] W.G. Holtz, and H.J. Gibbs, Engineering properties of expansive clays: Transactions. ASCE; 121:pp. 641–77, 1956.

[13] R. Proctor, Fundamental principles of soil compaction. Eng News Record; 111(9):pp. 245–8, 1933.

[14] AS 1289.6.1.1. Methods of testing soils for engineering purposes – Soil strength and consolidation tests – Determination of the California Bearing Ratio of a soil – Standard laboratory method for a remoulded specimen. Australian Standards, Sydney, Australia; 2014.

[15] ASTM. Standard test method for California bearing ratio (CBR) of laboratory compacted soils. American Society for Testing and Materials, West Conshohocken. California 2002

[16] AS 1289.5.1.1. Methods of testing soils for engineering purposes – Soil compaction and density tests – Determination of the dry density/moisture content relation of a soil using standard compaction effort. Australian Standards, Sydney, Australia; 2017.

[17] E.E. Alonso, J. Vaunat, and A. Gens, Modelling the mechanical behaviour of expansive clays. Eng Geol; 54: pp. 173–83, 1999.

[18] J.K. Kodikara, New framework for volumetric constitutive behaviour of compacted unsaturated soils. Can Geotech J 49: pp. 1227–43, 2012.

[19] E.C. Lawton, R.J. Fragaszy, and J.H. Hardcastle, Collapse of compacted clayey sand. Of Geotechnical Eng.;115(9): pp. 1252–67, 1989.

[20] E.E. Alonso, A. Gens, and A. Josa, A constitutive model for partially saturated soils. Ge’otechnique; 40(3): pp. 405–30, 1990.

[21] G.T. Houlsby. The work input to an unsaturated granular material. Géotechnique; 47(1): 193–6. https://doi.org/10.1680/geot.1997.47.1.193. 1997.

[22] S. Bhuvaneshwari, R.G. Robinson, and S.R. Gandhi, Resilient Modulus of Lime Treated Expansive Soil, Geotech Geol Eng 37:305–315,https://doi.org/10.1007/s10706-018-0610-z, Published online: 18 June 2018.

[23] Al-T. Asmaa, M. M. Disfanic, R. Evansa, A. Arulrajaha, and S. Horpibulsukd, Swell-Shrink Cycles of Lime Stabilized Expansive Subgrade, Advances in Transportation Geotechnics 3. The 3rd International Conference on Transportation Geotechnics, Procedia Engineering Volume 143, pp. 615–622, 2016.

[24] M. Mavroulidou, J. Gunn, and Z. Cabarkapa, Water Retention and Compressibility of a Lime-Treated, High Plasticity Clay, Geotech Geol Eng 31:1171–1185 DOI 10.1007/s10706-013-9642-6, 2013.

[25] F. Rogers, and S. Glendinning, Deep stabilization using lime, Technology and Engineering, Loughborough University Civil Building, 1996.

[26] A. A. Firoozi, C. G. Olgun, A. A. Firoozi, and M. S.Baghini, Fundamentals of soil stabilization, International journal of Geo-Engineering, https://doi.org/10.1186/s40703-017-0064-9, 2017

[27] J. Hauser, Standard ČSN 72 1006 Inspection of the soil and crushed stone compaction, Český normalizační institut, kat. number 97558, ICS 91.100.20; 93.020, 2015.

[28] F. Kresta, ARCADIS Geotechnika, a.s., TP 94, Soil treatment, technical conditions, Ministry of Transportation, Roadways Department, 2013.

[29] Prague geo-mechanical laboratory of ARCADIS Geotechnika, a.s., Laboratory test results protocols, 2015.