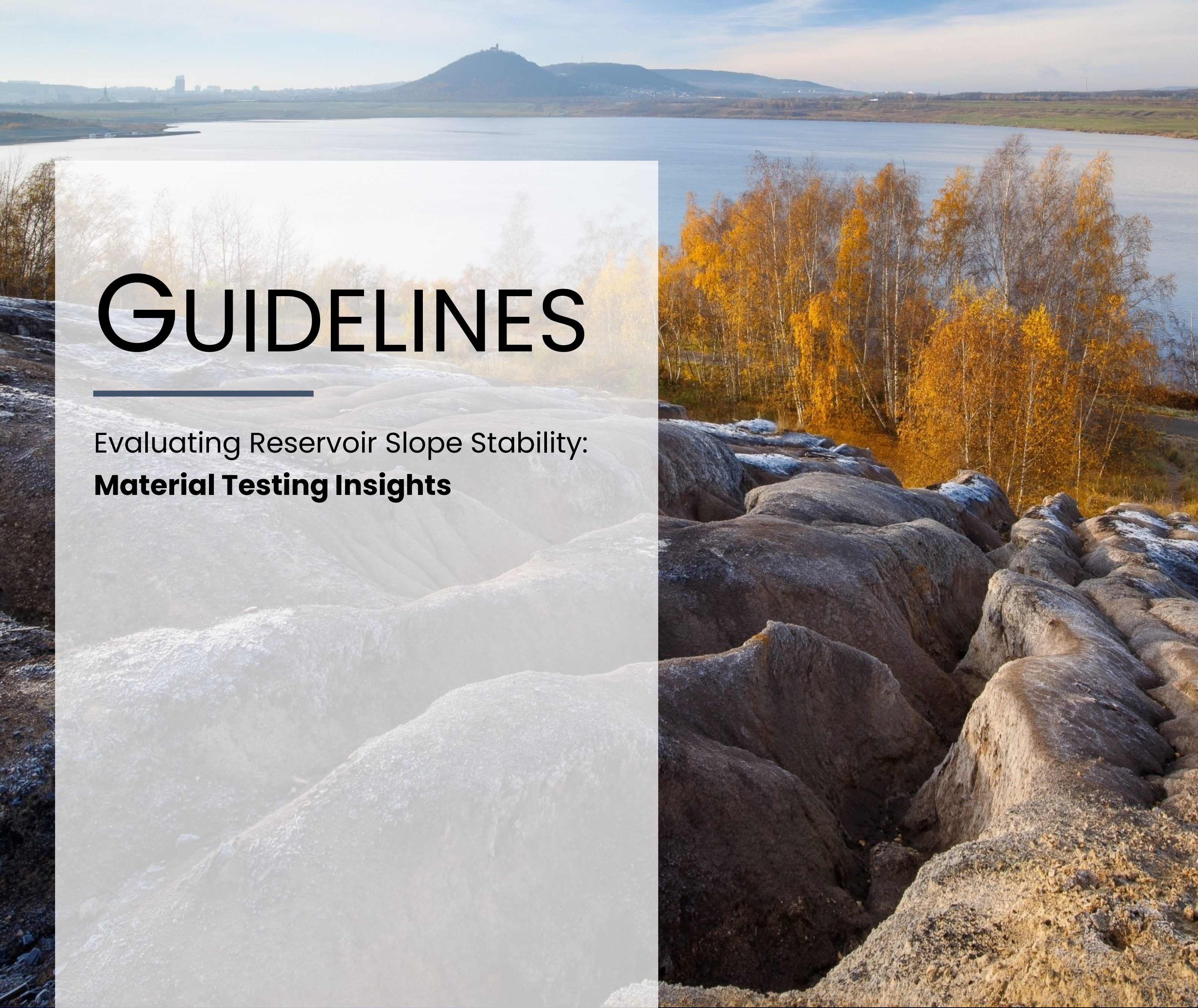

GUIDELINES

Evaluating Reservoir Slope Stability:
Material Testing Insights

RAFF

Risk Assessment
of Final pits
during Flooding

European
Commission

RFCS Project:
Risk Assessment of Final pits during Flooding (847299 RAFF)

Deliverable 2.1

GUIDELINES FOR APPROPRIATE TESTING ON MATERIALS THAT AFFECT THE STABILITY OF RESERVOIR SLOPES

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Preface

Reservoir slopes represent crucial components of infrastructure that are central to the management and distribution of water resources. Ensuring the stability of these slopes is not only vital for safeguarding lives, property, and the environment but also for protecting the interests of stakeholders and ensuring the seamless functioning of various civil operations. To address these multifaceted concerns, the "Guidelines for Testing Materials Affecting Reservoir Slope Stability" have been developed. These guidelines are intended to serve as a comprehensive reference manual for a wide range of individuals, including geotechnical engineers, engineering geologists, stakeholders, and civil operators.

The scope of these guidelines extends beyond the realm of geotechnical professionals. They have been designed to cater to a diverse audience of stakeholders and civil operators who are associated with reservoir projects and related civil infrastructure. The guidelines offer a holistic approach to understanding and managing reservoir slope stability, addressing a broad range of geological conditions and project requirements. Whether you are an engineer, a project investor, a regulator, or anyone with a vested interest in reservoirs, these guidelines are a valuable resource for you.

Geotechnical engineering plays a pivotal role in safeguarding reservoir slopes, a role that affects everyone connected to these structures. This section highlights the central role of geotechnical engineers and engineering geologists in the evaluation, analysis, and design of reservoir slopes. It emphasizes that geotechnical professionals act as critical partners in ensuring the safety and stability of reservoir slopes and, by extension, the well-being of stakeholders and the smooth functioning of civil operations

Definitions and Terminology

Recognizing the diverse audience for these guidelines, we provide clear and concise definitions of key terms and concepts related to geotechnical engineering, slope stability, and materials testing. This section is indispensable for establishing a common language and understanding, ensuring that all stakeholders and civil operators can actively engage in the discussions and decisions pertaining to reservoir slope stability.

- *USC, Unified Soil Classification*
- *CPT, Cone Penetration Test*
- *GC, Geotechnical Category*
- *QA, Quality Assurance*
- *CAD, Computer-Aided Design*
- *BIM, Building Information Modeling*
- *FR, Friction Ratio*
- *DEM, Digital Elevation Model*
- *GIS, Geographic Information System*
- *LiDAR. Light Detection And Ranging*
- *V:H, Vertical: Horizontal*
- *FoS, Factor of Safety*
- *FEM, Finite Element Method*

About the RAFF project

Project *Risk Assessment of Final pits during Flooding* (RAFF) aims to research issues related to pit lakes, which is one of the most common uses of post exploitation voids. The RAFF projecr is supported by Europian Research Fund for Coal and Steel (RFCS) and an extensive team of experts from Poltegor-Instytut, Central Mining Institute, CTL MACZKI-BÓR (POL), Brown Coal Reseach Institute, DIAMO (CZE), The French National Institute for Industrial Environment and Risks (FRA), Technical University of Crete, The Centre for Research & Technology - Hellas (GRE), University of Nottigham (UK), Univeseity of Petrosani, Complexul Energetic Oltenia (ROM) and Subterra Ingenieria Sl (ESP) are involved in its solution.

RAFF is the first RFCS project that deals with the geotechnical risk associated with flooding open-pit coal mines in Europe. Many scientific and operational questions will be answered in the project. Up to now, in Europe, there is no precedent for the creation of a pit lake of a brown coal open pit mine of the volume c.a. 1,5 billion m^3. There are many examples of flooded smaller final pits and in some of these, during the process of filling with water, serious geotechnical problems have been encountered. It is expected that during reclamation of open pits of volume 1,5 billion m^3 the scale of geotechnical problems will be significant and may impede the process of filling the voids with water.

The project will allow for further development of technological methods to minimize the amount of earth works necessary during the preparation of the final reservoir slopes, and to decrease the costs and duration of reclamation works. The project aims to produce a coherent system of risk assessment for post exploitation of open pits during flooding with water. Risk assessments will be prepared in final stages of the project for lignite open cast mines scheduled for flooding.

The project aims to research issues in areas related to pit lakes as the most common use of post exploitation voids. The main planned achievements in the project are connected with the creation of a comprehensive model that can be used for risk assessment purposes.

The model will be based on the three-step development (gathering information and identification of hazards, creation and validation of sub-models, creation of a comprehensive model). It will describe geotechnical, geological and hydrogeological conditions and it will be the basis for risk evaluation.

Preparation of comprehensive models of pit lakes will allow for development of technological methods to minimize the amount of earth works necessary during preparation of the final reservoir slopes.

Chapter I

Introduction – geotechnical issue in lake flooding process

Within the context of reservoir slope stability, this section offers guidelines delineating the expected levels of investigation and design efforts across various stages of project development. It's important to recognize that the necessary level of effort can fluctuate significantly, contingent upon the intricacies embedded within the geotechnical model and the risk mitigation imperatives stipulated by the project owner.
The geotechnical model forms the foundational framework for open pit mine design and thus even for pit lakes design. This model encompasses the essential components of geotechnical engineering: geology, hydrogeology, material properties, and the structural model, which, in the context of mining, is represented by the mining model (refer to Table 1).

Table 1 Basic overview of the parts that enter the geotechnical model (modified according to: Read a Stacey, 2009).

Type of model	***Key data***	***Type od study***
Geological model	Depth, thickenss and compostion of strata	Planning/preliminray investigation, definition of needs, desktop study
Mining model	Extentnt and character of undergrround excavation, final shape of residual pit, depth and thickness of dumps	Planning/preliminray investigation, definition of needs, desktop study, post-mining hazard mitigation
Hydrogeological model	piezometric levels, hydrogeol. units, rock permeability, distribution of pore pressures, groundwater flow regime, water quality	Water balance of lakes, monitoring, inspections, detailed site investigation, water management
Material model	Soil classification, material model, physical soil properties	Monitoring, development studies

Type of model	***Key data***	***Type od study***
Geotechnical model	Distribution soils classes, strenght parametres, anisotropy and weakness, hydrogeological factors	Detailed site investigation, monitoring, gevelopment studies

The geotechnical model, together with its four components, the geological, structural, rock mass and hydrogeological models, is the cornerstone of residual pit slope design. As illustrated in Table 1, the model must be in place before the successive steps of setting up the geotechnical domains, allocating design sectors and preparing the final slope designs can commence. Populating the geotechnical model with relevant field data requires not only keen observation and attention to detail, but also strict adherence to field data gathering protocols from day one in the development of the project. Those who are responsible for project site investigations must be aware of the mainstream technologies available to them, and how and when they should be applied to provide a functional engineering classification of the rock mass for slope design purposes. Irrespective of the specific project stage, the residual slope design process fundamentally encompasses the following sequential steps:

- **Formulation of a Geotechnical Model for the Pit Area**: The process commences with the construction of a comprehensive geotechnical model that encapsulates the geological, hydrogeological, material, and structural elements essential to the project.
- **Population of the Model with Relevant Data**: The model is then enriched with pertinent data, ensuring that it accurately reflects the unique geological and geotechnical characteristics of the pit area.
- **Division of the Model into Geotechnical Domains**: The model is subsequently dissected into distinct geotechnical domains, each characterized by a set of geotechnical attributes.
- **Subdivision of the Domains into Design Sectors**: Within these domains, further subdivision occurs, creating distinct design sectors that enable a more precise and detailed approach to slope design.
- **Design of Slope Elements in Respective Sectors**: In these defined sectors, the actual design of slope elements takes place, adhering to the specific geotechnical requirements and constraints.
- **Assessment of Slope Stability**: The resulting slopes are rigorously assessed for stability, scrutinizing them against project-specific acceptance criteria. This step is pivotal in ensuring the safety and longevity of the slopes.
- **Definition of Implementation and Monitoring Requirements**: Finally, the design process concludes with the articulation of implementation plans and monitoring prerequisites necessary for translating the designs into tangible, operational structures.

These steps together constitute the foundational process of open pit mine slope design, a process that demands a holistic grasp of geological intricacies, geotechnical nuances, and pragmatic design execution. It is through these rigorous procedures that reservoir slope stability is not only envisioned but, more importantly, realized, safeguarding lives, property, and the environment.

1.1 Geotechnical Engagement

- Geotechnical involvement is a critical component in ensuring the stability and success of projects related to pit lakes, residual pit lakes, or final pit lakes. This involvement encompasses two distinct approaches for acquiring geotechnical data, each of which carries specific implications for project design and implementation:
    - Traditional Approach: In this approach, ground conditions are viewed as a parameter to be considered after determining the structure or development's design location and configuration. Subsequently, relevant ground conditions are obtained to design in accordance with or against these predetermined parameters. While this approach has been conventionally practiced, it may not always be the most cost-effective option.
    - Recommended Approach: The recommended approach, especially when aiming to minimize overall project costs, involves integrating geotechnical considerations into the project from its inception. In this method, the design, layout, and configuration are fundamentally influenced by the ground conditions. It emphasizes proactive planning and a dynamic, adaptive design process that takes the unique geotechnical characteristics of pit lakes, residual pit lakes, or final pit lakes into account right from the outset.
- Geotechnical involvement should occur throughout the life of the residual-pit lake. The input varies depending on phase of project.
- The phasing of the investigation provides the benefit of improved quality and relevance of the geotechnical data to the project.

1.2 Geotechnical requirements for the different flooding phases

- A strategic phasing of geotechnical study is vital to maximize its benefits, which are approximately distributed evenly across the entire project's lifecycle. Traditionally, in the context of post-mining sites, the predominant approach has been to allocate the bulk of geotechnical efforts and associated expenses (>90 %) to the investigation and mining phases. However, it's crucial to recognize that a more balanced distribution of effort throughout the project's life can yield considerable advantages.

- The comprehensive mining-investigation phase may result in the redundancy of certain preliminary investigation data. Acknowledging this, embracing iterative processes in geotechnical investigations becomes invaluable for refining efforts and maintaining data relevance during the post-mining stage.
- The geotechnical input at any stage has a different type of benefit. The Quality Assurance (QA) benefit during flooding stage, is as important as optimising the final lake parameters in the desktop study. The volume of testing as part of QA, may be significant.
- The observational approach during flooding can lead to more efficient application of factors of safety, potentially reducing overall project costs. This approach is particularly relevant in areas of lesser criticality, allowing a nuanced consideration of safety factors. In critical areas, the observational approach may still be necessary, even without reducing safety factors, to ensure the utmost stability and safety of the residual pit.
- By optimizing geotechnical study phases, distributing efforts more evenly, and embracing an iterative approach, projects related to pit lakes, residual pit lakes, or final pit lakes can achieve enhanced cost-effectiveness, safety, and long-term sustainability while aligning with the specific geotechnical nuances of these water bodies.

1.3 Planning prior to ground truthing

- Prepare preliminary site investigation and test location plans prior to any ground truthing. This may need to be adjusted on site.

Table 2 Geotechnical category (GC) of investigation

Geotechnical category	*GC1*	*GC2*	*GC3*
1. Nature and size of construction	Small & relatively simple – conventional loadings.	Conventional structures – no abnormal loadings.	Large or unusual structures.
2. Surroundings	No risk of damage to neighbouring buildings, utilities, etc.	Risk of damage to neighbouring structures	Extreme risk to neighbouring structures.
3. Ground conditions	Straightforward. Does not apply to refuse, uncompacted fill, loose or highly compressible soils.	Routine procedures for field and laboratory testing.	Specialist testing.
4. Ground water conditions	No excavation below water table required.	Below water table. Lasting damage cannot be caused without prior warning	Extremely permeable layers.
5. Seismicity	Non Seismic	Low seismicity	High Seismic areas.
6. Cost of project	<$0.5M (Aus – 2005)		>$50M (Aus – 2005)
7. SI Cost as % of capital cost	0.1%–0.5%	0.25%–1%	0.5%–2%
8. Type of study	Qualitative investigation may be adequate.	Quantitative geotechnical studies.	Two stage investigation required.
9. Minimum level of expertise	Graduate civil engineer or engineering geologist under supervision by an experienced geotechnical specialist.	Experienced Geotechnical engineer/ Engineering geologist.	Specialist geotechnical Engineer with relevant experience. Engineering geologist to work with specialist.

Case studies: Insights from RAFF Project Best Practice

Within the realm of material testing and its profound impact on the stability of reservoir slopes, understanding and implementing good practices is of paramount importance. Best practices, often established as standard guidelines with a proven track record of yielding favorable outcomes, play a pivotal role in ensuring the integrity and safety of these critical structures.

Within the scope of the RAFF project, a wealth of knowledge has been acquired through an in-depth analysis of five case study areas across Europe. In this chapter, we embark on an exploration of exemplary best practices in material testing, unveiling a trove of wisdom and expertise that can be effectively applied to enhance the stability of reservoir slopes.

Figure 1 Open-pit mine Ležáky – Most, today´s lake Most (1970s).

Figure 2 Lake Most (Czech Republic), an example of a good practice of hydric reclamation and successful transition of post-mining area.

Chapter II

Site investigation

2.1 Terrain evaluation

- Terrain evaluation is particularly useful in large post mining lakes with high and steep slopes.
- This involves an extensive desktop study of aerial photos, geology maps, topography, etc, before any need for extensive ground truthing. Phasing of the study is important here.
- In the context of the majority of mining sites, we commonly identify two fundamental categories of slopes that shape the lake's banks:
    - ***open-pit (in-situ) slopes****: created by shaping the original relief in the soils in-situ.*
    - ***dump slopes****: resulting from excavation of the overburden soils and their reloading into the dump bodies, which created a completely new relief with a chaotic and incoherent composition of soils, dependent purely on the method of foundation of the dump.*

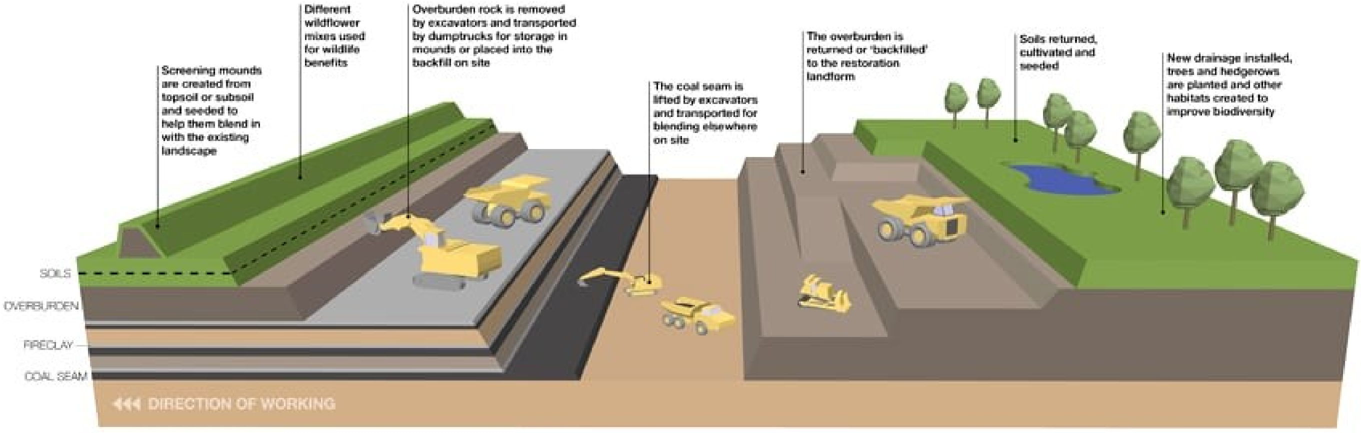

Figure 3 Open-pit and dump slopes.

- Both types represent two specific types of terrain, the uniqueness of which must be specifically taken into account in all subsequent steps.
- Terrain evaluation is particularly useful in large post mining lakes with high and steep slopes.
- This involves an extensive desktop study of aerial photos, geology maps, mining blueprints, topography maps, etc, before any need for extensive ground truthing. Phasing of the study is important here.
- Historically, the mapped data were recorded by hand on paper sheets and/or field notebooks, but advances in electronic software and hardware mean that this is increasingly replaced by electronic data recording directly into handheld tablets and/or laptop computers. Both systems have their merits, but the electronic system has the advantage that it eliminates the tedious transfer of paper data into an electronic format. It produces data that can be almost instantly transmitted for further analysis and checking in Autocad or similar systems.
- This data can be of varying quality, from archival map sheets, to pre-CAD models, to modern BIM architecture.
- Experience says that the quality is rather worse in post-mining locations

Table 2 Terrain evaluation considerations.

Consideration	***Terrain evaluation***	***Comments***
Accuracy of data scale	Mining blueprints Geology maps Aerial photos Orthophotos	Archives of mining comapnies. geological surveys, „coal reserch institutes“, map repositories, etc. The maps are likely to be at different accuracy scales and digitization and veri fication is recommended.
Development	Grades Size	Construction/Access as well as long term.
Geology	Lithology Structure	Rock/soil type. Dip/orientation with respect to proposed slope.
Drainage	Surface Ground Erosion Catchment area	Hydrology considerations. Also affected by vegetation and land cover.
Slope	Transverse batters Longitudinal grades	Affects horizontal resumptions/stability measure required.
Height	Above flood levels Cuttings	Affects vertical alignments, which could mean a horizontal alignment shift if significant cut/fill/stability issues.
Aspect of slope	Orientation	With respect to development as well as true north. southern aspect wetter in southern hemisphere (Greater landslide potential).
Land use	Existing proposed	Roads, rails, services, and developments. Environmental considerations. Adjacent affects considered here.
Vegetation	Type, intensity	Forested, agricultural, barren

- Optimally, the collection, sorting and eventual digitization of this data will enable the creation of a digital terrain model (DEM) which is the basic input for design, advanced GIS analysis and geotechnical assessment.
- Specificity of the evaluation of residual pits is the need to know the shape of the terrain even below the surface, the DEM must be supplemented with a bathymetric model. Especially for older pits, it can be a problem to obtain it, as the data from older maps are often no longer up-to-date and do not correspond to reality.

2.2 Mapping phase

- Field mapping is fundamental to all the activities pursued by the teams responsible for designing and managing the pit slopes.
- It includes regional and minescale surface mapping during development prior to mining and bench mapping once mining has commenced.
- Structural data are a key input for kinematic, limit equilibrium and numerical slope design analyses. Gathering these data and estimating how the orientation and spatial distribution characteristics of dumps, landslides, cracks and faults vary across the residual pit slopes is thus one of the most important modelling activities.
- Mapping techniques used for detailed structural data gathering usually fall into one of the following three types:
    - *line mapping*
    - *window (cell) mapping*
    - *digital imaging*
- Preferably it should be carried out by properly trained geologists, engineering geologists, geological engineers or specialist geotechnicians, assisted by specialists from other disciplines as needed,

Case study: (Post)Mining and Digital Elevation Model as a basic inmput

The lake Most (Czech Republic) was selected, as a case study, to perform a largescale stability analyses based on in-situ observation, a 3D geometric model and a large-scale numerical model. The lake was established in the former pen-pit mine and the its dumps. The flooding began in October 2008 and was finished in September 2014 (surface = 309 ha, volume = 70 hm^3).

Terrestrial mapping A 3D numerical model is time consuming, however the study area must, at least, integrate the areas of ground movement already identified and must consider the heights of geometric anomalies (valleys or hills). The maximum depth of the lake is 75 m, the highest hill in the immediate vicinity of the lake has a height of 70 m and the boundaries of the 3D model were positioned at more than 6 times 70 + 75 m from the shores of lake Most.
Additionally, a large LiDAR campaign was carried out in 2019. The final data point cloud was used to create the digital terrain model to build the 3D volumetric mesh.

Figure 4 In September 2019, Ineris performed a LiDar 3D survey of the shores of the artificial Most lake. Seventy-five metres deep, this lake was created by flooding an old open-pit lignite mine.

Bathymetry survey The CAPEREA measuring ship - equipped with modern technologies for measuring a continuous digital model of the bottom, monitoring sediments and scanning underwater structures. It was created according to the own design of the company VARS BRNO a.s. and its parameters bring qualitatively new possibilities.
A multibeam interferometric sonar was used for measurments to depths of about 45 m, while singlebeam parametric sonar was used for greather depths. Measurements grid was in distance of 0.5 m.

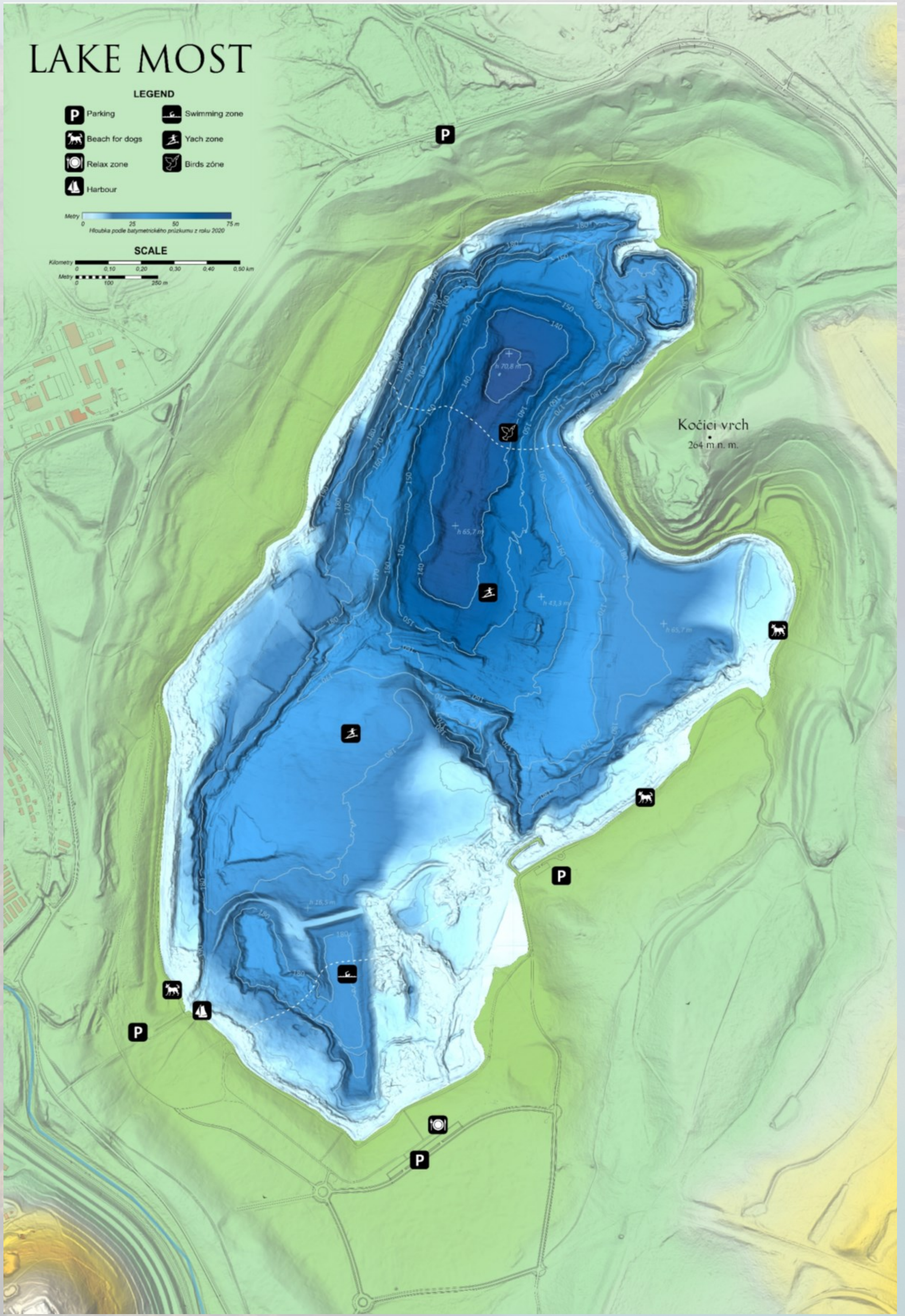

Figure 5 Final Digital Elevation Model (DEM) combining terrestrial LiDAR mesh with bathymetric sonar mesh was the first and key step in creation of geotechnical model of the site.

2.3 Development grades

- When planning the future slopes of the pit lake, it is crucial to bear in mind that different types of development necessitate different grades.
- This entails having a clear understanding of the intended land use before shaping the slopes.
- The required grades for typical development categories are outlined in the table below.

Table 3 Grades required for development (part from Cooke and Doornkamp, 1990).

Development type	*Grade %*	*Deg. ◦*	*Vert. : Horiz.*
International airport runways	1	0.6	1V : 100H
Main line passenger and freight rail transport Local aerodrome runways	2	1.2	1V : 50H
To minimize drainage problems for site development Acceptable for playgrounds			
Major roads	4	2.3	1V : 25H
Agricultural machinery for weeding, seeding Soil erosion begins to become a problem Land development (construction) becomes difficult	5	2.9	1V : 20H
Industrial roads Upper limit for playgrounds	6	3.4	1V : 17H
Housing roads Acceptable for camp and picnic areas	8	4.6	1V : 12.5H
Absolute maximum for railways	9	5.1	1V : 11.1H
Heavy agricultural machinery Large scale industrial development	10	5.7	1V : 10.0H
Site development Standard wheel tractor Acceptable for recreational paths and trails Upper limit for camp and picnic areas	15	8.5	1V : 6.7H
Housing site development	20	11.3	1V : 5.0H
Lot driveways Upper limit for recreational paths and trails Typical limit for rollers to compact	25	14.0	1V : 4.0H
Benching into slopes required	33	18.4	1V : 3.0H
Planting on slopes become difficult without mesh/benches	50	26.6	1V : 2.0H

2.4 Development procedures

- Slope is a fundamental factor in geotechnical engineering and earth sciences because it has a direct and significant influence on the stability and behavior of geological formations and engineered structures.
- Understanding the relationship between slope and stability is crucial for making informed decisions in construction, mining, environmental management.
- Of course, stability also depends on other factors such as geology, aspect, drainage, etc., but here we describe procedures based only on slope gradients.

Table 4 Development procedures based on slope gradients

Vert. : Horiz.	*Deg. ◦*	*Grade %*	*Slope risk*	*Comments on site development.*
>1V : 2H	>27	>50	Very high	Not recommended for development
1V : 2H to 1V : 4H	27 to 14	50 to 25	High	Slope stability assessment report
1V : 4H to 1V : 8H	14 to 7	25 to 12.5	Moderate	Standard procedures apply
<1V : 8H	<7	<12.5	Low	Commercially attractive

2.5 Landslide classification

- Landslides pose significant challenges during mining operations and remain a substantial risk even after mining activities cease. This is primarily due to alterations in the natural stability of the terrain brought about by excavation.
- Post-mining activities should prioritize effective rehabilitation and stabilization efforts to minimize the long-term risks associated with landslides.
- For these measures, it is crucial to understand some fundamental characteristics of landslides. For instance, it's important to recognize that varying slopes have different potentials for landslides (Table 6), and that different velocities (Figure 6) of slope movement can lead to varying levels of final damage.

Table 5 Typical landslide dimensions in soils (Skempton and Hutchinson, 1969).

Landslide type	***Depth/Length ratio (%)***	***Slope inclination lower limit (Deg. ◦)***
Debris slides, avalanches	5 – 10	22 – 38
Slumps	15 – 30	8 – 16
Flows	0.5 – 3.0	3 – 20

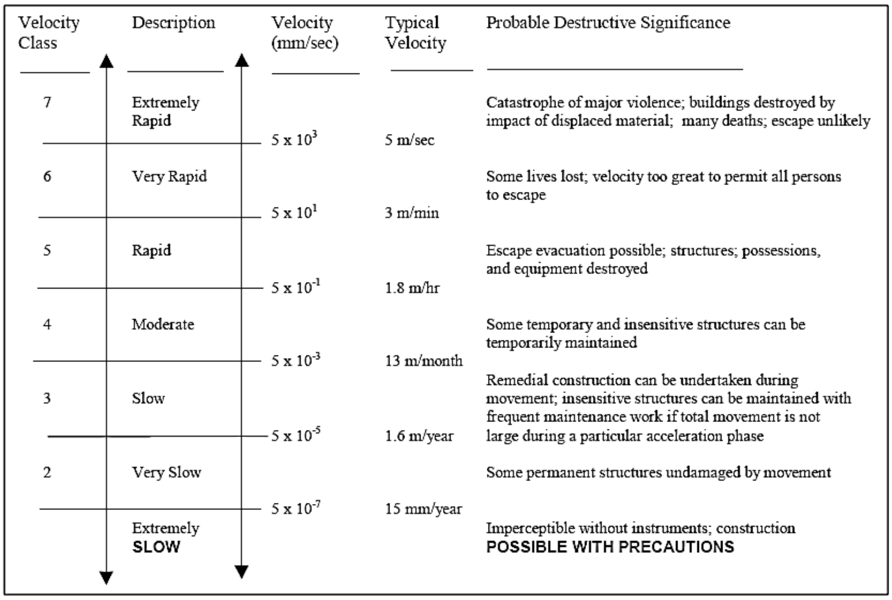

Figure 6 Landslide velocity scale (Cruden and Varnes, 1996).

2.6 Slope erodibility

- Erosion exerts a significant impact on slope stability. It can lead to the weakening of subsoil through topsoil drift, heightening the risk of slope failures. Additionally, erosion may induce changes in slope characteristics, resulting in the formation of unstable overhangs, further destabilizing the slope.
- Various factors, including slope gradient Table 7, vegetation cover, and soil type Table 8, play pivotal roles in determining slope erodibility.

Table 6 Slope erodibility with grades.

Erosion potential	***Grade %***
High	>10 %
Moderate	10−5 %
Low	<5 %

- The ability of a soil to reduce erosion depends on its compactness.
- The soil size (gradation characteristics), plasticity and cohesiveness also affect its erodibility.
- Fine to medium sand and silts are the most erodible, especially if uniformly graded.

Table 7 Typical erosion velocities based on material.

Soil type	***Grain size***	***Erosion velocity (m/s) particle size only***
Cobbles, cemented gravels, conglomerate. Soft sedimentary rock	>60 mm	3.0
Gravels (coarse)	20 mm to 60 mm	2.0
Gravels (medium)	6 mm to 20 mm	1.0
Gravels (fine)	2 mm to 6 mm	0.5
Sands (coarse)	0.6 mm to 2 mm	0.25
Sands (medium)	0.2 mm to 0.6 mm	0.15
Sands (coarse)	0.06 mm to 0.2 mm	0.25
Silts (coarse to medium)	0.006 mm to 0.06 mm	0.5
Silts (fine)	0.002 mm to 0.006 mm	1.0
Clays	<0.002 mm	3.0

Chapter III

Soil classification

Soil classification plays a pivotal role in safeguarding the stability, safety, and sustainability of development projects, particularly in regions marked by residual excavation sites. This classification system not only informs decision-making across various domains but also holds paramount importance in evaluating the efficacy of remediation strategies. A key aspect of this evaluation lies in the ability to discern between ***in-situ*** and **dumped** soils.

- Different types of soil have varying levels of stability. Understanding the soil's classification helps in assessing the potential for landslides, erosion, and other risks in residual pits.
- Soil classification informs the design of foundations for structures. Different types of soil have different bearing capacities and settlement characteristics, which are critical considerations in construction.
- Soil permeability affects drainage. Knowing the soil type helps in designing effective drainage systems to mitigate water-related risks like flooding and saturation.
- Soil classification informs decisions about suitable land uses. For example, expansive clays may not be suitable for certain types of construction due to their swelling and shrinking properties.

3.1 Soil borehole record

- A soil borehole record, also known as a borehole log or a geotechnical borehole record, is a detailed documentation of subsurface soil and rock conditions at a specific location.
- The record contains information obtained from drilling a borehole or excavation and is described using the following data:
  - *Drilling Information*
  - *Soil Type*
  - *Unified Soil Classification (USC) Symbol*
  - *Colour*
  - *Plasticity/Particle Description*
  - *Structure*
  - *Consistency (Strength)*
  - *Moisture Condition*
  - *Origin*
  - *Water Level*

Table 8 Borelog -based on the visual examination, description of samples, laboratory test and driller´s daily report.

Drilling information				***Soil description***						***Field testing***				***Strata information***			
Depth	Drilling method	Water level	Sample type	USC symbol/soil type	Colour	Plasticity/particle description	Structure	Consistency	Moisture	Standard penetration type	Shear vane test	Pocket penetrometer	Dynamic cone penetrometer	Origin	Graphic log	Elevation	Depth

- Identification of the Test log is also required with the following data:
    - *Client*
    - *Project Description*
    - *Project Location.*
    - *Project Number*
    - *Sheet No. – of –*
    - *Reference: Easting, Northing, Elevation, Inclination.*
    - *Date started and completed.*
    - *Geomechanical details only. Environmental details not covered.*

3.2 Borehole record in the field

- In the field, the borehole record takes on various formats. The example provided above serves as a template for the final log that designers use. The sequence, level of detail, and relevance of field data entry may vary.
- There are several advantages to employing different borehole templates in the field:
    - *A dedicated field log provides ample space for capturing pertinent field information, including both quality-related data and administrative details that may not be pertinent to the designer's final version.*
    - *Design engineers often require a distinct sequence of information and different details compared to those in the field log. For instance, the field log may encompass certain administrative particulars for billing purposes, which may not be relevant to the designer.*
    - *Designers typically review borelog information from right to left, initially focusing on key issues on the right-hand side before examining details to the left. Conversely, field supervisors log information from left to right, progressively adding more details as they move across the page.*
    - *Given these preferences, a landscape layout is better suited for recording field logs, while a portrait layout is more conducive to creating the final report.*
- However, many individuals prefer the field log to mirror the format of the final produced borehole record.

Table 9 Borehole record in the field.

Drilling information				***Sampling and testing***			***Soil description***						***Comments and origin***
Depth	Drilling method	Time of drilling	Water level	Sample type	Amount of recovery	Field test – type (PP< SPT, SV, PP, DCP)	USC symbol/soil type	Colour	Plasticity/particle description	Structure	Consistency	Moisture	

3.3 Drilling information

- Drilling information refers to data and details gathered during the process of drilling.
- The following table shows typical features, which may vary slightly depending on the consultant.

Table 10 Borehole record in the field.

Symbol	***Equipment***
BH	Backhoe bucket (rubber tyred machine)
EX	Excavator bucket (tracked machine)
HA	Hand auger
AV	Auger drilling with steel “V” bit
AT	Auger drilling with tungsten carbide (TC) bit
HOA	Hollow auger
R	Rotary drilling with flushing of cuttings using:
RA	*– air circulation*
RM	*– bentonite or polymer mud circulation*
RC	*– water circulation*
	Support using:
C	*– casing*
M	*– mud*
W	*– water*

3.4 Water level

- Underscoring the significance of this measurement on all sites is crucial.
- Additionally, the weather and rainfall conditions during the investigation period hold relevance.

Table 11 Water level.

Symbol	***Water measurement***
$\underline{\nabla}$	Measurement standing water level and date
∇	Water noted
▷	Water inflow
◁	Water/drilling fluid los

3.5 Soil type

- As mentioned in the introduction, soil type is one of the most important parameters in soil profile assessment.
- Individual particle sizes <0.075 mm (silts and clays), are indistinguishable by the eye alone.
- Some codes use the 60 µm instead of the 75 µm, which is consistent with the numerical values of the other particle sizes.

Table 12 Soil type and particle size.

Major Divisions		***Symbols***	***Subdivision***	***Particle size***
Coarse grained soils (more than half of material is larger than 0.075 mm).	Boulders			> 200 mm
	Cobbles			60 mm–200 mm
	Gravels (more than half of coarse fraction is larger than 2 mm).	G	Coarse	20 mm–60 mm
			Medium	6 mm–20 mm
			Fine	2 mm–6 mm
	Sands (more than half of coarsemfraction is smaller than 2 mm).	S	Coarse	0.6 mm–2 mm
			Medium	0.2 mm–0.6 mm
			Fine	75 mm–0.2 mm
Fine grained soils (more than half of material is smaller than 0.075 mm).	Silts	M	High/low plasticity	< 75 m
	Clays	C		
	Organic	O		

3.6 Unified soil classification

- Field assessments can provide valuable initial information about soil characteristics. However, laboratory testing serves as a means to verify and supplement these observations, ensuring a comprehensive understanding.
- Laboratory analyses are conducted according to standardized procedures and methodologies, ensuring consistency and comparability of results. This allows meaningful comparisons between different sites or projects.
- Lab testing becomes crucial in cases where the distinction is subtle, such as between silty sand and sandy silt.
- In case of disputes or claims related to construction projects, having validated soil classification through laboratory testing provides a solid basis for resolving any legal issues.

Table 13 Unified soil classification (USC) group symbols.

Soil type	***Description***	***USC symbol***
Gravels	Well graded	GW
	Poorly graded	GP
	Silty	GM
	Clayey	GC
Sands	Well graded	SW
	Poorly graded	SP
	Silty	SM
Inorganic silts	Clayey	SC
	Low plasticity	ML
	High plasticity	MH
Inorganic clays	Low plasticity	CL
	High plasticity	CH
Organic	with silts/clays of low plasticity	OL
	with silts/clays of high plasticity	OH
Peat	Highly organic soils	P

- For soils with medium plasticity, a combination of symbols like CL/CH or CI (Intermediate) is commonly employed.

3.7 Soil plasticity

- It influences the selection of appropriate foundation types and helps in determining the depth and dimensions of foundations for structures.
- Understanding soil plasticity is crucial in assessing the stability of slopes and embankments, helping to prevent landslides and failures.¨

Table 14 Soil plasticity.

Term	***Symbol***	***Field assessmen***
Non plastic	–	Falls apart in hand
Low plasticity	L	Cannot be rolled into (3 mm) threads when moist
Medium plasticity	L/H	Can be rolled into threads Shows some shrinkage on drying
High plasticity	H	when moist. Considerable shrinkage on drying.
		Greasy to touch. Cracks in dry material

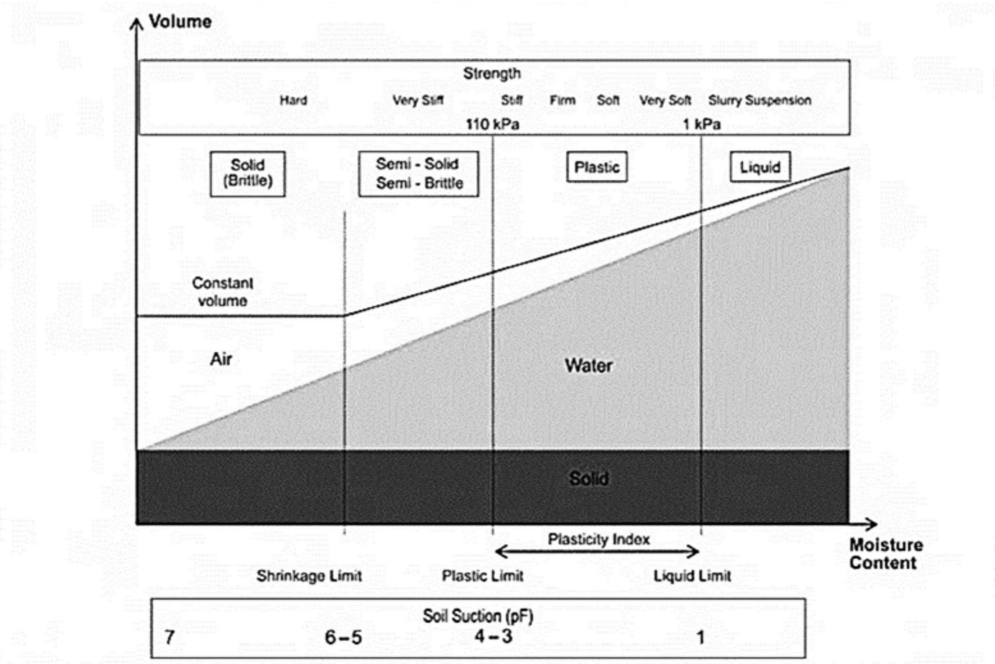

Figure 7 Consistency limits.

3.8 Atterberg limits

- The Atterberg limits are a set of three specific moisture content levels that define the behavior of fine-grained soils (such as clays and silts) under varying states of consistency.
- These tests are performed on the % passing the 425 micron sieve. This % should be reported. There are examples of "rock" sites having a high PI, when 90 % of the sample has been discarded in the test.

Table 15 Atterberg limits.

Symbol	***Description***	***Comments***
LL	Liquid limit – minimum moisture content at which a soil will flow under its own weight.	Cone penetrometer test or casagrande apparatus.
PL	Plastic limit – Minimum moisture content at which a 3 mm thread of soil can be rolled with the hand without breaking up.	Test
SL	Shrinkage limit – Maximum moisture content at which a further decrease of moisture content does not cause a decrease in volume of the soils.	Test
PI	Plasticity Index = LL–PL.	Derived from other tests.
LS	Linear shrinkage is the minimum moisture content for soil to be mouldable.	Test. Used where difficult to establish PL and LL. PI = 2.13 LS.

Case study: Soil types on the shore

As part of the solution to the pedological problem of lake Most, regular monitoring of the properties of the soil profile of 9 selected probes was carried out, contamination with risky trace elements was detected in areas of interest, detailed mapping of the area was carried out, soil properties of soils from another 11 probes in areas of interest from the point of view of the occurrence of remarkable flora were determined.

Compared to the properties of the soils of the old reclaimed wastedump and the properties of the growing soils taken from the well near the Ore Mountains, the properties of the soils of Lake Most are relatively the most favorable, which is due to the youth of the reclamation and the sufficient distance from the Ore Mountains. The shores and slopes of Lake Most were divided into 3 main pedological areas, small isolated phytotoxic areas (totaling less than 1 % of the area of the entire area) and small slips and landslides. A comprehensive basic pedological map is shown in Figure 4.

The first area (approx. 80 % of the bank) consists of kaolinitic illitic brown clay suitable for recultivation. These are mostly original soils, the western slopes are partially made up of the soils of the Střimice dump. The soils are fine-grained, have a favorable mineralogical composition, a neutral to weakly alkaline soil reaction, lower calcite and oxidizable carbon contents, good reserves of acceptable nutrients and good sorption capabilities. It is a soil very suitable for recultivation.
Very small areas without vegetation appear locally in the area (their occurrence has been mapped). The cause is usually the occurrence of phytotoxic acid soils of the coal seam, less often the occurrence of hard, siderite-enriched soils.

The second area (approx. 5 % of the shore) consists of a former aggregate (phonolite) quarry. It consists of variously weathered whitish phonolites, from practically solid gravel to kaolinically weathered soil. Depending on the degree of weathering, these soils are extremely coarse-grained to slightly fine-grained, have a rather unfavorable mineralogical composition, weakly alkaline soil reaction, minimal calcite and oxidizable carbon contents, minimal reserves of acceptable nutrients and poor sorption capacity. In terms of recultivation, these soils are completely unsuitable, from the point of view of the landscape, the former quarry is an interesting phenomenon, which is recommended to be left in controlled succession.

The third area (approx. 15 % of the bank) is the steep slope of the Pařidel lobe. Similar to the case of area 1, the soils here are composed of kaolinitic-illitic clays suitable for recultivation. They are fine-grained to medium-grained, have a favorable mineralogical composition, neutral to slightly alkaline soil reaction, lower calcite content and lower to medium oxidizable carbon content, good reserves of acceptable nutrients and good sorption capabilities.

Due to the danger of erosion and landslides, organic materials from the former Štětí paper mill - bark from debarking and cellulose sludge - were applied here in the past as part of technical reclamation. In the course of pedological mapping, up to 15 small phytotoxic areas occurring in individual pedological areas were found. Occurrences are plotted in the soil map.

It should be noted that due to long-term remediation and recultivation works, the number of phytotoxic areas and especially slippery areas is gradually changing. As part of pedological research, considerable space was devoted to phytotoxic areas due to their properties, but from the point of view of reclamation, their significance is minimal, as they do not even make up 1 % of the assessed area.

Table 16 Soil characteristics of selected soil types.

Type of soil	N_c (%)	Organic substances C_{ox} (%)	$CaCO_3$ (%)	pH KCL	Acceptable nutrients ($mg.kg^{-1}$)			Sorption capacity		
					P	K	Mg	S [mmol/100g]	T [mmol/100g]	V (%)
S1	0,07	2,2	1,7	6,8	4	311	812	17	17	100
	0,10	2,4	1,8	7,1	6	319	867	18	18	100
S2	0,05	2,4	1,8	7,0	3	295	763	15	15	100
	0,09	2,4	2,0	7,2	5	320	835	18	18	100
S3	0,09	2,7	2,1	6,9	6	325	855	17	17	100
	0,10	2,4	2,3	6,9	6	354	921	18	18	100
S4	0,07	1,9	1,7	6,7	3	256	711	15	15	100
	0,08	2,3	1,9	6,8	5	311	901	17	17	100
S5	0	5,6	0,8	3,9	0	75	198	5	25	20
	0,01	5,3	0,7	4,5	0	82	201	6	24	25
S6	0	0	0,4	7,1	1	95	211	3	3	100
	0	0,2	0,5	7,2	21	101	244	7	7	100
S7	0,01	0,2	0,7	7,3	1	105	223	5	5	100
	0,02	0,3	1,3	7,2	3	125	231	7	7	100
S8	0,08	3,3	2,0	6,8	4	265	724	17	17	100
	0,11	3,3	2,2	7,1	5	291	822	17	17	100
S9	0,07	2,9	1,8	6,8	3	248	699	15	15	100
	0,12	3,3	2,0	7,1	6	282	731	17	17	100

S1 – brown clay, S2 – brown clay, S3 – brown clay, S4 – gray clay, S5 – gray clay with coal mass, S6 – fonolite gravel, S7 – kalinically weathered phonolite, S8 – brown clay, S9 – brown clay

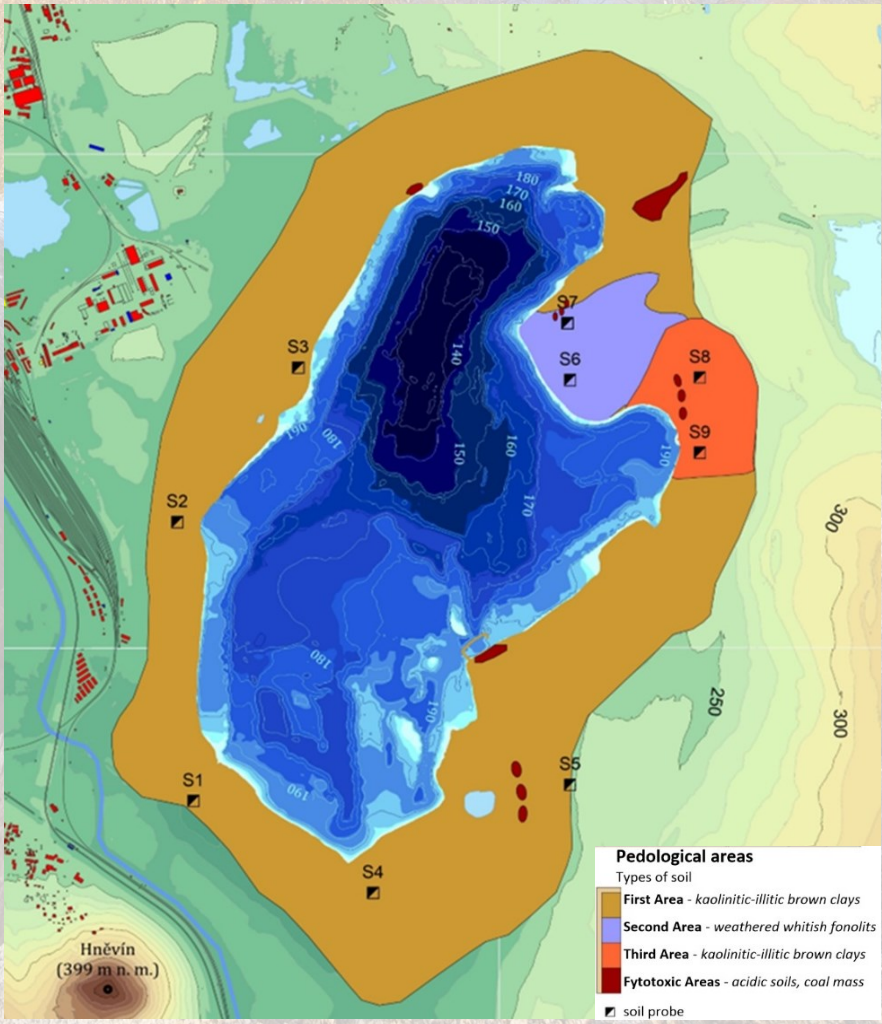

Figure 8 Lake Moste – pedological map of soil types.

Chapter IV

Field sampling and testing

Field sampling and testing serve as foundational pillars within the realms of soil science and geotechnical engineering. Their role is paramount in ensuring the stability, safety, and enduring sustainability of development projects, particularly in locales marked by residual excavations. These practices yield indispensable data, empowering informed decision-making across a diverse spectrum of applications. Furthermore, the distinction between in-situ and dumped soils assumes critical importance when evaluating the effectiveness of remediation strategies. In this section, we delve primarily into penetration tests, with a focus on the Cone Penetration Test (CPT). These tests, renowned for their significance and reliability, assume a pivotal role in the site investigation process, delivering essential information vital for the secure and cost-effective design and execution of various engineering endeavors. Below are some of the key data points that can be gleaned from these tests:

- CPT data is used to evaluate settlement and consolidation characteristics of soils. This is crucial for predicting settlement of structures and ensuring their stability over time.
- CPT can be used to estimate soil strength and shear parameters. This information is vital for designing foundations, retaining walls, and other structural elements.
- CPT provides direct measurements of the soil's friction angle and cohesion, which are essential in slope stability analyses and the design of earth-retaining structures.
- CPT can measure the pore pressure in the soil. This information is critical for assessing groundwater conditions, predicting liquefaction potential, and designing dewatering systems.

Table 17 Cone penetration tests.

Symbol	***Test***
qc	Measured cone resistance (MPa)
qT	Corrected cone tip resistance (MPa): $q_T = q_c + (1 - a_N)\, u_b$
aN	Net area ratio provided by manufacturer: *$0.75 < a_N < 0.82$ for most 10 cm^2 penetrometers* *$0.65 < a_N < 0.8$ for most 15 cm^2 penetrometers*
Fs	Sleeve frictional resistance
FR	Friction ratio = F_s/q_c
u0	In – situ pore pressure
Bq	Pore pressure parameter – excess pore pressure ratio $Bq = (u_d - u_0)/(q_T - P'_o)$
P´o	Effective overburden pressure
ud	Measured pore pressure (kPa)
△u	$\triangle u = u_d - u_0$
T	Time for pore pressure dissipation (sec)
t50	Time for 50 % dissipation (minutes)

- There are several variations of the cone penetration test (CPT). Electric and mechanical cones should be interpreted differently.
- The dissipation tests which can take a few minutes to a few hours has proven more reliable in determining the coefficient of consolidation, than obtaining that parameter from a consolidation test.

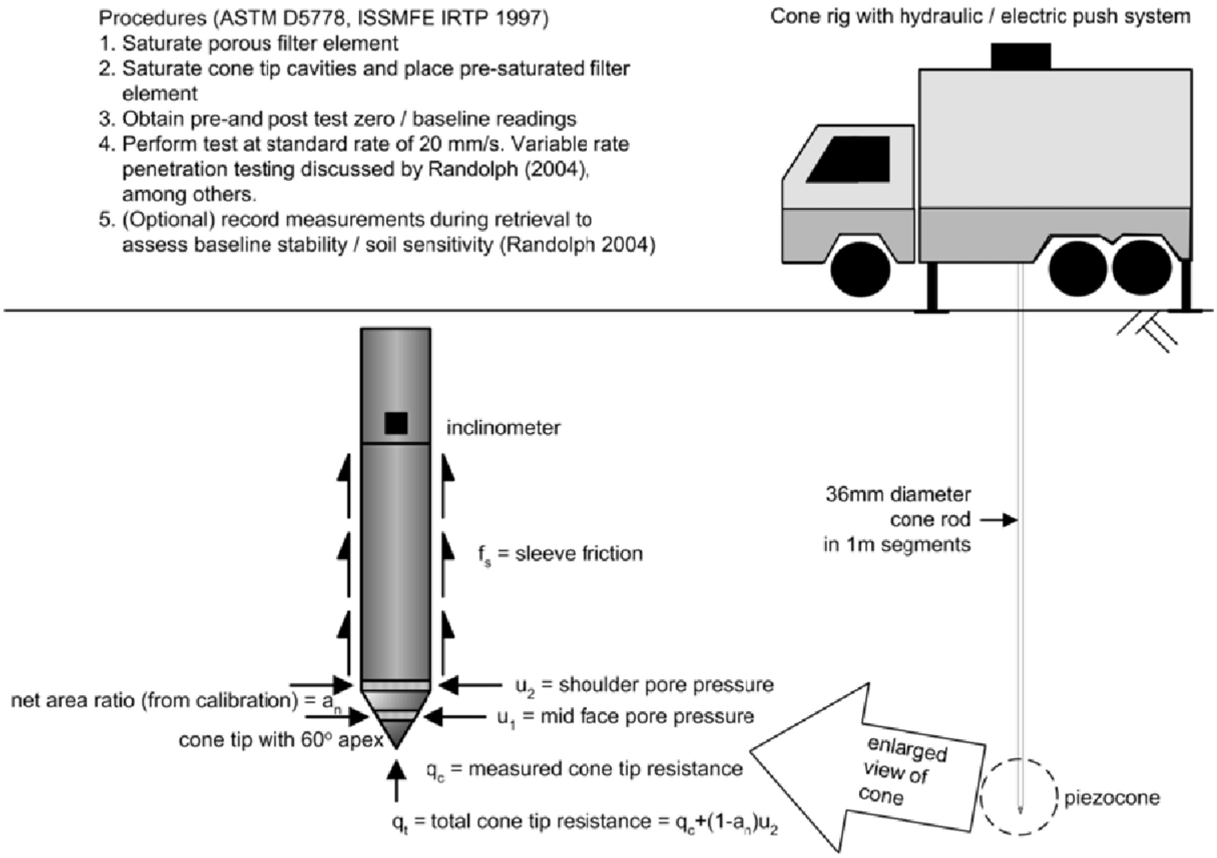

Figure 9 Principle of cone penetration test

5.1 Soil classification from CPT

- This is an ideal tool for profiling to identify lensing and thin layers.
- The table shows simplified interpretative approach. The actual classification and strength is based on the combination of both the friction ratio and the measured cone resistance, and cross checked with pore pressure parameters.

Table 18 Soil classification (adapted from Meigh, 1987 and Robertson et al., 2010).

Parameter	***Value***	***Non cohesive soil type***	***Cohesive soil type***
Measured cone resistance, q_c	<1.2 MPa	-	Normally to lightly overconsolidated
	>1.2 MPa	Sands	Overconsolidated
Friction ratio (FR)	<1.5%	Non cohesive	–
	>3.0%	-	Cohesive
Pore pressure parameter B_q	0.0 to 0.2	dense sand (q_T > 5 MPa)	hard/stiff soil (O.C) (q_T > 10 MPa)
	0.0 to 0.4	medium/loose sand (2 MPa < q_T < 5 MPa)	Stiff clay/silt (1 MPa < q_T < 2 MPa)
	0.2 to 0.8	-	firm clay/fine silt (q_T < 1 MPa)
	0.8 to 1.0	-	soft clay (q_T < 0.5 MPa)
	>0.8	-	very Soft clay (q_T < 0.2 MPa)
Measured pore pressure (u_d – kPa)	~0	dense sand (q_T – P'_o > 12 MPa)	-
		medium sand (q_T – P'_o >5 MPa)	-
		loose sand (q_T – P'_o > 2 MPa)	-
	50 to 200 kPa	-	silt/stiff clay (q_T – P'_o> 1 MPa)
	>100 kPa	-	soft/firm clay (q_T – P'_o< 1 MPa)

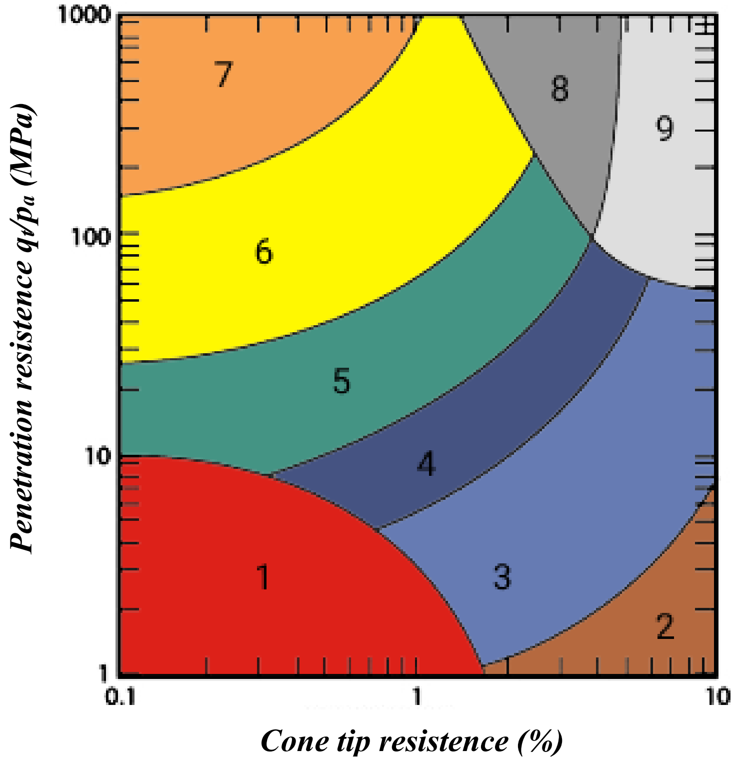

Figure 10 Non-normalized CPT Soil Behavior Type (SBT) chart according to Robertson et al, (2010). Soil Behavior Type (SBT): 1 - Sensitive, fine-grained; 2 - Organic soils – clay; 3 - Clay - silty clay to clay; 4 - Silt mixtures - clayey silt to silty clay; 5 - Sand mixtures - silty sand to sandy silt; 6 - Sands - clean sand to silty sand; 7 - Gravelly sand to dense sand; 8 - Very stiff sand to clayey sand; 9 - Very stiff fine-grained*.*
** Heavily overconsolidated or cemented*

5.2 Soil type from friction ratios

- Soil type from friction ratios refers to the classification of soil based on the ratio of sleeve friction to cone resistance obtained during a Cone Penetration Test (CPT). The CPT measures the resistance encountered by a cone-tipped probe as it is pushed into the ground. This test provides valuable information about the subsurface soil conditions.
- Different soil types exhibit characteristic ranges of friction ratios, which can be used to classify the soil. Here are some general guidelines for soil classification based on friction ratios:

Table 19 Soil type based on friction ratios.

Friction ratio (%)	***Soil type***
<1	Coarse to medium sand
1–2	Fine sand, silty to clayey sands
2–5	Sandy clays. Silty clays, clays, organic clays
>5	Peat

- It's important to note that these are general guidelines and actual ranges may vary depending on specific site conditions, soil composition, and other factors.

5.3 Clay parameters from CPT

- The conversion factor for the cone can greatly impact the interpretation of results.
- In situations requiring critical conditions and realistic designs, it is essential to calibrate this testing with laboratory strength testing.

Table 20 Clay parameters from cone penetration test.

Parameter	***Relationship***	***Comments***
Undrained strength (C_u – kPa)	$C_u = q_c/N_k$ $C_u = \triangle u/N_u$	Cone factor (N_k) = 17 to 20 17–18 for normally consolidated clays 20 for over-consolidated clays Cone factor (N_u) = 2 to 8
Undrained strength (C_u – kPa), corrected for overburden	$C_u = (q_c – P'_o)/N'_k$	Cone factor (N'_k) = 15 to 19 15–16 for normally consolidated clays 18–19 for over-consolidated clays
Coefficient of horizonta lconsolidation (c_h – sq m/year)	$c_h = 300/t_{50}$	t_{50} – minutes (time for 50 % dissipation)
Coefficient of verticalconsolidation (c_v – sq m/year)	$c_h = 2\ c_v$	Value may vary from 1 to 10

5.4 Clay strength from CPT

- CPT data can be used to estimate the undrained shear strength C_u of clay. This parameter is essential for analyzing stability against shear failure, particularly in situations where rapid loading or excess pore water pressure can occur.
- By utilizing CPT data to understand clay strength, engineers and geotechnical experts can make informed decisions to mitigate risks associated with construction and development in clayey soil environments.
- The following relationships are used to determine the likely strength of the clay.

Table 21 Soil strength from cone penetration test.

Soil classification		***Approximate qc (MPa)***	***Assumptions. Not corrected for overburden***
V. Soft	C_u = 0–12 kPa	<0.2	N_k = 17 (Normally consolidated)
Soft	C_u = 12–25 kPa	0.2–0.4	N_k = 17 (Normally consolidated)
Firm	C_u = 25–50 kPa	0.4–0.9	N_k = 18 (Lightly overconsolidated)
Stiff	C_u = 50–100 kPa	0.9–2.0	N_k = 18 (Lightly overconsolidated)
V. Stiff	C_u = 100–200 kPa	2.0–4.2	N_k = 19 (Overconsolidated)
Hard	C_u => 200 kPa	>4.0	N_k = 20 (Overconsolidated)

5.5 Shear strength of the dump materials

- CPT data can be used to estimate the undrained shear strength c_u of clay. This parameter is essential for analyzing stability against shear failure, particularly in situations where rapid loading or excess pore water pressure can occur.
- By utilizing CPT data to understand clay strength, engineers and geotechnical experts can make informed decisions to mitigate risks associated with construction and development in clayey soil environments.

5.6 Simplified sand strength assessment from CPT

- Just as the parameters for clays have been characterized, sands have their own testing.
- The assessment may vary depending on the depth of the effective overburden and type of coarse grained material.

Table 22 Preliminary sand strength from cone penetration tests.

Relative density	***Dr (%)***	***Cone resistance, qc (MPa)***	***Typical φ◦***
V. Loose	D_r < 15	<2.5	<30◦
Loose	D_r = 15–35	2.5–5.0	30–35◦
Med dense	D_r = 35–65	5.0–10.0	35–40◦
Dense	D_r = 65–85	10.0–20.0	40–45◦
V. Dense	D_r > 85	>20.0	>45◦

5.7 Dump classification and assessment from CPT

- While CPT (Cone Penetration Test) results are valuable, they may not be directly compatible with numerical models assuming a Mohr-Coulomb-type elastoplastic constitutive model for geological units.
- To use CPT data in such models, it is necessary to calculate both cohesion (c) and the friction angle (φ) from the available data.
- Several empirical relationships exist for assessing the friction angle, as outlined in CPT data interpretation theory manuals. These relationships are useful but have limitations, particularly in assessing cohesion and for soils beyond sands and fine-grained soils.
- The guidelines recommend using the system of equations proposed by Motaghedi & Eslami in 2014. These equations provide a means to calculate both cohesion (c) and friction angle (φ) from CPT data:

$$\begin{cases} u_2 + \gamma B \tan\phi + (\sigma_v - u) N_q + \gamma B N_q \tan^2\phi + c \dfrac{N_q - 1}{\tan\phi} = q_t \\ c = f_s - 0.000789(1-\sin\phi)(\sigma_v - u)\tan\dfrac{2\phi}{3}\left(\dfrac{q_t - \dfrac{\sigma_v}{3}(1+2(1-\sin\phi))}{\dfrac{\sigma_v - u}{3}(1+2(1-\sin\phi))}\right)^{1.44} \end{cases}$$

with the bearing capacity factor $N_q = \dfrac{(\sin\phi + 1)^2}{\cos\phi}\exp\left[\left(0.0061\dfrac{q_t}{P_a} + 2.4\right)\tan\phi\right]$

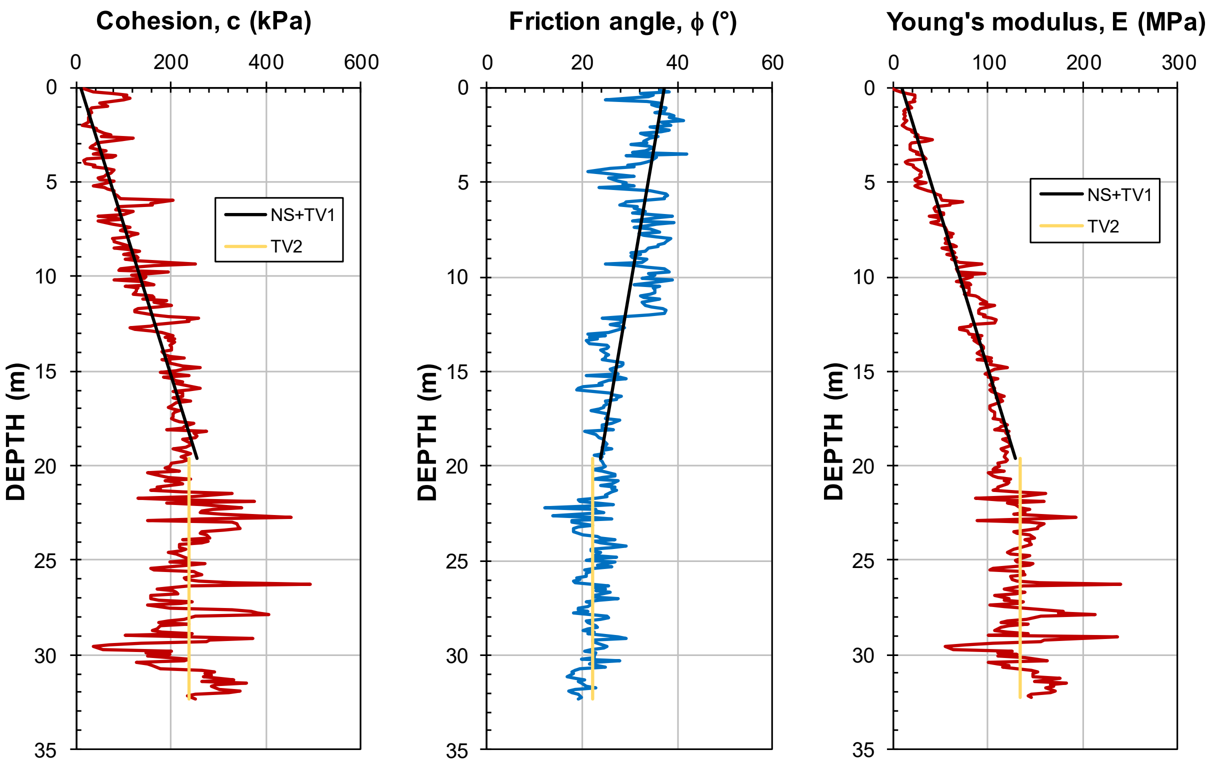

Figure 11 Cohesion, friction angle and Young's modulus computed from CPT

5.8 Evaluation of cohesion and friction angle from CPT

- The primary equation for calculating friction angle contains a single unknown variable (ϕ). This equation can be solved numerically, such as by using software like Mathematica or Excel.
- This approach automates the resolution process and can be conveniently implemented in a universal spreadsheet, making it accessible and user-friendly.
- Plot regression results on the same figure for clear visualization.
- Apply statistical processing to estimate the means, minimum, and maximum bounds of key mechanical properties, including cohesion, friction angle (φ), density (ρ), Young's modulus (E).
- Assess these properties for each geological unit within the dump.
- Analyze the measurements to determine the spatial dependence of mechanical properties.
- Recognize that, in some cases there may be a vertical correlation, indicating that properties change with depth.
- For units exhibiting depth-related variations, consider the following observations:
    - Cohesion and Young's modulus tend to increase with depth.
    - Friction angle (φ) tends to decrease with depth.
- The observed behavior is consistent with known soil behavior principles.
- alculate correlation coefficients (e.g., r values) to quantify relationships between mechanical properties. Note that anti-correlation values between properties, such as φ and cohesion (c), are commonly observed in the literature.
- Ensure that data analysis considers these correlations when interpreting results.

Case study: Evaluation of cohesion and friction angle

During the geotechnical inestigation of the lake Most and its adjacent areas, a comprehensive survey campaign involving CPT was conducted on the expansive dump formations that comprise critical elements for addressing the stability of the site's slopes.
In 2021, an extensive CPT campaign was conducted within the vicinity of Lake Most. This campaign yielded a dataset comprising 9538 CPT measurements in 23 CPT sampling places.

Technical data of the CPT unit HYSON-200 kN

CPT unit manufactured :	A.P. van den Berg, The CPT factory (Holland)
Pushing force :	200 kN
Pulling force :	260 kN
CPT unit chassis :	MAN TGS 33-420, 6x6 truck - weight 21,5 tons
Pushing force generation :	hydraulic
Balance of the pushing force :	weight of the truck
Sounding speed :	2 cm/s
Data registration step :	1 cm

Electric CPT cone with pore pressure sensor

Parameters tip cone :	diameter 35,7 mm, cross-section 10 cm^2, tip angle 60°
Sleeve friction sensor :	surface 150 cm^2
Pore pressure sensor :	type U2 (by standard EN ISO 22476-1)

Analysis of geomechanical parameters

The analysis aimed to identify the interface between NS* and TV1** units within a specific depth range, which proved challenging. To simplify the geomechanical model, these two units were combined. The interface between NS-TV1 and TV2*** was successfully identified in 20 out of 23 CPTs, with corresponding depth values. Additionally, a statistical approach helped clarify the data, allowing the estimation of cohesion, friction angle, and Young's modulus for each dump unit. Notably, differences in properties were observed between the NS-TV1 unit and the underlying TV2 unit, likely due to increased consolidation in the lower layers. The analysis also revealed that spatial dependence was primarily limited to the vertical dimension (depth), with no significant correlation observed between mechanical parameters and the horizontal position of boreholes (x, y).

** upper (younger) layers of the dump, with expected higher porosity (see Chapter 7.5)*
*** older dump layers, with expected lower porosity (see Chapter 7.5)*
**** oldest layers with expected reduced infiltration capacity (see Chapter 7.5)*

Figure 12 CPT In Situ truck (equipped by Hydraulic Unit HYSON-200 kN) at the site of lake Most.

This approach allows raw data to be denoised that is unusable without statistical processing. Thus, we can estimate what are the means and the minimum and maximum bounds of cohesion, friction angle and Young's modulus for each dump unit:

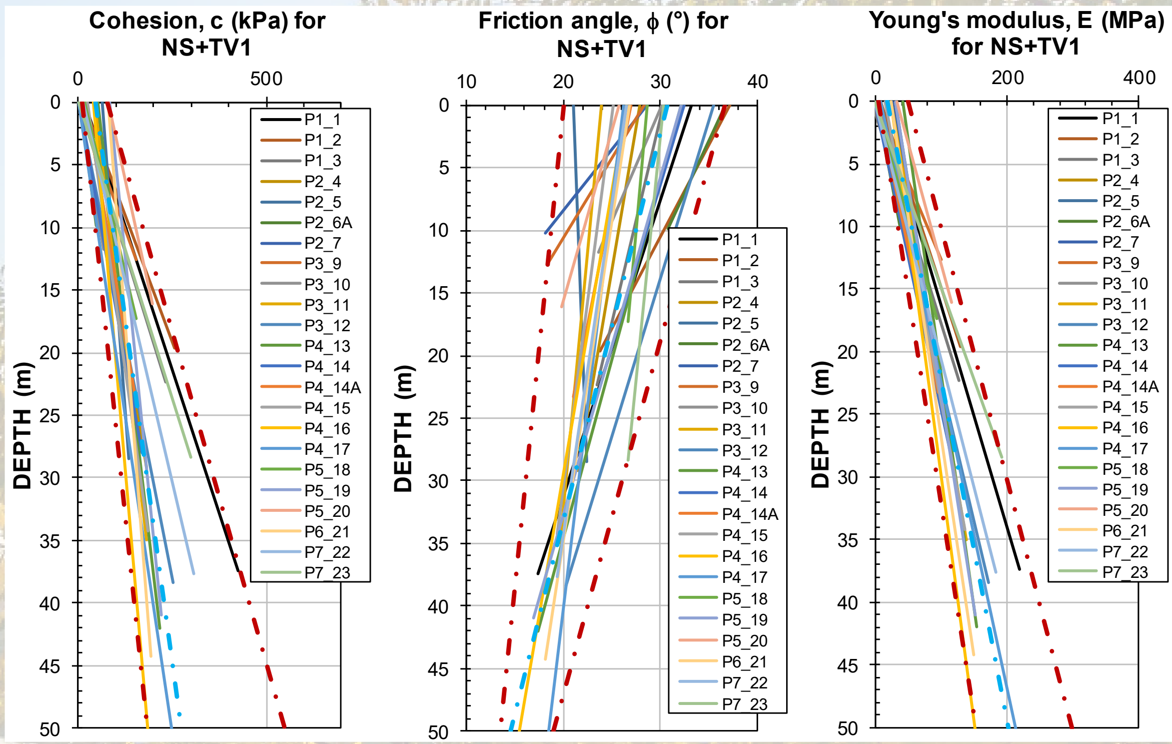

Figure 13 Linear variation of cohesion, friction angle and Young's modulus for all CPT profiles for NS-TV1 unit; minimum and maximum limits in red and mean value in cyan.

Analysis of the measurements for each profile showed a dependence of the values of *C, f, r* and *E* at depth for the unit NS-TV1 unlike the measurements attached to the underlying unit TV2. This can be explained by a greater consolidation of the lower layers which tends to limit the variability of mechanical properties. Therefore, the minimum and maximum bounds of cohesions, friction angles, density and Young's modulus are considered constant for TV2 while they vary linearly for NS-TV1.

The results of this statistical analysis are reported in the table. Note that the lognormal distribution is the one that best represents variations in cohesions and Young's modulus. The same is true for the friction angle except for the TV2 unit which correlates better with a Birnbaum-Saunders distribution. On the other hand, the normal distribution makes it possible to represent well the spatial distribution of the density masses in the 23 CPTs.

The probability density function of Birnbaum-Saunders distribution is:

$$pdf = \frac{\sqrt{\frac{x}{\beta}}+\sqrt{\frac{\beta}{x}}}{2\sqrt{2\pi}\gamma x}\, e^{-\frac{\frac{x}{\beta}+\frac{\beta}{x}-2}{2\gamma^2}} \quad \text{with } \beta=\frac{\mu}{3}\left(4-\sqrt{3c^2+1}\right),\ \gamma=\sqrt{\frac{2\left(c^2-1+\sqrt{3c^2+1}\right)}{5-c^2}} \quad \text{and } c=COV=\frac{\sigma}{\mu}<\sqrt{5}$$

Figure 14 Hydraulic pusher of CPT Unit HYSON-200 kN inside the CPT truck

Table 24 Statistic of geotechnical properties of the lead geotechnical units (lake Most).

Properties	***Geotechnical unit***		
	NS + TV1	***TV2***	***Contact Layer***
cohesion: C (kPa)	mean : 4.59 d+46.15 min : 3.5 d+10 LN dist ∫ [0.6 , 650]: μ_{LN}=0.0324 d+4.034 s_{LN}=-0.0063 d+0.598	mean : 247.4 min : 4 d-60 LN dist ∫ [6 , 1098]: μ_{LN}=5.31 & s_{LN}=0.626	6.0
friction: φ (°)	mean : -0.323 d+30.69 min : -0.13 d+20 LN dist ∫ [7 , 44]: μ_{LN}=-0.241 d+28.73 s_{LN}=-0.071 d+6.074	mean : 22.7 min : 16.8 BS dist ∫ [8.2 , 38.6]: b=22.019 & g=0.244	6.0
dilation angle: y (°)	0	$\psi = \frac{\phi}{3}$	
tension: Rt (kPa)	$R_t = \frac{R_c}{10} = \frac{C}{5}\tan\left(\frac{\pi}{4}+\frac{\phi}{2}\right)$		
Young's modulus: E (MPa)	mean : 3.67 d+18.51 min : 2.96 d+5 LN dist ∫ [1 , 354]: μ_{LN}=0.037 d+3.577 s_{LN}=-0.004 d+0.377	mean : 193.9 min : 3.6 d-52 LN dist ∫ [29.6 , 636.5]: μ_{LN}=3.855 d+32.046 s_{LN}=0.6195 d+28.295	70
Poisson's ratio: n	0.35	0.35	0.3
saturated density: r (t/m^3)	mean : 0.0124 d+1.739 (d≤12.5 m) 0.0035 d+1.869 (d>12.5 m) min : 0.0035 d+2.0 N dist ∫ [1.3 , 2.2]: μ=0.0053 d+1.813 s=-0.0008 d+0.0919	mean : 2.023 min : 0.0017 d+2.079 N dist ∫ [1.55 , 2.285]: μ=2.023 & s=0.098	2.0

Chapter VI

Soil properties and state of the soil

Soil properties refer to the characteristics and behaviors exhibited by a particular type of soil. These properties can include grain size distribution, composition, moisture content, density, shear strength, and permeability, among others.
The state of the soil refers to its current condition, which can range from loose or unconsolidated to dense or compacted. This state can be influenced by factors like loading history, drainage conditions, and natural processes like weathering.
The influence of soil properties and state on slope stability is significant.

6.1 Soil behaviour

- Geotechnical models integrate knowledge of soil behavior with mathematical and computational methods to provide engineers with a tool to make informed decisions about the design, construction, and safety of civil engineering projects. These models are constantly refined and validated based on field observations and laboratory testing to improve their accuracy and reliability. As a result, they can be used to predict how soils will respond to different loading conditions and environmental factors.
- Soil behaviour is therefore a very important input in the development of geotechnical models.
- Geotechnical models are often constructed for the behaviour of sand (granular) or clay (fine-grained) therefore the following table describes these two models with many variations.
- In cases of uncertainty of clay/sand governing property, the design must consider both geotechnical models. The importance of simple laboratory classification tests becomes evident.
- Given the distinct behaviour of the two types of soils, then the importance of the soil classification process is self-evident. The requirement for carrying out laboratory classification tests on some samples to validate the field classification is also evident.
- Yet there are many geotechnical reports that rely only on the field classification due to cost constraints

Table 25 Comparison of behaviour between sands and clays.

Property	***Sands***	***Clays***	***Comments***
Permeability (k)	High k. Drains quickly (assumes <30 % fines).	Low K. Drains slowly (assumes non fissured or no lensing in clay).	Permeability affects the long term (drained) and short term (undrained) properties.
Effect of time	Drained and undrained responses are comparable.	Drained and undrained response needs to be considered separately.	Settlement and strength changes are immediate in sands, while these occur over time in clays.
Water	Strength is reduced by half when submerged.	Relatively unaffected by short term change in water.	In the long term the effects of consolidation, or drying and wetting behaviour may affect the clay.
Loading	Immediate response. Not sensitive to shape.	Slow response. 30 % change in strength from a strip to a square/circular footing.	See. N_c bearing capacity factor(shape influenced).

Strength	Frictional strength governs.	Cohesion in the short term often dominates, while cohesion and friction to be considered in the long term.	In clay materials both long term and short term analysis are required, while only one analysis is required for sands.
Confinement	Strength increases with confining pressure, and depth of embedment.	Little dependence on the confining pressure. However, some strain softening may occur in cuttings and softened strength (cohesion loss) then applies.	If overburden is removed in sands a considerable loss in strength may occur at the surface. N_q bearing capacity factor (becomes significant at $\varphi > 30°$).
Compaction	Influenced by vibration. Therefore a vibrating roller is appropriate.	Influenced by high pressures. Therefore a sheepsfoot roller is appropriate.	Deeper lifts can be compacted with sands, while clays require small lifts. Sands tend to be self compacting.
Settlement	Occurs immediately (days or weeks) on application of the load.	Has a short and long term (months or years) settlement period.	A self weight settlement can also occur in both. In clays the settlement is made up of consolidation and creep.
Effect of climate	Minor movement for seasonal moisture changes.	Soil suction changes are significant with volume changes accompanying.	These volume changes can create heave, shrinkage uplift pressures. In the longer term this may lead to a loss in strength.

6.2 State of the soil

- As mentioned in the introduction to this chapter, soil condition implies a certain state of the soil.
- The values is for a given soil as a clay in a wet state can still have a higher soil suction than a sand in a dry state.

Table 26 Some influences of the state of the soil.

Soil property		***State of soil***		***Relative influence***
Strength	Dry	High compaction	High OCR	Higher strength
	Wet	Low compaction	Low OCR	Reduced strength
Colour	Dry			Lighter colour.
	Wet			Dark colour
Suction	Dry	High compaction	High OCR	High suction
	Wet	Low compaction	Low OCR	Low suction
Density		High compaction	High OCR	High density
		Low compaction	Low OCR	Lower density

Table 27 Plasticity characteristics of common clay minerals (from Holtz and Kovacs,1981).

Clay mineral	***Plot on the plasticity chart***
Montmorillonites	Close to the U – Line. LL = 30 % to Very High LL> 100 %
Illites	Parallel and just above the A – Line at LL = 60 % ± 30 %
Kaolinites	Parallel and at or just below the A – Line at LL = 50 % ± 20 %
Halloysites	In the general region below the A – Line and at or just above LL = 50 %

6.3 Plasticity characteristics of common clay minerals

- Soils used to develop the plasticity chart tended to plot parallel to the A – Line (Refer Figure).

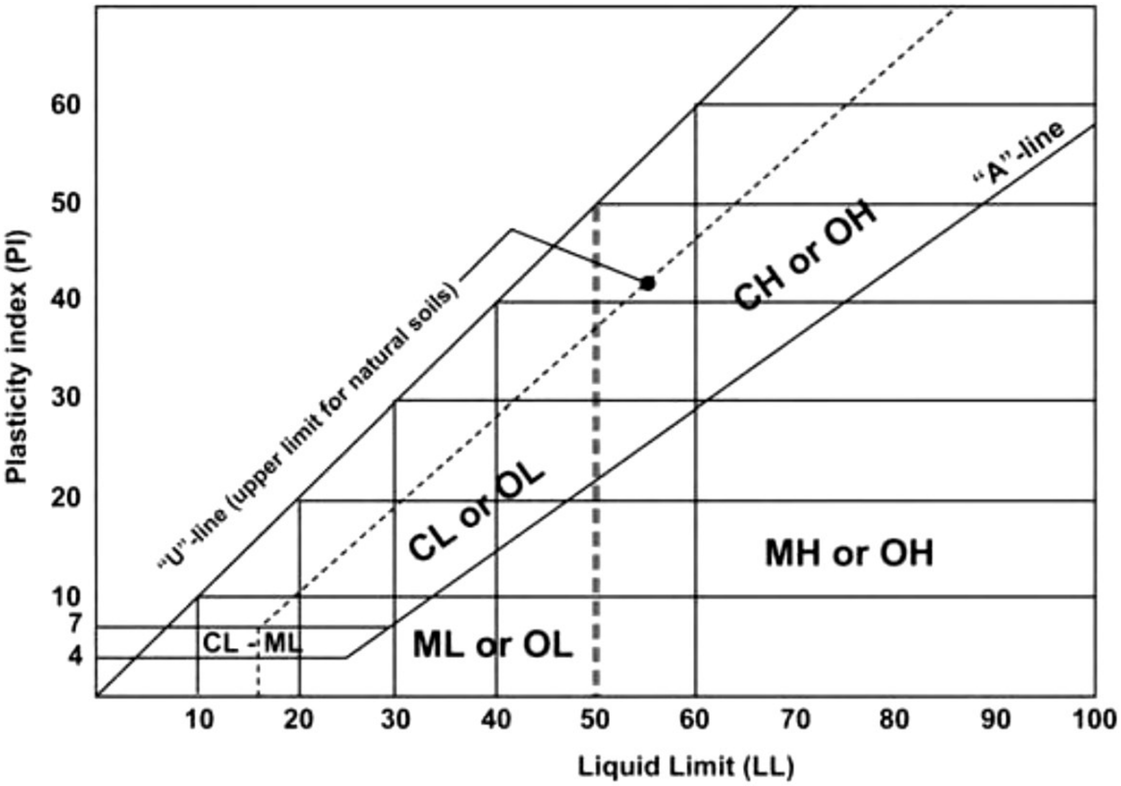

Figure 15 Soil plasticity chart.

- A – Line divides the clays from the silt in the chart.
- A – Line: PI=0.73 (LL – 20).
- The upper limit line U – line represents the upper boundary of test data.
- U – Line: PI=0.9 (LL – 8).
- Volcanic and Bentonite clays plot close to the U Line at very high LL.

6.4 Effective friction of granular soils

- The friction angle is a fundamental property that influences the behavior of soil under shear loading.
- Friction is contingent on factors such as the material's size and type, as well as its level of compaction and grading.
- Particle shape (rounded vs angular) also has an effect, and would change the above angles by about 4 degrees.

Table 28 Typical friction angle of granular soils.

Type	*Description/state*	*Friction angle (degrees)*
Cohesionless	Soft sedimentary (chalk, shale, siltstone, coal)	30–40
Compacted	Hard sedimentary (conglomerate, sandstone)	35–45
Broken rock	Metamorphic Igneous	35–45 40–50
Cohesionless	Very loose/loose	30–34
Gravels	Medium dense Dense Very dense	34–39 39–44 44–49

Cohesionless Sands	Very loose/loose	27–32
	Medium dense	32–37
	Dense	37–42
	Very dense	42–47
Cohesionless Sands	*Loose*	
	Uniformly graded	27–30
	Well graded	30–32
	Dense	
	Uniformly graded	37–40
	Well graded	40–42

6.3 Effective strength of cohesive soils

- It is important to account for the gradual softening of the clay over time, leading to a decrease in its effective cohesion.
- The remoulded strength and residual strength values are expected to exhibit a reduction in both cohesion and friction.

Table 29 Effective strength of cohesive soils in-situ.

Type	***Soil description/state***	***Effective cohesion (kPa)***	***Friction angle (degrees)***
Cohesive	Soft – organic	5–10	10–20
	Soft – non organic	10–20	15–25
	Stiff	20–50	20–30
	Hard	50–100	25–30

- Shear strength of clayey dump materials cannot be directly derived from shear strength values determined on compact hard soil samples. These differences are particularly evident in cohesion investigations. In general, a notable finding is that the more cohesive the hard soil of the overburden, the less cohesive its dump material tends to be.
- Shear tests on clayey dump materials at normal pressures up to 3 MPa reveal distinct behavior. Beyond a certain normal stress threshold, typically in the range of 1-1.5 MPa (depending on the soil type), corresponding to geostatic pressure at depths of 50 to 75 meters, there is a collapse of the material's structure.
- This phenomenon is associated with a decrease of half to one-third in the internal friction angle and a doubling of cohesion. This behavior is most noticeable in clayey dump materials with a maximum moisture content of 25-30 % (in dry matter). If the moisture content is higher, this phenomenon shifts to lower pressure ranges or may disappear entirely. A relatively small increase in moisture content (5-7 %) is sufficient to induce this behavioral change.
- In the range of low stress (0 to 3 MPa), the dump material exhibits low values of the internal residual friction angle, typically within the range of only 2-6°.

6.4 Overconsolidation ratio

- The Overconsolidation ratio (OCR) offers insight into the stress history of the soil. It represents the proportion of its highest prior overburden pressure to its present overburden pressure.
- The material might have undergone greater past stresses owing to fluctuations in the water table or the removal of previous overburden during erosion.
- For sand and gravel the maximum pressure at pile base $p_{max,base}$ is reduced depending on the value of overconsolidation OCR as follows:
    - *for all cohesionless soils the maximum pressure at pile base $p_{max,base}$ is 15 MPa*
    - *for OCR $\leqslant 2$ no reduction is performed*
    - *for $2 < OCR \leqslant 4$ the maximum pressure at pile base $p_{max,base}$ is multiplied by 0.67*
    - *for $OCR > 4$ the maximum pressure at pile base $p_{max,base}$ is multiplied by 0,50*

Table 30 Overconsolidation ratio.

Overconsolidation ratio (OCR)	$OCR = P'_c/P'_o$
Preconsolidation pressure = Maximum stress ever placed on soil	P'_c
Present effective overburden	$P'_o = \sum\gamma' z$
Depth of overlying soil	z
Effective unit weight	γ'
Normally consolidated	OCR ~ 1 but < 1.5
Lightly overconsolidated	OCR = 1.5–4
Heavily overconsolidated	OCR> 4

- In the case of mature glacial clays, the Overconsolidation Ratio (OCR) ranges between 1.5 and 2.0 for Plasticity Index (PI) values exceeding 20 % (Bjerrum, 1972).
- Soil that is normally consolidated can gain strength over time under load.
- Overconsolidated soils may experience a reduction in strength over time when not under load (such as during excavation) or when subjected to high strains.

6.5 Preconsolidation stress from CPT

- The preconsolidation stress signifies the highest stress encountered in its prior history.
- The present strength is influenced by both its historical and current overburden pressures.

Table 31 Preconsolidation pressure from net cone tip resistance (from Mayne et al., 2002).

Net cone stress	$q_T - P'_o$	kPa	100	200	500	1 000	1 500	3 000	5 000
Preconsolidation pressure	P'_c	kPa	33	67	167	333	500	1 000	1 667
Excess pore water pressure	$\triangle u_1$	kPa	67	133	333	667	1 000	2 000	3 333

- For intact clays only.
- For fissured clays $P'c$=2 000 to 6 000 with $\triangle u_1$ = 600 to 3 000 kPa.
- The electric piezocone (CPTu) only is accurate for this type of measurement. The mechanical CPT is inappropriate.

6.6 Effect of climate on soil suction change

- The impact of climate on variations in soil suction and its influence on stability is a crucial factor in geotechnical engineering. Changes in climate, particularly fluctuations in moisture levels, can significantly alter the suction (matric potential) within the soil matrix. This, in turn, affects the soil's mechanical properties and ultimately its stability.
- In regions experiencing wetter conditions, an increase in soil moisture content leads to a decrease in soil suction. This can result in reduced shear strength and increased compressibility, potentially compromising the stability of structures founded on or within the affected soil.
- Conversely, in drier climates, soil suction tends to rise due to reduced moisture levels. This can lead to an increase in shear strength and reduced compressibility. However, in the long term, prolonged dry conditions may lead to desiccation and consequent loss of strength and volume, potentially affecting stability.
- Therefore, understanding the relationship between climate-induced changes in soil suction and their effects on soil behavior is critical in the design and maintenance of engineering structures to ensure their long-term stability and safety.

Table 32 Soil suction based on climate (AS 2870, 1990).

Climate description	Soil suction change ($\triangle u$, pF)	Equilibrium soil suction, pF
Alpine/wet coastal	1.5	3.6
Wet temperate	1.5	3.8
Temperate	1.2–1.5	4.1
Dry temperate	1.2–1.5	4.2
Semi arid	1.5–1.8	4.4


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6.7 Effect of climate on active zones

- Changes in precipitation patterns due to climate change can significantly impact landslide activity. Intense or prolonged rainfall can saturate soil and increase the likelihood of landslides. Conversely, prolonged droughts can lead to soil desiccation and increased susceptibility to landslides once rain returns.
- The deeper active zones are expected in drier climates.
- The Thornthwaite Moisture Index (TMI) is used to characterize the climatic conditions in the area in terms of rainfall and evaporation.
- It can be useful for planning and decision-making related to soil and water management. The Thornthwaite Moisture Index is among the tools that can assist in monitoring and adapting to changes in climate and its impacts on soil moisture availability.

Table 33 Active zones based on climate (Walsh et al., 1998).

Climate description	***H_s(metres)***	***Thornwaithe moisture index (TMI)***
Alpine/west coastal	1.5	>40
Wet temperate	1.8	10 to 40
Temperate	2.3	−5 to 10
Dry temperate	3.0	−25 to −5
Semi arid	4.0	<−25

6.10 Variability of soils

- The importance of considering soil parameter variability should always be prioritized when assessing its significance, with a focus on its actual value.
- When it comes to index parameters versus strength and deformation parameters, one can have higher confidence in the former.
- It's worth noting that relying on index parameters for strength correlations doesn't necessarily imply greater accuracy, as it introduces another variable in the correlation.

Table 34 Variability of soils (Kulhawy,1992).

Property	***Test***	***Mean COV without outliers***
Index	Natural moisture content,w_n	17.7
	Liquid limit, LL	11.1
	Plastic limit, PL	11.3
	Initial void ratio, e_o	19.8
	Unit weight, γ	7.1
Performance	Rock uniaxial compressive strength, qu	23.0
	Effective stress friction angle, φ	12.6
	Tangent of φ	11.3
	Undrained shear strength C_u	33.8
	Compression index C_c	37.0

- Deformation processes in dump materials begin during backfilling. Initially, fragments absorb moisture, but the scope and speed of changes depend on water saturation.
- Water content is critical; dry materials are vulnerable to external water, while moisture during backfilling can make the material soft and soggy.
- Deformation varies with depth; deeper parts undergo classical filtration consolidation, while shallow areas have significant, non-uniform deformation.

6.11 Variability of in-situ tests

- It is important to acknowledge the constraints of in-situ testing equipment.

- The likely measurement error needs to be considered with the inherent soil variability.
- The Standard Penetration Test (SPT) exhibits considerable variability as an in-situ test.
- Among the in-situ testing methods, the Electric Cone Penetrometer and the Dilatometer demonstrate the lowest variability.
- The table illustrates the combined impact of equipment, procedure, and random factors.

Table 35 Variability of in – situ tests (From Phoon and Kulhawy, 1999).

Test	***Coefficient of variation (%)***
Standard penetration test	15–45
Mechanical come penetration test	15–25
Self boring pressure meter test	15–25
Vane shear test	10–20
Pressure meter test, prebored	10–20
Electric cone penetration test	5–15
Dilatometer test	5–15

6.12 Soil variability from laboratory testing

- Soil density can be reliably measured.
- There is considerable variability in the results of shear strength tests for clays and the Plasticity Index.

Table 36 Variability from laboratory testing (Phoon and Kulhawy, 1999).

Test	***Property***	***Soil type***	***Coefficient of variation (%)***	
			Range	***Mean***
Atterberg tests	Plasticity index	Fine grained	5–51	24
Triaxial compression	Effective angle of friction	Clay, silt	7–56	24
Direct shear	Shear strength, C_u	Clay, silt	19–20	20
Triaxial compression	Shear strength, C_u	Clay, silt	8–38	19
Direct shear	Effective angle of friction	Sand	13–14	14
Direct shear	Effective angle of friction	Clay	6–22	14
Direct shear	Effective angle of friction	Clay, silt	3–29	13
Atterberg tests	Plastic limit	Fine grained	7–18	10
Triaxial compression	Effective angle of friction	Sand, silt	2–22	8
Atterberg tests	Liquid limit	Fine grained	3–11	7
Unit weight	Density	Fine grained	1–2	1

- Be aware of the difficulties in laboratory investigations of shear strength for clayey dump materials, especially at high values of normal stress. At low stress levels, the dump material is aerated. However, as the normal stress increases to a certain limit (1-1.5 MPa), there appears to be an emergence and subsequent increase in the porous pressure of air and water. When porosity disappears in the dump material, any further increment in normal stress is primarily attributed to water in the pores. As a result, the mobilized shear stress in the dump material practically no longer increases.

Case study: Laboratory Test Results

This chapter provides a comprehensive analysis of laboratory test results concerning the composition of waste rock and post-mining waste from coal mining operations. The waste comprises a mixture of crumb rocks, including sandstones of varying grain sizes and mudstones, as well as clay rocks, primarily calystones. These rocks originate from different lithostratigraphic series within the Upper Carboniferous layers of the Upper Silesian Coal Basin.

The focus of this study is the uniaxial compressive strength of waste rocks that accompany coal seams. These rocks range from the youngest layers of the Krakow sandstone series to the oldest layers of the paralytic series. The results, presented for both the air-dry and water-saturated states, reveal a decrease in strength influenced by the age and composition of the rocks that are typically deposited in opencast mine dumps.

Key findings from the laboratory tests include:

Uniaxial compressive strength values demonstrate a decrease when transitioning from an air-dry state to water saturation, with coefficients of strength reduction as follows:

- Sandstones: 0.59 - 0.82
- Mudstones: 0.95 - 0.98 (with one case at 0.54)
- Claystones: 0.50 - 0.87.

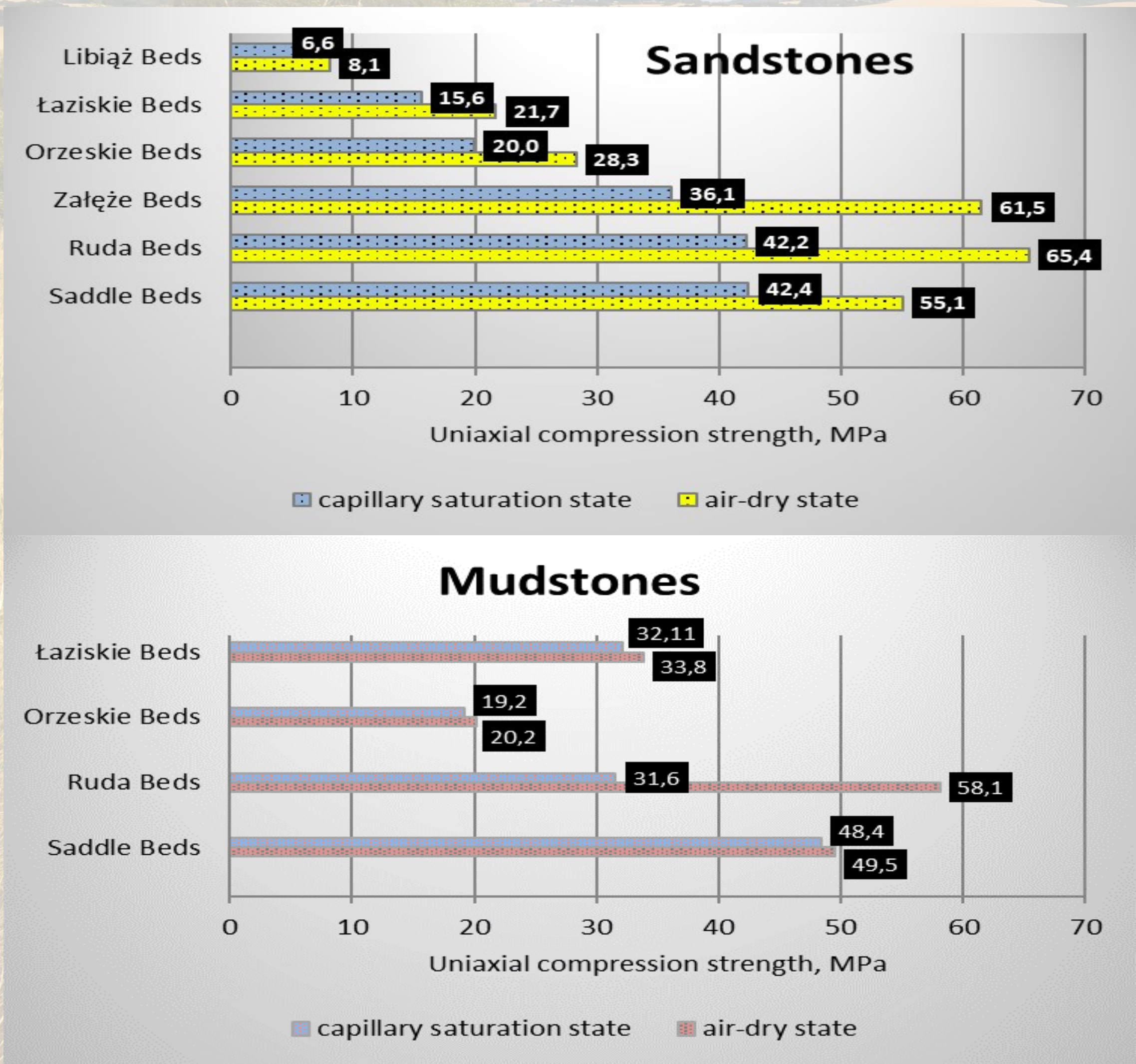

Figure 16 Uniaxial compressive strength of Carboniferous rocks in USCB in Poland – air-dry state and water saturation state of Sandstones and Mudstones

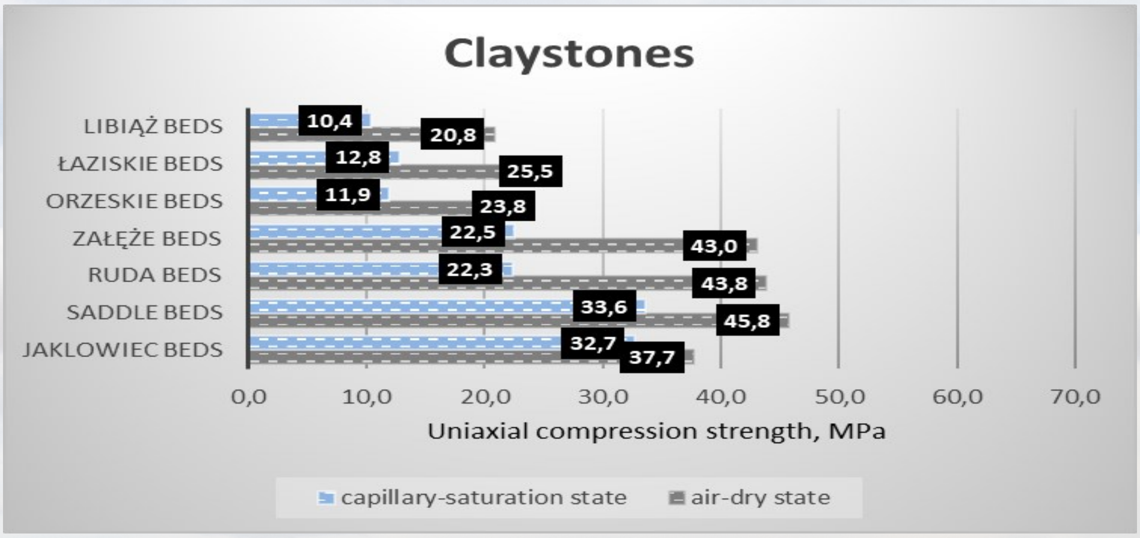

Figure 17 Uniaxial compressive strength of Carboniferous rocks in USCB in Poland – air-dry state and water saturation state of Claystones.

Cohesion values measured under conventional triaxial compression conditions show a reduction in water-saturated states compared to air-dry states. Cohesion values and structural weakening coefficients are outlined for sandstones, mudstones, and claystones.

The angle of internal friction in the air-dry state ranges from 41 to 50 degrees for all lithostratigraphic cells and rock types. These values remain relatively constant or experience slight increases after rocks are saturated with water. The geotechnical characteristics of post-mining waste in watedump sites are influenced by various factors, including the composition of waste rock, proportions of rock types, and storage duration. This research encompasses the examination of waste mixtures collected from different locations on the watedump's surface and stored for varying lengths of time.

The insights provided in this chapter offer valuable data for professionals involved in reservoir slope stability and waste management, shedding light on the geotechnical properties of post-mining waste and their implications for stability and safety.

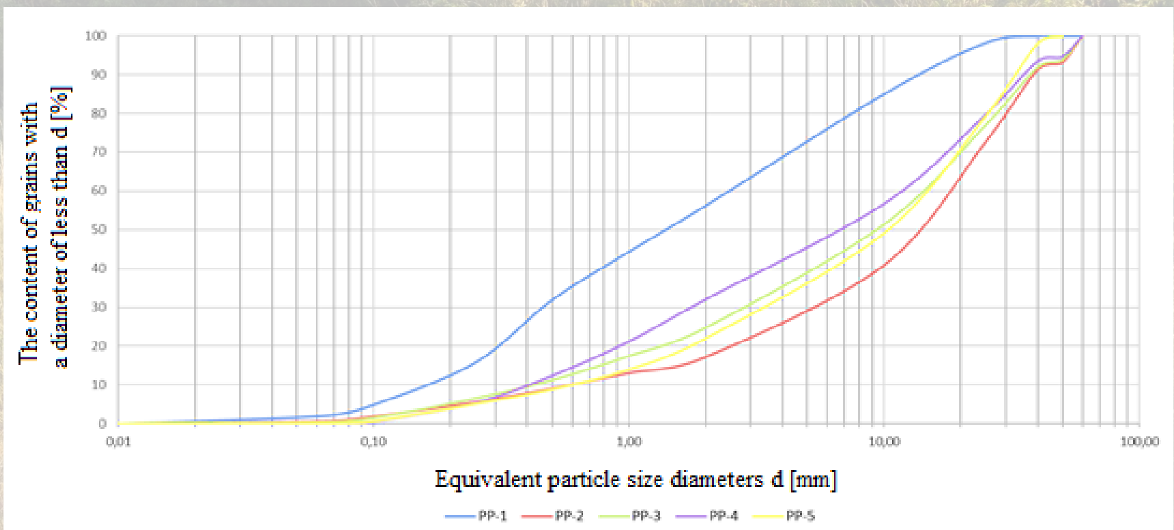

Figure 18 Particle size distribution curves of rock material stored in the dump (storage time: PP-1 -> 35 years, PP-2 - <6 months, PP-3 - about 5 years, PP-4 - 15 years and PP-5 - 15 years after coal recovery).

Chapter VII

Permeability

Permeability is a critical property in geotechnical engineering as it influences water movement within soils. Understanding and considering the permeability of soil is essential for designing stable foundations, assessing slope stability, managing groundwater, and preventing erosion.
The permeability of a soil is determined by factors such as pore size distribution, void ratio, and the connectivity of voids.
High permeability allows water to flow through soil easily, while low permeability restricts water movement. This affects drainage and seepage patterns in slopes and foundations. In high-permeability soils, water tends to drain quickly, reducing the risk of saturation and associated stability issues. In low-permeability soils, water may accumulate, potentially leading to instability.
For example, specific drainage measures can be implemented based on knowledge of the permeability characteristics of a particular soil.

7.1 Typical values of permeability

- The void spaces between the soil grains allow water to flow through them.
- Laminar flow is assumed.

Table 37 Typical values of coefficient of permeability (k).

Soil type		***Description***	***k, m/s***	***Drainage***
Cobbles and boulders	Flow may be turbulent, Darcy's law may not be valid		1	Very good
Gravels	Coarse	Uniformly graded coarse aggregate	10^{-1}	
	Clean		10^{-2}	
			10^{-3}	
Gravel sand mixtures	Clean	Well graded without fines	10^{-4}	Good
Sands	Clean, very fine	Fissured, desiccated, weathered clays	10^{-5}	
	Silty		10^{-6}	
	Stratified clay/silts		10^{-7}	
Silts	Homogeneous below zone of weathering	Compacted clays – dry of optimum	10^{-8}	Poor
			10^{-9}	
			10^{-10}	
Clays		Compacted clays – wet of optimum	10^{-11}	Practically impermeable
			10^{-12}	
Artificial	Bituminous, cements stabilized soil Geosynthetic clay liner / Bentonite enriched soil concrete			

7.2 Comparison of permeability with various engineering materials

- Materials exhibit varying densities.
- Generally, materials with a higher density (relative to their type) tend to have lower permeability.

Table 38 Variability of permeability compared with other engineering materials (Cedergren, 1989).

Material	***Permeability relative to soft clay***
Soft clay	1
Soil cement	100
Concrete	1,000
Granite	10,000
High strength steels	100,000

7.3 Permeability based on grain size

- Grain size is a fundamental factor influencing permeability.
- The Hazen Formula is a valuable tool for estimating permeability based on grain size distribution, particularly for coarse-grained soils (0,1 mm až 3 mm) with relatively uniform particle sizes.
- Ideally for uniformly graded material with U < 5.
- However, it may not provide accurate results for soils with more complex grain size distributions. In those cases, additional testing and analysis may be required to determine permeability accurately.

Table 39 Permeability based on Hazen's relationship.

Coarse grained size	***>Fine sands***		***>Medium sands***				***>Coarse sands***			
Effective grain size d_{10}, mm	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Permeability ($k = Cd^2_{10}$)	10^{-4} m/s		10^{-3} m/s						10^{-2} m/s	
C = 0.10 (above equation)	1	4	0.9	1.6	2.5	3.6	4.9	6.4	0.8	1.0
C = 0.15	1.5	6	1.4	2.4	3.8	5.4	7.4	9.6	1.2	1.5

7.4 Permeability based on soil classification

- Knowing the soil classification can serve as an initial assessment of permeability, but it doesn't consider factors like structure or stratification.

Table 40 Permeability based on soils classification.

Soil type	***Description***	***USC symbol***	***Permeability [m/s]***
Graels	Well graded	GW	10^{-3} to 10^{-1}
	Poorly grated	GP	10^{-2} to 10
	Silty	GM	10^{-7} to 10^{-5}
	Clayety	GC	10^{-8} to 10^{-6}
Sands	Well grated	SW	10^{-5} to 10^{-3}
	Poorly grated	SP	10^{-4} to 10^{-2}
	Silty	SM	10^{-7} to 10^{-5}
	Clayety	SC	10^{-8} to 10^{-6}
Inorganic silts	Low plasticity	ML	10^{-9} to 10^{-7}
	High plasticity	MH	10^{-9} to 10^{-7}
Inorganic clays	Low plasticity	CL	10^{-9} to 10^{-7}
	High plasticity	CH	10^{-10} to 10^{-8}
Organic	with silts/clays of low plasticity	OL	10^{-8} to 10^{-6}
	with silts/clays of high plasticity	OH	10^{-7} to 10^{-5}
Peat	Highly organic soils	Pt	10^{-6} to 10^{-4}

7.5 Permeability of compacted clays and dumps

- Under high pressures, the alteration in permeability experiences only a marginal shift. This relatively minor modification is often disregarded in the majority of analytical assessments due to its limited influence on overall behavior.

Table 41 Laboratory permeability of compacted cooroy clays – CH classification (Look, 1994).

Stress range (kPa)	***40–160***	***160–640***	***640–1280***	***1280–2560***
Typical soil depth (m)	2.0–8.0 m	8.0 m–32 m	32–64 m	>64 m
Permeability, k (m /s)	$0.4–70 \times 10^{-10}$	$0.4–6 \times 10^{-10}$	$0.2–0.7 \times 10^{-10}$	$0.1–0.4 \times 10^{-10}$
Median value, k (m /s)	2×10^{-10}	0.8×10^{-10}	0.4×10^{-10}	0.2×10^{-10}

- Permeability of dump soils directly influences water infiltration and capture on the dump's surface.
- Newly established dumps tend to accumulate most precipitation due to their characteristics such as high porosity and negative pore pressure in newly backfilled claystones, resulting from the relief of original geostatic stress in the overburden.
- Older dumps have typically reduced infiltration capacity. However, in such cases, several groundwater horizons (aquifers) often exist, frequently with confined groundwater levels. This phenomenon arises from the noticeable inhomogeneity of the dump, particularly in the vertical direction, caused by variations in backfilling time intervals.
- The previously dumped benches may develop low-permeability interfaces due to weathering and compaction caused by mining technology. Positions along former shear surfaces may be practically impermeable, while positions with sand, seams, burnt clays, etc., tend to be well permeable.
- Coefficients of permeability vary with depth within clayey dumps.

Table 42 Laboratory permeability of clayey dumps (Vanicek, Schrofel, 1991).

Depth horizont	***Permeability** [m / s]*
At the dump surface (up to 10 m)	9.10^{-4} - 4.10^{-5}
In the depth of 10-50 m	2.10^{-5} - 6.10^{-6}
> 50 m	1.10^{-6} - 9.10^{-11}

- The value of the coefficient of filtering at 1.10^{-6} m.s^{-1} is generally considered the limit for the technical (gravitational) drainage capacity of the dump. If the dump material remains permeable, it may also be collapsible.
- When dealing with potentially collapsible dump material, conduct tests to determine the level of settling collapsibility. Note that the hardest claystone in specific conditions may have a settling collapsibility of 4.5-6.5 %. Loose material may already be impermeable and non-collapsible at normal pressures of 2.5 and 3.5 MPa.

7.6 Relationship between coefficients of permeability and consolidation

- The coefficient of permeability influences the rate at which excess pore water pressure dissipates during consolidation. Soils with higher permeability tend to consolidate more quickly, while soils with lower permeability exhibit a slower consolidation process. Recognizing this relationship is essential for making accurate predictions and engineering decisions in geotechnical projects.
- The coefficient of consolidation (c_v) is influenced by both the permeability and compressibility of the soil.
- Compressibility is a parameter that significantly varies with stress levels. Consequently, c_v is contingent on the stress level.

- Permeability can be derived from the coefficient of consolidation, but this is based on a limited sample size and doesn't consider the overall mass structure.

Table 43 Relationship between coefficients of permeability and consolidation.

Parameter	***Symbol and relationship***
Coefficient of vertical consolidation	$c_v = k/(m_v\gamma_w)$
Coefficient of permeability	K
Unit weight of water	γ_w
Coefficient of compressibility	m_v
Coefficient of horizontal consolidation	c_h = 2 to 10 c_v
Coefficient of vertical permeability	k_v
Coefficient of horizontal permeability	k_h = 2 to 10 k_v

7.7 Typical values of coefficient of consolidation

- The smaller value of the coefficient of consolidation produces a longer time for consolidation to occur.

Table 44 Typical values of the coefficient of consolidation (Carter and Bentley, 1991).

Soil	***Classification***	***Coefficient of consolidation, c_v, m^2/yr***
Boston blue clay	CL	12 ± 6
Organic silt	OH	0.6–3
Glacial lake clays	CL	2.0–2.7
Chicago silty clays	CL	2.7
Swedish medium	CL–CH	0.1–1.2 (Laboratory)
Sensitive clays		0.2–1.0 (Field)
San francisco bay mud	CL	0.6–1.2
Mexico city clay	MH	0.3–0.5

7.8 Variation of coefficient of consolidation with liquid limit

- The coefficient of consolidation is influenced by the liquid limit of the soil.
- As strength improves and structure is lost in remolding, cv decreases.
- LL > 50 % is associated with a high plasticity clay/silt.
- LL < 30 % is associated with a low plasticity clay/silt.

Table 45 Variation of coefficient of consolidation with liquid limit (NAVFAC, 1988).

Liquid limit, %	***30***	***40***	***50***	***60***	***70***	***80***	***90***	***100***	***110***
			Coefficient of consolidation, c_v, m^2/yr						
Undisturbed – virgin compression	120	50	20	10	5	3	1.5	1.0	0.9
Undisturbed – Recompression	20	10	5	3	2	1	0.8	0.6	0.5
Remoulded	4	2	1.5	1.0	0.6	0.4	0.35	0.3	0.25

Case study: Infiltration Test

As part of the processed hydrogeological study of the ČSA residual pit, field infiltration tests were carried out in the area of the internal dump. The aim of these tests was to determine the hydraulic parameters of dump soils and approximate calculation of the future seepage when filling the ČSA lake.Infiltration tests were carried out and evaluated using the method of N. S. Něstěrova Determination of the filtration coefficient is carried out using two concentric cylinders, pushed to a depth of approx. 10 cm so as to minimally disturb the soil structure. Water is poured into the inner cylinder and into the annulus, and a constant level is maintained with the help of Mariotta containers until the time of steady infiltration. The filtration coefficient is calculated according to the formula:

$$K = \frac{Q}{F} \quad [\mathrm{m.s^{-1}}]$$

K . . . *filtration coeficient [$m.s^{-1}$]*
Q . . . *volume of soaked water [$m^3.s^{-1}$]*
F . . . *the area of the bottom of the inner cylinder [0,1 m^2]*

The course and result of infiltration tests can be affected to some extent by weather conditions. The permeability of the soil depends mainly on its geomechanical properties. However, the current air temperature, amount of precipitation, or long-term drought or permanent frost also have a certain influence.
After a detailed reconnaissance of the terrain of the internal dump, 11 (IT01 – IT011) locations were selected for infiltration tests. At the same time, the fact that infiltration tests were already carried out on the internal dump.

Figure 19 Procedure for carrying out infiltration tests on the internal wastedump of the ČSA mine.

The results of the infiltration tests show that the partially consolidated clay rocks placed in the upper floors of the dumps in the area of the Most basin are characterized by poorly permeable or impermeable soils. Filtration coefficients of these soils range in the order of n.10^{-5} – n.10^{-6} $\mathrm{m.s^{-1}}$. Based on these results, we can assume that the resulting absolute water infiltration (loss) when filling the future CSA lake will probably be disposable and in the order of percent (max. 5%) of the total volume of the lake.

Table 46 Location of infiltration test sites and filtration coeficient.

Test site	***Filtration coeficient [m.s⁻¹]***
IT 01	$7{,}96.10^{-5}$
IT 02	$1{,}33.10^{-5}$
IT 03	$2{,}92.10^{-4}$
IT 04	$4{,}08.10^{-5}$
IT 05	$2{,}04.10^{-5}$
IT 06	$5{,}50.10^{-5}$
IT 07	$1{,}21.10^{-5}$
IT 08	$1{,}88.10^{-6}$
IT 09	$1{,}36.10^{-4}$
IT 010	$2{,}56.10^{-6}$
IT 011	$2{,}67.10^{-5}$

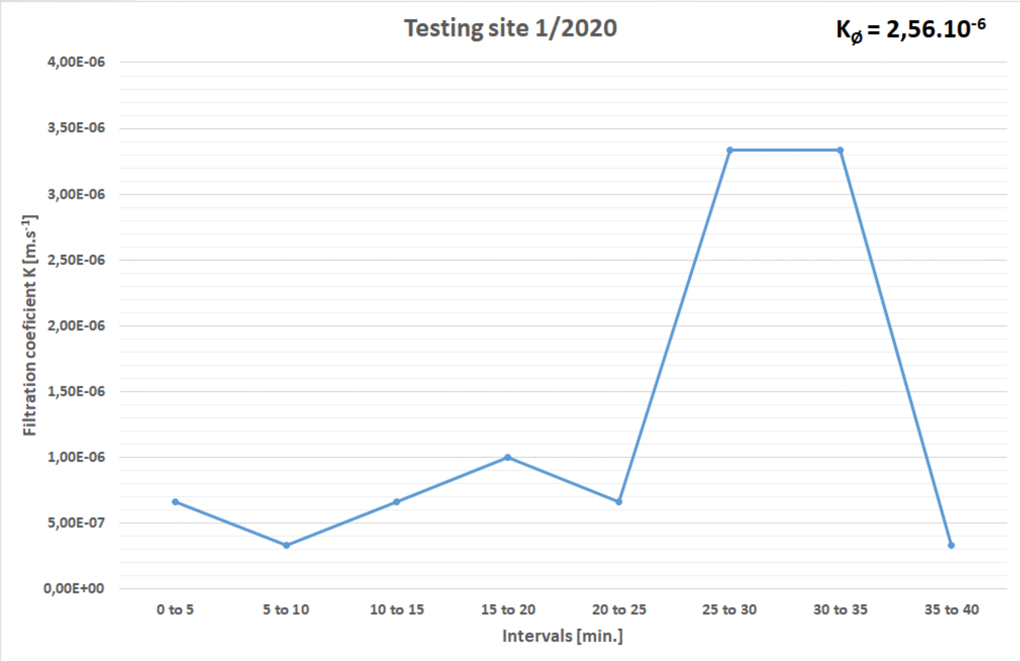

Figure 20 Time course of the infiltration test (1/2020).

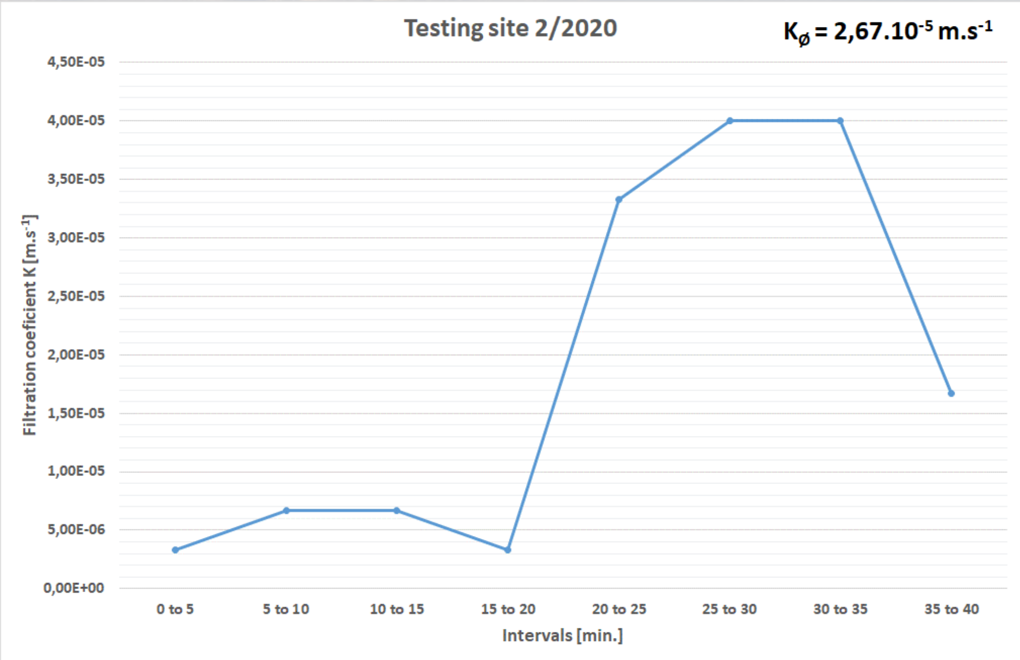

Figure 21 Time course of the infiltration test (2/2020).

7.9 Time factors for consolidation

- The time to achieve a given degree of consolidation = $t = T_v\, d^2/c_v$.
- Time Factor = T_v.
- D = maximum length of the drainage path = ½ layer thickness for drainage top and bottom.
- Degree of Consolidation = U = Consolidation settlement at a given time (t)/Final consolidation settlement.
- $\alpha = u_0(top)/u_0(bottom)$, where u_0 = initial excess pore pressure.

Table 47 Time factor values (from NAVFAC DM 7-1, 1982).

Degree of consolidation	***Time factor T_v***		
	$\alpha = 1.0$ (two way drainage)	$\alpha = 0$ (one way drainage – bottom only)	$\alpha = \infty$ (one way drainage – top only)
10 %	0.008	0.047	0.003
20 %	0.031	0.100	0.009
30 %	0.071	0.158	0.024
40 %	0.126	0.221	0.048
50 %	0.197	0.294	0.092
60 %	0.287	0.383	0.160
70 %	0.403	0.500	0.271
80 %	0.567	0.665	0.440
90 %	0.848	0.940	0.720

7.10 Time required for drainage of deposits

- The duration of drainage is influenced by both the coefficient of consolidation and the length of the drainage path.
- t90 – time for 90 % consolidation to occur
- The presence of silt and sand lenses within clays has an impact on the length of the drainage path.
- Vertical drains containing silt and sand lenses can markedly shorten the drainage paths, consequently accelerating the consolidation process.
- Conversely, in the absence of such lensing, wick drains may prove ineffective for thicker layers, as the installation process may lead to smearing of the wicks, potentially reducing permeability.

Table 48 Time required for drainage.

Material	***Approximate coefficient of consolidation, Cv (m^2/yr)***	***Approx. time for consolidation based on drainage path length (m)***			
		0.3	***1***	***3***	***10***
Sands & Gravels	100,000	<1 hr	<1 hr	1 to 10 hrs	10 to 100 hrs
Sands	10,000	<1 hr	1 to 10 hrs	10 to 100 hrs	1 to 10 days
Clayey sands	1000	3 to 30 hours	10 to 100 hrs	3 to 30 days	1 to 10 mths
Silts	100	10 to 100 hours	3 to 30 days	1 to 10 mths	10 to 100 mths
CL clays	10	10 to 100 days	1 to 10 months	1 to 10 yrs	10 to 100 yrs
CH clays	1	3 to 30 months	1 to 10 yrs	30 to 100 yrs	100 to 1000 yrs

Chapter VIII

Deformation parameters

Deformation parameters are another important factor in addressing the risk management of residual lakes. They assist in predicting the behavior of the soil environment under various loads and deformations, which is essential for the design of stable and reliable geotechnical structures. These material properties have a significant impact on the design and stability of structures such as foundations, dams, tunnels, and other geotechnical constructions.
The deformation parameters use modulus, which quantifies how the material responds to the applied load or deformation.
The modulus can be used, for example, to predict the degree of compression of the rock mass during filling. This allows engineers to predict how the rock mass will behave under the influence of water pressure.
Deformation parameters can be employed to assess the stability of residual pits. Analyzing how quickly and to what extent the rock mass can deform can provide crucial information about potential risks to structures above the pit.

8.1 Modulus definitions

- The stiffness of a soil or rock is defined by its modulus. This modulus represents the ratio of stress to strain at a specific point or within a defined area.
- Even materials with similar strength may exhibit different stiffness characteristics.
- The appropriate modulus depends on the range of strain being considered.
- Fine-grained soils display a significant disparity between their long-term and short-term moduli, while granular soils show only a slight variation. The latter is generally regarded as being nearly equivalent for practical purposes.
- Modulus is typically inferred from strength correlations, with the two most prevalent being:
    - Secant modulus, commonly employed in models concerning soil-structure interaction.
    - Resilient modulus, pertinent to roads.

Table 49 Modulus definitions.

Modulus type	***Definition***	***Strain***	***Comment***
Initial tangent modulus	Slope of initial stress concave line	Low	Due to closure in micro-cracks from sampling relief (laboratory) or existing discontinuities (in-situ).
Elastic tangent modulus	Slope of linear point (near linear)	Medium	Also elastic modulus. Can be any specified on the stress strain curve, but usually at a specified stress levels such as 50 % of maximum or peak stress.
Deformation modulus	Slope of line between zero and maximum or peak stress	Medium to high	Also secant modulus.
Constrained modulus	Slope of line between zero and constant volume stress	High	This is not mentioned in the literature. But values are lower than a secant modulus, and it is obtained from odeometer tests where the sample is prevented from failure, therefore sample has been take to a higher strain level.

Recovery modulus	Slope of unload line	High	Insitu tests seldom stressed to failure, and unload line does not necessarily mean peak stress has been reached. Usually concave in shape.
Reload modulus	Slope of reload line	High	Following unloading the reload line takes a different stress path to the unload line. Usually convex in shape. Also resilient modulus.
Cyclic modulus	Average slope of unload/reload line	High	Strain hardening can occur with increased number of cycles.
Equivalent modulus	A combination of various layers into on modulus	Various	A weighted average approach is usually adopted.

8.2 Modulus applications

- There is considerable uncertainty surrounding modulus values and their application.
- The table offers a tentative ranking of relative moduli. A rank of 1 corresponds to the smallest values, with increasing numbers indicating larger moduli. However, this can differ among different materials. For instance, an initial tangent modulus in a clay sample without micro cracks might have a higher value than the secant modulus at failure, deviating from the rankings in the table.
- The relative values are contingent on the type of material, the condition of the soil, and various loading factors.
- Certain applications (such as pavements) may involve high stress levels but relatively low strain levels. In such cases, a strain-based criterion is applicable. Conversely, for other applications like foundations, a stress-based criterion is used in design.
- Typically, only one modulus is utilized in the design process, even if the structure experiences multiple modulus ranges.
- Modulus values between applications with small strains and those with large strains can vary by a factor of 5 to 10.
- In terms of dynamic modulus, for granular, cohesive materials, and rock, it can exceed the static modulus value by more than 2, 5, and 10 times, respectively.

Table 50 Modulus applications.

Rank	***Modulus type***	***Application***	***Comments***
1 (Low)	Initial tangent modulus	• Fissured clays. • At low stress levels. Some distance away from loading source, eg at 10 % $q_{applied}$ • Low height of fill	Following initial loading and closing of micro-cracks, modulus value then increases significantly. For an intact clay, this modulus can be higher than the secant modulus.
2	Constrained modulus	• Wide loading applications such as large fills • Wide embankments	Used where the soil can also fail, ie exceed peak strength.
3	Deformation (secant) modulus	• Spread footing • Pile tip	Most used “average” condition, with secant value at ½ peal load (ie working load).
4	Elastic tangent modulus	• Movement in incremental loading of a multi-storey building • Pile shaft	The secant modulus can be 20 % the initial elastic tangent modulus for an intact clay.
5	Reload (resilient) modulus	• Construction following excavation • Subsequent loading from truck/train	Difficult to measure differences between Reload/Unload or cyclic. Resilient modulus term interchangeably used for all of them. Also called dynamic modulus of elasticity.

6	Cyclic modulus	• Machine foundations • Offshore structures/ waveloading • Earthquake/blast loading	
7	Recovery (unload) modulus	• Heave at the bottom of an excavation • After loading from truck/train • Excavation in front of wall and slope	
Varies	Equivalent modulus	• Simplifying overall profile, where some software can have only 1 input modulus	Uncertainty on thickness of bottom layer (infinite layer often assumed). Relevant layers depend on stress influence.

8.3 Typical values for elastic parameters

- Metals possess notably greater strength compared to the ground. Consequently, the behavior of the structure is primarily influenced by movements originating from the ground.
- For industrial concrete floors, modulus values of 30,000 MPa would be applicable.

Table 51 Typical values for Young's modulus of various materials (after Gordon, 1978).

Classification	***Material***	***Young's modulus, E (MPa)***
Human	Cartilage	24
	Tendon	600
	Fresh bone	21,000
Timber	Wallboard	1,400
	Plywood	7,000
	Wood (along grain)	14,000
Metals	Magnesium	42,000
	Aluminium	70,000
	Brasses and bronzes	120,000
	Iron and steel	210,000
	Sapphire	420,000
	Diamond	1,200,000
Construction	Rubber	7
	Concrete	20,000
Soils	Soft clays	5
	Stiff clays, loose sands	20
	Dense sands	50
Rocks	Extremely weathered,soft	50
	Distinctly weathered, soft	200
	Slightly weathered, fresh, hard	50,000

8.4 Elastic parameters of various soils

- Foundation design relies on secant modulus values, which can vary based on the magnitude of strain levels, potentially being either higher or lower.
- It is crucial not to apply these modulus values to different contexts, such as non-foundation uses. For instance, if we consider the modulus values for similar soils used as backfill around a pipe in a trench, they would be considerably lower than the values mentioned above.

Table 52 Elastic parameters of various soils.

Type	***Strength of soil***	***Elastic modulus, E (MPa)***	
		Short term	***Long term***
Gravel	Loose	25–50	
	Medium	50–100	
	Dense	100–200	
Medium to coarse sand	Very loose	<5	
	Loose	3–10	
	Medium dense	8–30	
	Dense	25–50	
	Very dense	40–100	
Fine sand	Loose	5–10	
	Medium	10–25	
	Dense	25–50	
Silt	Soft	<10	<8
	Stiff	10–20	8–15
	Hard	>20	>15
Clay	Very soft	<3	<2
	Soft	2–7	1–5
	Firm	5–12	4–8
	Stiff	10–25	7–20
	Very stiff	20–50	15–35
	Hard	40–80	30–60

8.5 Deformation parameters from CPT results

- The CPT results provide the means to obtain the Coefficient of Volume Change and the constrained modulus (i.e., under large strain conditions).
-

Table 53 Deformation parameters from CPT results (Fugro, 1996; Meigh, 1987).

Parameter	***Relationship***	***Comments***
Coefficient of volume change, m_v soils	$m_v = 1/(\alpha\ q_c)$	For normally and lightly overconsolidated α = 5 for classifications CH, MH. ML α = 6 for classifications CL, OL α = 1.5 for classifications OH with moisture >100 % for overconsolidated soils α = 4 for classifications CH, MH. CL, ML α = 2 for classifications ML, CL with q_c > 2 MPa
Constrained modulus, M	$M = 3\ q_c$	$M = 1/m_v$
Elastic (Young's) modulus, E	$E = 2.5\ q_c$	Square pad footings – axisymetric
	$E = 3.5\ q_c$	Strip footings – plane strain

8.6 Drained soil modulus from cone penetration tests

- The table furnishes an approximate correlation between CPT values and the drained elastic modulus for sands.

Table 54 Preliminary drained elastic modulus of sands from cone penetration tests.

Relative density	***Cone resistance, q_c , (MPa)***	***Typical drained elastic modulus E′, MPa***
V. loose	<2.5	<10
Loose	2.5–5.0	10–20
Med dense	5.0–10.0	20–30
Dense	10.0–20.0	30–60
V. dense	>20.0	>60

8.7 Drained modulus of clays based on strength and plasticity

- The drained modulus of soft clays is correlated with both its undrained strength (Cu) and its plasticity index.

Table 55 Drained modulus values (from Stroud et al., 1975).

Soil plasticity (%)	***E'/C_u***
10–30	270
20–30	200
30–40	150
40–50	130
50–60	110

8.8 Undrained modulus of clays for varying over consolidation ratios

- The undrained modulus Eu depends on the soil strength, its plasticity and overconsolidation ratio (OCR).

Table 56 Variation of the undrained modulus with overconsolidatio ratio (Jamiolkowski et al., 1979).

Overconsolidation ratio	***Soil plasticity***	***E_u/C_u***
<2	PI < 30%	600–1500
2–4		400–1400
4–6		300–1000
6–10		200–600
<2	PI = 30–50%	300–600
2–4		200–500
4–10		100–400
<2	PI > 50%	100–300
2–10		50–250

8.9 Poisson ratio in soils

- The Poisson's ratio is important because it provides crucial information about how the soil will behave under loading. More specifically, it indicates how the material will deform.
- The Poisson's ratio also affects the transmission of loads and the distribution of stresses in the soil, which can have a significant impact on the overall bearing capacity and safety of the structure. Therefore, it is considered an important parameter in the analysis and design of geotechnical structures.

- In an undrained state, clay exhibits a Poisson ratio of 0.5. However, during the Oedometer test, where lateral strain is nearly negligible (close to zero), the Poisson ratio effectively becomes 0.0.

Table 57 Poisson's ratio for soils (Industrial floors and pavements guidelines, 1999).

Material	***Short term***	***Long term***
Sands, gravels and other cohesionless soils	0.30	0.30
Low PI (<12 %)	0.35	0.25
Medium PI (12 % < PI < 22 %)	0.40	0.30
High PI (22 % < PI < 32 %)	0.45	0.35
Extremely high PI (PI > 32 %)	0.45	0.40

8.10 Significance of modulus

- The applicable modulus value is contingent on the relative influence of stress.

Table 58 Significance of modulus (Deere et al., 1967).

Modulus ratios for rock	***Comments***
$E_d/E_{conc} > 0.25$	Foundation modulus has little effect on stresses generated within the concrete mass.
$0.06 < E_d/E_{conc} < 0.25$	Foundation modulus becomes significant with respect to stresses generated within the concrete mass.
$0.06 < E_d/E_{conc}$	Foundation modulus completely dominates the stresses generated within the concrete mass.

Chapter VII

Slope stability assessment

Assessment of slope stability in the event of residual pit flooding is essential to minimise potential hazards. The increased presence of water can significantly affect soil properties, which in turn affects the stability.

Factors such as pore water pressure, soil saturation levels and changes in material properties need to be carefully considered. In addition, it is important to conduct a thorough geotechnical investigation, as mentioned earlier in this paper, and then apply appropriate engineering techniques such as drainage systems or reinforcement measures, which are key steps to ensure the stability and safety of these pits. Regular monitoring and maintenance are also necessary to address any evolving stability concerns over time. The following are some practices that should be considered when evaluating slope stability.

9.1 Slope measurement

- Slope measurement involves the assessment and quantification of the inclination or gradient of a terrain or structure. Accurate slope measurements provide valuable data for ensuring stability, safety, and effective design in projects involving elevated or sloping surfaces.
- Slopes are commonly expressed as 1 Vertical: Horizontal slopes as highlighted. This physical measurement is easier to construct (measure) in the field, although for analysis and design purpose the other slope measurements may be used.

Table 59 Slope measurements.

Descriptor	***Degrees (°)***	***Radians***	***Tangent***	***Percentage (%)***	***1Vertical: Horizontal***	***Design considerations***
Flat	0	0.000	0.000	0	∞	Drainage
Moderate	5	0.087	0.087	9	11.43	
	10	0.174	0.176	18	5.67	
Steep	11.3	0.197	0.200	20	5.00	Slope design
	15	0.262	0.268	27	3.73	
	18.4	0.322	0.333	33	3.00	
	20	0.349	0.364	36	2.75	
	25	0.436	0.466	47	2.14	
Very steep	26.6	0.464	0.500	50	2.00	
	30	0.524	0.577	58	1.73	
	33.7	0.588	0.667	67	1.50	
	35	0.611	0.700	70	1.43	
	40	0.698	0.839	84	1.19	
Extremely steep	45	0.785	1.000	100	1.00	Reinforced design if a soil slope
	50	0.873	1.192	119	0.84	
	55	0.960	1.428	143	0.70	
	60	1.047	1.732	173	0.58	
	63	1.107	2.000	200	0.50	
	65	1.134	2.145	214	0.47	
Sub-Vertical	70	1.222	2.747	275	0.36	Wall design if a soil slope
	75	1.309	3.732	373	0.27	
	76	1.326	4.000	400	0.25	
	80	1.396	5.671	567	0.18	
	85	1.483	11.430	1143	0.09	
Vertical	90	1.571	∞	∞	0.00	

9.2 Causes of slope failure

- The upcoming table will outline the micro scale effects that lead to slope movement.
- Slope instability can result from either a decrease in soil strength or an increase in stress levels.
- Influential Factors on Slopes:
    - *Load*
    - *Strength*
    - *Geometry*
    - *Water Conditions*
- The load may be permanent, such its own weight or transient (dynamic from a blast).

Table 60 Causes of slope failure (adapted from Duncan and Wright, 2005).

Decrease in soil strength	*Increase in shear stress*
• Increased pore pressure (reduced effective stress). Change in water levels. High permeability soils have rapid changes. This includes coarse grained soils, clays with cracks, fissures and lenses.	• Loads at the top of the slope. Placement of fill and construction of buildings on shallow foundation near crown of slope.
• Cracking. Tension in the soil at the ground surface. Applies only in soils with tensile strength. Strength is zero in the cracked zone.	• Water pressure in cracks at the top of the slope. Results in hydrostatic pressures. If water in cracks for extended periods seepage results with an increase in pore pressures.
• Swelling. Applies to highly plastic and overconsolidated clays. Generally a slow process (10 to 20 years). Low confining pressures and long periods of access to water promote swell.	• Increase in soil weight. Change in water content due to changes in the water table, infiltration or seepage. Increasing weight of growing trees and wind loading on those trees.Vegetation has a stabilising effect initially (cohesion effect of roots).
• Development of Slickensides. Applies mainly to highly plastic clays. Can develop as a result of tectonic movement.	• Excavation at the bottom of the slope. Can be man made or due to erosion at base of slope.
• Decomposition of clayey rock fills. Clay shales and claystone may seem like hard rock initially, but when exposed to water may slake and degrade in strength.	• Change of slope grade. Steepening of slope either man made (mainly) or by natural processes.
• Creep under sustained load. Applies to highly plastic clays. May be caused by cyclic loads such as freeze – thaw or wet – dry variations.	• Drop in water level at base of slope. Water provides a stabilising effect. Rapid drawdown effect when this occurs rapidly.
• Leaching. Change in chemical composition. Salt leaching from marine clays contributes to quick clays, which have negligible strength when disturbed.	• Dynamic loading. Usually associated with earthquake loading or blasting. A horizontal or vertical acceleration results. This may also result in a reduction in soil strength.
• Strain Softening. Applies to brittle soils. • Weathering. Applies to rocks and indurated soils. • Cyclic Loading. Applies to soils with loose structure. Loose sands may liquefy.	

- The perceived stability is significantly influenced by the analytical model employed and its subsequent interpretation.
- Shallow (surficial) failures frequently manifest in the aftermath of rainfall events. For these cases, an infinite slope analysis is conducted, accounting for steady-state seepage parallel to the slope. It's important to note that surficial failures can involve a substantial mass of soil and are not necessarily confined to small slides.
- Deep-seated failures necessitate the application of both translational and rotational slope stability analyses.

- Water plays a pivotal role in most of the aforementioned factors that contribute to instability.

9.3 Factors of safety for slopes

- The factor of safety is used in the analysis of slope stability, including in the case of residual pits, to ensure that the slope can withstand the various forces acting upon it without failure.
- Residual pits may be in use for extended periods, and long-term factors like weathering, erosion, and changes in water table levels can influence stability over time. The factor of safety helps assess the long-term stability of the pit.
- This underscores the importance of comprehending the safety factor, as it enables engineers and stakeholders to evaluate the extent of risk linked to a specific slope design.
- The factor of safety is determined by comparing the restoring condition to the activating condition.
- The condition under consideration may involve forces or moments.
- For the analysis of rotational slides, moment equilibrium is typically employed, involving the examination of circular slip surfaces.
- In the case of rotational or translational slides, force equilibrium is generally applied, and analysis may involve circular, planar, wedge, or polygonal slip surfaces.
- Various factors of safety are necessitated based on the specific facility and its impact on the surrounding environment.

Table 61 Factor of safety dependency.

Variable	***Effect on Factor of safety***	***Comment***
Strength • Lowest value • Lower quartile • Median	Lower quartile should be typically used. Higher or lower should have corresponding changes on acceptable factor of safety.	Mean values should not be used due to the non normality of soil and rock strength parameters.
Geometry • Height • Slope • Benching • Stratification/discontinuities	Higher slopes at a given angle would be more unstable than a low height slope. Dip of weakness plane towards slope face influences result.	Benching also useful to reduce erosion, provides rock trap area, and as a maintenance platform.
Load • Weight • Surcharge • Water Conditions	Water is the most significant variable in design. Buoyant unit weight then applies at critical lower stabilizing part of slope, i.e. soil above is heavier than soil below.	The weight acts both as an activating and restoring force.
Analytical methods • Method of slices • Wedge methods	Different methods (and some software programs) give different outputs for the same data input. Moment equilibrium and force equilibrium methods can sometimes produce different results, especially with externally applied loads.	Probability of failures/ displacement criteria should also be considered in critical cases. Factor of safety for 3 – dimensional effect ~15% greater than 2-D analysis.

- The selection of the factor of safety is also influenced by the quality of accessible geotechnical data and the choice of parameters, whether opting for the worst credible scenario, probabilistic mean, or a conservative best estimate.
    - *Temporary works might employ lowered factors of safety.*
    - *Projects in critical areas would necessitate the application of elevated factors of safety.*

9.4 Factors of safety for existing slopes

- In general, existing slopes tend to possess a lower factor of safety compared to newly constructed slopes.
- Existing slopes have typically been exposed to various environmental influences and have undergone a process of equilibration.

Table 62 Factors of safety for existing slopes (adapted from GEO, 1984).

Risk	***Required factor of safety with loss of life for a 10 years return period rainfall***
Negligible	>1.1
Low	1.2
High	1.3 – 1.5

9.5 Economic and environmental risk

- Balancing economic and environmental risks in slope stability assessments involves considering the costs associated with stabilization measures against the potential economic and environmental losses that may result from slope failure. It also requires taking into account sustainable and environmentally-friendly engineering solutions to mitigate these risks.
- Environmental risk, in the context of slope stability or any project, can extend beyond just ecological concerns. It can also encompass political risk and how the project is perceived by the public or stakeholders.

Table 63 Economic and environmental risk (adapted from GEO, 1984).

Situation	***Risk***
Open farmland, country parks, lightly used recreation areas of low amenity value	Negligible
Country roads and low traffic intensity B roads, open air car parks	Negligible
Facilities whose failure would cause only slight pollution	Negligible
Essential services (eg gas, electricity, water, whose failure would cause loss of service)	Low
Facilities whose failure would cause significant pollution or severe loss of amenity (cultivated public gardens, with established and mature trees)	Low
High traffic density B roads and all A roads, residential, low rise commercial, industrial and educational properties	Low
Facilities whose failure would cause significant pollution	High
Essential services whose failure would cause loss of service for a prolonged period	High
All A Roads, by- passes and motorways, including associated slip roads, petrol stations and service areas	High
Buildings storing hazardous goods, power stations (all types), nuclear, chemical, and biological complexes	High

9.6 Stable slopes underwater

- Solely relying on slope stability analysis may not accurately depict the stability of a slope submerged in water.
- Fully submerged slopes often maintain stability at considerably shallower angles than what slope stability analysis suggests.
- This is attributed to the dynamic forces of water and the ongoing erosive effects beneath the water's surface.

Table 64 Typical slopes under water (ICE, 1995).

Type of material	***Description***	***Slopes in still water***		***Slopes in active water***	
Rock		Nearly vertical		Nearly vertical	
Clay	Stiff	45°	1V: 1H	45°	1V: 1H
	Firm	35°	1V: 1.4H	30°	1V: 1.7H
	Sandy	25°	1V: 2.1H	15°	1V: 3.7 H
Sand	Coarse	20°	1V: 2.7H	10°	1V: 5.7H
	Fine	15°	1V: 3.7H	5°	1V: 11.4H
Silt	Mud	0–1°	1V: 5.7H to 57H	<5°	1V: 11.4 H or less

9.7 Variability in design and construction process

- The design and construction process of slopes exhibits a notable degree of variability, influenced by a range of factors. These may include the geological and geotechnical characteristics of the site, environmental considerations, project objectives, and budget constraints. Additionally, local regulations and codes play a pivotal role in shaping the approach taken in designing and constructing slopes. Engineers and geotechnical experts employ diverse methodologies and materials, tailoring their approach to each specific project's unique requirements. This variability underscores the need for a comprehensive understanding of site-specific conditions and a flexible, adaptive approach to slope design and construction.
- The variability components and their associated coefficient of variation percentages represent the range of uncertainty and potential variation in different aspects of the design and construction process.

Table 65 Variations in Design and construction process based on fundamentals only (Kay, 1993).

Variability component	*Coefficient of variation*
Design model uncertainty	0–25%
Design decision uncertainty	15–45%
Prototype test variability	0–15%
Construction variability	0–15%
Unknown unknowns	0–15%

9.8 Tolerable risk for new and existing slopes

- The probabilities of failure are often more comprehensible to professionals from various disciplines and clients compared to factors of safety.
    - *A factor of safety of 1.3 does not automatically imply a lower probability of failure than a factor of safety of 1.4 or 1.5.*
- Different criteria must be employed when assessing existing slopes as opposed to new ones.

Table 66 Tolerable risks for slopes (AGS, 2000).

Situation	*Tolerable risk probability of failure*	*Loss of life*
Existing slope	10^{-4}	Person most at risk
	10^{-5}	Average of persons at risk
New slopes	10^{-5}	Person most at risk
	10^{-6}	Average of persons at risk

9.9 Acceptable probability of slope failures

- An "acceptable probability of slope failures" refers to the level of risk that is deemed tolerable or permissible in relation to potential slope instabilities. It represents the likelihood or chance that a slope may experience failure or instability within a specified time frame or under certain conditions.
- This acceptable probability is determined based on various factors including engineering standards, regulatory requirements, environmental considerations, and the potential consequences of a slope failure. In practice, it represents a balance between ensuring safety and the costs associated with implementing stabilization measures.
- For example, in critical areas such as near populated areas, infrastructure, or environmentally sensitive regions, a lower acceptable probability of failure may be set to prioritize safety. In less critical areas, where the consequences of a failure may be less severe, a slightly higher acceptable probability might be considered.
- Ultimately, defining an acceptable probability of slope failures is a critical aspect of risk assessment and management in geotechnical engineering. It helps guide decision-making and

informs the design and construction process to ensure that slopes are engineered to meet established safety standards.

Table 67 Tolerable risks for slopes (AGS, 2000).

Conditions	***Risk to life***	***Costs***	***Probability of failure (Pf)***
Unacceptable in most cases			$<10^{-1}$
Temporary structures	No potential life loss	Low repair costs	10^{-1}
Nil consequences of failure bench slope, open pit mine	No potential life loss	High cost to lower Pf	1 to 2×10^{-1}
Existing slope of riverbank at docks. Available alternative docks	No potential life loss	Repairs can be promptly done. Do – nothing attractive idea.	5×10^{-2}
To be constructed: same condition			$<5\times10^{-2}$
Slope of riverbanks at docks no alternative docks operations.	No potential life loss	Pier shutdown threatens	1 to 2×10^{-2}
Low consequences of failure	No potential life loss	Repairs can be done when time permits. Repair costs < costs to lower P_f.	10^{-2}
Existing large cut – interstate highway	No potential life loss	Minor	1 to 2×10^{-2}
To Be constructed: same condition	No potential life loss	Minor	$<10^{-2}$
Acceptable in most cases	No potential life loss	Some	10^{-3}
Acceptable for all slopes	Potential life loss	Some	10^{-4}
Unnecessarily low			$<10^{-5}$

9.10 Approach to slope stability analysis

- Two types of approaches to the stability analysis use to be - classical analysis according to the factor of safety and the analysis following the theory of limit states:
    - The verification methodology based on the "**Limit states**" theory proves the safety by comparing a resisting variable (resisting force, strength, bearing capacity) and a variable causing failure (sliding force, stress).

$$X_{pas} > X_{act}$$

where: Xpas - A variable resisting the failure (resisting force, strength, capacity)
Xact - A variable causing the failure (sliding force, stress)

    - The verification methodology of structure safety based on the "**Safety factor (FS)**" is historically the oldest and most widely used approach. The principal advantage is its simplicity and lucidity. In general, safety is proved using the safety factor:

$$FS = \frac{X_{pas}}{X_{act}} > FS_{req}$$

where: FS - Computed safety factor
Xpas - A variable resisting the failure (resisting force, strength, capacity)
Xact - A variable causing the failure (sliding force, stress)
FSreq - A Required factor of safety

- When performing the analysis using the "Safety factor", neither the load nor the soil parameters are reduced by any of the design coefficients.
- The verification based on "Limit states" is a more modern approach than the "Safety factor". However, it is less lucid.

9.11 Analysis in 3D vs 2D

- Slope stability analysis can be analyzed using both by **2D** and **3D**. In general, 2D approach is simpler than 3D analysis (volume and quality of spatial data, quality of geotechnical model, computational power etc.).
- However, if it is seen from its width of slope assumption, 2D analysis can be unrepresentative because of the infinite width of the slope assumption - the conservative (most critical) shape of the shear surface is considered.
- By analyzing a slope in 3D, the result can be more acceptable because it is considered the limit of the width of a failure.
- Moreover, the volume of failure can also be estimated so it can be used as a consideration in the decision making according to the slope function.

9.12 Limit equilibrium vs Finite Element Method (FEM)

- Two finite element method approach allows model and analyze a wide range of geotechnical problems, including terrain settlement, sheet pile/diaphragm walls, slope stability, excavation analysis.
- It offers several material models for soils and a variety of structural elements such as walls, anchors, geotextiles or geogrids.
- The FEM is used to compute displacements, internal forces in structural elements, stresses and strains and plastic zones in the soil and other quantities in every construction stage.
- FEM also performs the Tunnel excavation analysis, the steady sate or transient Water Flow analysis, the coupled Consolidation analysis, or the dynamic impact of an Earthquake.states.
- In the stability (safety factor) analysis the program reduces the original strength parameters - the angle of internal friction and cohesion - until failure occurs. The analysis then results in a factor of safety that corresponds to the classical methods of limit equilibrium.
- Since plastic slip is the main failure mechanism we also require that the Mohr-Coulomb, the modified Mohr-Coulomb, or the Drucker-Prager plasticity model be used for all soils.

Case study: Factor of safety

To develop a reliability methodology for assessing the long-term stability of flooded open pit mines, a large-scale numerical model of the lake was carried out and was applied on lake Most, which is one of the largest mining lakes in Europe. The large-scale numerical model was built, based on the site observations, large scale LiDAR data and geotechnical data. The results highlighted the reliability of the methodology to combine the geometric model with the geological model to create a large-scale numerical model, and to identify local and potentially instable zones.

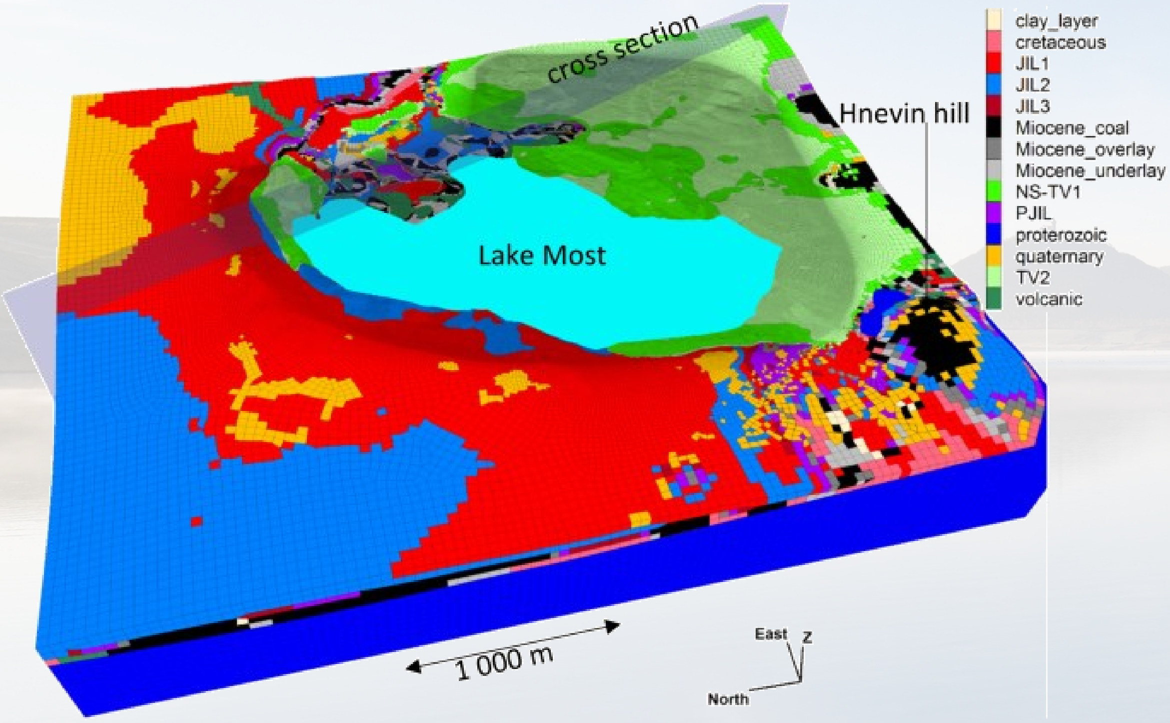

Figure 22 3D geological model of lake Most (11,826,069 elements); rectangular limit: area where the FS is minimal.

The use of the strength reduction method produces one global minimum stability state per simulation. This method is applied with the Mohr-Coulomb failure criterion by progressively reducing the shear strength of the material to bring the slope to a state of limiting equilibrium. However, along a complex slope profile, it is interesting to be able to compute multiple minimum states. Instead of excluding different regions of the slope when performing the strength reduction calculation, we have used the ability of Flac3D to compute multiple local stability surfaces in a single simulation.rming the strength reduction calculation, we have used the ability of Flac3D to compute multiple local stability surfaces in a single simulation.

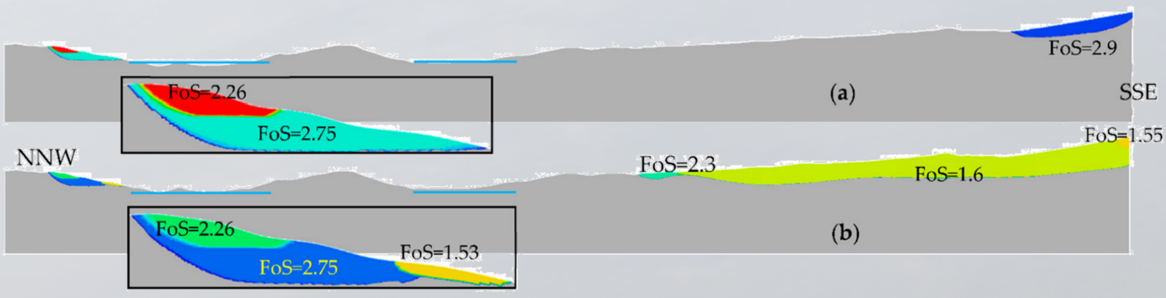

Figure 23 FS contours for the 4629 m length NNW-SSE cross section with distributions of C, E, φ and ρ in the dump units (a) without contact layer. (b) with contact layer.

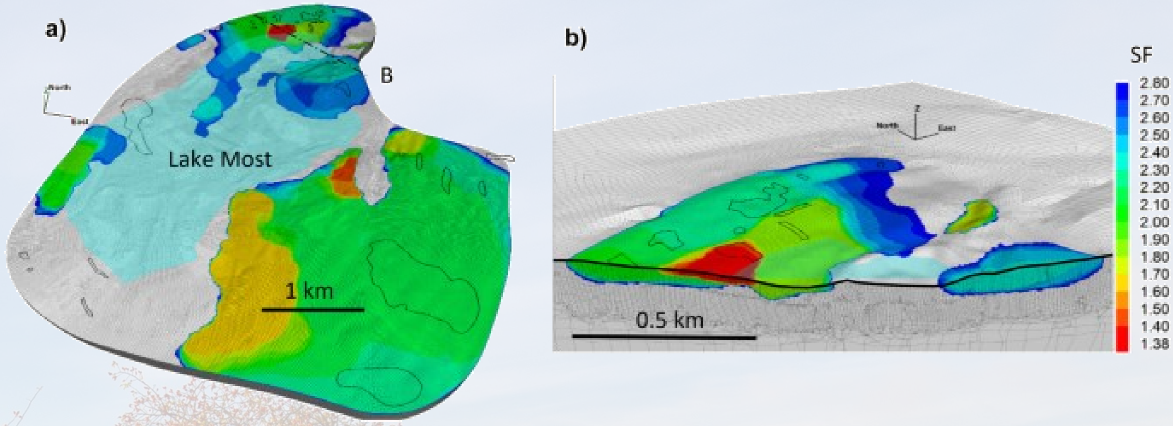

*Figure 26 Safety factors contours – red zones are the critical slope zones (FS < 1.5). (**a**) 3D view, (**b**) cross section.*

Six calculations were made to estimate the safety factor of the Most site in its current (short-term) situation. The 6 calculations were established from the scenarios of geomechanical properties of dump units (mean values, minimum bounds or statistical distribution) and the presence or not of the contact layer at the bottom of the dump bodies. The contact layer is characterized by very low geomechanical parameters ($c = 6$ kPa and $\varphi = 6^{o}$).
First, the 3 scenarios without contact layer produce the same isovalues of FS because these 6 scenarios differ only in the properties of the dump units and because dump units are not the weakest geologic units. On the other hand, the 3 scenarios including the contact layer change the stability of all dump units (younger dump soils: NS-TV1; older dump soils: TV2 and contact layer).
This is explained by the very low properties of the contact layer. Areas with FS of 2.75 (NNW) and 2.9 (SSE) without contact layer have a safety factor of between 1.14 and 1.53 at NNW and between 1.5 and 1.6 at SSE when contact layer is present.
The 3D calculations give results compatible with this 2D cross section: FS = 2.2 and 1.38 without or with contact layer respectively. The global 3D FS is located on the north bank of lake Most (Figure 20) at the location where the majority of slope failure stabilization operations took place in the past. In those zones earth and stabilization work were carried out to insure long-term stability.

The results of the 2D and 3D numerical modelling were analyzed as a large scale by calculating global and local safety factors. The results highlighted the reliability of the methodology to combine the geometric, geological and hydraulic models to create a large-scale numerical model, and to identify local and potentially unstable zones. The hypothesis of the presence of a very weak contact layer (at the bottom of the dump bodies) is therefore a strong hypothesis, capable of questioning the stability of the slopes of the site (lake Most). It should be noted that the contact layer was not detected by the CPT campaign measurements.

Chapter X

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Chapter XI

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