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MECHANICAL BEHAVIOUR OF VARIABLY SATURATED CLAY FILLS

MECHANICKÉ CHOVÁNÍ PROMĚNLIVĚ NASYCENÝCH JÍLOVITÝCH
VÝSYPEK

Ph.D. Thesis

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Declaration:

I declare that I have worked out the dissertation thesis on my own and that I have referenced all used information sources and literature. This thesis or its substantial part has not been submitted to any other or the same academic degree.

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Abstract

The excavated overburden from open-cast coal mines is end-dumped in fills. Clay-type soil prevails among the filled soils in North-western Bohemia. As a consequence of the character and state of the soil, together with the mining and dumping technology, a typical lumpy structure of the clay fill is formed. The filled lumps range in size and vary in mechanical properties, but stiff to solid lumps up to 0.5 m in size are prevalent. Large and open macrovoids between the single lumps are a characteristic feature of the clay fills. Clay fills exhibit a double porosity structure, consisting of intergranular porosity between the lumps and intragranular porosity inside them. The total porosity of the freshly filled soils can reach 70 %. This type of clay fill structure is metastable.

This thesis focuses on the problematic behaviour of clay fill soils due to their metastable structure and their partly saturated state. Long-term monitoring of matric suction at depths up to 2 metres in two reclaimed clay fills revealed two different trends. Seasonally-dependent suction oscillations were recorded in the moderate slope of the clay fill in the Bílina mine, while no suction pointing to fully saturated conditions was measured in the clay fill of the former 5. Květen mine. The hydraulic characteristics for undisturbed lumpy specimens were determined and were used in the inverse analysis of the matric suction. The vertical profiles of matric suction were modelled in different times. High matric suctions and their frequent oscillations at shallow depths were simulated.

Mechanical behaviour of the clay fill soils from two sites was investigated. The compressive strength, matric suction and overconsolidation ratio of individual lumps were investigated. High matric suction acts in the microvoids inside the lumps, even when they are fully saturated. The degradation of the lump structure subjected to wetting-drying cycles was observed. A single flooding event applied to the lump at natural water content is not sufficient to break the structure, while the application of wetting-drying cycles leads to disintegration of the lump. Consequently, suction oscillations contribute to the degradation of the structure and macrovoid closure in the field. Standard laboratory experiments (direct shear box, oedometer) on the

lumpy soil have been supplemented by large scale testing. The mechanical behaviour of the fragmentary soil is influenced by the presence of macrovoids and shows further size and time effects, which makes it difficult to describe the behaviour of the clay fill. The initially lumpy structured soil changes with time back to a homogeneous material, due to weathering processes and through geostatic load. Partial saturation of the fragments also contributes to the peculiar behaviour of the clay fill material.

A literature review and our own experiments defined the thresholds for the collapse potential of the two typical collapse-susceptible soils – low plasticity clays and sands. The collapse potential depends on the degree of saturation, the density and the applied load. The collapse potential of two clay fills was investigated on two specimen types from two clay fills using double oedometer tests. Different collapse potentials was measured for lumpy and undisturbed specimens, respectively.

Abstrakt

Vytěžené nadloží z povrchových hnědouhelných dolů se ukládá na výsypky. V Severozápadních Čechách převládají mezi ukládanými zeminami jílovité zeminy. Následkem charakteru a stavu vytěžené zeminy, dohromady s technologií těžby a ukládáním, vzniká typická hrudkovitá struktura jílovitých výsypek. Ukládané hroudy se liší svou velikostí a mechanickými vlastnostmi, ale převládají tuhé až pevné hroudy o velikostech do 0.5 metru. Velké a otevřené makropóry mezi jednotlivými hrudkami jsou charakteristickým jevem jílovitých důlních výsypek. Výsypky vykazují strukturu s dvojitou pórovitostí skládající se z mezerovitosti (intergranulární pórovitosti) mezi hrudkami a intragranulární pórovitosti uvnitř hrudek. Celková pórovitost čerstvě nasypaných výsypek může tedy dosahovat až 70%. Tento typ struktury výsypkových zemin je metastabilní.

Tato práce se zaměřuje na problematiku chování jílovitých výsypek kvůli jejich metastabilnímu struktuře a částečně nasycenému stavu. Dlouhodobý monitoring kapilárního sání v hloubkách do 2 metrů na dvou rekultivovaných výsypkách odhalil rozdílné trendy. Sezónně závislé oscilace kapilárního sání byly namodelovány na výsypce dolu Bílina, zatímco žádná sání ukazující na plně nasycený stav byla naměřena na výsypce bývalého dolu 5.květen. Určily se hydraulické charakteristiky pro neporušené a hrudkovité vzorky, které byly využity v inverzní numerické analýze vertikálních profilů kapilárního sání. Vysoké hodnoty kapilárního sání a jeho častá oscilace byly namodelovány v mělkých hloubkách.

Zkoumalo se mechanické chování výsypkových zemin ze dvou výsypek. Pevnost v tlaku a kapilární sání se stanovily na jednotlivých hroudách. Vysoká kapilární sání činkují v mikropórech hrud i když jsou plně nasycené. Pozorovala se degradace hrud vystavených cyklům zvlhčování a sušení. Jedno zvlhčení hroudy při její přirozené vlhkosti je nedostatečné, aby rozrušilo její strukturu, zatímco aplikace cyklů vede k rozrušení hroudy. Oscilace sání přispívají k degradaci hrudkovité struktury a uzavírání makropórů v poli. Klasické laboratorní zkoušky na hrudkovitém materiálu (smykové a oedometrické zkoušky) byly doplněny o velkorozměrové zkoušky. Mechanické chování výsypkového materiálu je ovlivněno přítomností makropórů a

také reaguje na vlivy velikosti hrud a času, což dále znesnaduje popis jeho chování. Původně hrudkovitá struktura se mění v čase činky zvětrávání a kvůli geostatickému napětí. částečně nasycený stav hrudek také přispívá ke zvláštnímu chování výsypkové zeminy.

Pomocí rešerše literatury a na vlastních experimentech se definovaly meze pro prosedavost u dvou typicky prosedavých zemin. Index prosedavosti závisí na stupni nasycení, objemové hmotnosti a aplikovaném napětí. Metoda dvojích křivek byla použita pro zhodnocení potenciálu prosedavosti u neporušených a hrudkovitých vzorků odebraných ze dvou výsypek. Prosedavost snižuje otevřenou makropórovitost, což způsobuje vznik ulehlejší struktury. Byly naměřeny dva rozdílné trendy u prosedavosti neporušených vzorků ze starší výsypky a hrudkovitých vzorků.

Chapter 1

Introduction

Brown coal or lignite mining produces large amounts of spoil that have to be deposited into fills. Open cast mining is the most effective exploitation method for brown coal or lignite, because 90% or more of the coal can be recovered. However this method has a very destructive environmental impact. Not only the mine pit but also the artificially created earth structures change the landscape. The efficiency of the open-cast method of brown coal production follows from the depth, thickness and quality of the coal seam. The so-called “overburden ratio” relates the amount of exploited overburden to the coal. For cost-effective brown coal production the ratio typically does not exceed 6:1. In North-western Bohemia, the overburden ratio is in general about 3.2:1, but it can vary significantly. For example, the Nástup Tušimice mine overburden ratio is 2:1 while at the Bílina mine the ratio approaches 5:1 [4]. However, the surfaces of fills are mainly reclaimed; further development is required mainly on aged fills. A term “fill” or “clay fill” is used for the landfill of spoil from the open cast mines within my thesis.

Deposition of fine-grained overburden materials during open-cast coal mining results in clay fills with an intricate mechanical behaviour. This is demonstrated in the region of North-western Bohemia, where the brown coal seams are overlain by natural sedimentary layers varying in thickness between 80 m and 200 m. The clay fills from the displaced and disintegrated clay can reach heights greater than 100 m. An area of about 100 km² has been affected by the clay fills in North-western Bohemia.

The deposition process takes place without any artificial compaction. Consequently, the clay fills that are formed have a loose structure, particularly in their upper part in the range of small overburden pressures. Clayey types of overburden often form lumps, resulting in dumped soil with double porosity. With increasing depth, the compression induced by the own weight of the soils squeezes the individual fragments, destroys them and creates a new compacted clayey material without

macrovoids. Even at shallow depths close to the surface, the structure of the clay fill varies in time. The destruction of the clay fragments is caused by weather-influenced suction cycles.

It is important to investigate the clay fill behaviour, since societal demands require clay fill areas to be opened up to the public, especially for transportation infrastructure. As an example, the motorway between Dresden and Prague has been recently constructed over a clay fill of this kind. The behaviour of engineering structures founded on clay fills must be predicted within a certain tolerance. This is necessary not only for bridges, which are sensitive to settlement, but also for motorway embankments and cuts, where slope instabilities can occur. When dealing with clay fill materials, the standard geotechnical approach for determining conventional soil parameters comes to its limits or may even fail.

1.1 Motivation and aims

The clay fills of open cast mines result from filling of the excavated clayey overburden. The material is initially highly inhomogeneous and contains large macrovoids. Furthermore the filling is typically dry and therefore the filled material is partly saturated, which makes its mechanical behaviour even more complex and problematic.

There are several aspects to consider in studying the clay fill material. For example the role of matric suction is to be considered in any investigation of the strength of the lumps at their natural water content. Particular attention should be paid to the influence of water penetrating the clay fill material. The influence of the wetting-drying cycles on the clay fill structure should be studied and the disintegration of the lumps due to these cycles should be described and evaluated. Since the typical laboratory devices cannot accommodate the full scale lumps forming the clay fills a further task is to consider and evaluate the consequences of scaling-down the size of the investigated lumps on the shear strength and compressibility of the studied soil.

The main aim of this thesis is to extend the knowledge about the mechanical behaviour of the clay fill soil at its natural water content and to assess the influence of change in moisture on its structure and the mechanical properties. Since the partly saturated clay fills experience gradual wetting, with likely wetting-drying cycles, it is important to study the changes in the matric suction in the clay fills over a longer time.

A further aim is to investigate the collapsibility of the clay fill soil. With this aim the thresholds for the typical collapse susceptible soils will be identified, which

could serve as a tool for the evaluation of the collapse potential of the undisturbed specimens from the aged clay fills and of the lumpy specimens prepared in the laboratory as a model material.

Two reclaimed clay fills of different age are chosen for the investigation and for a long term monitoring of matric suction. The numerical model used in hydrogeology for the water flow in variably saturated porous media, which include the approach of the dual permeability, is used for simulating the vertical profiles of matric suction in the clay fills, dependent on the seasonal water balance.

1.2 Thesis structure

The character and the problematic mechanical behaviour of the open cast mine fills are summarized in the Chapter 2. The factors, which influence the behaviour of fills are described. The aspects of the geomechanical behaviour connected with the partly saturated state of fills and their open structure are emphasized. Possible methods of the investigations of the clay fills are listed.

The results of the investigations are presented in Chapters 3 – 7. In Chapter 3 the geology and geotechnical and mineralogical characterisation of the investigated soils are described. The monitoring of matric suction of the two reclaimed clay fills for the period of four and a half years, is presented in Chapter 4. For the clay fill from the Bílina mine, where the suctions were high, the suction profiles were numerically simulated. Hydrometeorological data (supplied by the Czech Hydro-Meteorological Institute - CHMI) and the hydraulic characteristics of the clay fill soil are used in the modelling of the vertical profiles of matric suction. The concept of double porosity is simulated using a phenomenological approach of dual permeability with the preferential flow domain. The results of the monitoring were also dealt with in [55, 54, 53, 56].

Chapter 4 presents the properties of clay lumps. Uniaxial compressive strength, matric suction and overconsolidation are studied. The structural decay of intact lumps subjected to wetting-drying cycles is observed. Some of the results can be found in [58].

An extensive laboratory investigation carried out in standard and large-scale devices is presented in Chapter 5. The soils of the clay fills from two different mines in North-western Bohemia are investigated. The mechanisms leading to the structural changes in the lumpy soil were investigated for various lump sizes. The differences in the mechanical behaviour following from the properties of the lumps and from the character of the original sedimentary soil are distinguished. The influence of the

change in the state of the clay fill soil on its compressibility and on the shear strength is presented. The results of these investigations can be also found in [59, 53, 96, 58]. The collapse potential of typical collapse susceptible soils is analysed using a literature review and own experiments in the first part of the Chapter 7. The collapse potential at different stress levels is quantified for the different degrees of saturation and dry unit weights. These results can be found in [57]. In the second part of Chapter 7 the collapsible response of the clay fill material is studied on lumpy and undisturbed specimens from the aged clay fill. The collapse potential of the clay fill soil is compared with the collapse potentials of the typical collapse susceptible soils. These results were published in [54, 56].

Chapter 2

Character and behaviour of mine fills

More man-made soils are formed as a consequence of brown coal or lignite production than through any other activity. These fills can basically be divided into outer and inner fills, according to their position in relation to the mine pit. The outer fills are dumped outside the mine pit in the early stages after the mine is opened, due to lack of space inside the pit. Further space is therefore required.

For example, the Radovesice fill, one of the biggest outer fills in central Europe with an area of about 1400 ha, filled a valley in the surroundings of Radovesice, and required the destruction of 5 villages. With a volume of about 500 million m³ it reached a height of 50-70 m [5]. It began to be filled in 1964. The excavated soil was transported on conveyor belts to a distance of about 6 km. A new bridge was built over the existing roadway to transport the material. The filling process has now been brought to an end, and surface reclamation is proceeding.

Another example of a famous outer fill is Sophienhöhe, the outer fill of the Hambach mine in the Rhineland brown coal mining district (see Fig. 2.9–top). With an overall height of 400 metres, Sophienhöhe is the highest fill in Europe. It forms an artificially filled mountain which stands on an average 200 m above the flat surroundings of the Tertiary coal basin. Around 1.1 billion m³ of sand, gravel and clay were filled on this pile. Nowadays, most of Sophienhöhe has been reclaimed, and bicycle and walking paths with look-out points have been opened for tourists (see [1]).

In the later stages after landfilling, the inner fills are deposited inside the mine pit. No further space is required. As a consequence of open cast mining technology, a mine pit remains behind after mining has been brought to an end. This is normally flooded and forms residual lakes. The inner fills, in particular, should therefore be

engineered in a way that ensures stability even when the lakes are flooded to their maximum height.

2.1 Factors affecting the mechanical behaviour of fills

The heterogeneous character of the mine fills follows from the properties of the excavated overburden, and is further affected by various influences. The only common feature of all the fills is their high overall porosity in the early stages after landfilling. The mechanical behaviour of the fills cannot be characterized generally, but depends on various factors, which will be discussed below.

2.1.1 Exploitation, transport and filling

The overburden is mainly excavated using bucket wheel excavators. This is nowadays the most efficient type of open-cast soil exploitation, and enables continuous operation. When the soil cannot be extracted mechanically, a blasting method is used. During coal production, the excavated overburden is disaggregated to blocks, lumps or even single grains, depending on the character of the excavated soil. The maximum size of the excavated lumps is limited by the size of the extracting shovel. Subsequent transport of the disaggregated overburden, most typically on conveyor belts, leads to further material disaggregation and mixing. In the Hambach mine, for example, the excavated soil is disturbed during transportation, and reaches a remoulded state [62]. In addition, the materials of better quality and of poorer quality are separated, and transported to target deposition locations. The overall length of the conveyor belts is 120 km, and their average speed reaches 27 km/hour. The soil is transported at the distances up to 5 km [1, 114]. Finally, the soil is dumped using dumping machines. The soil falls from various heights at the kinetic energy proportional to the height from which it falls and the velocity of the conveyor belt. This causes its slight compaction, but the freshly-filled soils are typically loose. Some segregation of the fill material also takes place due to this process. Bigger lumps or grains roll down and are deposited at the toe of the heap. The lump size distributions taken from different positions on the slope of the model lumpy material with scaled lump size was studied by Najser et al. [102] and Najser [100]. This filled overburden forms spoil heaps under the angle of repose (reaching as high as 40°, see Fig. 2.1– top), which is governed by the granulometry and the form of the lumps or grains. This distinguishes its stability in the very early stages after landfilling.

Two types of filling are commonly distinguished: high filling and deep filling [93]. High filling means that a dumping machine is placed below the heap and fills the soil in the direction above itself. In deep filling, by contrast, the dumping machine is placed above the heap and fills in the downward direction. The maximum fill height is influenced by the height of the arm of the dumping machine, which is the limiting factor as far as the high filling method is concerned. However, the final height is governed by the strength of the filled grains or lumps. The falling height reaches 20 m [126]. The type of filling and the falling height also influence the variation in the density of the filled material.

Matschak [93] studied the variations in density of the sands in the fills in the Lausitzer mining district. He observed that deep filling causes better densification of the material in the centre of the heap. This is due to a higher falling height. For both types of filling, the centre of the heap is denser than the slopes and toes where the material rolls down. This results in loosening. In the centres of the heaps, a bulk density of $\rho_n = 1.7\text{--}1.9 \text{ g/cm}^3$ was reached, while the slopes were formed at a bulk density of $\rho_n = 1.4\text{--}1.6 \text{ g/cm}^3$. A typical bulk density of $\rho_n = 1.5\text{--}1.6 \text{ g/cm}^3$ ($\rho_d = 1.2\text{--}1.23 \text{ g/cm}^3$ assuming $w_n = 30\%$) for freshly filled lumpy clay was reported by Vaníček and Vaníček [126] for fills in North-western Bohemia. However no variation in density depending on the position in the heap was presented. Najser [100] filled the air-dried and scaled-down lumps into heaps in laboratory conditions to model the lump segregation. The dry densities of $\rho_d = 1.06\text{--}1.07 \text{ g/cm}^3$ and $\rho_d = 1.15\text{--}1.13 \text{ g/cm}^3$ were determined for the toes and the centres of the heaps, respectively. This is well in agreement with the trend of the in-situ observations of Matschak [93] on sandy heaps.

Fills can also be transported hydraulically as slurries and filled in so-called polders in a semi-fluid state (see, e.g. [18, 27, 51]). This type of filling will not be discussed in detail in this thesis. However, some soils excavated mechanically have to be filled into polders. These soils, mainly with a high water content and a soft-to-liquid consistency, are not able to form stable heaps ([81], Fig. 2.1–bottom).

2.1. FACTORS AFFECTING THE MECHANICAL BEHAVIOUR OF FILLS



Figure 2.1: Filling the dams, mine Hambach (top, by S. Gesellmann); Process of filling the soft (not for construction usable) soil inside the polder, mine Hambach (bottom, [1]).

2.1.2 Type of filled soil

The mechanical properties of the filled material also depend on the type of filled soil. As all lignite was originally deposited in tertiary sedimentary basins, its overburden is formed from sedimentary layers of different granulometry. The grain size distribution is directly related to the rate of the flowing water in rivers or stagnating water in lakes in the Tertiary era. The lignite overburden is therefore formed mainly from clayey, silty and sandy (occasionally gravelly) soils and their admixtures. Possible cementation of sedimentary layers cannot be excluded, but it is typically not very strong. The degree of disaggregation during exploitation is also related to the soil type. The coarse-grained overburden can be disaggregated to single grains, while silty and clayey soils mainly form lumps of various sizes and with variable mechanical properties.

2.1.3 Overconsolidation

The overconsolidation is an important factor influencing the void ratio, shear strength and compressibility of the excavated lumps, in the case of fine-grained soils. In the case of sandy or gravelly overburden types, however, the original soil is mainly disintegrated to single grains and deposited in a fully distorted state. Thus the density of the original sedimentary soil (governed by overconsolidation) has no impact on the mechanical behaviour of coarse-grained fills.

The maximum stress that the soil can experience is not influenced only by the original depth. Four factors influencing overconsolidation of original sedimentary soils are:

- **original depth** The void ratio of the sedimentary layers is related to the burial depth. The deeper in the mine pit the excavated overburden is located, the stiffer will be the lumps that are formed.
- **lowering of the ground water table** Due to open cast mining, the ground-water table has to be lowered substantially over large areas to make soil excavation possible in the staged mine cut. This leads to subsequent loading of the soil due to the change from the effective (under water) weight to the total unit weight, which causes soil compression and surface settlements. However, pore pressure dissipation is a long-term process, especially for fine-grained soils and for large drainage paths. During repeated rises of the groundwater table, further settlement can occur due to ongoing dissipation of the pore water pressures [139]. Some undissipated pore water pressures can be present in

the soil at the time of excavation, and this can have a negative influence on the strength of the lumps.

- **thickness of geological denudation** Overconsolidation was caused by the denudation of overlying layers in the geological past. In the North Bohemian brown-coal basin the thickness of the overlying complex on Miocene layers between 70 m and 300 m was reconstructed by Hurník [68] (see Sect. 6). Overconsolidation can also be affected by the glacier load during a glaciation episode.
- **creep and diagenetic processes** These time dependent processes decrease the void ratio of the sedimentary soil without the change of effective stress. Creep settlements are important for the clays of high plasticity. Diagenetic bonding strengthen the soil by adding a true cohesion.

2.1.4 Consistency of the filled soil

Due to the lowering of the groundwater table, the excavated soil is in a partly or fully saturated state, depending on the air entry value of the soil. In both cases, negative pore water pressures can influence the strength of the excavated soil, if the soil forms lumps when excavated. In this case, the consistency of the filled lumps is a governing mechanical property which influences the mechanical behaviour of the filled lumps.

The soil is typically dumped in a dry way, which causes partial saturation of the filled soil, even if the original overburden was fully saturated. This is due to the fact that the soil is disintegrated, and the voids between the single fragments are dry. In some cases, the disaggregated overburden is filled into polders, or directly into the sea for land reclamation purposes [51, 86]. Unlike dry way filling, this causes the voids between the filled lumps or grains to be water-saturated initially. In this case, the negative pore water pressures in the lumps of dredged soil start to dissipate when the lumps are placed in water. The lumps soften and undergo plastic straining, so the voids between the lumps are progressively filled by these strained lumps. The pore water pressure development in dredged clay lumps and the consolidation characteristics of lumpy soils have been studied for example in [86, 110, 111, 135] for land reclamation in Singapore. Some of these effects are closely analyzed in this thesis on the lumpy fill material of the Bílina mine (see Sect. 6, [58]).

In some cases, fine-grained soils do not form lumps. They are mainly of soft consistency. During excavation and subsequent transport, their structure is disturbed

and remoulded to the almost reconstituted state. Such soils are typically clays of low plasticity with a high silt content, or silts, which change their consistency easily as a consequence of a small change in water content [62, 63]. These soils are not able to form heaps and are filled in to form polders ([81], see Section 2.1.1 and Fig. 2.1). Since the surface of the excavated soil is exposed in the mine cut or during transportation, weather conditions further affect the state of the filled soil. The consistency of the soil can further change after it has been deposited in a spoil heap (see Sect. 2.2.5).

2.2 Fill structure

As a result of all the phenomena discussed above, the fills from structured soils are heterogeneous and of variable mechanical properties. Their common feature is the high overall porosity, consisting of macropores and micropores. Fills cannot be considered stable, and due to further processes their structure changes in space and time. Two types of filled soils can be distinguished: coarse-grained and fine-grained ones. Different mechanisms of structural changes can be expected for the behaviour of each type.

2.2.1 Double porosity

The term double porosity is used for soils that contain two distinct pore systems. Such soils exhibit a bimodal pore size distribution, and are often encountered in the form of unsaturated aggregated fine-grained soils of natural or man-made origin (see for example [78, 31, 32]). A double porosity structure is often formed by soils compacted dry of optimum. The total porosity consists of the macrovoids between the aggregates and the microvoids inside them. The structure of such soils is open, favourable to collapse under wetting ([32, 56] see also Chapter 7) and favourable to structural changes due to plastic straining [78, 102, 58].

Also the lumpy (fragmentary after Feda [43]) material of clay fills can also be characterized by two porosities — intragranular and intergranular, see [42, 43] and [19], the latter porosity being equivalent to macrovoids. Fig. 2.3 shows different definitions of void ratios for a double porosity soil. The overall void ratio e_{tot} takes into account both, the macrovoids between clayey lumps (intergranular voids having volume V_e) and the voids within the lumps (intragranular voids having volume V_i). The intragranular void ratio e_i roughly corresponds to the original void ratio of the overburden clay under the assumption of no internal distortion of the fragments, and a

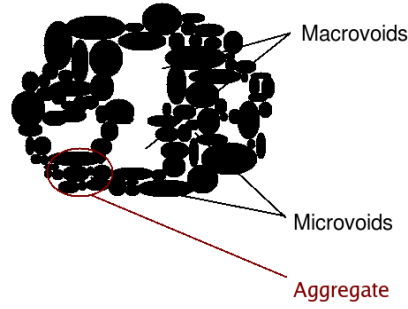


Figure 2.2: Double porosity - definition.

negligible unloading effect. The intergranular (also called extragranular) void ratio e_e is related to the voids between the clay lumps when considering the fragments as solid grains.

One can write

$$e_{tot} = \frac{V_e + V_i}{V_s} = \frac{e_e(V_i + V_s) + e_i V_s}{V_s} = e_e(1 + e_i) + e_i \quad (2.1)$$

subscripts $_{tot,e,i}$ denote total, intergranular (macrovoid) and intragranular (microvoid), respectively.

An analogous relationship between porosities is given by:

$$n_{tot} = n_i(1 - n_e) + n_e \quad (2.2)$$

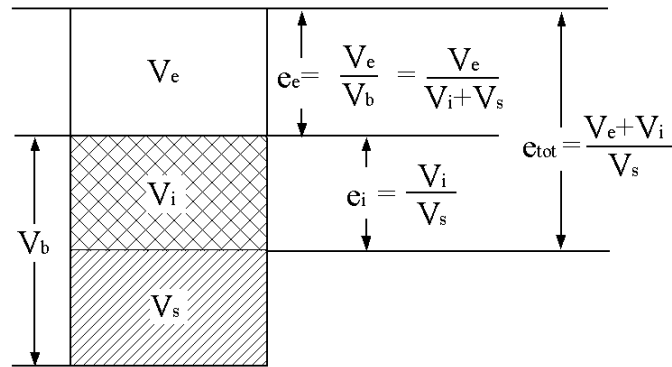


Figure 2.3: Void ratio relationships for a double porosity soil.

The term lumpy soil or lumpy clay is adopted further in the text for the clay fill soil which was sampled in the form of lumps or for the artificially prepared lumps. Fedá [43] used for the same soil a term fragmentary or granular clay.

2.2.2 Coarse grained

The capillary forces on the air-water menisci between grains of a partially saturated sandy type of overburden are typically low. This enables it to disintegrate to single grains and to be deposited in a granular state. However, these forces are often still able to glue a few grains in aggregates. During filling of this aggregated soil, an open structure containing macropores between the aggregates is formed. The density of this filled sand is much lower than the density of a loose dry filled sand, where no capillary forces are present. However this double porosity structure is highly unstable, and is prone to collapse settlement and liquefaction ([61, 33, 46, 133] see also Sect. 2.4.2 and 2.4.3).

For example, Herle et al. [64] made an optical study of the number and distribution of the macrovoids in a loosely-filled partly-saturated sand using computer image analysis. This type of soil is very unstable, and when it is flooded or loaded to small stresses, most of the macrovoids are destroyed. The distribution of the macrovoids and the collapse behaviour was found to be dependent on the grain size distribution and the moisture during filling.

2.2.3 Fine grained

While coarse-grained overburden is disintegrated to single grains during mining, fine-grained overburden typically forms lumps. These lumps are of various sizes, depending on the mining technology, the size of the excavating shovel, the type of transportation (see Sect. 2.1.1) and the soil type and state. Lump size distribution was studied for example by Dykast [36] on the fill material from the Merkur and “VCSA” mine, and by Najser [102, 100] on a model material with scaled-down lump size used for centrifuge modelling. The lumps of freshly dumped fill reach 0.5 m in diameter [36], while for example lumps used for land reclamation in Singapore can reach about 1.5 m in diameter [110].

Matric suction acts at the scale of microvoids inside the lumps as a consequence of unloading due to excavation and possible drying. Matric suction influences the mechanical behaviour, increasing the shear strength and reducing the compressibility in the freshly-filled spoil. However, there is typically very low matric suction acting in the macrovoids under low stresses (see [58]). This is caused due to the large size of macrovoids and the dry-way filling. The contact bonding develops under higher loads due to the matric suction, which act on the contacts between the lumps. The matric suction develops as a consequence of closer packing of lumps and moisture transfer through their contacts. Feda [42] observed adhesion of lumps at vertical

loads of $\sigma_v \geq 50$ kPa. However, the amount of contact bonding depends on the lump water content. When flooded, these bonds vanish and the open structure collapses (see [42, 56], Sect. 7).

As already mentioned, in some cases the excavated soil is soft, does not form lumps and is placed in a remoulded state into polders ([62], see sect. 2.2.4).

2.2.4 Spatial distribution

As a consequence of various grain size distributions and mechanical properties of the sedimentary overburden, and various mechanisms leading to structural changes in the filled soils, the mechanical behaviour of the fills varies in space.

In some mining districts, a distinct type of overburden prevails. For example, a sandy type of overburden prevails in the Lausitzen and Silesian mining districts [133, 46, 134], while clayey soil types are mainly exploited in the North-western Bohemian Tertiary basin [90, 122, 36, 102, 58]. Even in open-cast mines with a more or less uniform soil type, the mechanical behaviour of the filled soil is often spatially distributed. In general, lumps of fine-grained soils coming from shallower depths are softer, and are therefore more compressible, than soils from deeper deposits. This is attributed to the higher overconsolidation of the lumps coming from greater depths (see Sect. 2.1.3).

However, some mines have a heterogeneous overburden. The Hambach mine in the Rhineland brown coal-mining district is a typical example. With a depth of around 370 m [1], it is the deepest open-cast mine in Europe and also one of the largest. The stability of the fill has to be assured [105], despite the poor quality of parts of the excavated overburden placed in the fills. According to Pierschke [105], unstable soils are soils which are not able to form heaps, or which fail in heaps ≤ 7 m in height. These make up a remarkable percentage of the total production of excavated overburden at the Hambach mine. During excavation the overburden is therefore continuously classified and sorted according to its grain size and consistency into lower and higher quality soil classes. The soils are transported on many conveyor belts to target places for deposition, and so-called regular sections (“Regelprofile”, Fig. 2.4 [62]) are deposited. A dam (a) is created first from the coarse-grained type of soil M1 (mainly sands). Then the M2n mixed soil of poor quality (mainly of soft to liquid consistency) (b) is deposited inside the polder (see also Fig. 2.1 – bottom). On the polder, a mixed soil of better quality M2t (mainly of stiff consistency) (c) is dumped, and subsequently the one regular section is closed by the more permeable

M1 (d) soil type. In this way, a sequence of regular sections is deposited resulting in a heterogeneous fill.

The most typical spatial variation of fills involves the diminishing of open porosity with depth. Increasing the fill height during filling causes compression of the underlying soil layers and the closure of open macrovoids. This can be produced under various mechanisms.

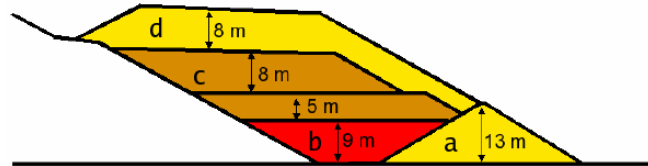


Figure 2.4: Hambach mine, Germany: an example of a regular section [62].

Based on a laboratory investigation of granulated clay with a scaled-down lump size distribution, Feda [42] studied the mechanisms leading to structural changes in this material: Four basic response mechanisms of fragmentary clay to loading were distinguished.

- crushing of fragments
- ductile squashing of fragments
- fragment rearrangement
- contact bonding

Based on these elementary mechanisms, a distinction was made between four more complex structural processes which change the structure from granular to cohesive [42]:

- diffusion. Besides the consolidation which is a typical example of the diffusion process, the diffusion of deformation at a constant effective stress was observed by the fragmentary specimens. For a so called diffusion collapse, the coefficient of diffusion was defined.
- percolation. If a system consists of the two components, their proportion governs the system response. This response may range from the typical of one component to that of the other one. The double-porosity geomaterials, which trigger different structural mechanisms are prone to develop the percolation

phenomena. The change from the granular to more cohesive response is typical for the fragmentary clay.

- normalized behaviour. The mechanical behaviour which can be described by a single constitutive function using a dimensionless form. A linear Mohr-Coulomb envelope is a typical example of the normalized (stress independent) parameter, contrary to a nonlinear envelope, which indicate the stress dependent phenomena.
- collapse. Two types of collapse can be defined:
 - breakdown – a total change of structure, for example a hydrocollapse. Collapse compression accelerates the process of macrovoid closure at the moment when the fill, filled in a dry way, is saturated for the first time. The phenomenon of hydrocollapse is described in Sect. 2.4.3 for various soil types. Collapse of clay fill soil is dealt with in Sect. 7 of this thesis.
 - bifurcation – a jump-like change in the constitutive parameter resulting in kinks. A garland-like time response to loading was observed for crushed fragmentary clay, Fig. 2.6. The change in the coefficient of secondary compression in time in Fig. 2.6 was attributed to the collapse of the bifurcation type. Bilinear shear strength envelopes for the low pressure range, or even trilinear shear strength envelopes, were found for the fragmentary clay, see Fig. 2.5. A collapse crushing of the bifurcation type and squashing the fragments were found to be responsible for this unusual behaviour.

The mechanisms leading to structural changes in the clayey fill soil from the Bílina mine are discussed in detail in Chapter 6.

2.2.5 Time variation

Not only the increasing fill height changes the character and the mechanical behaviour of filled soils. The fill structure also varies due to ageing after it has been deposited.

The self-weight consolidation of fills is a long-term process, and it can take years for the pore water pressures to dissipate. The drainage path is crucial for the consolidation rate. Yang et al. [135] found out that dredged clay placed in the sea in the form of lumps consolidates significantly faster than the hydraulically placed soil. This is due to the open macrovoids between the lumps, which shorten the drainage path. After the closure of macrovoids the consolidation rate significantly decreases.

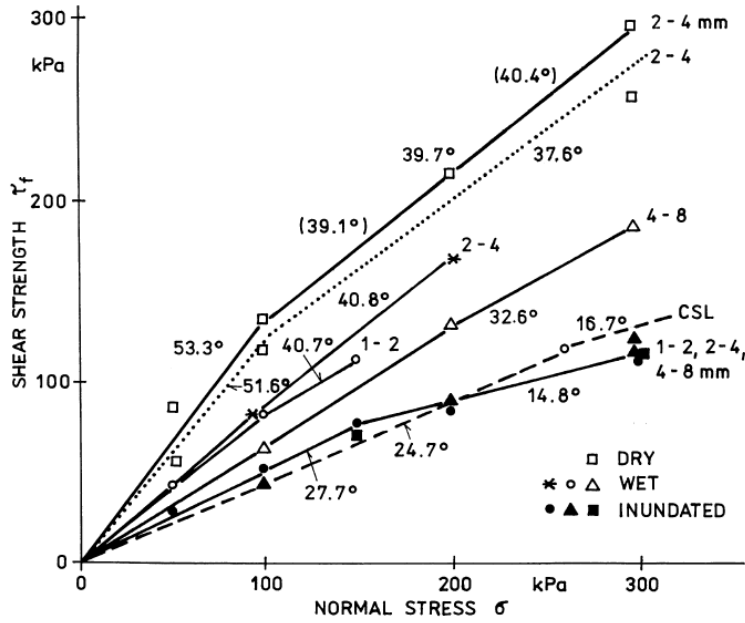


Figure 2.5: Shear strength envelopes for fragmentary clay sheared at lower pressure range [42].

The consolidation process results in a stiffer soil with higher shear strength. A laboratory study of the freshly filled, soft M2 soil type in [63] in the Hambach mine showed no dependence of the undrained shear strength on depth, Fig. 2.7, which is usually typical for natural in situ conditions. It proves that the tested soil has not been consolidated yet after the deposition. However, the logarithm of the undrained shear strength was found to be dependent on the logarithmic consistency index, Fig. 2.8. Logarithmic consistency index is defined as [63]:

$$I_{c_{log}} = \frac{\log w_L - \log w}{\log w_L - \log w_P} \quad (2.3)$$

Lines in Fig. 2.8 denote the prediction of the undrained shear strength using hypoplastic model for clays [94]. The six reconstituted specimens were used to represent the range of M2 soil types in the Hambach mine. A high degree of saturation during filling was assumed and an undrained shear strength as a function of water content was calculated. The prediction of the hypoplastic model was validated by numerous experimental results from undrained triaxial tests on undisturbed specimens sampled several weeks after the soil deposition (colour points in Fig. 2.8). This indicates that undrained shear strength still depends on the consistency of the soil in the time of filling, however the undrained shear strength is higher than the one in the time of the soil deposition.

2.2. FILL STRUCTURE

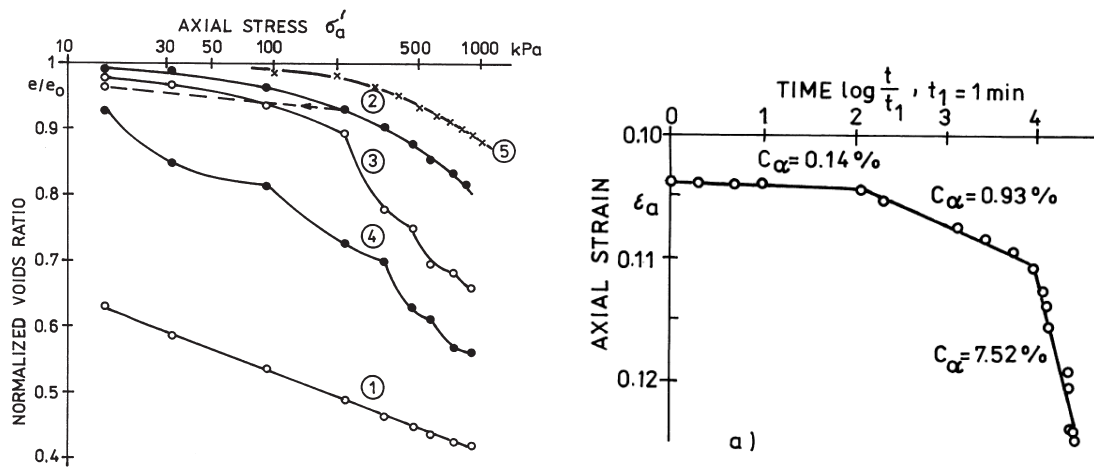


Figure 2.6: Oedometer compression lines of fragmentary clay of different water contents (right), Creep response for naturally wet specimen of fragmentary clay (left) [42].

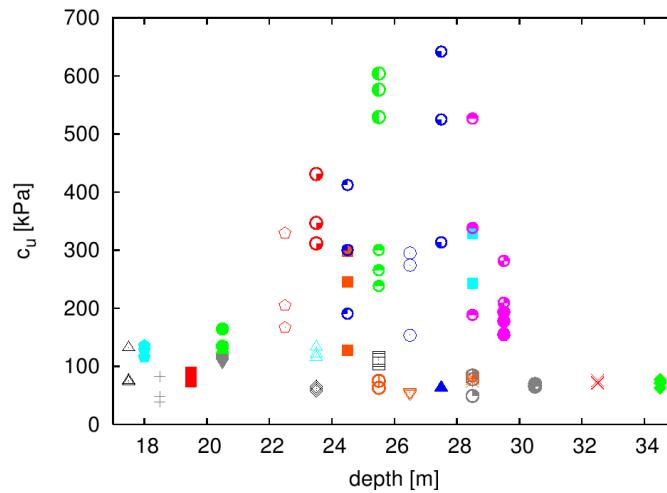


Figure 2.7: Undrained shear strengths of undisturbed samples from the material class M2n in dependence on the sampling depth [62].

Creep also contributes to the compression of fills. An unusual type of creep with the time dependent collapses taking the form of the diffusion processes can be observed by the lumpy structures, due to the ductile deformation of the contacts of lumps. Also creep, following a multilinear trend in a semi-logarithmic plot with the considerable change of the coefficient of secondary compression was found for a fragmentary clay [42], see Fig. 2.6 – right in previous Section.

Degradation due to weathering influences the structure of initially lumpy soils on the exposed surface and at a shallower depth, where suction oscillations take place

2.2. FILL STRUCTURE

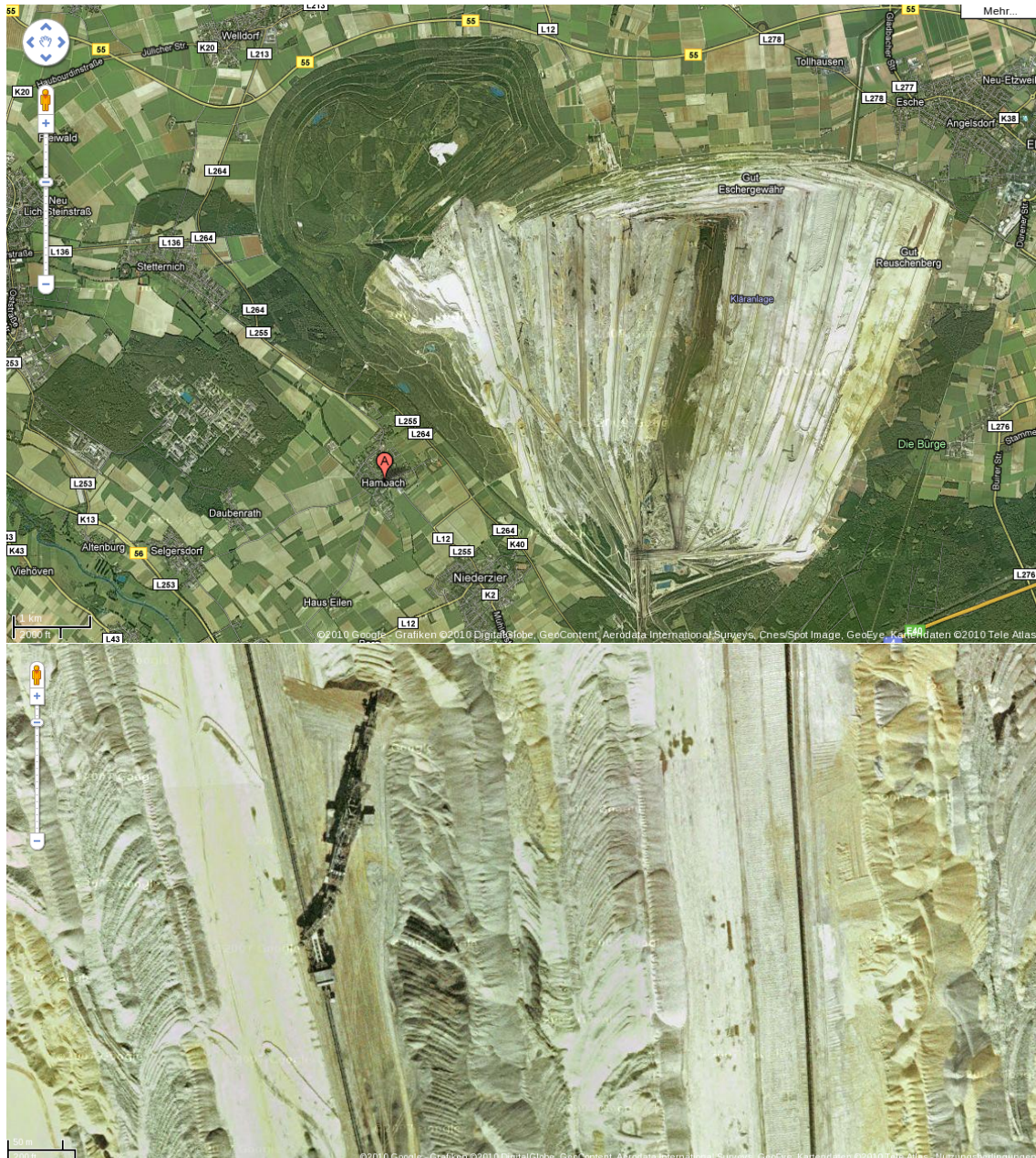


Figure 2.9: Satellite picture of the Hambach mine (top) one of the biggest open cast mines in Europe, Process of filling (bottom) [2].

2.3 Modelling of clay fills as structured soils

The similarity between the behaviour of structured soils, soils with double porosity and lumpy clay fills can be used for the mathematical description of the complex behaviour of clay fills.

2.3.1 Sensitivity concept

The mechanical behaviour of structured soils is a phenomenon that has been studied intensively. Burland [23] introduced the concept of relating the properties of structured clay to clay with intrinsic properties. Cottechia and Chandler [30] introduced a sensitivity framework for a structure applicable to many different natural clays. Within this framework, the structure of natural clays is due to a combination of fabric and bonding. Sensitivity s relates the size of a gross yield surface of a structured soil and a reference or intrinsic [23] yield surface of reconstituted clay. A number of numerical models include the effect of soil destructuring, e.g. [89, 78, 102].

The sensitivity framework developed by Cottechia and Chandler [30] was successfully implemented in the hypoplastic model for clays [94] with a metastable structure by Mašín [95] in the similar way as in the other models for structured soils (e.g. [14]). The hypoplastic model for structured clays by Mašín [95] is able to reproduce the behaviour of structured clays well, both qualitatively and quantitatively. An advantage is that it needs just three additional parameters to the five required for the basic hypoplastic model for clays, for describing the effect of the structure. The additional state variable sensitivity s , which defines the ratio between the sizes of the yield surface of the structured and reference (reconstituted) clays is defined in the rate form as:

$$\dot{s} = \frac{k_h}{\lambda^*} (s - s_f) \dot{\epsilon}^d \quad (2.4)$$

where k_h is a parameter controlling the rate of the structure degradation, s_f is the final sensitivity and $\dot{\epsilon}^d$ is strain damage rate. To define the later a parameter A , controlling the relative importance of volumetric and shear strain is needed. For details refer [95]. The three parameters (k_h , s_f and A) and the specification of initial sensitivity s_0 are required for the incorporation of the soil structure. These parameters can be calibrated using the standard oedometer and shear tests performed on a structured soil.

2.3.2 Modeling of double porosity soils

Koliji et al. [78, 76] introduced a model describing the evolution of the pore size distribution of an aggregated soil subjected to an increase in suction, and also a constitutive framework [77] for aggregated soils. A degree of soil structure quantifies the soil structure in terms of volumetric ratio of macrovoids. Based on the tomography analysis results, an evolution of the degree of soil structure was related to the plastic straining of the material.

The double porosity structure also has to be considered when studying the behaviour and when modelling the compacted bentonite buffer around the container of highly radioactive waste in repositories [115, 113]. Extensive and advanced laboratory investigations, large in situ testing in underground laboratories, and the development of complicated thermo-hydro-mechanical coupled models are used to capture all phenomena connected with this problematic issue (see, e.g. [48, 115, 128] and many others).

As in the case of structured clays, the yield surface of soils with double porosity exceeds the yield surface of the intrinsic soil [96, 77].

2.3.3 Application to clay fills

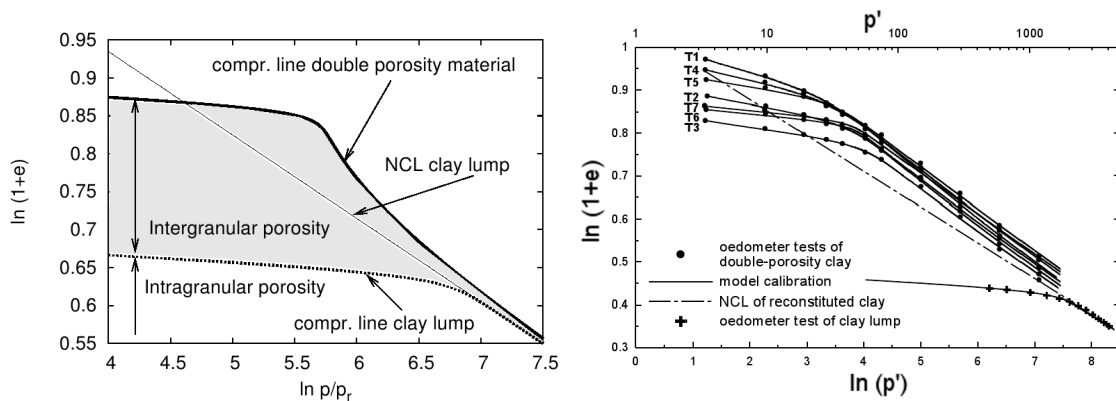


Figure 2.10: Concept of total void ratio for double porosity soils [101] modified after [96]; data from oedometer test on specimens consolidated in mini-centrifuge [102, 100] (right).

A parallel between numerical modelling of structured soils and numerical modelling of double porosity soils was successfully demonstrated on simulations of the consolidation settlements of two trial embankments for highway construction on a lumpy clay fill soil [101, 100]. Due to the double porosity nature, the compressibility line

of the lumpy clay exceeds the values for intrinsic clay (see Fig. 2.10). The hypoplastic model for clays with metastable structure by Mašín [95] was used. The model parameters describing the double porosity structure were calibrated using oedometer tests on specimens prepared from scaled-down lumpy material (see Fig. 2.10), while parameters for the basic hypoplastic model were calibrated using isotropic compression and triaxial tests on reconstituted material. The model was evaluated by comparison with the self weigh consolidation of the centrifuge model, and good performance of the constitutive model was achieved when modelling the embankment in a geotechnical (large) centrifuge. However, both the centrifuge [102, 100] and the numerical models [101] overpredicted the in-situ measured settlement beneath the monitored embankments. A phenomenon weathering destructuration was therefore introduced (see Fig. 2.11), which reduces the total void ratio of the fill due to weathering effects. The percentage of weathering destructuration was defined as:

$$D_s = \frac{e_{s100} - e}{e_{s100} - e_{s0}} \quad (2.5)$$

where e_{s100} refers to the void ratio of fully structured model and e_{s0} express the void ratio of the model without double porosity structure at the corresponding depth. Based on an inverse analysis, the intergranular porosity in the top 10 m of the aged fill was found to be degraded, and the degree of degradation due to weathering decreased with increasing depth.

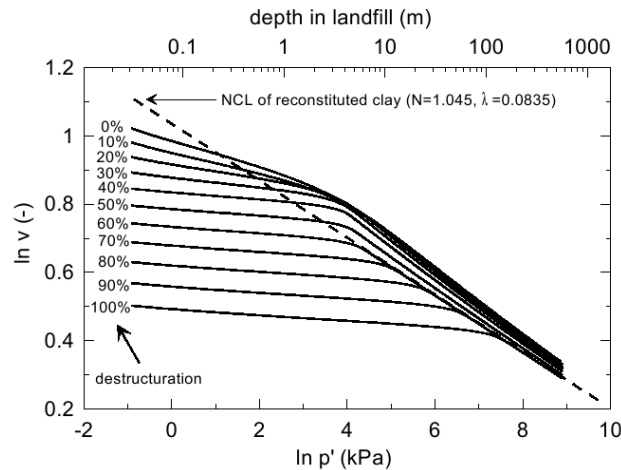


Figure 2.11: The modelled compression curves of the lumpy clay with different degrees of destructuration [101].

Another approach was applied by Doležalová and Kořán [35]. They used Discrete Element Modelling (DEM) to reproduce the mechanical behaviour of a model lumpy clay. This method simulates the lumps as distinct grains characterized by their

stiffness. However, the structural transitions from granular to clayey cannot be captured by this approach.

2.4 Geotechnical problems of fills

The different mechanical behaviour of various types of overburden can lead to different modes of failures of fills. In the early 1970s, Bishop [16] described the mechanisms of loss of stability of spoil heaps and tips, depending on the character of the fill material. A high void ratio of a freshly-filled material, which is in most cases placed without compaction (e.g. [16, 18, 17, 25]), is the common feature of the fills causing stability problems and excessive compressibility. Many fills exist in a so-called metastable state [58, 56], and a change in their state or an external factor can trigger instability. These fills are stabilized mainly locally, where further development is planned or where instabilities are anticipated.

2.4.1 Landslides

The excavated overburden is filled on heaps under the angle of repose, which is initially steep depending on the state and the granulometry of the filled material (see Fig. 2.1). However the high angles of repose are relevant for the short term stability of the single heaps only. Since the structure and therefore the mechanical properties of the filled material also vary in space, the final slopes should be much more moderate. For example, Blight [17] recommends that the final slopes should be flatter than a slope with an acceptable factor of safety, and should not exceed an inclination of 15° .

A typical inclination of the final slopes of fills in North-western Bohemia is 1:9 [126]. Dykast et al. [37] reported some huge landslides even in slopes with a general inclination of approximately 1:12 and 100 m in height. The landslides were attributed to the degradation process of the originally lumpy clay.

Liquefaction as a special case of loss of stability of fill slopes is discussed in the following Section.

2.4.2 Flowslides

Flowslides are typically due to a very rapid loss of stability of a saturated loosely-filled sandy type of overburden and occur mainly in poorly graded sandy fills [133,

116]. Slopes which are saturated and simultaneously have a high void ratio, are particularly prone to liquefaction [33].

This loss of stability is typically connected with rapid propagation of an increase in pore water pressure due to a possible single or repeated dynamic action (e.g. blasting). A pore water pressure increase causes a decrease in the effective pressure. When the effective stress tends to zero (the pore water pressure reaches the geostatic pressure), the soil loses its shear strength and behaves as a viscous fluid. The behaviour of liquefied mine tailings as a Bingham plastic fluid was analyzed by Jeypalan et al. [70, 71].

In some cases, loosely-filled sandy fills can even liquefy under static conditions [61]. Slopes which are metastable often liquefy under an inclination that is much lower than the critical state friction angle of sand, even without a downstream in the slope. After crossing the so-called instability line, which is defined by the maximum shear stresses during undrained shearing of loose sand, the effective stresses drop significantly. In the case of the extremely loose sands effective stresses can even drop to zero. Classic limit equilibrium analysis of slope stability using the effective critical state friction angle therefore fails. The undrained shear strength is therefore preferred for stability analysis of these fills, but this is very sensitive to soil density and stress level. Metastable fills having critical density, which is higher than the maximal density of the dry sand, should be densified to minimize the risk of liquefaction [33, 46].

Liquefaction of fills is connected not only with the flooding of mined-out pits in the early stages after landfilling, but can also occur with a certain time delay in the case of aged fills. Forster et al.[46] gave an example of a flowslide of a 25-year-old fill in the Lausitzer mining district. Wichter [134] confirmed that the age of the fill is not important for liquefaction.

Blight and Fourie [18] presented an overview of disastrous flow failures of mine and municipal wastes all over the world. They described more than 20 examples of failures. Two particularly catastrophic failures of coal solid wastes, connected with liquefaction of a spoil heap, occurred in 1966 in Aberfan, UK (144 fatalities) and in 1972 in Buffalo Creek, USA (118 fatalities).

A recent failure of a fill occurred in Nachterstedt in eastern Germany (Saxony-Anhalt) on 18th July 2009, and caused three fatalities. A flowslide (Fig. 2.12) 500 m in length disturbed some housing and infrastructure in Nachterstedt, parts of which had been built on a reclaimed old fill. No clear cause has yet been published, but a combination of several possible causes has been under discussion. The possible causes are [21]:

2.4. GEOTECHNICAL PROBLEMS OF FILLS

- the groundwater flow in the ancient galleries, which exact position was not known;
- the continuous rise of the groundwater table, connected with flooding of the mine pit. At the time of the event, the water table was still 20 m below the planned flooding height of Concordasee;
- a low-intensity earthquake was recorded in the Nachterstedt area at the time of the event, which may have triggered the flowslide mechanism.



Figure 2.12: Flowslide Nachterstedt, eastern Germany [3].

2.4.3 Collapse settlement

Only a collapsible response of soils to wetting or so called hydrocollapse is discussed in this section. A first wetting event triggers collapse settlements in loosely-filled soils containing macropores. Since most fills are dumped in a dry way, without any compaction, there is a high potential for collapse. The dry unit weight and the moisture content of the fill are the major factors controlling the collapse potential ([27] , see also Sect. 7). Charles and Watts [26] gave three criteria for fills to qualify

as particularly vulnerable to collapse compression:
the fill

- was not subjected to a heavy compaction,
- was placed relatively dry and has never been fully saturated or above a critical degree of saturation,
- is under relatively low stress.

According to these criteria many fills are in a metastable collapse susceptible state. Using oedometer tests, Feda [42, 44] demonstrated a collapsible response of fragmentary clay subjected to inundation at 60 kPa (see Fig. 2.13 left), and even the collapsible response of a specimen at the vertical load of 1 MPa (see Fig. 2.13 right).

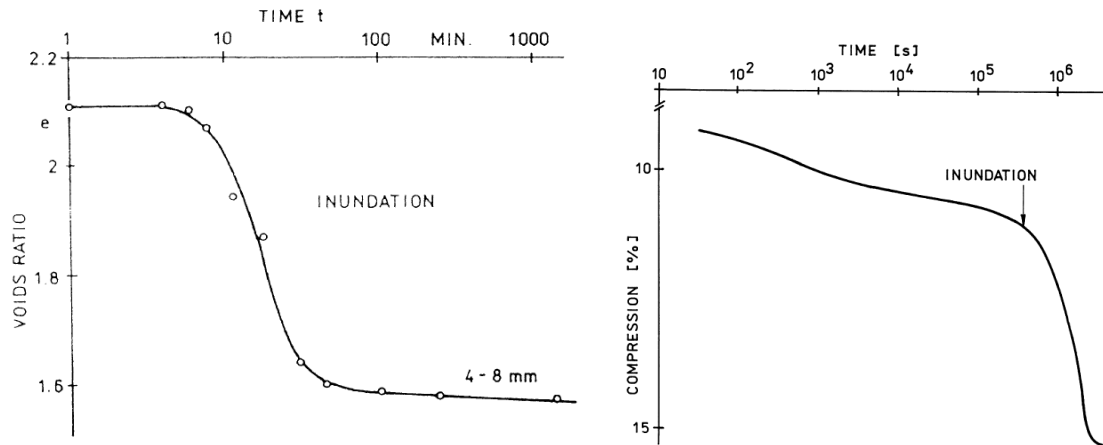


Figure 2.13: Collapse after inundation of naturally wet specimens of fragmentary clay loaded to 60 kPa after Záleský in [44] (left); and loaded to 1 MPa [42] (right).

A collapse settlement in a heterogeneous fill material can act as a triggering mechanism for flowslides [46] (see Sect. 2.4.2). A sudden decrease in pore volume can mobilize an increase in pore water pressures in the surrounding soil. This induces a drop in the effective stresses, which tend to zero, and liquefaction can occur.

Based on a literature review the phenomenon of hydrocollapse of different soil types is discussed in greater detail further in this Section.

Origin of collapsible soils

A distinction can be made between collapsible soils of natural origin and of man-made origin. Collapsible soils of natural origin can be either transported or residual.

2.4. GEOTECHNICAL PROBLEMS OF FILLS

The conditions in arid and semi-arid climates favour the formation of the most problematic collapsible soils [67]. In some cases, natural collapsible soils can also be found in a mild climate. According to Delage et al. [32], even in the rather wet climate of northern France, the geological disposition of the loess deposits and the structure of their profiles prevents water saturation during wet autumn and winter periods.

Most natural collapsible soils are eolian (wind-blown) loess deposits, which are geologically young or recently altered [112]. Loessial soils are fine-grained, and consist mainly of silt and smaller clay fractions. Their very loose double porosity structure with micro- and macropores developed as a result of eolian sedimentation under dry conditions. These soils experience high shear strengths in their natural state, due to the interparticle capillary forces (caused by matric suction) and possible cementation of grain contacts. However, when flooded, the capillary forces and the cementation disappear, and a sudden collapse can occur. Loessial soils are thus sensitive to variations in the degree of saturation (see Fig. 2.14, 2.15).

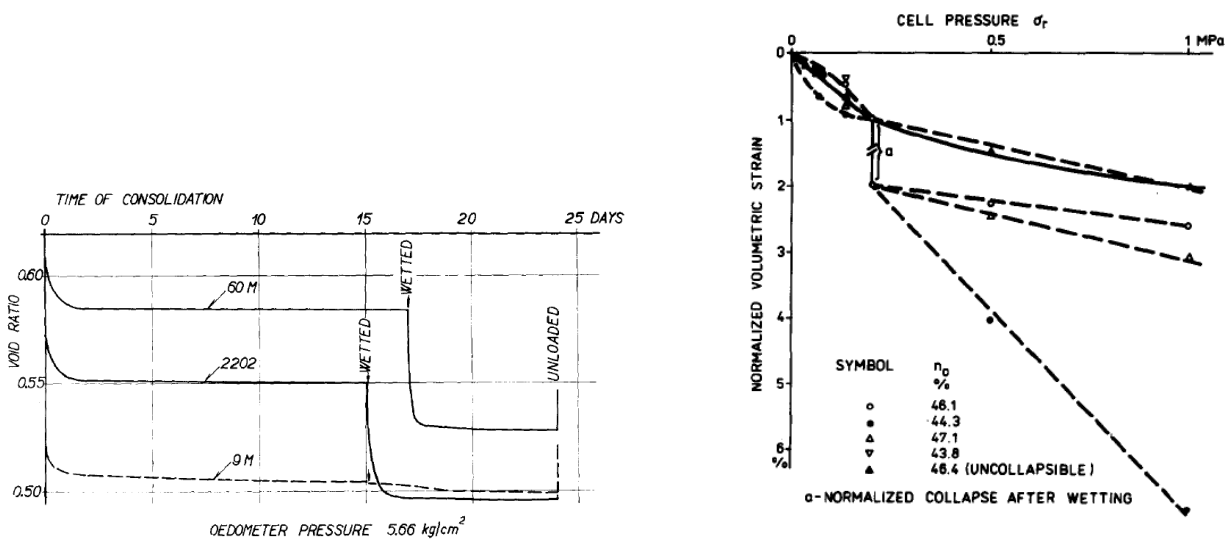


Figure 2.14: Subsidence of loess specimens from Prague - Dejvice in oedometer after wetting (left)[40]; Compression and collapse of loess (right) [41].

Collapsible soils of man-made origin are commonly either compacted at low densities dry of optimum [47], [104], [127] (Fig. 2.15 left) or are loosely filled [47], [58] (Fig. 2.15 right). Mine fills, typical collapse-susceptible soils of a man-made origin [26, 27, 56], are discussed in Sect. 7.

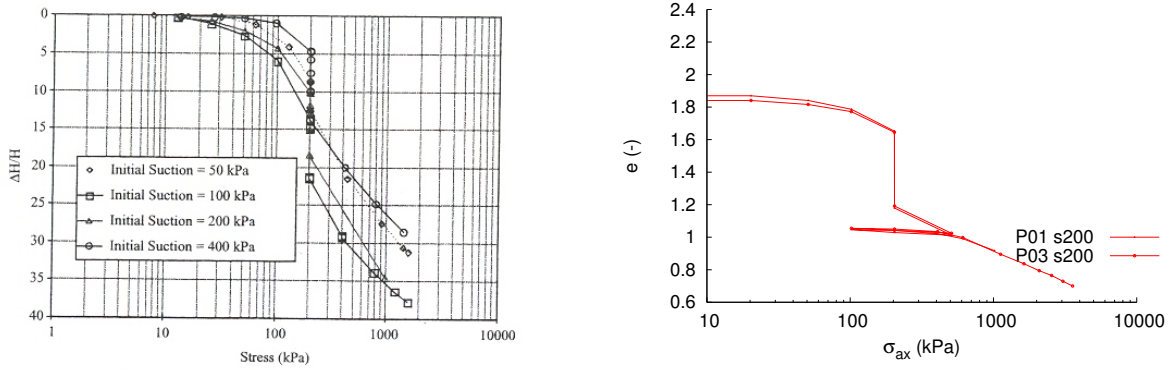


Figure 2.15: Settlement after wetting of compacted clayey specimens with different values of initial suction (left)[127]; Collapse of a model lumpy material after inundation at 200 kPa (right) [54].

Degree of saturation

Collapsible soils are in general unsaturated. The lower the degree of saturation (water content), the greater the collapse potential. Partly-saturated soils have water and air in their pore spaces, and act as a three-phase system. With different pore water and pore air partial stresses, capillary menisci arise at the contacts between the grains. These capillary forces between the grain contacts, also called matric suction, give the partly saturated soil an apparent strength, referred to as capillary cohesion. Matric suction is defined as:

$$s = u_a - u_w \quad (2.6)$$

where u_a and u_w are pore air and pore water pressure, respectively.

Matric suction increases not only the shear strength but also the stiffness. Thus, high matric suction at lower water content makes the soil less compressible (see Fig. 2.16 – left). The capillary effects vanish when the soil becomes fully saturated, and changes to a two-phase system.

According to Feda [41], the collapse surface can be found from the threshold degree of saturation at which collapse takes place in an oedometer test (see Fig. 2.16 right). This surface describes the collapsibility coefficient as a function of the applied vertical stress and the initial degree of saturation.

Suriol et al. [121] tested collapse compression of a compacted soil by various dry unit weights and the water contents. For a given dry unit weight they confirmed that the collapse potential increases with decreasing water content as suction increases (Fig. 2.17).

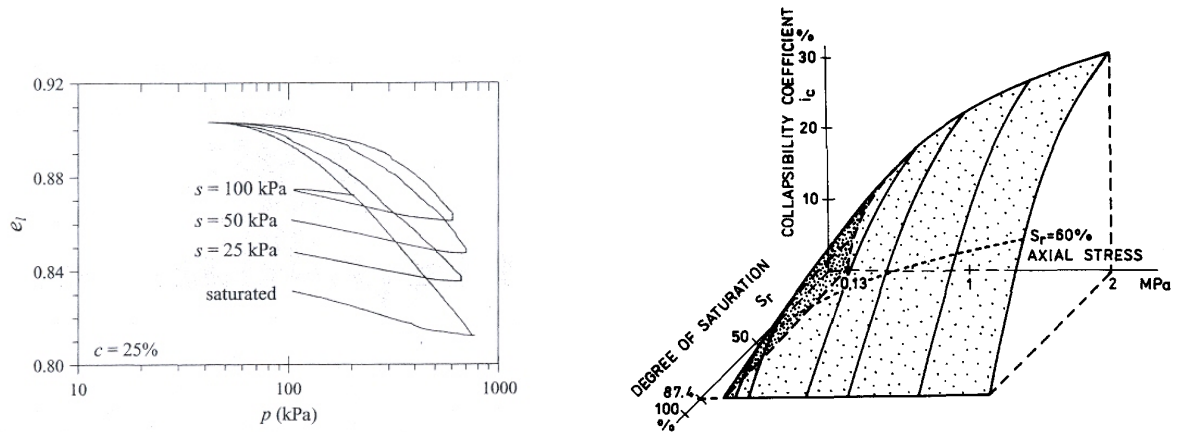


Figure 2.16: Void ratio versus mean net stress for different suctions of artificially cemented unsaturated sandy soil (left) [24]; Collapse surface of loess (right) [41].

Dry density

Relatively low dry density associated with a high amount of open voids in the structure (see Fig. 10) is typical for collapse susceptible soils. Unsaturated soils with dry unit weights below 16 kN/m^3 tend to be prone to collapse [91]. Low dry unit weight is not the only prerequisite for the collapse potential. Collapsible soils must have also open and large macrovoids in their structure. Cui et al. [31] and Delage et al. [32] studied the microstructure of loessial soils in northern France using intrusion porosimetry and electron microscopy, and confirmed the presence of large open macrovoids in collapse-susceptible loess. They also showed that collapse is connected with the closure of macrovoids as a consequence of distortion of the matric suction which acts at the macrovoid scale and retains the aggregates in the form of an open structure. However, the microvoids in the inter-aggregates remain almost unaffected.

Herle et al. [64] investigated the relationship between the magnitude of collapse settlement and the number and the size of macrovoids in a sandy soil. They showed that the collapse behaviour is linked to the granulometric properties of the tested soil. Relative settlements due to flooding increased with decreasing mean grain size and increasing non-uniformity of the grain size distribution for artificially dumped sands. Suriol et al. [121] found that dry density corresponding to the transition from swelling to compression behaviour is reasonably well defined and rather insensitive to water content. The denser the soil at a constant water content, the smaller collapse potential of the soil.

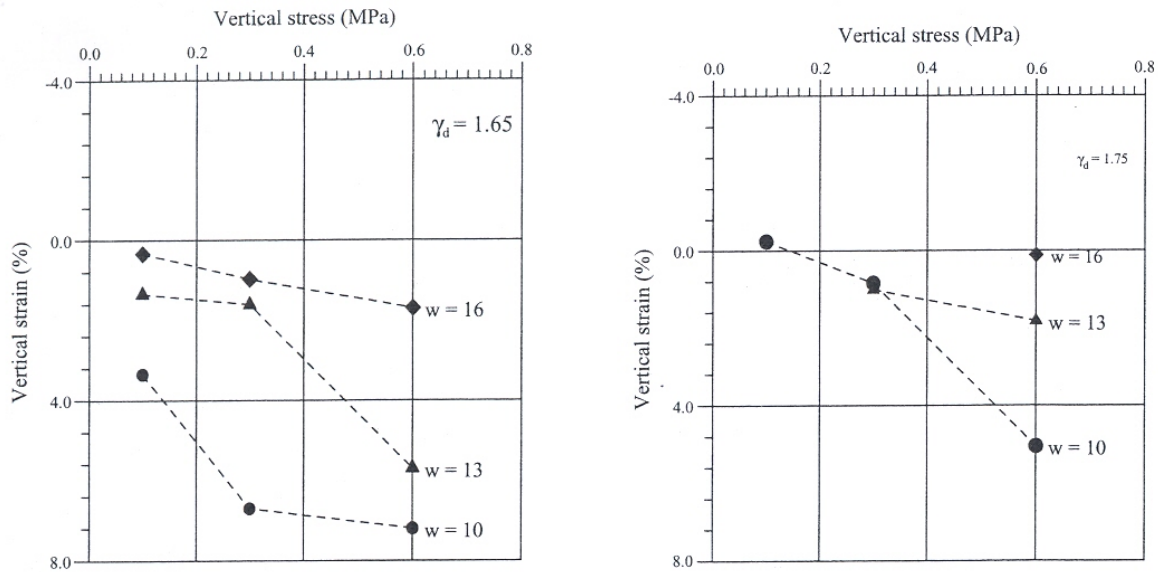


Figure 2.17: Dependence of collapse vertical strain on applied vertical stress for specimens compacted at constant dry density of 1.65 kN/m^3 (left) and 1.75 kN/m^3 (right) and various water contents [121].

Wetting – drying history

In order to produce a soil collapse, the water saturation event must be the first such event that the soil has ever been exposed to [26]. If the soil has already experienced a wetting event in the past, it most probably already collapsed in the course of that event, and no further excessive settlement can be expected. In mild climates geologically older collapse-susceptible soils could have collapsed in the past. Thus young or recently altered soils and soils of man-made origin (fills and compacted soils) are the most prone to collapse. Nevertheless, still in mild climate collapsibility has been often recorded, e.g. in loess deposits in northern France [31, 32] in Prague [40] and also in China and Russia [112].

If the degree of saturation of the soil has previously been higher but the soil has not reached full saturation, a potential collapse could still be anticipated [27]. Based on partial wetting tests in the field, El-Ehwany and Houston [38] proved that partial wetting causes just partial collapse. For the degrees of saturation above 65-70%, essentially full collapse occurs which is equal to the collapse after flooding.

Permeability

Collapsible soils usually have relatively high permeability resulting from their large open and connected macrovoids. Consequently, water can flow fast through the

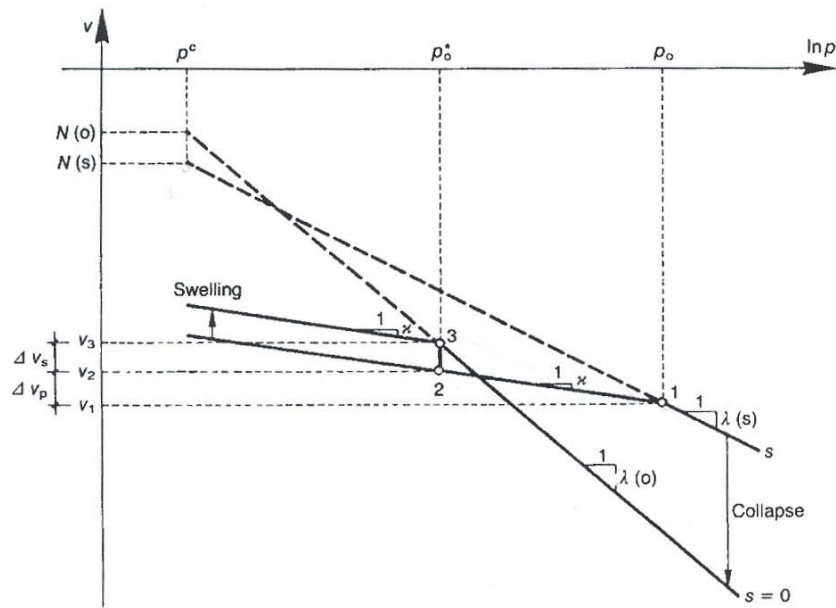


Figure 2.18: Relation between preconsolidation stresses p_0^* (for saturated) and p_0 for unsaturated soil and their relations to swelling or collapsible behaviour in Barcelona-Basic Model [9].

open structure of the soil, and collapse appears as a sudden event affecting a large soil volume [112].

Pressure for closure of macrovoids

The higher the initial stress level, the lower the amount of macrovoids in the soil structure. Therefore the collapse potential vanishes at higher stresses when the large voids become closed.

Lower pressure limit

If a fine-grained unsaturated soil is inundated under low vertical stresses, it can rather swell than collapse. A sufficiently high total stress is thus a necessary condition for observing a collapse in metastable soils [4]. The apparent preconsolidation pressure p_0^* [9] denotes a limit separating a quasi-elastic response (swelling) from collapsible (irreversible) behaviour due to wetting (see Fig. 2.18).

This condition is not valid for loose sands. Inundating loose sand causes a collapse even under vanishing total stresses. This is due to a very low matric suction acting in the relatively large macrovoids. Herle et al. [64] conducted oedometer tests with

dry and wet sand samples. Sand specimens were flooded at very low or vanishing vertical stresses. A sudden decrease in the void ratio was observed in all cases.

2.5 Methods of investigation of clay fills

It is difficult to investigate the mechanical behaviour of fills, due to their high degree of inhomogeneity and anisotropy. Large-scale laboratory testing, field testing and advanced numerical modelling are often combined to describe the complex behaviour of fills. A complex approach combining an extensive laboratory investigation, a statistical evaluation and numerical modelling used for evaluating a large-scale field test was recently successfully applied in the Hambach mine and helped in optimizing the geometry of the filled spoil heaps (see Sect. 2.5.1, [63]).

2.5.1 Laboratory tests

Standard laboratory testing is generally not sufficient for characterizing the lumpy soils from clay fills due to the typical dimensions of the lumps. Scaled-down models of lumps can be used, however the consequences of scaling down the lumps have to be taken into account. The scale effects are also considered in this thesis.

Even if non-standard large scale devices are used for testing, the lumps of reduced size must also often be used. Standard size laboratory tests can be used when characterizing the behavior of the original overburden soil or the material of lumps.

However, for sites where the lumpy structure is not significant and the soil is filled in a disturbed state, a standard laboratory investigation supplemented with field testing produces a useful tool for characterizing the fill behaviour. An evaluation of an extensive laboratory investigation resulted in a concept for determining the undrained shear strength for waste soils in the Hambach mine [63]. The scatter of the soil properties and states correlated with the scatter of the undrained shear strength. A log-normal distribution of the undrained shear strength was found for the M2 material class (see Fig. 2.4). This statistical distribution was considered in a numerical analysis using the hypoplastic model for clays [94]. The model takes into account the influence of soil consistency on undrained shear strength. Satisfactory agreement between the model prediction of consolidation behaviour and the in-situ hydrostatic survey of the large-scale field test confirmed the applicability of the model.

2.5.2 Centrifuge modeling

Centrifuge modelling can offer a helpful tool for simulating the behaviour of fills. The entire bodies of the fills and the structures on their surface can be simulated. However, the influence of scaling down the lumps on their stiffness and strength should be considered, in the similar way as in laboratory testing.

Centrifuge tests are relatively rare due to the unavailability of the centrifuge devices and the difficulty of performance. Najser [100] and Najser et al. [102] modelled the self-weight consolidation process of the clay fill under fully saturated conditions and the construction of a trial embankment on its surface in a geotechnical centrifuge. Scaled-down lumps were used to simulate the lump size distribution in-situ. The settlement of the model of an embankment was almost twice as great as the measurement from in-situ monitoring. This was attributed to the reduced porosity in-situ, due to the ageing process caused by weathering, which cannot be modelled in a centrifuge. Pooley et al. [106] compared in the centrifuge different ground improvement techniques on double porosity clay landfills.

2.5.3 Monitoring and field tests

Because the filled material deforms significantly, long-term field monitoring of pore pressures, the inclination and movements of slopes, settlements, etc., should be carried out not only during the fill operation but also after its closure (for example [17, 16, 25]). Since the behaviour of fills is unusual, and in many cases difficult to predict, field testing and monitoring are often essential for optimizing the design of fills, or when any construction work is planned on the surface of the fill.

Field testing should be preferred in cases where the material is heterogeneous and where its representative elementary volume exceeds the scale of standard and large-scale testing. However such tests are expensive, time-consuming and therefore scarce. In addition, a laboratory study is always necessary in order to supplement and evaluate the data recorded in the monitoring of large scale tests.

So-called large-scale field tests are regularly performed while the mine heaps are being filled at Hambach [81, 105] for optimizing the filling process and the geometries of the regular sections, and also for verifying the stability calculations. The field monitoring equipment (extensometers, inclinometers, pore water pressure gauges, hydraulic settlement gauges at various depths, etc.) for accurate characterization of the soil behaviour. For example, after analyzing a slope failure in a large field test, Kunsche [81] concluded that the shear strengths from small-scale laboratory tests overestimated the real in-situ strength.

Herštus et al. [66] used unique sensors for monitoring the anisotropic tensor of the stresses and strains in clay fills. These sensors were sufficiently robust to record data for a period of 7 years after installation. The results of a large-scale loading test, equipped with these sensors, showed that a 10 m deep layer of soil with open and connected macrovoids is responsible for the recorded settlement at this particular clay fill.

Long-term monitoring of the movements of an extensive reclaimed slope (approximately 25 ha) was presented by Záleský et al. [136, 137]. The slope was formed by an excavated open pit and also by backfilled soil. The combination of monitoring the pore water pressures and movements (using inclinometers, extensometers and geodetical measurements) helped in identifying the shear zones.

The so-called *Ervěnice* corridor, finalized in 1983, is a pioneer construction project on a fill in the Czech Republic. On the surface of a 130 m high spoil heap, infrastructure consisting of a railway, a motorway and even the *Bílina* river in four pipelines had to be transferred in order to facilitate further progress of an open-cast mine [36, 37, 126]. The unique long-term monitoring of the surface movements confirmed large and uneven settlements.

Two fully instrumented trial embankments were constructed in 1998 and 2001 on the route of the planned D8 highway, connecting Prague and Dresden, at the aged fill. The sections crossing the clay fills have a total length of 5.9 km [20, 83]. The long term monitoring of the settlements and the pore water pressure distribution at different depths beneath the embankments were used for predicting the settlement of the trial motorway embankments, and in investigating the behaviour of this aged fill (e.g. [20, 131, 83]). Najser et al. [102] used the monitoring data for a comparison with the settlements of the centrifuge model. A large in-situ loading test of vibrated stone columns for the foundation of the bridge structure was performed at a nearby site. The data were used in the design of the D8 highway [82, 83].

Chapter 3

Studied clay fills - geology and soil classification

3.1 Geology

Open cast coal mining requires excavating the whole thickness of overburden overlying a coal seam. The Bílina mine with depths up to 200 m is the deepest open cast coal mine in the Czech Republic. It is a part of the Tertiary brown coal sedimentary basin Most (Fig. 3.1), which is the largest basin of the Eger (Ohře) rift system. The Eger rift system consists of five sub-basins (Most, Sokolov, Cheb, Zitau and Berzdorf-Radomierzyce), which were separated from each other by a tertiary volcanism (Fig. 3.3) [107]. The Most Basin occupies approximately 840 km² of the total area of the Eger rift system which is about 1400 km² [90].

The deposition of the Most Formation in the basin started in the late Oligocene [84]. Sandy clays and coarser facies of the proluvial sediments of the Duchcov Series were deposited at first. During the Early Miocene, the subsidence of the basin caused an increase of the area flooded by rivers and a flatland with swamps and shallow lakes developed. Coal seam and bodies of Žatec and Bílina river delta sedimented in this environment. The ongoing orogenesis caused further subsidence of the basin, the landscape changed and a large and shallow lake about 1-7 m deep developed. The Lake Clay Horizon (LCH) of the Libkovice Member sedimented in this lake [84]. Fig. 3.2 shows the sequence of the sedimentary layers. The position of the layers excavated in the Bílina mine is displayed as well in the stratigraphy chart.

Mostly soils ranging between silty clays and sandy silts overlie the brown coal seam which is 25-40 m thick. The Bílina delta which is characteristic due to a high degree of grain-size segregation include sand-dominated, fluvial channel fills, sand-

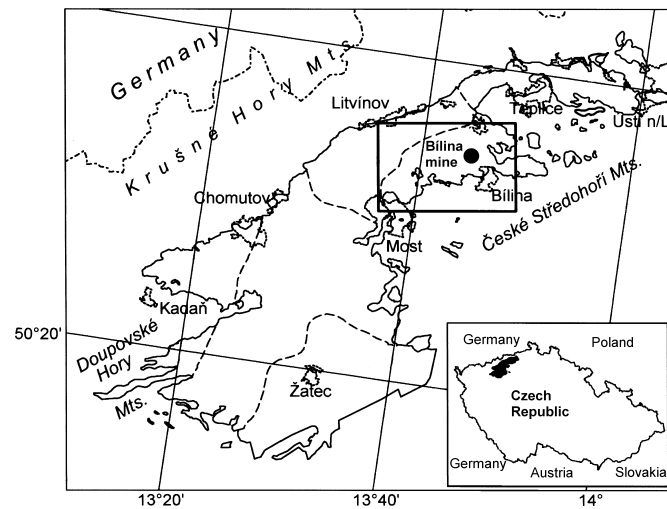


Figure 3.1: Geographic position of the Most Basin and Břilina mine, after [90].

dominated mouth-bar wedges and heterolithic sheets of prodeltaic deposits [107]. These deltaic sediments gradually pass into a Libkovic Series. The sediments of the Libkovic Series are the thickest and can be characterised as monotonic lacustrine silty clays. In the deepest parts of the Most Basin the Libkovic series reach the thickness of about 300 m [90].

After the past geologic uplift, an unknown extent of erosion took place in the basin. According to the reconstruction of the overlying complex in the Most basin by [68], the original thickness of the sedimentary layers is estimated to be as much as 550 m. The subsequent denudation could range between 70 and 300 m.

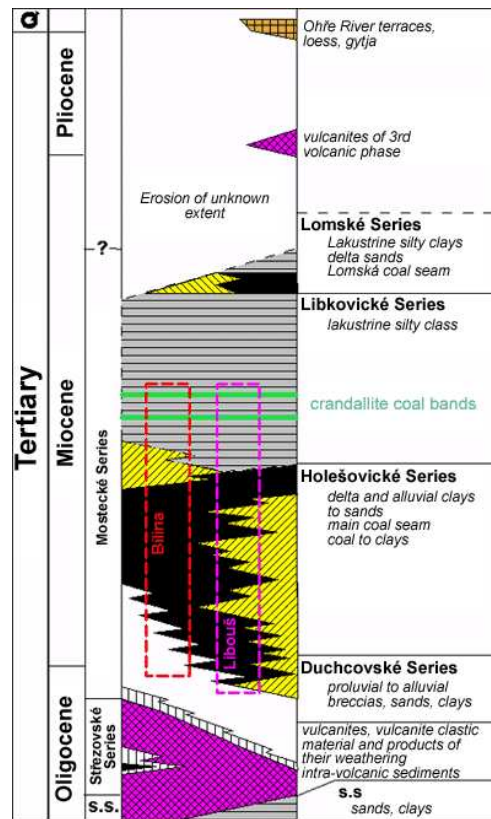


Figure 3.2: Stratigraphic layering of the Most basin, after [90].

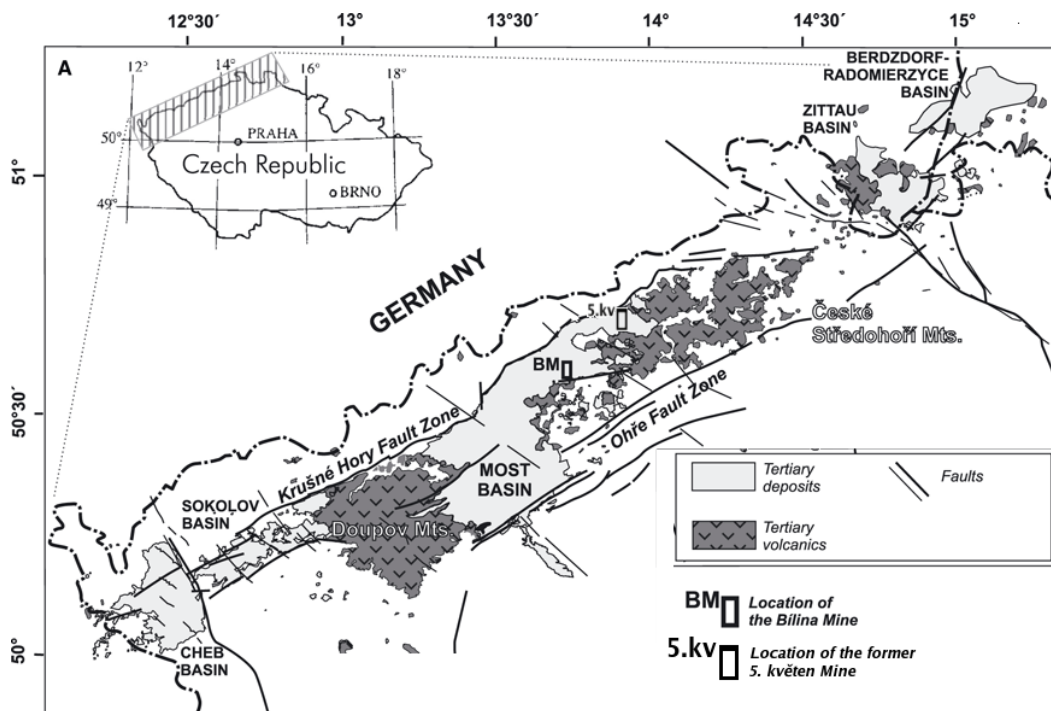


Figure 3.3: Extent of tertiary basins in the North-western Bohemia, after Rajchl et al. [107] - modified.

3.2 Soil classification



Figure 3.4: Extent of non-engineered spoilheaps within the Tertiary basins in the Czech Republic.

The clayey material, investigated in the study, originates from two sites: Bílina mine and former 5. květen mine, near Ústí nad Labem (see Fig. 3.4). The age of fills is different.

The fill at the Bílina mine had been dumped approximately 4 to 5 years prior to taking samples for this study. Some supplementary investigations were done on the lumps which were about 4 to 5 months after dumping. The original soil forming the lumps can be characterized as an overconsolidated Neogene clay of high plasticity with the natural water contents of about 30-34%. Disturbed samples in the form of lumps were taken.

The soil from 5. květen clayey fill was sampled about 40 years after filling and reclaiming of the clay fill surface. Undisturbed samples were taken using a standard pushed-in sampler from three rotary cored boreholes.

Both soils are clays of high plasticity, Table 3.1 summarizes their index properties¹. Table 3.2 summarizes the results of the soil classification and mineralogy for the clay fills soils from three different mines. The mineralogy was determined by X-ray diffraction (XRD) analysis. The soil classification and mineralogical composition of the clay from the fill of 5. květen mine was analyzed by Najser [100]². Available data from another site at the Marie mine in the Sokolov basin by Henriquez [52] is presented in Table 3.2 for comparison.

¹Soil classification of the soil from 5. květen mine was provided by Stavební Geologie, Geotechnika, a.s - (Arcadis Geotechnika a.s. in present)

²The index properties of the soil from 5. květen mine by SG – Geotechnika a.s. and Najser slightly differ (Compare Table 3.1 and 3.2). This can be attributed to the natural inhomogeneity of the excavated overburden.

Table 3.1: Index properties of the original clay from both tested sites.

Site	Plastic Limit	Liquid Limit	Plasticity Index	Actitivity	Particle Density
	$w_P(\%)$	$w_L(\%)$	$I_p(\%)$	A (-)	$\rho_s(\text{g/cm}^3)$
Bílina	36	94	58	1.6	2.675
5. květen	28 (27–31)	59 (56–62)	32	0.9 (0.77-1.03)	2.7

Table 3.2: Properties of the original clay from different sites.

Site		Bílina mine	former “5.kveten” mine [100]	former Marie mine [52]	
Sedimentary basin		Most	Most	Sokolov	
Index properties					
	w_L	in %	94	72	71
	I_P	in %	58	46	32
	A	-	1.6	1.39	1.07
	ρ_s	in g/cm^3	2.675	2.71	2.66
Grading curve					
	Sand	in %	5	11	13
	Silt	in %	61	56	57
	Clay	in %	34	33	30
Mineralogical composition					
	Quartz	in %	46	21	5
	Kaolinite	in %	7	36	36
	Illite	in %	43	12	10
	Smectite	in %	1	25	20
	Chlorite	in %	-	-	3
Accessory minerals					
	Siderite, K-Feldspar, Anatase	in %	<3	<3	-
	Calcite, Pyrite, Anatase	in %	-	-	<3

Although all three soils from different sites have very similar grain size distribution (see Table 3.2), they differ in mineralogy and index properties. Consequences of these differences on the mechanical behaviour is further analyzed in Sect. 6.5.

3.2. SOIL CLASSIFICATION

The material from the Bílina mine forming the clayey lumps, see Fig. 3.7 was described as an illite-kaolinitic clay with an admixture of silt particles of quartz. The montmorillonitic and siderite admixtures were also recorded, however they were classified as minor components. The possible organic content of the soil can not be determined by (XRD). But thermal gravimetric analysis (DSC-TGA) confirmed some admixture of organic content. The gravimetric loss of the organic content of 2% was determined.

The clay from 5. květen mine and Marie mines can be described as kaolinite-smectitic clays with an admixture of illite and quartz. According to the geotechnical classification, the soils from Bílina and 5. květen mine are highly plastic or organic clays (CH-OH) and soil from Marie mine can be characterized as the organic clay or highly plastic mud (OH-MH), see plasticity chart in Fig. 3.5.

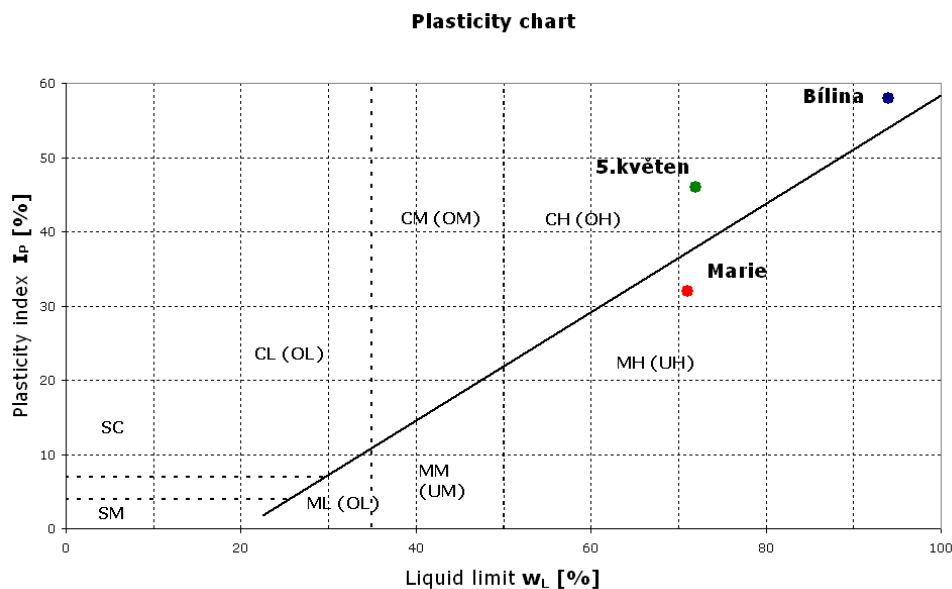


Figure 3.5: Classification of the three clays from different mines.

3.2.1 Size distribution of lumps

The clay fill soil from the Bílina mine was sampled in the form of lumps. The lumpy structure of the samples had already been affected by weathering. Some blocks were disintegrated to a finer mass and smaller fragments, see Fig. 3.6. Rare excessively large soil blocks (greater than 15 cm in diameter) were excluded during sampling and are not present in Fig. 3.6.

It is difficult to determine a representative grain size distribution of the clay fill. If the initially lumpy material is disintegrated and homogenized into a clayey soil, it

has a mean grain size $d_{50} = 0.005$ mm. However, grains of the tested lumpy material of d_{50} of about 10 mm would correspond to gravel, if disregarding a low hardness of the clay macrograins. The grain distribution curve obtained by sieving of the clay fill material can be compared with the grain size distribution of the original soil composing the clayey matrix in Fig. 3.7.



Figure 3.6: Original lumps of the investigated clayey material (left), Sieved lumps (< 5 mm, right).

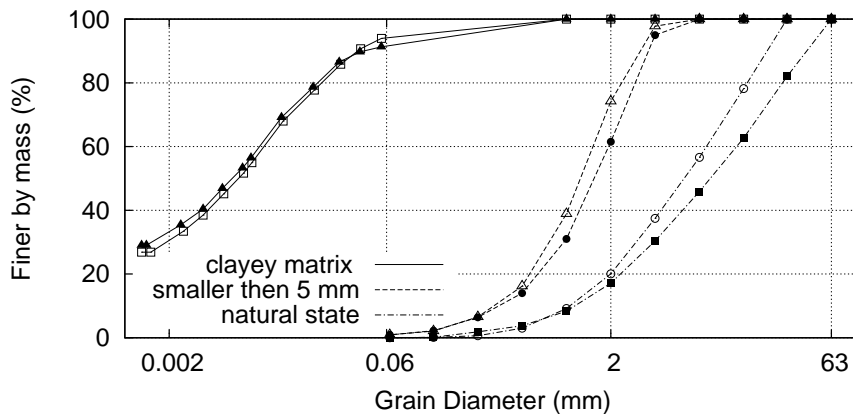


Figure 3.7: Grain size distribution of the clayey matrix (fine), lumpy clay smaller than 5 mm (medium) and natural lumpy clay (coarse).

When testing standard laboratory specimens, the maximum grain size should be smaller than $1/5$ of the smallest specimen dimension, see e.g. [34]. Considering a typical specimen height of 25 mm (shear box, oedometer cell), the maximum grain size should be limited to 5 mm, which is considerably less than the d_{50} of the lumpy clay. Extracting large grains from the samples can cause substantial differences between the behaviour investigated in laboratory experiments and that observed in the field.

3.2. SOIL CLASSIFICATION

For example Shackelford et al. [117] studied the effect of sample size on permeability of compacted samples including particle aggregates or clods by comparing small scale laboratory tests with large scale and in-situ tests. They stated that small scale tests performed on specimens which represent only the smaller particle clod size are probably not representative of the permeability of the overall soil and underestimate the results. However, studies on the mechanical behaviour of clayey soil with different sizes of the fragments are rare and less conclusive, see e.g. [86, 111]. Thus the question of size effects is also addressed in this thesis by comparing the test results for different specimen sizes.

Three types of specimens are referenced further in the text. A term “lumpy specimen” is used for the soil tested either at natural water content or saturated in the form of lumps. Two lump sizes were used natural for large scale testing and reduced (smaller than 5 mm) for standard tests, respectively, see Fig. 3.7. If it is not further defined in the text the term lumpy specimen refers to the specimen with the lumps of reduced size (smaller than 5 mm). Undisturbed and reconstituted specimens were also investigated. Mechanical behaviour of the clay fill soil from 5. květen mine was investigated on the undisturbed and reconstituted specimens, respectively.

Chapter 4

Matric suction in reclaimed clay fills

The long-term field monitoring of matric suction on two sites of reclaimed clay fills is presented and analyzed. The two hydraulic characteristics: saturated hydraulic conductivity and water retention curve were determined on lumpy and undisturbed specimens, respectively. These characteristics and the hydrometeorological data provided by CHMI were used in 1-D inverse analysis of in-situ matric suction in the variably saturated porous media. A concept of dual permeability was used to characterize the double porosity clay fill.

4.1 Role of water in clay fills

Freshly filled clay fills are basically unsaturated. Water enters clay fill material easily in its early stages after landfilling due to the open and large interconnected macrovoids between lumps, while microvoids (natural porosity) inside lumps do not contribute to water flow. Such an open clay fill structure forming interconnected flow channels is highly permeable to both water and air. According to Větrovský [132], the surface of fills can be exposed many months or even years before it is reclaimed or covered by another layer and thus the infiltration into the clay fill is high. Based on field monitoring, Herštus [65] reported an unconsolidated depth with connected macrovoids of 10-20 m, which is also permeable to air. His research was carried out at fills of Merkur, Libouš and Stodola mines. Karpíšková [72], using hydraulic conductivity measurements on saturated specimens, suggested the depth of closure of macrovoids to 12 – 25 m. A threshold for complete closure of macrovoids for naturally wet lumpy specimens of 1500 kPa (which corresponds to the depth of

75 m) tested in oedometer is determined in Chapter 6. This is also in agreement with the laboratory investigations of Herštus [65], who found out the threshold for complete closure of macrovoid of 50 m and more. On the other hand according to the observations in mini-centrifuge, Najser [100] defined no clear limit for closure of macrovoids and suggested their gradual closing with increasing load.

However clay fill structure varies in space and time (see Section 2.2). Initially open and fragmentary structure changes to more or less homogeneous clayey soil due to the plastic straining of lumps under the overburden load (see Chapter 6). The closure of macrovoids is a complex process influenced by many factors. Water entering clay fill bodies causes further changes in their mechanical behaviour. The partially saturated structure formed from originally saturated clay lumps and open, partly saturated (or air dry) macrovoids gradually saturate, with the following consequences:

- disaggregation of lumps induced due to wetting-drying cycles - weathering due to the matric suction oscillations (more details in Sect. 5.2)
- weakening of the lumps due the decrease of matric suction within them (intragranular suction) (details in Sect. 6.)
- collapse under wetting due to the drop of suction on the contacts between the lumps and due to the weakening of the lumps (more details Sect. 7)

All these effects lead to closure of macrovoids, decrease in total porosity and decrease in hydraulic conductivity. Due to the heterogeneity of clay fills however more permeable layers can occur in greater depths and vice versa impermeable positions in lower depth can be formed. Therefore complicated hydrological situation with artesian ground water and water horizons which do not hydraulically communicate can be formed [65]. Herštus [65] reported a formation of two distinct aquifers, which are separated by an isolator from the soil without interconnected macrovoids. The flow regime in upper aquifer follows the preferential paths formed from interconnected macrovoids and depends on the rainfall intensity.

The disaggregation of bigger lumps near the surface due to wetting-drying cycles can increase the hydraulic conductivity, since more open preferential flow channels can be formed due to the disintegration of the fragments. This effect is expected to prevail in the early stages of disintegration, later the hydraulic conductivity at shallow depths starts to decrease as a consequence of filling the macrovoids, plastic straining of weakened lumps and material homogenization. The layers of homogenized clay of plastic consistence were observed also by Větrovský [132] up to the depths of 4 m. Also Charles and Watts [27] reported forming a more compact layer of less permeable clay at shallow depths.

4.2 Monitoring of matric suction

The classic type of jet-fill tensiometers was used for the long-term monitoring of in-situ matric suction at two clay fills: Bílina mine and former 5. květen mine (at the site of the D8 motorway construction).

This type of tensiometers is used in agriculture or soil science [47]. A high-air-entry porous ceramic cup is sealed at one end of a PVC tube filled with deaired water. The other end of the tube is air-tight closed with a rubber membrane. High air-entry ceramics should be saturated prior to installation in-situ. When the tensiometer is inserted into the required depth in a small-diameter hole and its porous tip comes in contact with the soil, water tends to be drawn out of the tube into the soil. The potential for water to be drawn out of the tensiometer can be recorded at the other end of tensiometer by penetrating the membrane by a needle and measuring the vacuum under the tip (see Fig. 4.1). After reaching equilibrium, the recorded vacuum is a measure of the matric suction present in the soil. Proper measurement of matric suction requires careful maintenance of tensiometers, especially re-filling of water column after measuring, changing the rubber membrane regularly and sealing it properly after each re-filling.



Figure 4.1: Process of rubber membrane penetration a measurement of the vacuum using a manometer with digital display.

4.2. MONITORING OF MATRIC SUCTION

The measuring limit of standard type of tensiometers is 90 kPa due to the cavitation of water column in tensiometer (see e.g. [47]). Furthermore the maximal recorded matric suction depends on the height of water column in the tube. The maximal measurable matric suction of a 2 m long tensiometer is approximately 70 kPa. Water column of approximately 2 m (depending on actual water column height during reading) acting on the gauge in the depth should be subtracted from the maximal recorded vacuum of 90 kPa. Positive pore water pressures can be also recorded by tensiometer analogously to piezometer.

Tensiometers with the lengths between 0.5 – 2 metres were used. The in-situ suction measurements started in April 2004 (former 5. květen mine) and in July 2004 (Bílina mine) and lasted 4.5 years till the end of 2008. Readings proceeded manually in 14 days to three weeks intervals.

At the clay fill of the Bílina mine the tensiometers were installed in a moderate slope in three groups (see Fig.4.2). The measured suctions reached maximum values of about 90 kPa in late autumn. It is however the limit of negative pore pressures that can be measured by this type of gauges due to cavitation. In late winter and spring the suction dropped to the values close to zero even in the deeper gauges. The seasonally dependent cyclic suction oscillations were recorded. The two uphill groups (L01 and L02) show consistently higher suctions even in winter period (Fig. 4.3). On the contrary, just the shallow tensiometers (Fig. 4.3- bottom, gauges T051, T113, T114) from the bottom group (L03) at this site exhibited moderate and high suctions in summer and autumn (85 kPa at maximum), while during the wet winter period the suctions vanished. The tensiometers monitoring the suctions in 190 cm depths (Fig. 4.3- bottom, gauges T223, T224, T227) show consistently lower suctions. The groundwater level is in the depth of 17 meters and does not influence the suction measurements. The higher suctions in uphill groups during dry periods may be explained by the surface and shallow subsurface runoff.

At the site near Ústí and Labem (aged clayfill of former 5.květen mine), the matric suction was monitored next to the D8 motorway construction site. Monitoring went on also when the construction was finished and the motorway was in service. Despite the protecting of the measuring points (see Fig. 4.4 – left top) some tensiometers were destroyed. Some spoil was deposited next to one of the tensiometer location (P101,P151 and P201) and a free runoff was restrained. A pond of surface water developed and the location point was flooded (see Fig. 4.4 – right top). Thus the measurement was possible just on a reduced number of tensiometers.

Very low or zero suctions and even positive values of pore water pressure were measured at this clay fill (Fig. 4.5). The fill remained saturated during the whole

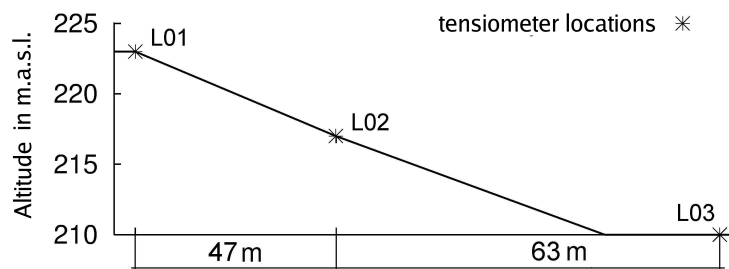


Figure 4.2: Scheme of tensiometer locations - Břilina mine.

monitored period. The only rise of matric suction was measured in the tensiometer installed in shallow depth, due to the landscaping of the surface after the construction of highway was finished. The surface around the tensiometers position was lowered of approx 75 cm (see Fig. 4.4 – bottom). Surrounding of these tensiometers was thus exposed and significant suctions were recorded just by the exposed gauges in summer and autumn period.

4.2. MONITORING OF MATRIC SUCTION

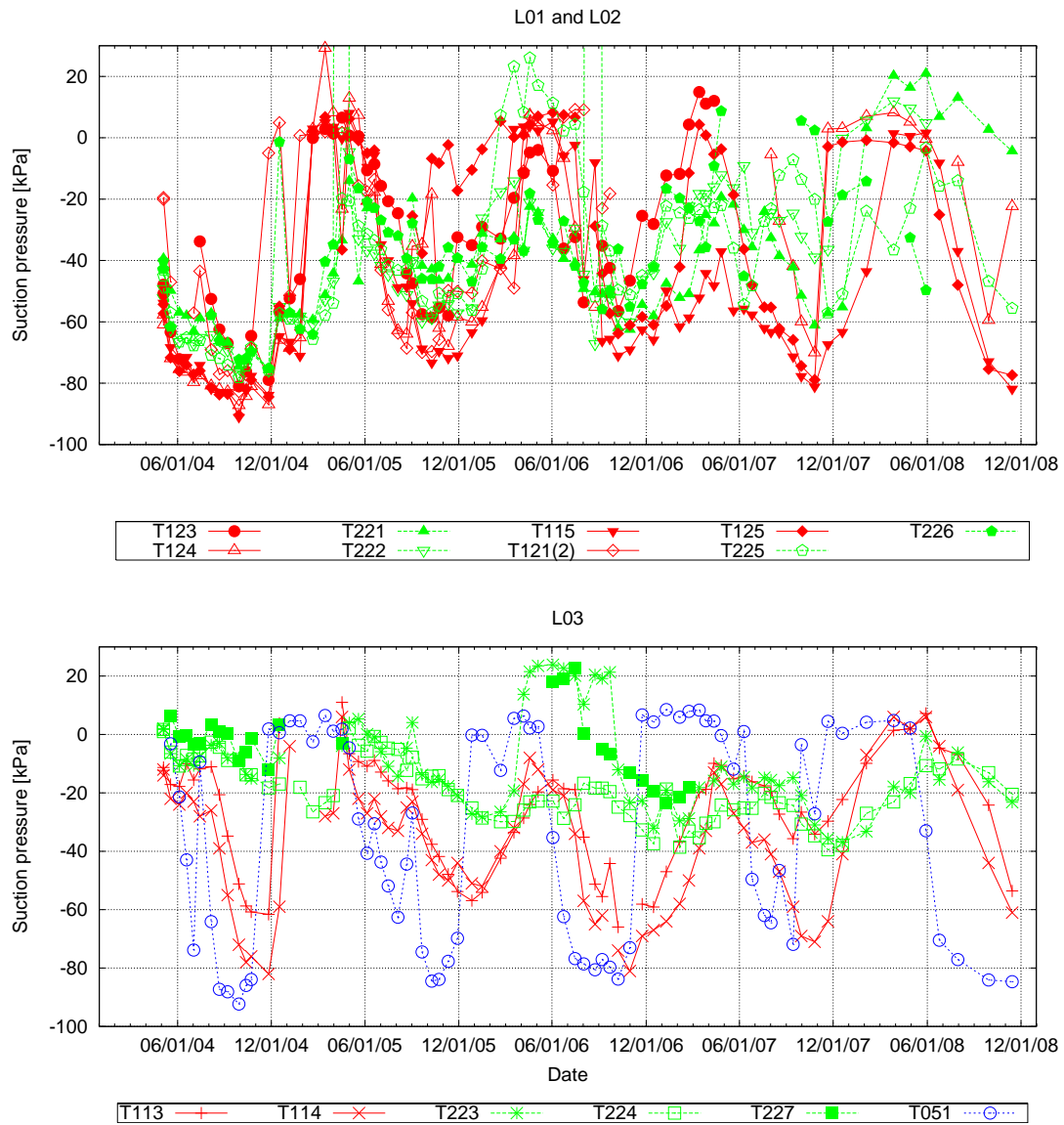


Figure 4.3: Suction pressures recorded at the clay fill of the Bílina mine. Blue depicts 0.5 m, red 1.0 m and green 2.0 m long tensiometers.



Figure 4.4: Measuring points at the site of highway construction (former 5. květen mine).

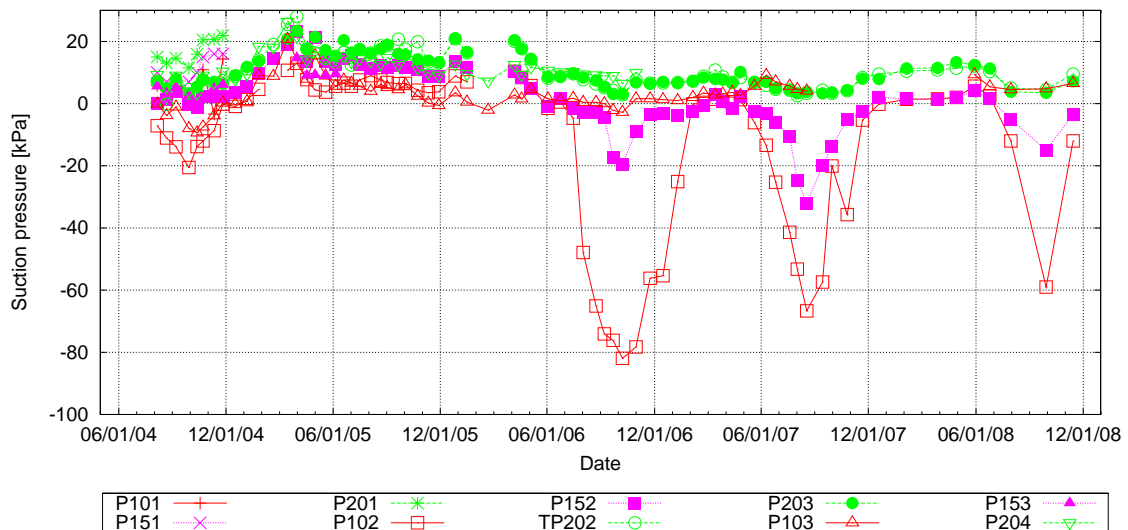


Figure 4.5: Suction pressures recorded at the site of highway construction (former 5. květen mine). Red depicts 1.0 m and violet 1.5 m and green 2.0 m long tensiometers.

4.3 Hydraulic characteristics

Saturated hydraulic conductivity and water retention curve, the two hydraulic characteristics necessary for the description of water flow in variably saturated pore space, were determined on two types of samples for the material from the clay fill of the Bílina mine. For the first test series the lumpy specimen with reduced size distribution was used. The specimens were compacted to the void ratio of $e_{init} = 1.8$ ($\rho_d = 1.25 \text{ g.cm}^{-3}$) at a natural water content of $w = 34\%$. In this manner, the similar density as for the compacted specimens tested in oedometer (see Sect. 6) was achieved. Undisturbed specimens sampled from the depth of about 2 – 3 m were tested as well. Both hydraulic characteristics of the two specimen types are discussed in the following subsections.

4.3.1 Saturated hydraulic conductivity

Hydraulic conductivity strongly depends on the soil structure and its interconnected voids. Hydraulic conductivity of clay fills cannot be considered as a parameter but varies as the structure changes. As the originally open and interconnected intergranular voids close, the hydraulic conductivity decreases by several orders of magnitude. Tests to simulate the changes of hydraulic conductivity in the saturated state with increasing stress level in the laboratory conditions have been performed on the lumpy specimens. The lumps were compacted in the triaxial cell. This stabilized the specimen which enabled the tests also under relatively low effective stresses. The specimens was saturated using the back pressure of about 5 kPa and than consolidated to different effective stresses of 30, 60, 100, 200 and 500 kPa. At each effective stress a constant head permeability test was performed. At lower cell pressures of 30 and 60 kPa the a lower pressure head of about 4 kPa was maintained, by greater cell pressures the pressure head of about 8 – 10 kPa was chosen. The hydraulic conductivity was strongly stress dependent (see fig. 4.6). It dropped from about $K = 4 * 10^{-6} \text{ m.s}^{-1}$ by the stress of 30 kPa, which corresponds to the hydraulic conductivity of sands, to $K = 2 * 10^{-11} \text{ m.s}^{-1}$ for the specimen consolidated to 500 kPa, which corresponds to hydraulic conductivity of clays used in landfill mineral linings. The decrease in hydraulic conductivity of 5 orders of magnitude was caused by the progressive filling of intergranular macrovoids. As the specimen was left to consolidate to the certain effective stress the softened clayey lumps and strained lump contacts progressively filled the intergranular voids. Due to progressive filling of macrovoids, the low intragranular porosity of the lumps starts to contribute to flow more significantly only at higher stresses.

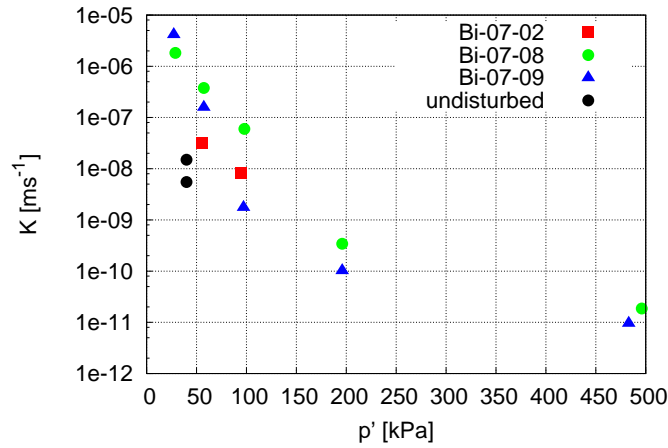


Figure 4.6: Dependence of saturated hydraulic conductivity on stress of the lumpy specimens and of the undisturbed specimens.

The closure and filling of the intergranular macrovoids by finer mass was confirmed not only by the decreasing hydraulic conductivity but also by visual examination of the sample after the test. Fig. 4.7 shows the specimen, which was consolidated the effective confining stress of 500 kPa, after the test. A clearly visible lumpy structure was observed but the intragranular macrovoids were already filled with clayey finer mass. Therefore the hydraulic conductivity of the specimen consolidated to 500 kPa, which is $K = 2 \cdot 10^{-11} \text{ m.s}^{-1}$, also corresponds to the hydraulic conductivity of impermeable clayey soil.

Hydraulic conductivity was investigated by Karpíšková [72] testing lumpy specimens from the clay fill of Bílina mine in oedometer and triaxial device. The samples were consolidated in minicentrifuge and the specimens were cut to obtain specimens consolidated to different stress levels. The consolidation coefficients c_v evaluated using t_{50} according to Cassagrande or t_{90} after Taylor were different resulting in different hydraulic conductivities from the oedometer tests. This was attributed to permeabilities of macrovoids at t_{50} which were more permeable and microvoids at t_{90} . Despite the scatter of the measured hydraulic conductivities, the threshold for contribution of macrovoids to water flow was established to about 12.5–25 m. This threshold is linked to the change in trend of the decrease of hydraulic conductivity. However, the samples of Karpíšková [72] were prepared by another technique than in my study. The samples were air-dried at 50°C and then crushed to achieve the same lump size distribution as used by Najser [100, 102] for centrifuge modelling (see also Sect. 6.5). Subsequently the lumps were pre-consolidated in a mini-centrifuge and specimens with different overconsolidation ratios were cut. While the lumps in

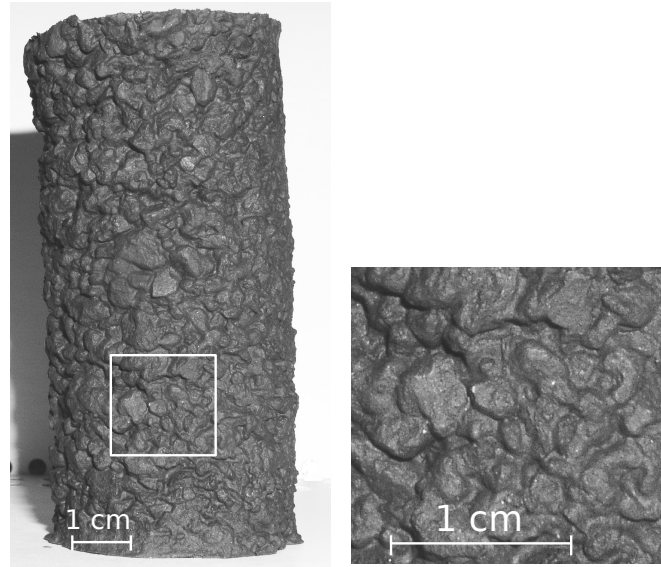


Figure 4.7: Dismantled specimen after permeability test.

my study were sieved from the material of natural water content of approximately 34 %. As shown in Sect. 5.2 air drying and subsequent flooding of lumps causes more rapid lump disintegration than flooding at the natural water content, after which the lumps remain intact. Nevertheless a similar trend of decrease of hydraulic conductivity of 5–6 orders of magnitude from 10^{-5} do 10^{-11}m.s^{-1} was measured, which compares well with my study.

Hydraulic conductivity of the lumpy material is furthermore influenced by time. Hydraulic conductivity of the specimen, which was left for about 1 month at the effective stress of 60 kPa, dropped from $1.7 \cdot 10^{-6}\text{m.s}^{-1}$ to $4.3 \cdot 10^{-9}\text{m.s}^{-1}$ (see Fig. 4.8). Furthermore, the decrease of hydraulic conductivity was observed by subsequent stress levels (see Fig. 4.8). A 1-D creep of individual contacts of lumps (discussed in Sect. 6) can explain this effect. This behaviour add further difficulties in describing this material. The hydraulic conductivity cannot be considered constant even for a given stress level.

4.3.2 Water retention curve

The water retention curve WRC (often also denoted as soil water characteristic curve - SWCC) defines a series of equilibria between the soil water content and the applied matric suction.

Similarly to the hydraulic conductivity, the soil water characteristic curve is linked to the size and geometry of pores. Water retention curve for soils with double porosity

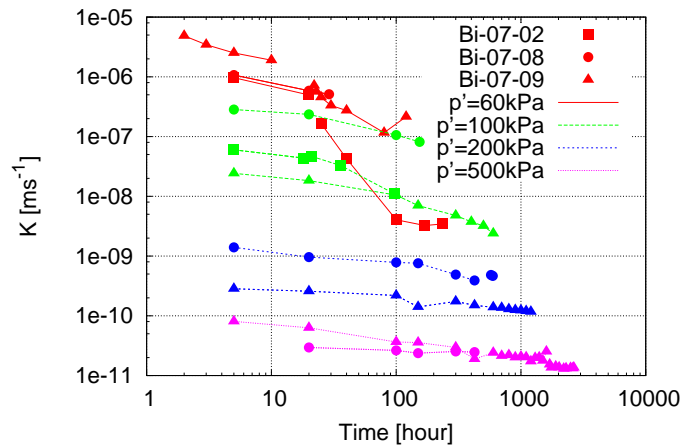


Figure 4.8: Dependence of saturated hydraulic conductivity on time for different effective stresses.

depends on the amount of open macrovoids present in the soil structure. According to Vanapali and Fredlund [125] the WRC of compacted fine-grained soils is mainly dependent on structure (aggregates) and the stress history. The WRC is governed by the macrostructure for soils compacted dry of optimum. A bimodal character of WRC was found by Bagherieh et al. [13] for compacted double porosity kaolin loaded to low and intermediate stresses. WRC for the same soil but loaded to high stresses approached the one of the non-aggregated soils, because the macro pores were closed. The principle of effective stresses as proposed by Khalili [75] accurately predicted the volume change in this aggregated soil.

Water retention curves were determined on two specimens types in the standard pressure plate apparatus¹. The maximal matric suction of 1500 kPa was applicable. Results are depicted in Fig. 4.9. Just the drying path of WRC was measured and therefore the hysteresis was not taken into account.

Four lumpy specimens were compacted to the density of $\rho_d = 1.25 \text{ g.cm}^{-3}$ at the natural water content of $w = 34\%$ (denoted as *BiCom*), similarly to the specimens for measuring of hydraulic conductivity. Two WRC were determined for undisturbed specimens sampled from the depth of about 2–3 m.

A difference reaching $\theta = 0.19$, or $w = 0.32$ respectively, was observed comparing the initial volumetric and gravimetric water contents of all specimens. No obvious difference was observed between the compacted lumpy *BiCom* and undisturbed *BiUn* specimens comparing the WRCs expressed using volumetric water content.

¹The tests were performed at the Institute of Hydropedology at the Technical University in Prague

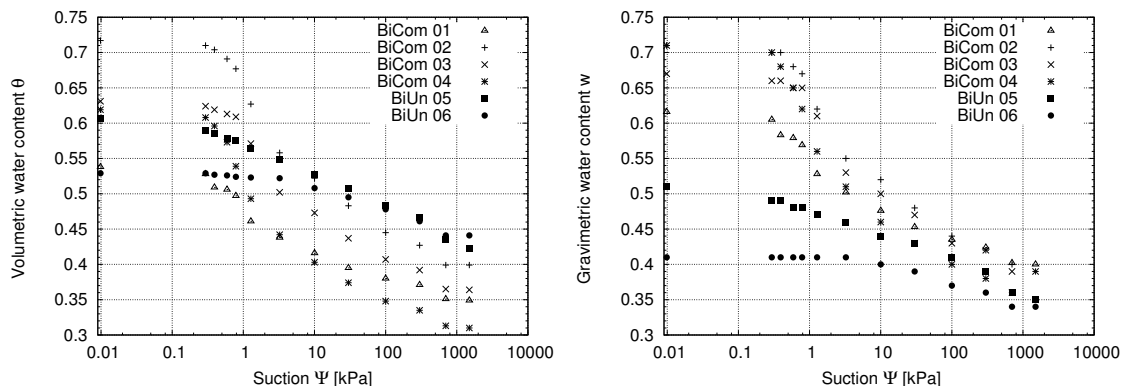


Figure 4.9: WRCs for the the compacted lumpy model *BiCom* and undisturbed *BiUn* specimens. Volumetric water content (left) and gravimetric water content (right) are displayed.

Gravimetric water content expression of WRC clearly differentiates between *BiCom* and *BiUn* specimens. *BiUn* have significantly lower saturated water content and thus the initial void ratio than the *BiCom* specimens. This discrepancy could be caused due to an error in estimating the initial volume of the specimen. Furthermore WRC using the volumetric water content may not reflect the real volume of the specimen. After reaching the equilibrium with the adjusted matric suction the specimen is weighted and the volumetric water content is back calculated using the mass and density of water. The volume of the specimen is usually not determined for each point of WRC in the particular laboratory. This could cause a discrepancy between the real and measured volumetric water contents mainly for soft and metastable soils, which change their volume under the application of suction.

The lower initial water contents and also the flatter course of WRC for the undisturbed specimens indicates the structure degradation in shallower depths and diminishing the macroporosity in the aged fills. However the void ratio (and thus the water content) of standard undisturbed specimens does not have to depict the total void ratio of the sampled soil. Undisturbed specimens are always cut from the intact part of the sample and thus have smaller void ratio. It is difficult to capture the possible present macroporosity by standard size sampling.

Parameters of WRC

A freeware program RETC [109, 124] was used for the estimation of model parameters of WRC. RETC uses a nonlinear least-squares optimization to estimate the model parameters from observed retention and/or hydraulic conductivity or diffusivity data. Either Brooks-Correy or Van Genuchten parametric models can be used

to represent the soil water retention curve. Van Genuchten retention model [123] was chosen for representing the WRC of both lumpy and undisturbed specimens. This model approximates the WRC using:

$$S_e(h) = 1 \text{ for } h \geq 0 \quad (4.1)$$

$$S_e(h) = \frac{1}{(1 + (\alpha h)^n)^m} \text{ for } h < 0 \quad (4.2)$$

where $S_e(h)$ denotes the effective degree of saturation expressed as

$$S_e(h) = \frac{\theta - \theta_r}{\theta_s - \theta_r} \quad (4.3)$$

θ_s and θ_r are saturated and residual volumetric water contents, h denotes suction. α , n and m are empirical parameters characterizing the shape of WRC. For m the Mualem approximation for pore tortuosity was used giving $m = 1 - 1/n$. For a detailed description of both retention models see [124].

Fig. 4.10 displays the Van Genuchten model curves of the two chosen WRC. Their parameters are summarized in Table 4.1. The WRC is a necessary soil characteristics for the description of water transport in variably saturated pore space. Using the parameters of the retention model of WRC and the saturated hydraulic conductivity, a function of the unsaturated hydraulic conductivity which depends on the suction, can be calculated (for details refer [124, 29]).

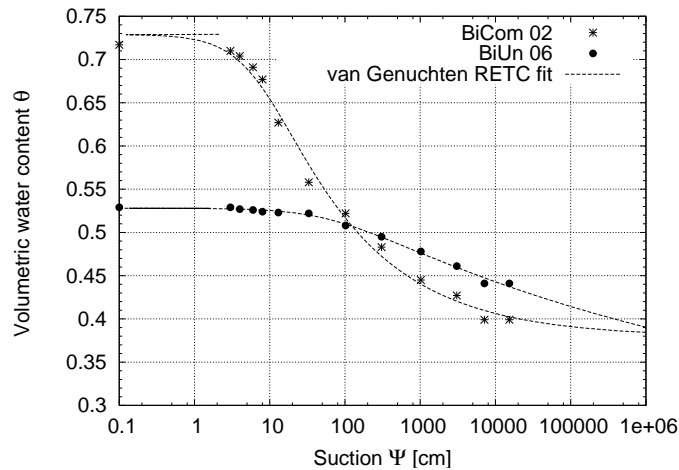


Figure 4.10: WRCs of the selected lumpy *BiCom02* and undisturbed *BiUn06* specimen and Van Genuchten model fit using RETC program.

Specimen Nr.	Θ_r	Θ_S	α	n
BiCom 02	0.37890	0.72909	0.13568	1.35338
BiUn 06	0.26609	0.52796	0.01675	1.07661

Table 4.1: Parameters of Van Genuchten retention model for the two selected specimens.

4.4 Inverse analysis of matric suction using dual permeability approach

Almost all natural soils have preferential paths for water flow. They can be formed by natural pore systems, roots, worm holes etc. Therefore there exists a standard approach in modelling of unsaturated water flow in double porosity media concerning the dual permeability (see e.g. [15, 49, 87, 118, 129, 130]). These models use a phenomenological concept for modelling the water flow. However hydro-pedological models are restricted to a stable porous media, which does not deform in time. When solving a deformation problem linked with unsaturated water flow a coupled hydro-mechanical model should be used.

Water flow in dual-permeability models is described using separate flow and transport equations for the fracture (macropores) and for the matrix (micropores). Two domains with different hydraulic and transport properties are assumed for the system of the structured porous media, both domains can hydraulically communicate with each other [129].

The program S1_D_Dual (Vogel, 2000) was used for the simulation of the water flow. The model considers variably saturated pore space and dual permeability. In the program just the vertical water flow in the soil profile in time is assumed. Matrix domain w_m and the preferential flow domain w_f are prescribed per unit volume of the bulk soil with $w_m + w_f = 1$. The total water content is thus the composition of water content in preferential flow and matrix domains:

$$\theta = w_f \theta_f + w_m \theta_m \quad (4.4)$$

A pair of Richard equation of flow for both preferential flow and matrix domains is solved in every iteration step:

$$\frac{\partial w_f \theta_f}{\partial t} = \frac{\partial}{\partial z} \left(w_f K_f \left(\frac{\partial h_f}{\partial z} + 1 \right) \right) - w_f S_f - \Gamma_w \quad (4.5)$$

$$\frac{\partial w_m \theta_m}{\partial t} = \frac{\partial}{\partial z} \left(w_m K_m \left(\frac{\partial h_m}{\partial z} + 1 \right) \right) - w_m S_m - \Gamma_w \quad (4.6)$$

The detail description of the model is out of scope of this thesis and for details please refer [29].

4.4.1 Problem definition

The flow in variably saturated clay fill at the site of Bílina mine was simulated. An inverse analysis of optimizing the input parameters for preferential flow and matrix domains was used to simulate matric suctions recorded in tensiometers. Two different boundary conditions for the simulations in the slope and under the slope were used to differentiate between each other when using S1_D_Dual.

4.4.2 Hydraulic characteristics for dual permeability model

The saturated hydraulic conductivity and Van Genuchten parametrization of the two selected water retention curves studied in Sect. 4.3 were used in simulation. Hydraulic characteristics of undisturbed specimens were assumed for the matrix and compacted model lumpy specimens for the preferential flow domain. However these hydraulic characteristic just approximate the real in situ conditions.

4.4.3 Boundary conditions

Top and bottom boundary conditions of the modelled soil profile have to be defined. For the bottom boundary condition a free drainage in the depth of 5 m was chosen, because the position of groundwater table in about 17 m depth does not effect the simulated pressure conditions in the depths up to 2 m.

The top boundary condition was defined as a Neuman boundary condition with the prescribed water flux. The daily sums of rainfalls and potential evapotranspiration were used. All hydrometeorological data were provided by the CHMI. To simulate the vertical flow in the slope, the surface and shallow subsurface runoff should be taken into account. According to Větrovský [132] the runoff from the freshly filled clayfills is almost negligible. All surface water infiltrates to the bodies of fills. However the water flow in already reclaimed fill with the grass cover on its surface was simulated. The surface and shallow subsurface runoff for the moderate slopes covered by grass is approximately 0.2–0.25 of the rainfalls [28]. The 75% of the daily precipitation was therefore used for the simulation of the vertical flow in the slope.

Potential evapotranspiration

The Penman-Monteith method [99] was used to estimate the daily potential evapotranspiration from the meteorological data provided by CHMI. The data of three nearby meteorological stations Vaňov, Kópisty and Kočkov had to be combined, because none of the stations provided the complete data needed for the estimation of potential evapotranspiration. The Penman-Monteith equation can be written as follows:

$$Q_\lambda = \lambda ET = \frac{\Delta (R_n - Q_c) + \rho_a c_p \frac{p_{vs} - p_v}{r_a}}{\Delta + \gamma \left(1 + \frac{r_s}{r_a}\right)} \quad (4.7)$$

where: Q_λ latent heat flux (W/m²)

ET evapotranspiration intensity (m/s)

R_n net radiation supply (W/m²)

Q_G ground heat flux (W/m²)

p_v vapour pressure (Pa)

p_{vs} saturated vapour pressure (Pa)

ρ_d water density (kg/m³)

ρ_a air density (kg/m³)

λ specific latent heat of vaporisation (J/kg)

c_p specific isobaric heat capacity of air (J/kg/K)

r_a aerodynamic resistance (s/m)

r_s surface resistance (s/m)

Δ slope of the saturated vapour pressure curve (Pa/K)

γ psychrometric constant (Pa/K)

Several assumptions had to be taken into account to estimate each term of this equation. The closer methodology of estimating of the potential evapotranspiration using hydrometeorological data is described in [8]. As inputs are required:

- daily mean temperature,
- daily amount of net solar radiation,
- air humidity,
- wind speed,
- air pressure.

In step-by-step procedure described in [8] was used to evaluate the above listed inputs and daily average potential evapotranspiration ET was estimated. It is out

of scope of this these to describe the process in detail. The values of ET were used as the input upper boundary condition in the part of the program S1_D_Dual (*root.in*). The root water uptake due to transpiration was modelled using the a generalized approach of the Fedes sink term [129]. The data of the standard grass cover were assumed.

4.4.4 Simulation results

An inverse problem of the vertical flow in variably saturated pore space was analysed. The simulation results were fitted to the recorded matric suctions in the different depths. The volumetric fraction of the preferential flow domain and a transfer coefficient between both domains were optimized in the inverse simulations to reach a better match with recorded matric suctions. The simulations resulted in time dependent vertical profiles of matric suction.

Preliminary simulations of the monitored period

When simulating the whole measured period of 4.5 years (1.5.2004 – 31.12.2008), the poor response of the simulation to wetting during winter period was observed (red and blue lines in Fig. 4.11). There is no vegetation activity during winter and more water infiltrates when snow slowly melts. The suctions reach their minimum in winter and early spring because the soil profile saturates. However the evaluation of the potential evapotranspiration resulted in some positive records also during winter period. Therefore some unlikely root water uptake was activated using S1_D_Dual and matric suction of the model did not dropped to zero values in winter. The potential evapotranspiration was set to zero in winter time. This resulted in a better reaction to the drop of suction in winter period (violet and turquoise lines in Fig. 4.11). But the rise of suction in further dry seasons was underestimated. Furthermore the simulation often failed to converge by changing the parameters of the model.

Optimization for one season

Only one vegetation season was used for the optimizing of the model response. Fig. 4.12 displays the optimized model prediction with the measured data. The suctions in both recorded depths of 90 cm and 190 cm were modelled sufficiently. This optimal model simulation was achieved by the volumetric fraction of the more permeable (fracture) domain of 0.39 in the depth 90 cm and 0.30 in the depth 190 cm,

4.4. INVERSE ANALYSIS OF MATRIC SUCTION

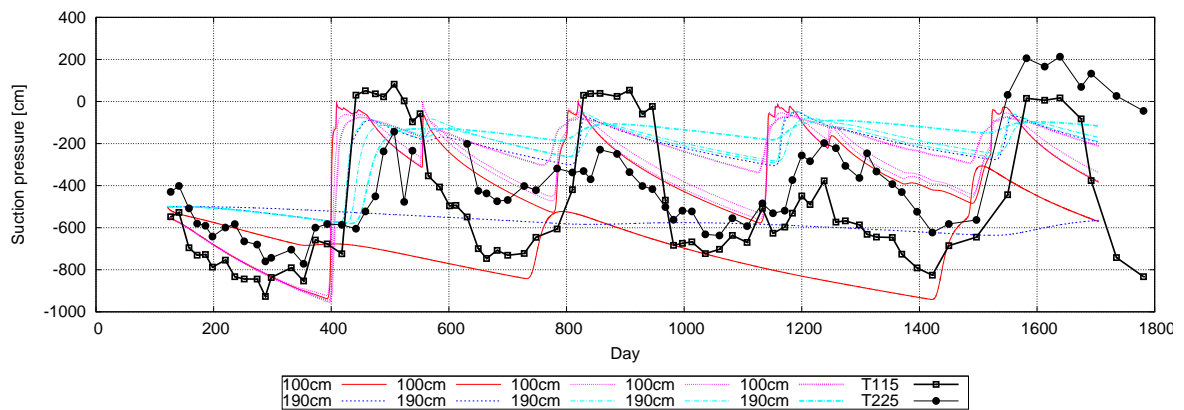


Figure 4.11: Preliminary results of the model response, parameter study (day 124 refers to 1.5.2004 and 1796 to 1.12.2008, respectively).

respectively. The model response without dual permeability approach is displayed for comparison (thin lines in Fig. 4.12), the modelled suctions are underestimated and more rapid reaction was simulated in shallower depths. The simulation resulted in the vertical profiles of matric suction for the selected times, see Fig. 4.13. The modelled suction near the surface reaches more than 1800 kPa in late summer. Furthermore the suction oscillations in the shallow subsurface layers were simulated.

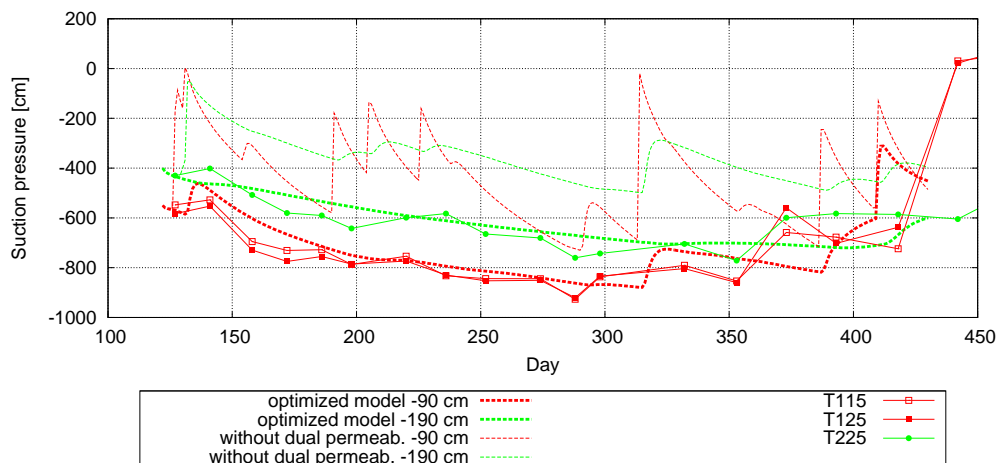


Figure 4.12: Optimized dual permeability model for one season in slope (L01 and L02); and simulation without dual permeability (day 124 refers to 1.5.2004 and 442 to 20.2.2005, respectively).

The parameters optimized for recorded suctions in L01 and L02 were used to simulate the suction profile at L03. The total daily rainfalls were used in this case. The poorer response of the model was achieved see Fig. 4.14.

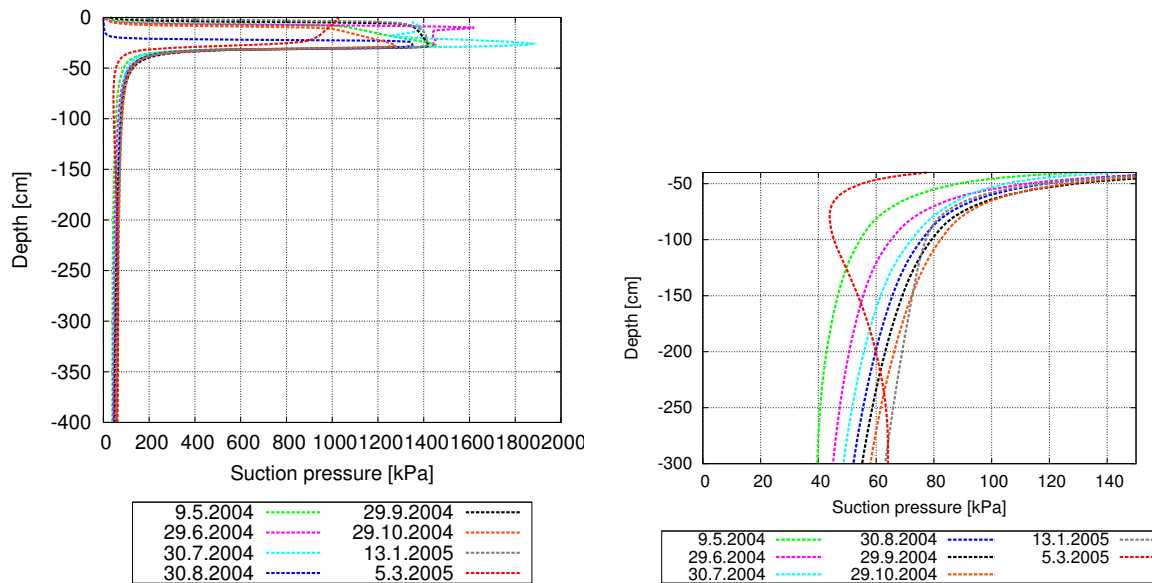


Figure 4.13: Suction profiles of the optimized model - whole profile (left), zoom in the depths of interest (right).

The results of presented simulations should be evaluated rather from the phenomenological point of view, particularly regarding the volumetric fractions of the matrix and the preferential flow domains. The hydraulic characteristics were determined on the lumpy soil with reduced size of lumps and small undisturbed specimens which may not necessarily reflect the characteristics of the soil in situ. Furthermore the data from the distant hydrometeorological stations were used for the input upper boundary conditions.

4.5 Summary

- The 4.5 year long monitoring of matric suctions by standard type jet-fill tensiometer in two clay fills of different age resulted in different records.
 - The seasonally dependent suction cycles were recorded at the clay fill of the Bílina mine. The ground water table position in approximately 17 metres did not influence the shallow subsurface suctions. In late summer, the limit of measurable matric suction of 90 kPa was reached in the shallower gauges. Lower suctions were recorded in the deeper gauges under slope than in the same depths in locations L01 and L02. This was presumably caused by the surface and shallow subsurface runoff. Under

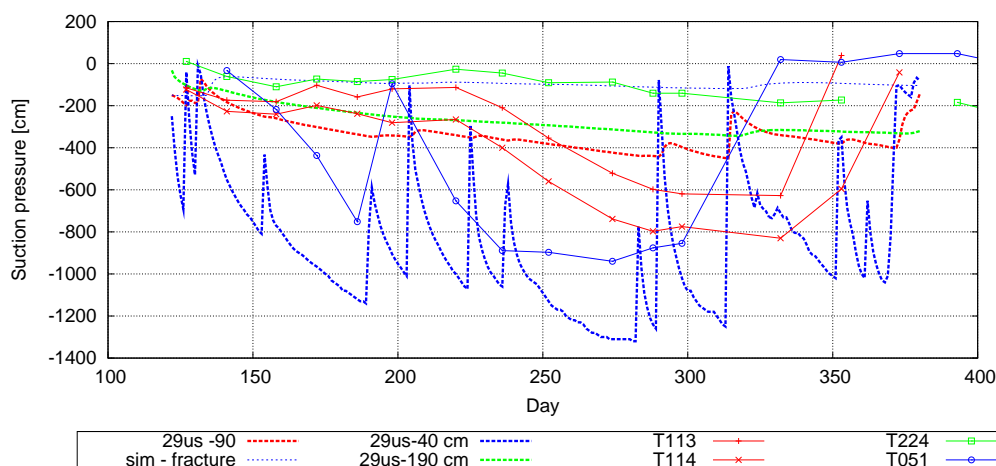


Figure 4.14: Dual permeability model for one season under slope (L03) (day 124 refers to 1.5.2004 and 442 to 20.2.2005, respectively).

slope (L03) the water infiltrates which results in lower matric suctions recorded in deeper gauges.

- At the site of the D8 motorway construction near Ústí nad Labem at the aged clay fill of former 5. květen mine, no or negligible suctions has been recorded through the whole monitored period. At this site the high position of the ground water table of about 2 m below the ground surface was recorded. The fill was nearly or fully saturated.
- The hydraulic conductivity of the lumpy specimens tested in triaxial cell, shows a clear stress dependence. The drop of about 5 orders of magnitude from 10^{-6} m/s to 10^{-11} m/s was measured during the increase of the effective confining stress from 30 to 500 kPa.
- The S1.D.Dual program (Vogel, 2000) was used to model of matric suction in the reclaimed clay fill of the Bílina mine. The hydraulic conductivity and the soil water retention curves, determined on the undisturbed and lumpy specimens, respectively, were used for hydraulic characteristics of matrix and preferential flow domains.
 - The vertical profiles of matric suction were modelled using the inverse analysis and fitting to the monitored data.
 - Different volumetric fractions of the matrix and preferential flow domains in different depths resulted in a better match with measured data.
 - High matric suction reaching 1.8 MPa in late summer and suction oscillations were modelled in the shallow depths.

- The simulated high matric suctions in shallow depths and frequent suction oscillations, even in the reclaimed clay fills, emphasize the importance of the wetting-drying cycles on the degradation of the structure of clay fills. However in freshly filled clay fills the suction can reach higher values and the suction oscillations can be more pronounced, since the surface of the fills is exposed.

Chapter 5

Properties of lumps

The properties of lumps sampled from the Bilina mine are investigated. The uniaxial compressive strength and matric suction acting in the lumps are determined. The response of unconfined lumps to wetting and wetting-drying cycles is investigated.

5.1 Strength of lumps

The strength of clayey lumps, which compose the granular structure of the fills, is influenced by matric suction, overconsolidation and possible diagenetic processes, see [68].

A rough uniaxial compressive strength σ_c was measured on irregular clayey lumps of similar dimensions ranging in diameter between 15 and 30 mm and having a height/diameter ratio of approximately two (Fig. 5.1). The measured values of σ_c between 0.53 and 1.06 MPa (Fig. 5.2) are below typical uniaxial compressive strengths of weak rocks and demonstrate the soil nature of the overburden layers.

5.1.1 Matric suction

The excavated overburden is deposited into fills in a "dry" way at its natural water content. Therefore, macrovoids between the lumps are typically air-dry while the intragranular voids in the lumps maintain their natural water content of about $w = 34\%$ (close to full saturation). The degree of saturation was determined from the measured volume, mass and water content of several clay lumps. The volume of lumps was measured by weighting under water. The resulting S_r of the lumps ranged between 0.98-1.00. Considering the size of macrovoids between the lumps typically $d_{15}/5 = 0.2$ mm (as applied in the design of granular filters), a crude estimation of



Figure 5.1: Testing of uniaxial strength of clay lumps.

the capillary pressure for a perfectly spherical menisci yields a maximum value of 1.5 kPa. With respect to the geostatic pressure and the shear strength of the lumpy clay fill material, this value can be neglected at least at lower stresses. However, a significant matric suction is present inside clayey lumps due to their excavation from the original depth. An indirect measure of the matric suction magnitude is the uniaxial compressive strength of the clay lumps, see Fig. 5.2.

Measuring of matric suction of lumps

The matric suction was determined using the filter paper method (according to ASTM [11]) on the lumps sampled about 4 – 5 months after landfilling. The filled material had the form of the lumpy soil with blocks up to 0.3 m and finer mass. The matric suction was determined on the lumps of about 15 - 20 cm. Each lump was halved by a cut wire and the filter paper Whatmann nr. 42 was sandwiched between two protective filter papers inside the halved sample. The sample was properly wrapped into a protective plastic sheet and placed into an air tight container. The equilibrium times of 14 days were used to ensure a steady state between the moisture of the filter paper and matric suction of the lump. The results are summarized in Table 5.1. The estimated matric suction of the lumps varies between 2440 kPa and

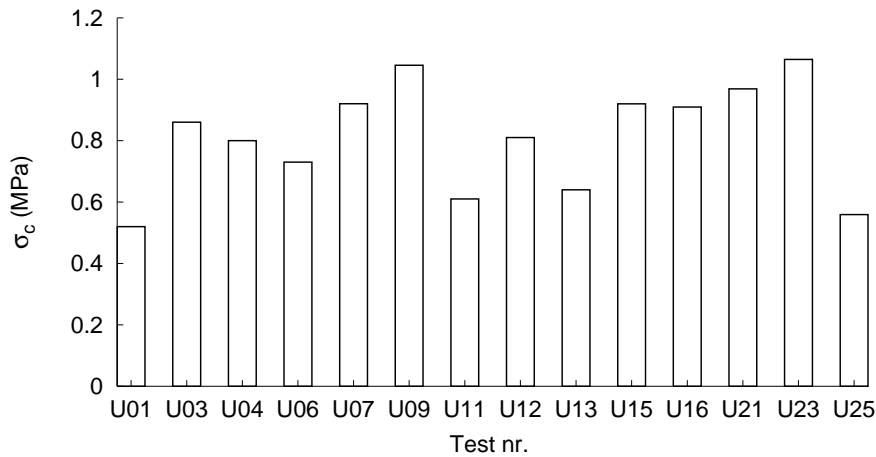


Figure 5.2: Uniaxial compressive strengths of the clay lumps.

1267 kPa by the moisture content variations of 1.6 %. This could be attributed to the overconsolidated nature of the lumps having small intragranular void ratio.

Table 5.1: Water content of the lumps and matric suction determined by filter paper method.

Water content w in %	Matric suction according to ASTM s in kPa
28.2	2440
29.5	1470
29.8	1267
29.3	1776

Assuming that only matric suction influences the shear strength of the lumps (any diagenetic bonding or extra strength due to overconsolidation will be neglected), the uniaxial compressive strength of these lumps can be roughly estimated. Assuming further the validity of the Bishop's relation for the effective stresses for unsaturated soils:

$$\sigma_{eff} = \sigma_{tot} - u_a - \chi(u_a - u_w) \quad (5.1)$$

where σ_{eff} and σ_{tot} are effective and total stresses, u_a is pore air pressure, $(u_a - u_w)$ matric suction and χ a parameter dependig on the degree of saturation. For example Khallili et al. [74, 73] presented a method for the determination of χ from the water retention curve. The soil water characteristic curve of the lump was not determined, the Bishop's parameter χ was assumed to be equal to the degree of saturation. This assumption is applicable because the lumps were nearly or fully saturated. The effective friction angle of the reconstituted soil of $\varphi' = 27^\circ$ was determined,

see Sect. 6. Further during the undrained uniaxial compression the pore water pressures were assumed equal to the matric suction and pore air pressure equal to the atmospheric pressure. Following from the shear strength envelope:

$$\sigma'_1 = \sigma'_3 \frac{1 + \sin\varphi'}{1 - \sin\varphi'} = (u_a - u_w) \frac{1 + \sin\varphi'}{1 - \sin\varphi'} \quad (5.2)$$

The uniaxial compressive strength is then:

$$\sigma_c = \sigma_1 - \sigma_3 = \sigma'_1 - \sigma'_3 \quad (5.3)$$

This very crude estimation results in the compressive strength of about $\sigma_c = 2 - 4$ MPa assuming the measured matric suctions of $s = u_a - u_w = 1.2 - 2.4$ MPa. The measured uniaxial strength of $\sigma_c = 0.6 - 1$ MPa is significantly lower. The slightly higher water content of the tested lumps and therefore lower suction acting in microvoids in the lumps could be the explanation. Equivalently, the matric suctions of $(u_a - u_w) = 0.25 - 0.4$ MPa corresponding to the measured unconfined compressive strength can be very roughly estimated. It should be noted that any build-up of pore water pressures during undrained shearing was not taken into account. Furthermore the anisotropy of the clay lumps can also influence the shear strength. The majority of the lumps are longer in one direction due to their disintegration along the sedimentary fabric. The uniaxial strength of the lumps compressed along their longer axis will be lower than of the lumps compressed in the direction perpendicular to this fabric.

5.1.2 Overconsolidation of the lumps

Clayey lumps deposited into the clay fills are overconsolidated not only as a result of their exploitation by mining from large depths, but also due to a considerable denudation of the overburden in the geological history, see [68].

The amount of the denudation can be estimated in the laboratory by comparing the void ratio in situ with results of one-dimensional compression tests on specimens prepared from slurries. Oedometer tests were carried out on samples prepared by reconstituting them from an initially granular material. The investigated clay fill material originates from the mine cut of the excavator k-74 from depths of about 25-40 m (personal communication by K. Mach, the main geologist of Bílina mine). Considering the unit weight of the undisturbed almost saturated lumps, $\gamma = 19.0$ kN/cm³, the vertical pressure in the depth of 40 m (above the water table) should be about 760 kPa. The normal consolidation lines (NCL) of reconstituted specimens yielded

the void ratios between 1.01 and 1.08 at the vertical pressure of 760 kPa (see the full horizontal lines in Fig. 5.3). This value of void ratio is significantly higher than the natural void ratio $e_i = 0.77$ of the clay lumps (see the dashed horizontal line in Fig. 5.3), thus indicating a considerable amount of overconsolidation of the original clay.

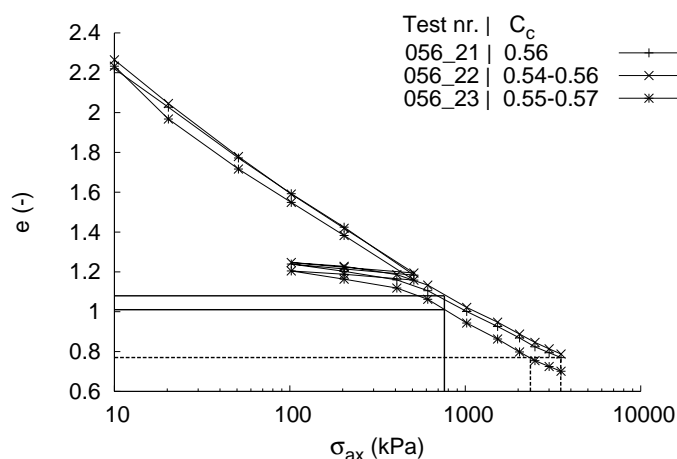


Figure 5.3: Normal compression lines (vertical pressure σ_{ax} versus void ratio e) of reconstituted soil compared with natural void ratio of the lumps. Dashed horizontal line depicts natural intragranular void ratio of undisturbed lumps, full horizontal lines depict void ratios of normally consolidated soil from a depth of 40 m.

In order to reach the void ratio e_i along the NCL, the vertical pressure should exceed 2500 kPa (lower curve) or 3500 kPa (upper curve), respectively. These values would correspond to a depth of the excavated soil from about 132 m to 185 m (considering only the natural unit weight of the soil without any water table). Taking into account the sedimentation in the sedimentary basin under the water table, the saturated unit weight of the submerged soil would be around $\gamma_{sat} = 9.0 \text{ kN/cm}^3$ and the thickness of the clayey overburden would reach about 277 to 389 m. The necessary thickness of the denudated layer between 237 and 349 m (considering also the excavation depth of 40 m) is well in agreement with Hurník (1978) who reported the denudation thickness between 70 to 300 m (see Sect. 3.1).

It should be noted that no ageing effects like creep or diagenetic strengthening, which decrease the volume of voids with time and shift the NCL vertically to smaller void ratios, were taken into account. It is possible that these effects are counterbalanced by swelling due to unloading. Moreover, considering the natural water content between 30 and 40%, the coefficient of secondary compression $C_{\alpha\varepsilon}$, after [97], can be estimated as 0.004 at maximum. This would produce an additional compression of

$$\Delta e = C_{\alpha\epsilon} \log\left(\frac{t}{t_0}\right) (1 + e) = 0.079 \quad (5.4)$$

(with $e_0 = 1.0$ and $t_0 = 1$ day) in 20 million years (the age of Miocene Libkovické layers after [122]), which could not explain the present intragranular void ratio of natural clay.

5.1.3 Diagenetic processes

The ageing of sedimentary soils results in diagenetic processes, mentioned e.g. by [68], which add a strength component to the lumpy soil. This component is independent of water content and resists water saturation. Naturally wet lumps inserted into water remain intact with time, which can confirm a kind of diagenetic strengthening (see Sect. 5.2. It will be discussed in Chapter 6 that a certain manifestation of this effect persists even after samples are reconstituted.

5.2 Changes of lumps due to weathering

As the surface of fills is exposed to climate conditions, weathering of shallow sub-surface layers contributes to changes in the clay fill structure. Due to the open and interconnected intergranular voids, the fill is highly permeable in the first tens of metres, and surface water can enter easily into clay fills. Herštus [65] reported the depth of the layer permeable to air to be about 10 – 20 metres for the Merkur, Libouš and Stodola mines. However the thickness of this highly permeable layer can vary depending on the stiffness and the void ratio of the filled lumps and the degree of closure of the intergranular porosity (see Sect. 6.5,4).

The contact between lumps and water causes weakening. There is a drop in the matric suction that is present in the intragranular voids inside lumps during dry-way filling, or the matric suction even vanishes, and the lumps soften. The oscillations of matric suction between 1.8 MPa and zero were simulated even in reclaimed clay fills (see Sect. 4.4). In addition, the matric suction that developed on the contacts between the lumps as a consequence of the load (contact bonding after Feda [42]), also vanishes. This causes more rapid closure of the intergranular porosity and a collapsible response of the filled material if there is a sufficient overburden load. The collapse behaviour of clay fills is dealt with in Section 7.

If the lumps are exposed to wetting-drying cycles, they disintegrate. The lumps split into smaller fragments that can fill the larger voids between them. However, more

open preferential paths for water to enter the filled material can be formed due to the opening of microcracks inside the lumps and their disintegration. The shallower parts can therefore remain more permeable in the early stages of disintegration of the lumps. According to Herštus [65], in one year a layer of very loose disaggregated small fragments a few tens of cm in thickness is formed on the fill surface that is exposed to weathering conditions. However, Najser [100] and Charles and Watts [27] reported the formation of a more compact layer of less permeable clay due to the weathering effect in the case of aged fills. Větrovský [132] also observed a layer of homogenized clay of plastic consistency down to a depth of 4 metres in an aged clay fill. Probably both effects could be observed, however at different time scales. First disintegration of lumps takes place, which can form more permeable shallow subsurface layer. Subsequently macrovoids are filled with disintegrated fragments, which further deform in a plastic manner and a layer of soft more homogenized and less permeable clay could be formed at later stages. The layers of homogenized clay of plastic consistence were observed also by Větrovský [132] up to the depths of 4 m. A simple test confirmed the important influence of wetting-drying cycles on the structure of a single unconfined lump. Two similar hard lumps with initial water content 30% and degree of saturation of $S_r = 1$ were selected. The two lumps were

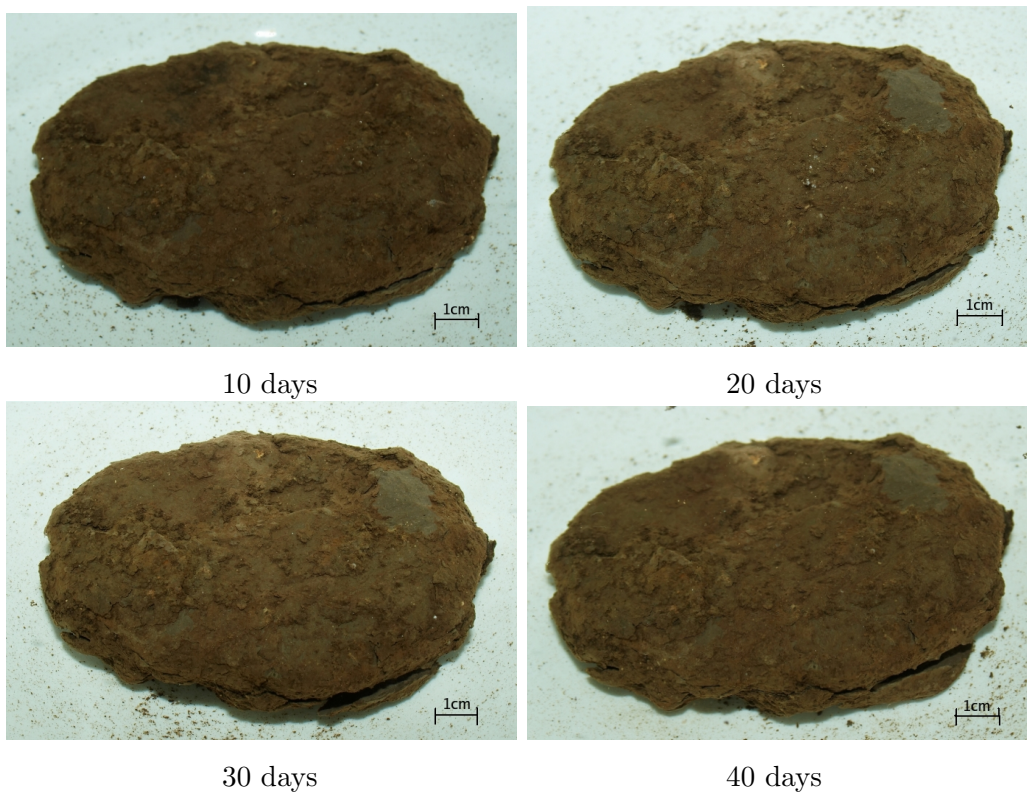


Figure 5.4: Submerged lump, no application of wetting-drying cycles.

5.2. CHANGES OF LUMPS DUE TO WEATHERING

submerged in a dish with tap water under room conditions ($22\pm 2^\circ\text{C}$). The first lump remained submerged for the whole testing period of approximately forty days (Fig. 5.4), while the second lump was exposed to 4 wetting-drying cycles (Fig. 5.4), while the second lump was exposed to 4 wetting-drying cycles (Fig. 5.5). The water in the dish was simply left to evaporate, and each cycle lasted approximately ten days. The first lump remained almost intact throughout the test, and a single wetting event did not affect its shape (see Fig. 5.4). On the other hand, already after two wetting-drying cycles, disintegration of the second lump was visible (see Fig. 5.5) and the first microcracks appeared. The ongoing disintegration during the third and fourth wetting-drying event caused the lump to separate into smaller angular platy fragments (see Fig. 5.5- bottom pictures).

The gradient between the matric suction in the natural moist lump and water under atmospheric pressure was not sufficient to break the lump structure. The matric

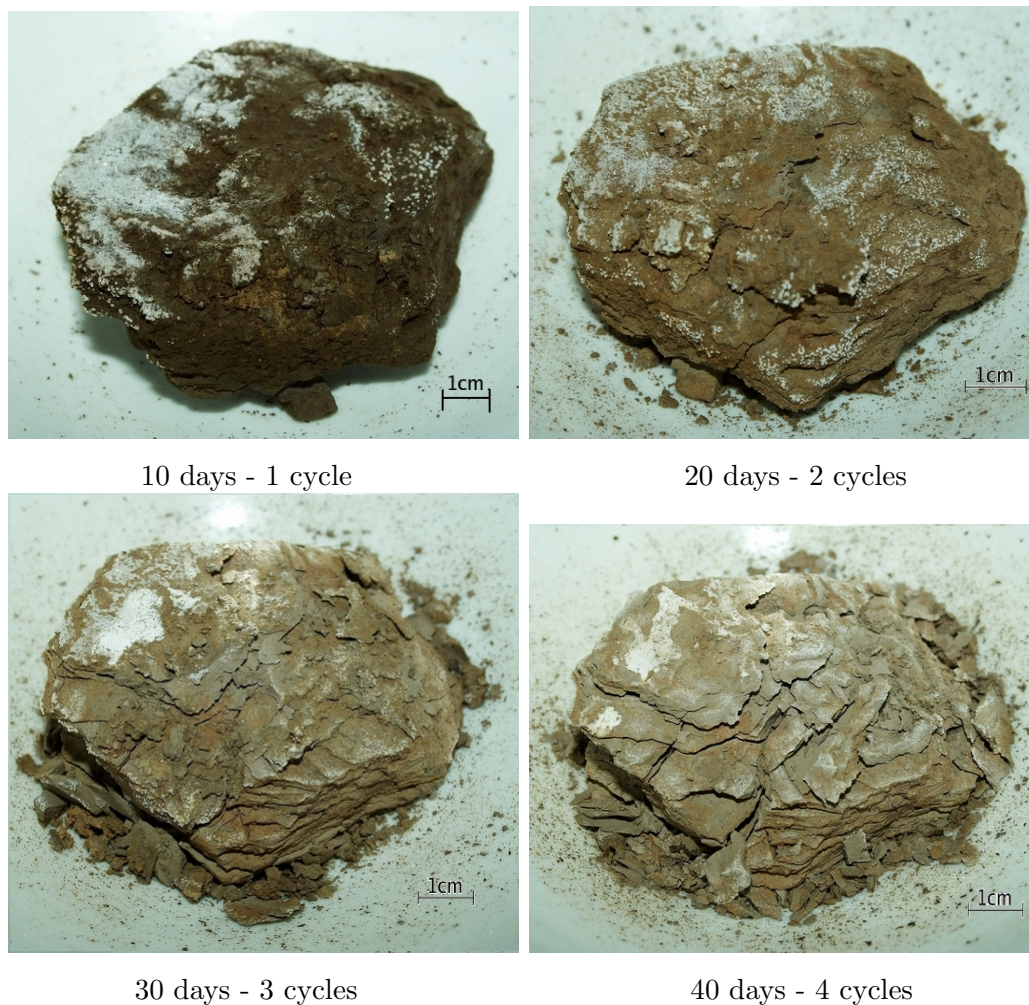


Figure 5.5: Lump exposed to four wetting drying cycles.

suction of similar lumps at their natural water content of about 30% which were close to saturation or saturated was determined using the filter paper method, see Sect. 5.1.1. The values varied between 1.2 and 2.4 MPa. The lump was fully saturated at its natural water content prior to submersion. Therefore, submersion just caused the gradient between the two pressures to equilibrate. The matric suction present in the lump prior to submersion vanished, without causing any flow between the inner pores of the lump and the surrounding water. This effect can furthermore manifest a certain diagenetic strengthening, which persists pronounced hydraulic gradients.

Drying to a water content corresponding to the room relative humidity (approx. 4-6%) and subsequent wetting caused a gradient between the matric suction in the lump and the water under atmospheric pressure surrounding the lump that was sufficient to break the lump structure. The matric suction of the air-dried lump was not determined, because the filter paper method is not applicable at such low water contents¹. However, it can be expected that the matric suction acting in the intragranular voids of the air-dried lump is high, because the water content of the lump is surely much lower than the water content at the air entry value for this soil, and probably also lower than the residual water content. Therefore the air-dried lump is partly saturated with a significant volume of air in the pores. A very rough estimate can be made from the Van Genuchten approximation of the water retention curve, determined on an undisturbed soil sampled in the Bílina mine, see Sect. 4.3.2. The model suggests a residual water content for this soil at suction about $s = 10^{10}$ MPa. These suctions are rather unrealistic for an air-dried lump, due the disconnected water-phase. However, suctions about one order higher than at the natural water content can be present.

This simple test was repeated using a lump that was first left to dry in room conditions and was then flooded. Fig. 5.6 depicts the time sequence of the disintegration of the lump. Rapid disintegration of the lump was observed. In addition, the whole process was dynamic, and the swelling of the lump was visible to the naked eye. Air bubbles escaping from forming minicracks were clearly visible. Single platy fragments developed due to the propagation of minicracks and were further separated from each other. This indicates that the soil was dried below the air entry value and a certain amount of air was entrapped in the pores.

If the material dries out to a water content below its air entry value, the process of

¹This method supposes a relation between the matric suction in soil and the water content of the filter paper after ensuring a hydraulic connectivity between the filter paper and soil. According to ASTM [11] adequate physical contact between the filter paper and soil is not always possible, when the soil is not sufficiently moist.

structure disintegration is more rapid and more significant. A single wetting, even applied to the lumps at their natural water content, is not sufficient to disintegrate them. Disintegration of the lumps apparently occurs only after cyclic changes in matric suction.

Disintegration after wetting-drying cycles is clearly visible in the field. Freshly filled soil from the Bílina mine is displayed in the Fig. 5.7 – top. Figs. 5.7 - bottom shows disintegrated lumps on the surface of the clay fill, which had been subjected to moisture oscillations for about 6 months. Fig. 5.8 – top shows the aged clay fill soil of about 10 years after filling from the Bílina mine and the soil about 40 years from filling from the 5. květen mine is displayed in Fig. 5.8 – bottom. This soil was excavated from a depth of about 1 – 2 metres. Some relicts of stiffer lumps are still clearly visible (darker brown), but they flow in the homogenized - weathered and strained - soft clayey mass (lighter brown). Oscillations of matric suction, and suctions reaching 1.8 MPa, in shallow depths of the reclaimed clay fill of the Bílina mine, were also simulated numerically (see Sect. 4.4).

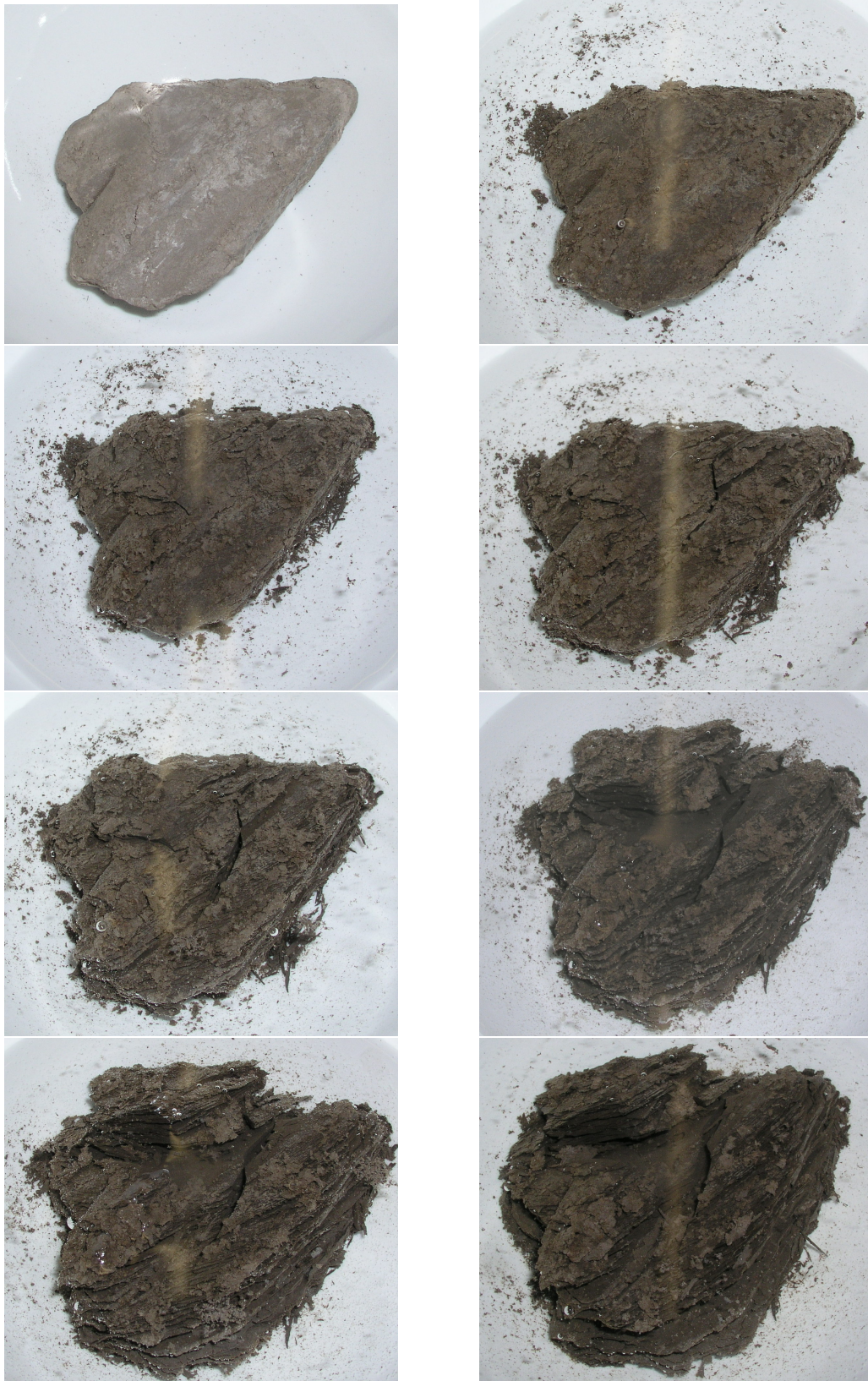


Figure 5.6: Air dry lump exposed to flooding, time sequence of: air dry, 30 sec, 1 min, 5min, 15 min, 30 min, 1 hour, 24 hours after flooding.



Figure 5.7: Clay fill just after filling (top); Surface of the clay fill exposed about 6 months (bottom), both from Břlina mine.



Figure 5.8: Surface of the clay fill about 10 years after filling (Bílina mine, top); Excavated soil from about 1 – 2 m depth, aged about 40 years (former 5. květen mine, bottom).

5.3 Summary

- The shear strength of the single lumps is related to the matric suction, overconsolidation and possible diagenetic bonding. The unconfined shear strength of the saturated lumps at the water contents of 34% varied between 0.8 – 1 MPa.
- The matric suction of the lumps at their fully or nearly saturated state at the water contents of about 30% was measured using the filter paper technique. It reached 1.2 – 2.4 MPa.
- A single wetting event applied to a lump at its natural water content was found to be insufficient to disintegrate the lump, which remains intact. This confirms a kind of diagenetic strengthening. Only the application of wetting-drying cycles causes disintegration of the lump into smaller angular fragments.
- Flooding the initially air-dried lump also leads to its disintegration. This process is rapid as a consequence of the high hydraulic gradient between the matric suction in the lump and the water under atmospheric pressure surrounding the lump. The water content of such lump is far below the air entry value, which causes certain amount of air to be entrapped in the microvoids. Air escaping from the propagating minicracks was visible to the naked eye. Matric suction of the air-dried lump could not be measured using filter paper technique, due to the low water content which disable the proper contact between the soil and filter paper.
- Structural changes due to weathering fill of the intergranular voids, due to softening and disintegration of the lumps. A structure with smaller fragments is formed first, and a more or less homogenized clayey mass at later stages. The void ratio of this soil is higher than the void ratio of the original lump, but it is lower than the void ratio of the freshly filled soil with the open structure. The depth of the layer influenced by the weathering can vary, but it does not go beyond the first metres.

Chapter 6

Structure transitions of clay fills

The mechanical behaviour of the lumpy clay fill material from Břilina mine was tested in the direct shear box and oedometer for different specimen sizes and lump size distributions (see Fig. 3.7). The effect of water saturation and soil reconstituting were also considered in the testing programme.

6.1 Shear strength

The shear strength was investigated in a large shear box with a rectangular cross-section 300×300 mm and a height approximately 150 mm. The lumpy clay with its original grain size distribution was used (Fig. 3.6). For comparison, a sieved material with a grain size less than 5 mm was tested in a standard shear box with a circular cross section (diameter 65 mm, height 25 mm). The ratio τ/σ gives the direct measure of the shear strength of the material at the shear plane. A persisting discussion on different ways of the evaluation of the direct shear test results, see e.g. [138, 88, 7], is not considered here. Possible errors which arise due to the different direct shear box construction when testing soft soils is analyzed in [79].

A conventional interpretation of the results (utilizing a best fit curve) from the large shear box yielded a linear limit stress envelope which can be defined by $\varphi = 29.8^\circ$ and $c = 25$ kPa (Fig. 6.1). Alternatively, assuming $c = 0$, one gets $\varphi = 41.7^\circ$ in the low pressure range and $\varphi = 33.6^\circ$ in the high pressure range (Fig. 6.2). Thus for $c=0$ the limit stress envelope can be assumed as curved and its linear approximation (Fig. 6.1) can overestimate the shear strength in the range of very low stresses and underestimate the shear strength at high stresses, respectively, see also [39]. It was assumed, for the calculations of φ , that the lumped clay behaves in a drained manner and there are no significant pore water pressures inside the macrovoids.

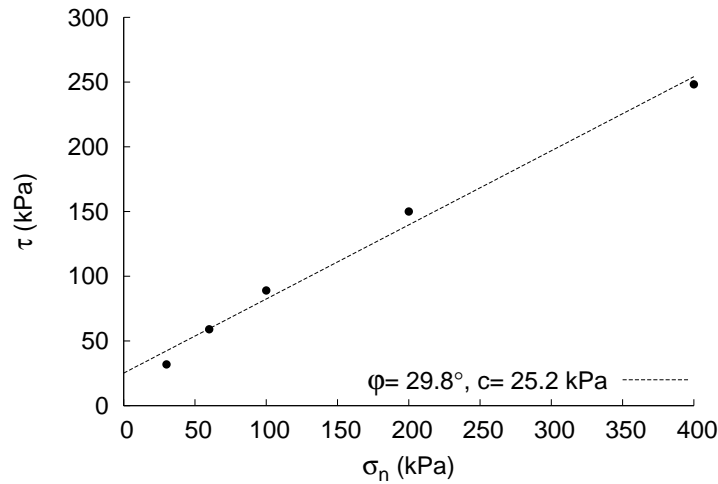


Figure 6.1: Conventional interpretation of shear strength (best fit straight line), τ : maximum shear stress, σ_n : normal stress.

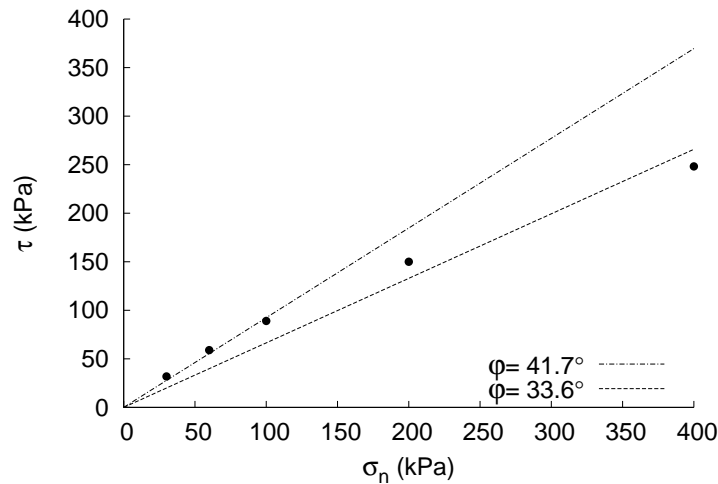


Figure 6.2: Conventional interpretation of shear strength assuming $c = 0$ kPa, τ : maximum shear stress, σ_n : normal stress.

The assumption of $c = 0$ seems to be meaningful: the clay lumps do not have any bonds between each other and can behave as a granular material, see the angle of repose in Fig. 6.3, at least up to a certain limit pressure. For $c = 0$ one can conveniently express the shear strength as a stress-dependent friction angle, see Fig. 6.4. A strong influence of the stress level can be observed. Moreover, a comparison between the finer (sieved) and coarse materials reveals a clear size effect: smaller specimens yield a higher shear strength.

It cannot be expected that the granular nature of the lumpy clay persists to arbitrary pressures. After dismantling the shear box after the tests, a granular nature of the



Figure 6.3: The angle of repose of the lumpy clay fill soil.

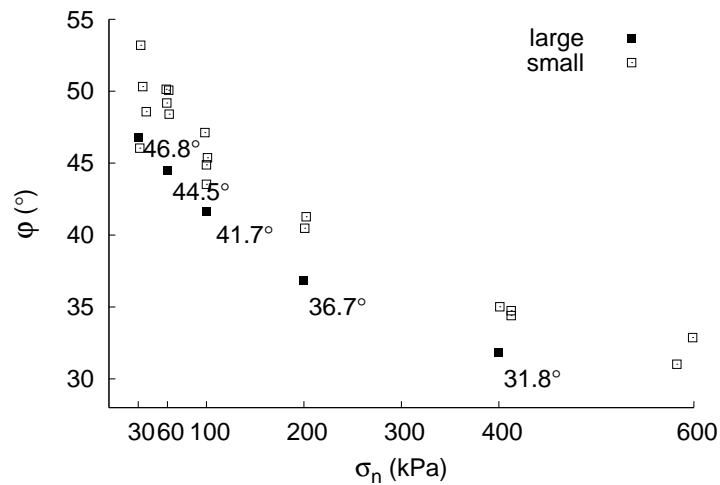


Figure 6.4: Comparison of friction angle φ for small and large shear box specimens at different normal stresses σ_n .

material remained clearly visible at lower stresses but a polished shear surface was created at higher stresses (approx. for $\sigma_n > 100$ kPa), see Fig. 6.5.

6.1.1 Water saturation

When flooded before shearing, a considerably smaller shear strength was measured (Fig. 6.6). This points out to changes in the strength of macrograins and/or to a decrease of the shear resistance at the contacts between clay lumps.

The clay lumps are practically fully saturated and there acts a matric suction in



Figure 6.5: Clay fill material before shearing (left). A polished shear surface after shearing at high normal stresses (right).

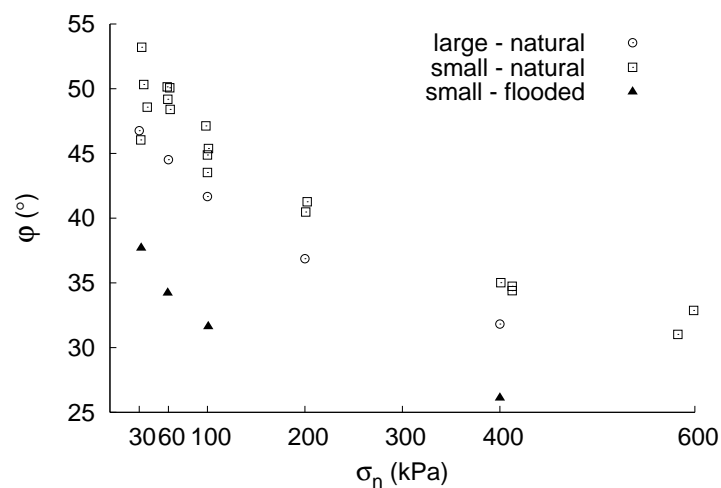


Figure 6.6: Influence of specimen flooding on friction angle φ at different normal stresses σ_n .

them. This negative water pressure is related to the mean effective stress in soil before removing the overburden by excavation.

The dissipation of negative pore water pressures inside the clay lumps in their natural state is prevented by the equilibrium conditions at the interface between lump surfaces and vapour in the air-filled macrovoids. When saturating a naturally wet specimen (filling the macrovoids with water), suction at the surface of clay lumps is destroyed, effective pressure in the lumps decreases and the lumps become softer.

[86] and [110, 111] described the influence of suction in the dredged clay lumps draining under fully saturated conditions when placed in sea for land-reclamation

purposes in Singapore. They showed that the matric suction in lumps, which is proportional to the mean effective pressure of overburden, dissipates when putting the lumps into water. Consequently, high effective stresses decrease and so the strength of the lumps. They become softer and one does not need a large energy for closing macrovoids. [86] stated that the consolidation process of the lumpy fill is significantly faster than that of homogeneous clay but slow down with increasing loading pressure, as the voids close up.

6.1.2 Soil reconstituting

Reconstituted samples were prepared from an initially lumpy structured soil by hand mixing at the water content of approximately two times w_L . Stiffer grains were destroyed by squeezing the soil through 0.5 mm sieve. Afterwards the soil was left to homogenize for at least 24 hours in an air tight container and afterwards properly mixed once more.

Soil reconstituting usually destroys diagenetic bonds in the macrograins and changes the mode of shear strength mobilization from a granular to a compact clay shearing. At low normal pressures, an extra strength component characteristic for a granular clay fill material is not present any more. The mobilized friction angle decreases markedly with increasing normal stress (Fig. 6.7).

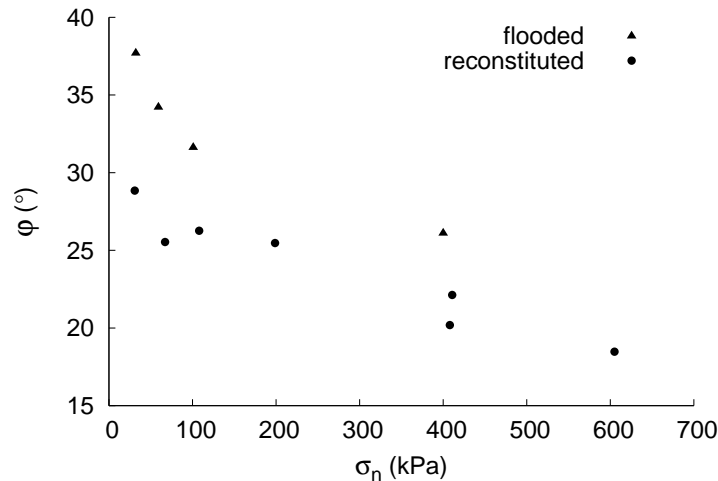


Figure 6.7: Influence of soil reconstituting on friction angle φ at different normal stresses σ_n .

One can expect that particle bonds in the lumps resistant to water flooding (diagenetic strengthening) vanish by remoulding. Shear strength of the reconstituted soil represented by friction angle φ should be independent on the stress level and with-

out a cohesion component. Nevertheless, one can still observe a stress-dependence of the friction angle φ producing a curved limit strength envelope. This implies a possible structural component persisting from the original clay macrograins which may come from diagenetically bonded clay aggregates at a mesoscale. The size of these aggregates remains unknown.

6.2 Compressibility

The behaviour in compression is crucial for the prediction of settlements. A large variety of oedometer rings, having diameters between 50 mm and 600 mm, has been used. The measured compression curves for different specimen sizes are shown in Fig. 6.8. The depicted total void ratio includes both, intra- and intergranular voids. The consolidation times for each load step was kept constant with $\Delta t = 1$ hour for the natural lumpy clay and $\Delta t = 24$ hours for saturated and reconstituted specimens respectively.

There is a significant scatter in the results between large scale and standard tests, especially comparing C_c values from the quasi-linear parts of the compression curves for the vertical pressures below 1500 kPa (C_{c1} in Table 6.2). A tendency for lower compressibility of larger specimens may be recognized (tests no. 056_13 to 056_16 in Fig. 6.8 and Table 6.2). This is a contradiction to the observed shear behaviour (Fig. 6.4) showing a higher shear strength for smaller specimens. However, larger specimens have lower initial void ratios due to a higher non-uniformity of the grain size distribution, which can produce a lower compressibility as well. Another explanation could be in the undrained response of the almost fully saturated clay lumps. They behave stiff in compression but positive excess of pore water pressures in the lumps induced by shearing is responsible for a reduction of an undrained shear strength. Decreasing the lump size, drained behaviour (i.e. zero excess pore water pressures) of the lumps with a higher internal shear strength prevails in the course of deformation.

To get an equal initial void ratio of about $e_{tot} = 1.8$ for samples with the original grain size and with the reduced maximum grain size distribution, the latter specimens had to be compacted (marked with c in Fig. 6.8). Assuming the same intragranular void ratios for all samples implies differences in the intergranular void ratios caused by removing greater lumps and thus changing the shape of the grain size distribution curve (see Fig. 3.7).

It may be noticed that at vertical pressures above 1500 kPa the curvature of the compression curves changes and so the character of the soil (see C_{c2} in Table 6.2

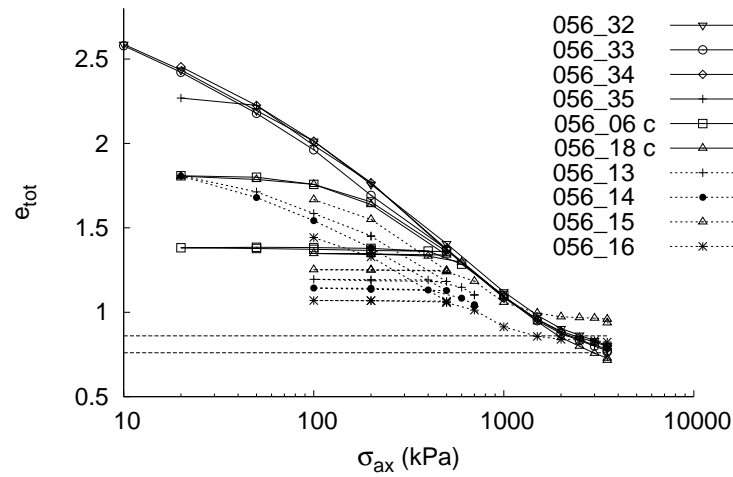


Figure 6.8: Influence of specimen size on compressibility. Total void ratio e_{tot} versus axial stress σ_{ax} .

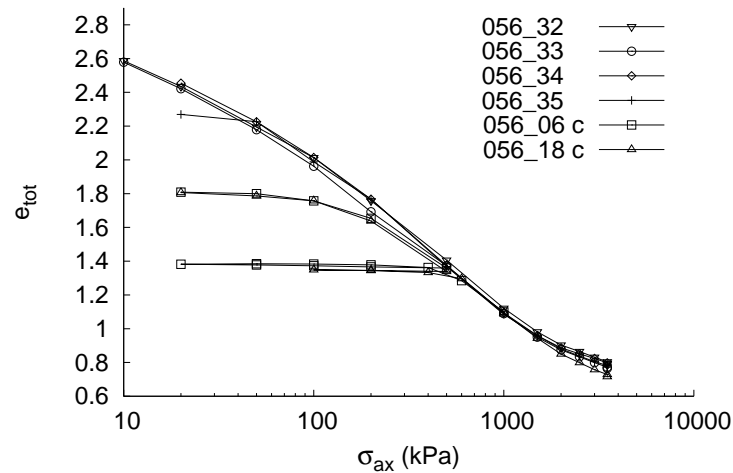


Figure 6.9: Compressibility of the specimens with reduced grain size. Total void ratio e_{tot} versus axial stress σ_{ax} .

determined at axial stresses $\sigma_{ax} > 1500$ kPa and Fig. 6.9). It is assumed that this pressure value is a threshold for closing the macrovoids in one-dimensional compression, which is well in agreement with [65], who reported the depths of about 50 m and more for the closure of macrovoids in clay fills. Applying shear stresses higher than in K_0 state, this threshold can be reached at lower normal stresses. In the range of $\sigma_{ax} \geq 1500$ kPa, compressibility is controlled mainly by intragranular porosity. After the sample dismantling at the end of testing, no obvious granular structure was visible and the initially lumpy material (Fig. 6.10 left) seemed homogenized (Fig. 6.10 right). Two dotted horizontal lines in Fig. 6.8 denote the range

Table 6.1: Compression indexes of lumpy material

Test Nr.	Ring Diameter r (mm)	Grain Size Distribution	Compression Index $C_{c1}(-)$	Compression Index $C_{c2}(-)$
056_32	50	reduced	0.87	0.41
056_33	50	reduced	0.88	0.42
056_34	50	reduced	0.91	0.37
056_35	50	reduced	0.89	0.40
056_06	70	reduced	0.85	-
056_18	50	reduced	0.83	0.53
056_13	600	natural	0.48	-
056_14	600	natural	0.62	-
056_15	250	natural	0.72	-
056_16	250	natural	0.59	-

of intragranular void ratios e_i inside the clay lumps suggesting to the destruction of macrovoids in the high pressure range. Additionally, a reduction of permeability at high stresses may produce some excess of pore water pressures contributing to a higher stiffness.

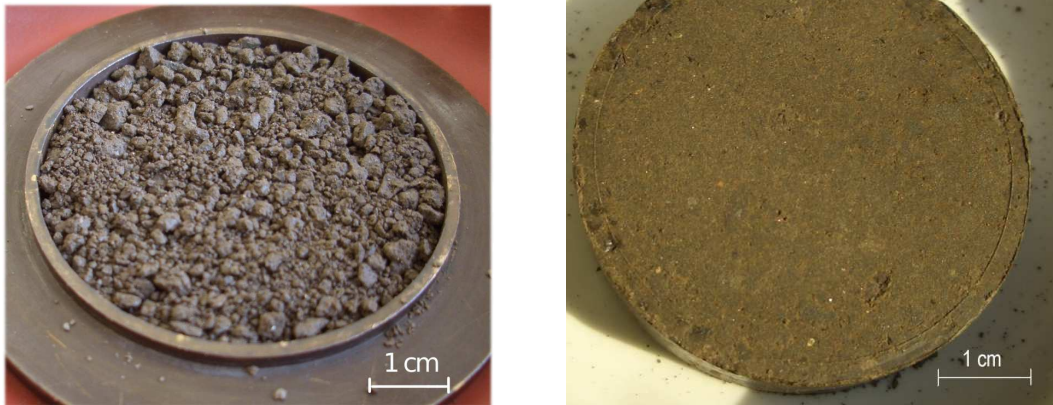


Figure 6.10: lumpy specimen with reduced grain size (left) and specimen after oedometric compression (right).

6.2.1 Water saturation

Water flooding of the naturally wet samples at low confining stress ($\sigma_{ax} = 10$ kPa) influences also the compressibility of the clay fill soil. The suction in lumps gradually

disappears and the lumps become softer. The difference between void ratios of natural and saturated (marked with s in Fig. 6.11) samples at the same vertical stress points out to the collapsible nature of the granular clay fill material. Collapse potential vanishes at vertical pressures of about $\sigma_{ax} = 1500$ kPa, where compression lines of wet and saturated samples come together. The collapse potential of this material is closer investigated in Sect. 7. It has been discussed in the preceding section that at this pressure limit the most macrovoids are already closed. Increasing the pressure further, compressibility is controlled by intragranular porosity only.

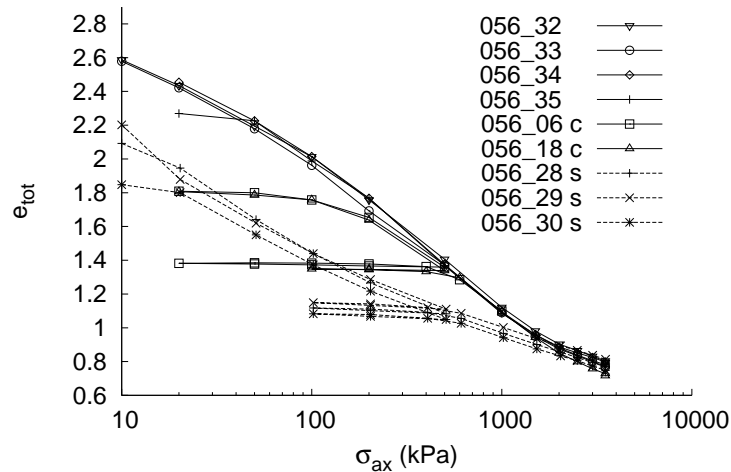


Figure 6.11: Influence of water flooding on compressibility (flooded specimens marked with s). Total void ratio e_{tot} versus axial stress σ_{ax} .

6.2.2 Soil reconstituting

Comparing the compressibility of the lumpy and reconstituted clay (Fig. 6.12), a lower C_c of the lumpy clay can be observed. Compression curves of both, reconstituted and lumpy clay come close together at the stress threshold for closing macrovoids, being consistent with the description given above. Horizontal lines (in Fig. 6.12) denote the range of intragranular void ratios inside the undisturbed lumps.

The NC lines of water saturated lumpy specimens (marked with s in Fig. 6.13) lie below the NC lines of reconstituted specimens (marked with r in Fig. 6.13). The destruction of the macrovoids by water saturation is accompanied by squeezing of clayey mass with a high water content into the macrovoids. The total porosity is given by both, a low intragranular porosity of the softened overconsolidated lumps and a high porosity of the paste, latter being close to the one of the reconstituted soil. Therefore, the flooded samples have a lower total porosity than the reconstituted specimens up to the mentioned threshold, where the lumpy structure vanishes.

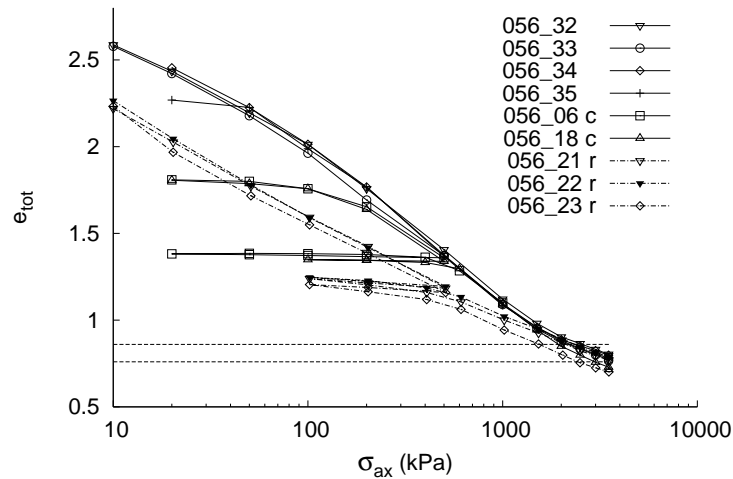


Figure 6.12: Influence of reconstituting on compressibility (reconstituted specimens 21, 22, 23). Total void ratio e_{tot} versus axial stress σ_{ax} .

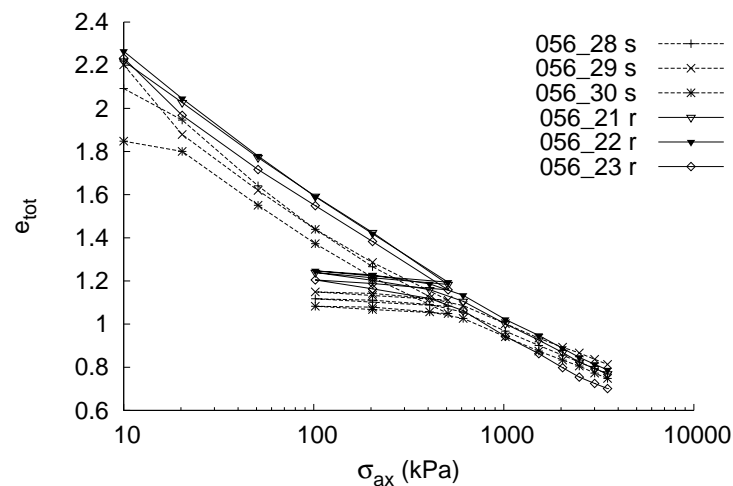


Figure 6.13: Comparison of compressibility of reconstituted (marked with r) and flooded specimens (marked with s). Total void ratio e_{tot} versus axial stress σ_{ax} .

6.3 Consolidation

The consolidation behaviour adds further difficulties to the material characterization. One can recognize a typical course of consolidation curves in a case of the flooded material (Fig. 6.14). However, the time behaviour of the lumpy clay at its natural water content is unusual. Due to the air connectivity of macrovoids, primary compression takes place fast, practically within 1 minute in the range of lower pressures, see Fig. 6.15. One can notice that for pressures larger than 1500 kPa the time component becomes more important. This confirms the previously stated

hypothesis on the closure of macrovoids at the threshold of 1500 kPa. A peculiar phenomena, however, appears for lower vertical stresses at larger time scale. A steady response with a negligible settlement of the specimen changes into an accelerated deformation after a certain time span, see Fig. 6.16. This kind of a secondary creep can be interpreted as a time-dependent collapse of relatively weak clay lumps. Some aspects of the secondary creep of granular materials as a diffusion process are described e.g. in [42, 44].

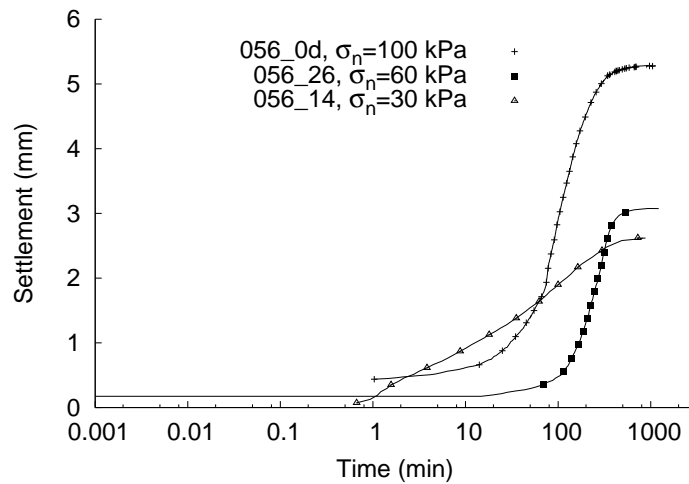


Figure 6.14: Time dependent settlement behaviour during consolidation of flooded specimens.

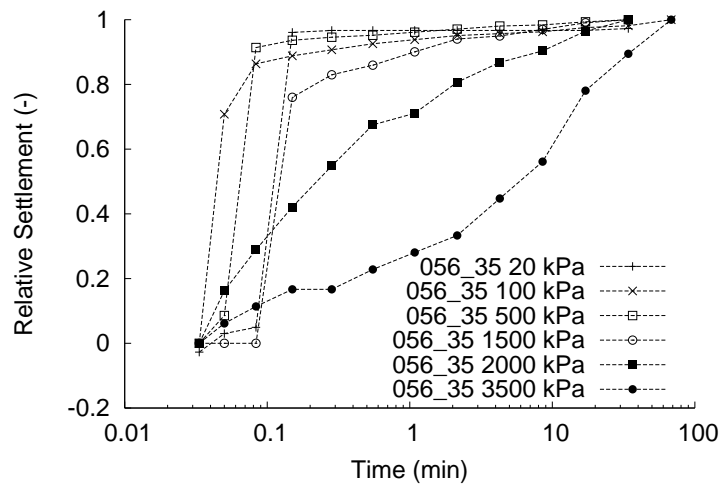


Figure 6.15: Time dependent relative settlement behaviour during consolidation of the specimens at natural water content.

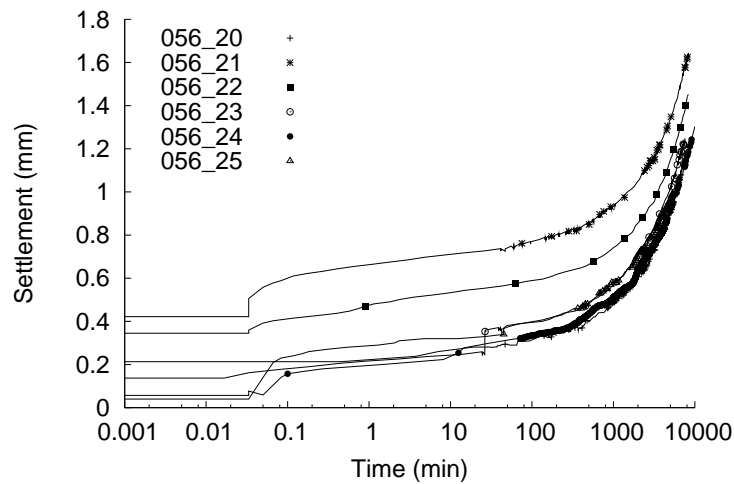


Figure 6.16: Time dependent settlement behaviour during consolidation of natural specimens at $\sigma_{ax} = 60$ kPa.

6.4 Rate-dependence

Already the consolidation behaviour shows complex time effects. Additional investigations of rate effects in the shear box apparatus revealed also a rate-dependent shear strength τ_{max} . An inspection of the results in Fig. 6.17 suggests a shear strength decrease with increasing shear velocity. Perhaps, the transition from the drained to the undrained behaviour of individual lumps could explain this effect. As the sample lumps are sheared at higher rates in undrained conditions, matric suction within the almost saturated lumps is affected by the build up of excess pore water pressures. Therefore, the strength of the lumps drops as well. The weaker response of the lumps is then responsible for the overall strength decrease of the soil.

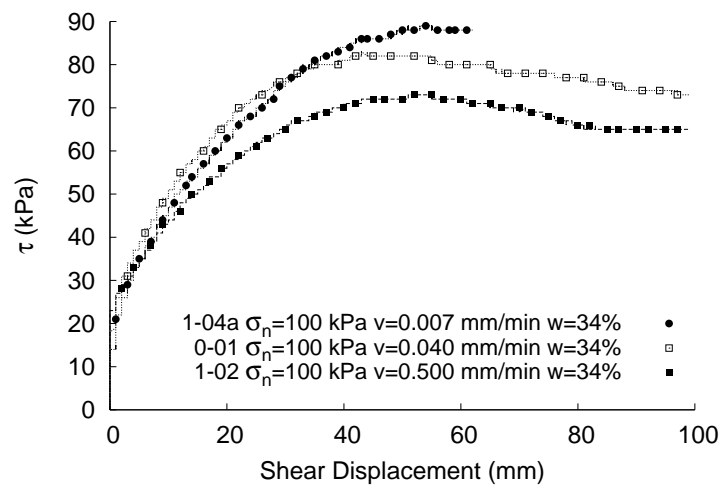


Figure 6.17: Influence of shear velocity on the shear strength τ in direct shear tests.

6.5 Compressibilities of scaled-down lumps from different sites

Oedometer tests on the soil with the scaled-down lump size distribution from the present study were compared with the data presented by Najser [100] and by Karpíšková [72]. Their oedometer tests were carried out on specimens consolidated in a mini-centrifuge.

The soils were sampled at different sites. Najser [100] used a soil from an aged clay fill of the former 5. květen mine, while the soil used by Karpíšková [72] and the soil in my study were sampled at the Pokrok clay fill of the Bílina mine (see Fig. 3.4). This soil was sampled in the form of lumps and some disaggregated finer mass about 4–5 years after filling.

Fig. 6.18 depicts the lump size distribution used in this study (crosses) and the distribution used by Najser [100] and by Karpíšková [72] (box symbols). Two techniques were used for preparing the lumpy soil. The lump size distribution of the specimens in [100] and [72] was prepared by crushing air-dried soil and sieving in order to obtain the same lump size distribution as in a freshly filled fill, which was presented by Dykast [36], taking into consideration the scale laws used in centrifuge modeling (details in [100]).

The lumpy specimens in my study were prepared by sieving at its natural content. Lumps below 5 mm were used for testing in standard oedometers. The lump size distribution of both samples, prepared by the different techniques, show a very similar trend and the lump size distribution at least for the used size of the clay lumps < 2.8 mm in [100, 72].

Oedometer tests were performed on the flooded lumps, prepared by these two techniques, and also on reconstituted specimens for both soils. The results are plotted in Fig. 6.19. The lumps in my study were flooded at their natural water content of 34% at low stresses (crosses in Fig. 6.19), and did not undergo any air-drying beforehand. The specimens of Najser [100] and Karpíšková [72] were air dried before centrifuge consolidation (box symbols and blue dashed line in Fig. 6.19).

A large difference between the void ratios of the two lumpy and reconstituted specimens was observed (see Fig. 6.19) comparing the compression lines of the soils sampled at the different sites. Blue denotes flooded lumps, and red denotes the reconstituted soil from the two sites in Fig. 6.19. A higher initial void ratio and also higher compressibility of the reconstituted soil was observed for the soil from the Bílina mine. On the other hand, the compression lines of the lumpy samples prepared by various techniques (sieving with natural water content and crushing

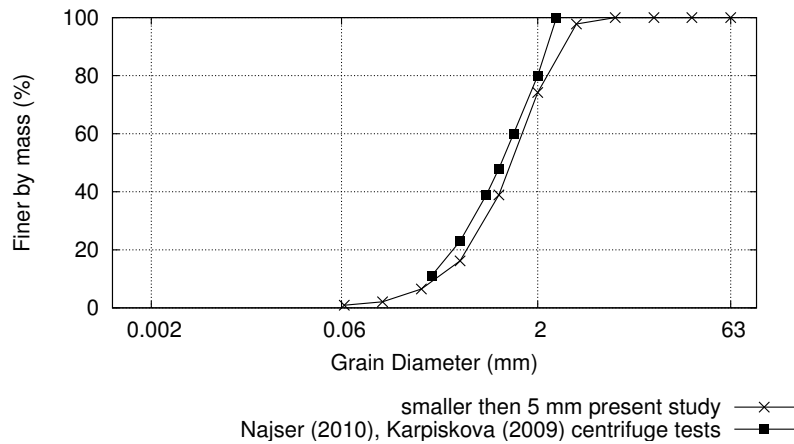


Figure 6.18: Grain size distributions of the model lumpy soils with reduced grain size distributions.

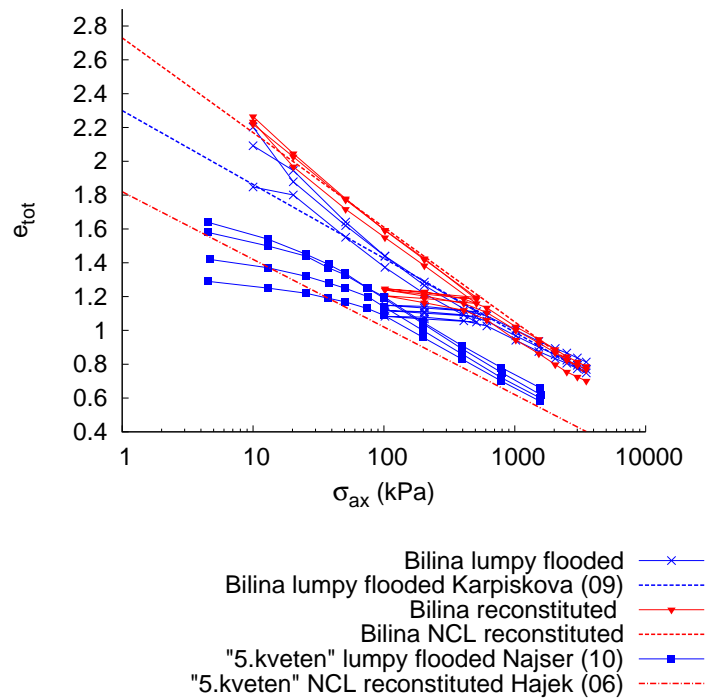


Figure 6.19: Comparison of compressibilities of flooded lumpy and reconstituted specimens from sites Bílina mine and former 5. květen mine.

of air-dried lumps) from the same site, show similar trends (Fig. 6.19 lines: Bílina lumpy flooded and Karpíšková [72]).

A comparison of the index properties and the mineralogical composition of the original clay can explain this difference. While the grain size distributions of the two original clays are very similar (Table 3.2), the mineralogical compositions vary. The

clay from the Bílina mine is more illitic (40% illite), while the clays from the former 5. květen and Marie mines are more kaolinitic (36%), see Table 3.2 ins Sect. 3.2. The compressibility of the water-saturated specimens increases in the order kaolinite < illite < smectite [98]. This is well in agreement with the observed behaviour of the two reconstituted clays (Fig. 3.2) and their mineralogical composition.

The clay from the Bílina mine also has by a 22% higher liquid limit and a 12% (or 26% for the Marie mine) higher plasticity index (Table 3.2). This also fits well with the higher compressibility of the reconstituted clay from the Bílina mine. Correlations between the index properties of reconstituted fine-grained soils and their compressibility behaviour have been published by many authors (for an overview, see e.g. [98, 120].) Figs. 6.20 and 6.21 display the compressibility indices and the initial void ratios of clayey fills from four different mines, three from North-western Bohemia and the fourth from the Rheinland in Germany (the data for the Hambach mine with permission of *RWE Power AG*), related to the water content at the liquid limit and the plasticity index, respectively. The best correlations for the depicted data and the well-known correlations between index properties and compressibility indices from the literature [80, 119] are also shown. Skempton [119] correlated the compression index with the liquid limit for remoulded clays and Mayne and Kulhavy [80] presented the correlation between compression index and plasticity index (I_P) for a number of different natural clays. The correlation of Mayne and Kulhavy [80] corresponds well with the correlation based on critical state theory, giving $c_c = \frac{I_P * \rho_s}{200}$ (see e.g. [12]). The data show a reasonable correlation, despite the broad range of their index properties and the different sampling sites. However the best fit functions differ slightly from the correlations published in the literature. A better correlation was found between the liquid limit and the compressibility index than for the plasticity index¹. A reasonable linear best-fit function was also found between the void ratio at 1 kPa e_0 and both the liquid limit and the plasticity index (see Fig. 6.21). However, no correlations between the position of 1-D NCL and the index properties were found in the literature.

This indicates that not only the mechanical properties of clay lumps, depending on their state and degree of overconsolidation, but also the properties of the original clay itself can vary significantly within one sedimentary basin and even within one mine (see also Sect. 2.2.4). As a consequence, the character of the original clay also influences the void ratio and the compressibility of the fills that are formed.

Disregarding the differences in the initial void ratios and compressibilities depending

¹The worse correlation between c_c and I_P could be possibly explained by the determination of index properties in different laboratories. The determination of water content at plastic limit is more sensitive to a workmanship than the dermination of the liquid limit.

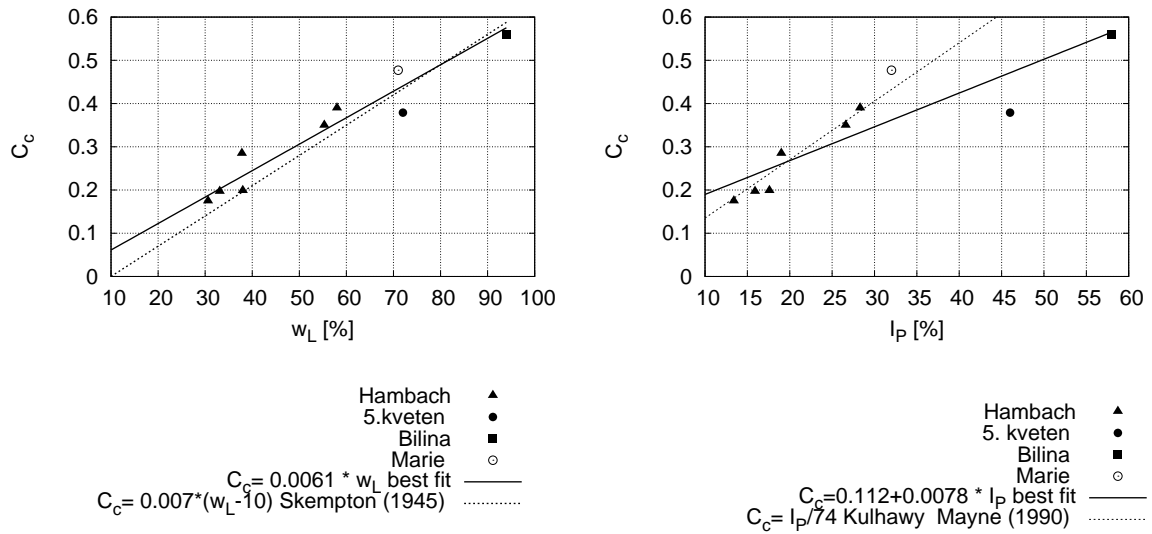


Figure 6.20: Compressibility indices of reconstituted clay fill soils from different sites related to water content at liquid limit (left), related to plasticity index (right).

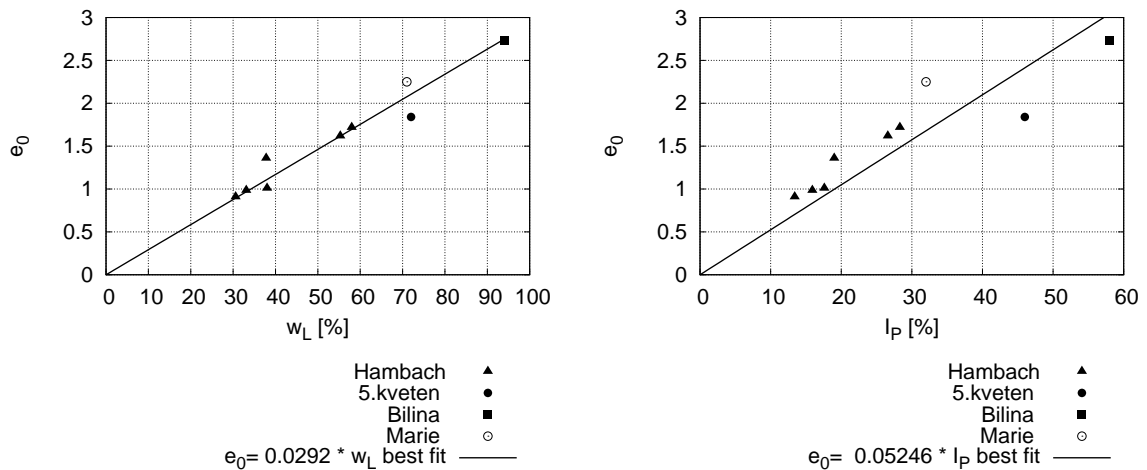


Figure 6.21: Void ratios at 1 kPa of reconstituted clay fill soils from different sites related to water content at liquid limit (left), related to plasticity index (right).

on the index properties, different compressibility tendencies between the reconstituted and the lumpy specimens can be distinguished for each site (Fig. 6.19). The compression lines of flooded specimens from the Bilina mine lie below the reconstituted lines until high stresses. This is caused by progressive filling of macrovoids by softened paste of high porosity from the strained weaker lumps and lump contacts. The porosity of the strained and softened clay lies close to the porosity of the reconstituted soil. A part of the structure is formed by stiffer lumps and the lump cores with a lower void ratio persisting the applied stresses even when flooded until higher applied loads (discussed further in Sect. 6.2.2). These stiffer lumps

form structural elements in the surrounding clayey mass of the higher void ratio filling the macrovoids. The compressibility line of the flooded lumps therefore lies below the NC-line of reconstituted specimen (see also Sect. 6.2), and thus shows a stiffer response for a certain stress level. At higher stress levels, the lumpy and reconstituted void ratios unify, and there are no differences in void ratios between the lump relicts and the strained and softened clay. The lumpy structure persisting until these stresses vanish, and the soil behaves as normally consolidated.

A soil structure with void ratio which surpasses that of the intrinsic (reconstituted) soil was observed only in the case of loose and compacted lumpy specimens compressed at their natural water content (see Figs. 6.11, 6.12 in Sect. 6.2) for the lumps from Bílina mine. These results fit well with the structure degradation after collapse on wetting, see Sect. 7.

Oedometer tests performed on lumpy specimens from the former 5. květen mine and consolidated in the mini-centrifuge [100] show a different trend (box symbols in Fig. 6.19). The initial void ratios of these specimens vary due to their consolidation at different stresses. The oedometer specimens were cut from different heights of the sample consolidated in a mini-centrifuge, corresponding to different simulated overburden pressures (details in [100]). Following the unloading-reloading line caused by self-weight consolidation in the centrifuge, the specimens exceed the NC line of the reconstituted specimen and reach an apparent overconsolidation pressure depending on the overconsolidation ratio. The higher void ratio of the lumpy specimens than of the reconstituted specimens for a certain pressure range results in the behaviour of a structured soil (see also Fig. 2.10).

This implies that some lumpy structures can exist at higher void ratios than the void ratio of the reconstituted clay at certain stress levels even after flooding, while the compressibility of some structured lumpy clay can lie below the level of reconstituted clay. The higher intragranular void ratio of the lumps prepared from the aged clay fill soil from the former 5. květen mine may explain this effect. The soil was sampled from a depth of about 2-3 m below the ground surface of the aged fill, where a high degree of homogenization under low stresses can be expected (Fig. 5.7 and Sect. 5.2). No information about the intragranular void ratio of the prepared lumps is given by Najser [100]. However the isotropic and oedometer compression tests on undisturbed specimens were performed on samples from different depths sampled at this site (see Fig. 6.22, the test results are analyzed in greater detail in Chapter 7 regarding the collapse potential at this site). The initial void ratios of the specimens sampled at a shallow depth up to 2.5 m were scattered between $e = 0.78$ and 0.87 . Considering this void ratio to lie close to the void ratio of the soil used for the preparation of lumps by Najser [100], the intragranular and intergranular void ratio of lumpy soils

from the two sites can be compared, see Table 6.2. The intragranular void ratio of the clay lumps from the Bílina mine was determined directly (see Sect. 5.1.2). Comparing the void ratios at 20 kPa from the two sites, see Table 6.2 (only the specimens consolidated to 18 and 20 kPa in the mini-centrifuge from [100] were considered), a higher intergranular void ratio for the flooded lumps of $e_e = 0.61 - 0.8$ was achieved for the Bílina site than for the 5. květen site, of $e_e = 0.34 - 0.44$.

The undisturbed samples were sampled using a thin-walled pushed sampler, for more details see Sect 7. Plotting the void ratios of all undisturbed specimens against the sampling depth, see Fig. 6.23, two regions of different trends can be distinguished despite a certain scatter of the data. At shallow depths up to 4 m, no evident decrease in the void ratio with increasing depth is observable. The scatter of the void ratios is also low for this region. The void ratio is expected to be influenced by weathering (see Sect. 5.2) at these depths, and there is a high degree of homogenization. The original intergranular and intragranular void ratios are both influenced by these processes, and consequently a more homogenized layer is formed. A decrease in the void ratio with increasing depth is clearly distinguishable in the region deeper than 4 m. It should be mentioned that possible persisting macroporosity is probably not captured well by standard sampling.

The compressibility curves of lumpy specimens of Najser [100] and reconstituted specimens of Hájek [50] are compared with the void ratios of undisturbed specimens at sampled geostatic pressures in the Fig. 6.22 - left. Compressibility curves of the selected undisturbed specimens are depicted in Fig. 6.22 - right. Void ratios of all undisturbed specimens lie below the CL of reconstituted and lumpy soil. Furthermore the compressibility curves of undisturbed specimens reveal a lower compressibility for the tested loads up to 1000 kPa. The stiffer response of the undisturbed specimens is well in agreement with the high degree of homogenization of the aged fill. The undisturbed specimens probably contained variable amounts of strained clayey mass and relicts of the fragments, resulting in the scatter in initial void ratios but in the overall stiffer behaviour. This results also fit well with the observations in the field on the monitored embankments. Much lower settlements were recorded in situ than for the centrifuge model of lumpy clay fill [100].

For a description of the structure degradation, two mechanisms should be distinguished regarding both the strength and the void ratio of the lumps and the structure degradation agents.

- The stiffer lumps swell and soften when wetted, shrink and strengthen when dried, and subsequently disintegrate due to cycles of wetting-drying. This causes the macropores to be filled by disintegrated lumps and the disturbed

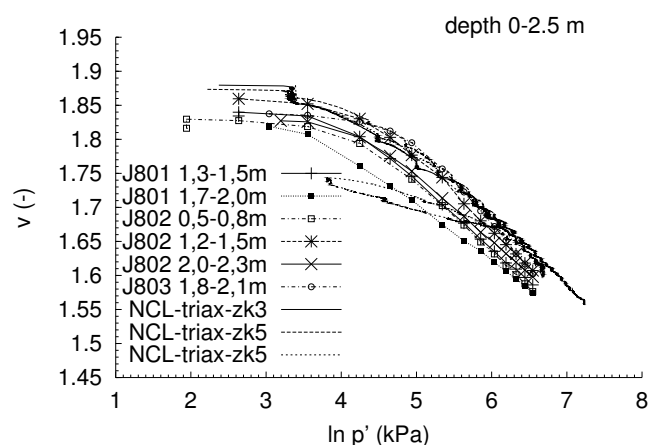


Figure 6.22: Compressibility curves performed on undisturbed specimens (former 5. květen mine).

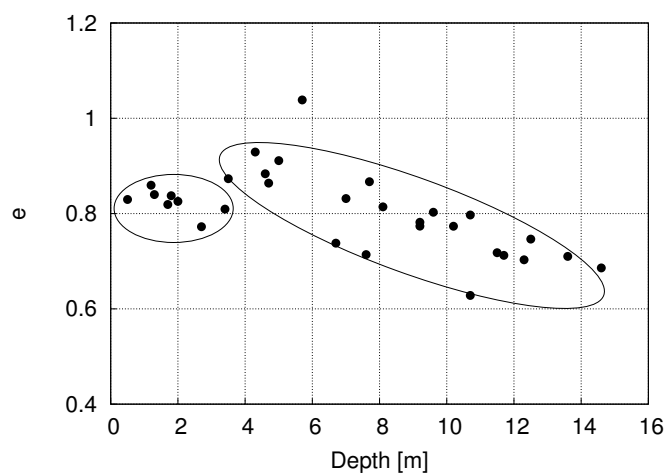


Figure 6.23: Void ratio of undisturbed specimens versus sampling depth (former 5. květen mine).

clay mass, and therefore there is a decrease in overall porosity at a shallow depth. Simultaneously, the lumps themselves change their structure and void ratio. The intragranular porosity increases due to this aging, and approaches the porosity of a reconstituted soil. Just the centers of bigger and stiffer lumps persist the aging effects (see also Fig. 5.7). However, weathering does not significantly influence the intragranular porosity of soft lumps from shallow depths, since they have already experienced pronounced effects.

- Both soft and hard lumps deform under load. Lumps can fail in shear, which follows different mechanisms. Hard lumps can dilate while soft lumps contract during shearing. This can cause an even more intensive decrease in macrovoids

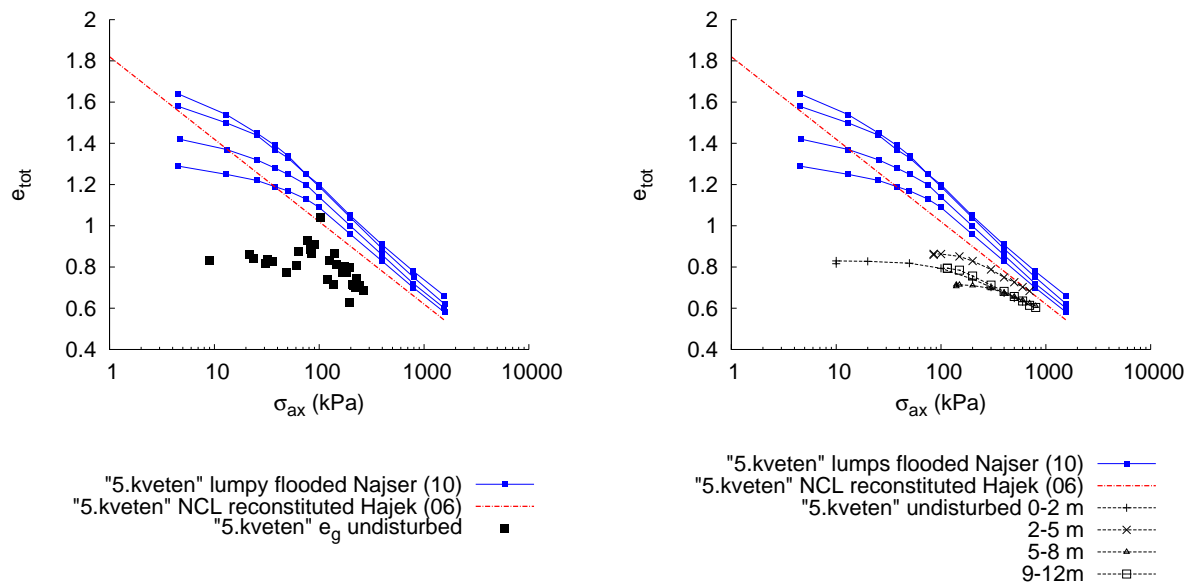


Figure 6.24: Compressibility curves of lumpy specimens and void ratio of undisturbed specimens - left; compressibility curves of undisturbed specimens - right (former “5.květen” mine).

Table 6.2: Void ratios of model lumpy material (sites 5.květen and Bílina).

Site	Intragranular void ratio	Total void ratio at 10 kPa	Intergranular void ratio after Feda [42]	Intergranular porosity after Feda [42]
5.květen	0.78–0.87	1.53–1.55 [100]	0.36–0.42	0.27–0.3
Bílina	0.76–0.78	1.8–1.95	0.65–0.67	0.4

in the case of a softened clayey mass for the dilative mode of shearing of stiff lumps.

As a consequence, two different types of structures of aged clayfills can be formed in terms of the strength and void ratios of the lumps.

- A fill structure containing a low void ratio and stiff fragment relicts, a higher void ratio and soft clayey mass from degraded and strained lumps, and macrovoids. This can suggest a tri-modal pore size distribution for certain stress levels. The compression line of this structure typically lies below the reconstituted line until both unify at higher stress levels.
- A fill structure containing soft lumps with a higher void ratio and macrovoids. This structure is more compressible and its void ratio unifies with the intrinsic

(reconstituted) void ratio at lower stress levels. However, due to the higher intragranular porosity the total void ratio can exceed the porosity of reconstituted soil at certain stress levels.

A metastable, hydrocollapse-susceptible structure can be formed by both types of structure when deposited at the natural water content.

Consequently, the structural changes leading to degradation of the fill structure should be evaluated, taking into account:

- lump size and lump distribution,
- the strength of the lumps,
- the intragranular porosity of the lumps,
- the state of the lumps with reference to their wetting-drying history.

6.6 Summary

- The initially granular nature of the clay fill material changes due to the external loads (increase of the clay fill height). The clay lumps are relatively weak when competing with rocks and adapt their shape to the anisotropic state of stress. For a rapid load change the clay lumps, which are close to its saturation or fully saturated, respond in an undrained manner and become elongated in the direction of the minor principal stress.
- Changes in the shape of the lumps contribute to the closure of macrovoids. Weak macrograins squeeze into the macrovoids and fill them up (Fig. 6.25). The destruction of the clay lumps is even more progressive by shearing. Even in K_0 -conditions the clay fill lumps are subjected not only to vertical (normal) stresses but also to shear stresses. These cause large irreversible strains, disintegration of the lumps and consequently intensive filling of macrovoids (see Fig. 6.25 right).
- The pressure threshold of 1500 kPa for closing the macrovoids in one-dimensional compression was determined for the lumpy specimens. At this threshold compression lines of both lumpy and reconstituted unite.
- If lumpy soil with the natural water content is flooded only once it collapses, but the bigger and stiffer lumps remain intact. These intact lumps are weaker than those possessing apparent cohesion due to matric suction. However, the

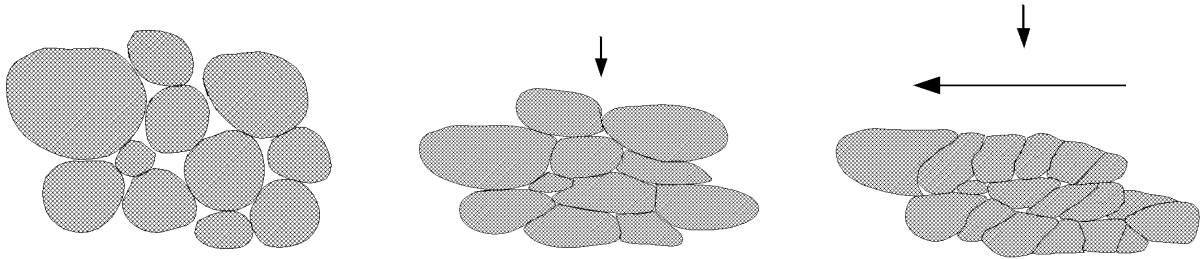


Figure 6.25: Clayey material in its natural state with inter and intragranular voids (left), closure of intergranular voids when compressed (centre), closure of intergranular voids and lumps disintegration when sheared (right).

lumps also have a smaller void ratio than the surrounding clayey mass, which is formed due to the plastic straining of the smaller and weaker lumps. This leads to a lower total void ratio for the soil structured in this way than for a soil without structure (the reconstituted soil). The implication is that this soil can be less compressible than a reconstituted soil for a certain stress level.

- The metastable structure with large and open macrovoids persists until high stresses only in the partly saturated state, due to the extra strength of the lumps. This strength is further influenced due to the matric suction acting in the lumps.
- If the lumpy structure is formed from the lumps having a higher void ratio, the total void ratio of the lumpy structure can exceed the void ratio of the reconstituted clay, even in a fully saturated state, as presented by Najser [100].

Chapter 7

Evaluation of collapse behaviour

In the first part of this chapter, the thresholds for the collapse potential of two types of typical collapse susceptible soils are defined. These thresholds were evaluated based on the literature review and own experiments. In the second part, the collapsibility of the clay fill soil is studied. Two sets of tests on undisturbed and lumpy samples have been carried out to investigate the collapse vulnerability of clay fills of a different age in North-western Bohemia. The collapse potential of clay fill soil is further compared with the thresholds defined for the typical collapsible soils.

7.1 Methods for estimating collapse potential

There exist several methods to estimate the magnitude of collapse that may occur when unsaturated soils are inundated. The collapse vulnerability can be conveniently evaluated on soil specimens in oedometer cell. Herein, a flat cylindrical specimen is constrained laterally by a rigid steel ring and thus allowed to deform only in vertical direction. ASTM [10] recommends to load the specimen to the load of interest (commonly 200 kPa) and measure its vertical deformation after flooding the specimen at the particular stress. Disadvantage of this method is its restriction to the single applied vertical load.

Double oedometer test, a method developed by Abelev [6] and reported e.g. by Jennings and Knight [69], covers the collapse potential at the full range of vertical loads applied in the laboratory. Two identical specimens are placed into oedometer cells. During the experiment, one specimen is maintained at its original water content, whereas the second one is inundated at low total vertical stress. The vertical distance between the two stress-void ratio curves indicates the magnitude of collapse which arises due to wetting.

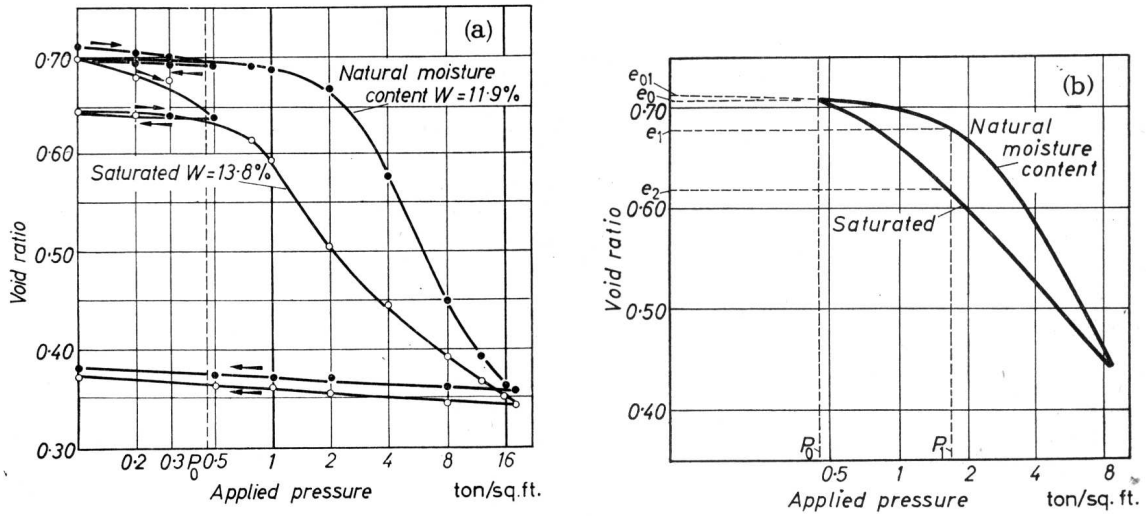


Figure 7.1: Principle of double oedometer tests after Jennings [69]. Compressibility curves of natural and flooded soil (left); transposed curves (right).

However in many cases, initial void ratio of two “identical” specimens differs (Fig. 7.1–left). Jennings and Knight [69] assumed that the specimens were in equilibrium with the overburden pressure and attributed any possible differences between the curves at the overburden pressure to the disturbance on sampling and rebound under release of load. They suggested to identify the probable in situ void ratio e_0 from the rebound curve and the overburden pressure p_0 and to transpose the laboratory curves to pass through the point $e_0 p_0$ (Fig. 7.1–right). The vertical difference between transposed curves was then attributed to the collapse on wetting.

Cui et al. [31] showed that double oedometer tests compare well with the standardized method of inundating specimens at a particular stress level (see [10]).

7.1.1 Collapse potential

According to ASTM [10] the relative magnitude of collapse determined at any constant stress level, a so-called collapse potential I_c in [%], is defined as:

$$I_c = d_h/h_0 * 100 \text{ [in \%]} \quad (7.1)$$

where

d_h = change in specimen height resulting from wetting, h_0 = initial specimen height.

The thresholds for classification of collapse indexes according to ASTM [10] are given in Table 7.1.

Table 7.1: Classification of collapse index I_c [10].

Degree of Specimen Collapse	Collapse Index I_c %
None	0
Slight	0.1 – 2
Moderate	2.1 – 6
Moderately severe	6.1 – 10.0
Severe	> 10

7.2 Thresholds for collapse of typical collapsible soils

Collapse under wetting is limited by thresholds in soil state and external load. In the section, typical collapse thresholds for two different soil types is presented. Loess (or, equivalently, low plasticity silty clay) and loose sand were chosen as representative soils for a literature review and laboratory investigations regarding collapse potential.

7.2.1 Unit weight

It is often claimed that unsaturated soils with dry unit weights below 16 kN/m^3 can be prone to collapse [91]. A review of published experimental results on loess and other low plasticity silts [[32], [103], [92], [121], [108]] has shown that for this type of soil the unit weight threshold for a collapse susceptible soil structure lies at about 17 kN/m^3 . Fig. 7.2 presents that only two specimens subjected to wetting at loads between 100 and 300 kPa and at different degrees of saturation, showed a slight collapse potential ($< 2\%$) at a dry unit weight 17 kN/m^3 and greater. All other specimen showed either no volumetric change or volume increase (swelling) after wetting. Partly saturated sand specimens prepared at dry unit weight about 16 kN/m^3 at different degrees of saturation have shown a slight collapse potential ($< 2\%$) (Fig. 7.3), which confirms again the threshold of 16 kN/m^3 suggested by [91].

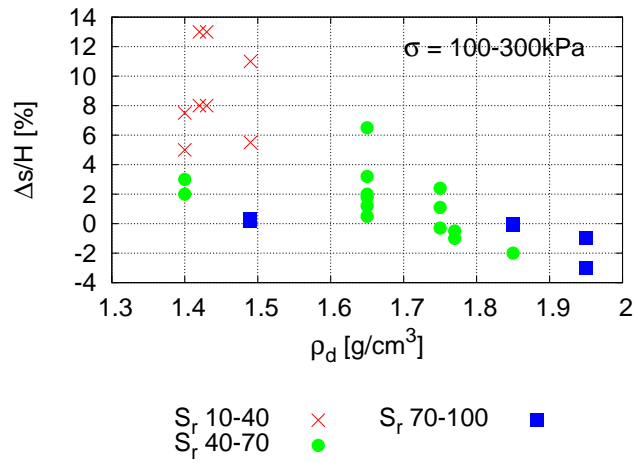


Figure 7.2: Dependence of collapse potential on dry unit weight at vertical stresses between 100 and 300 kPa and at different degrees of saturation (negative values denote swelling) for loess and low plasticity clays. Data from [[32], [103], [92], [121], [108]].

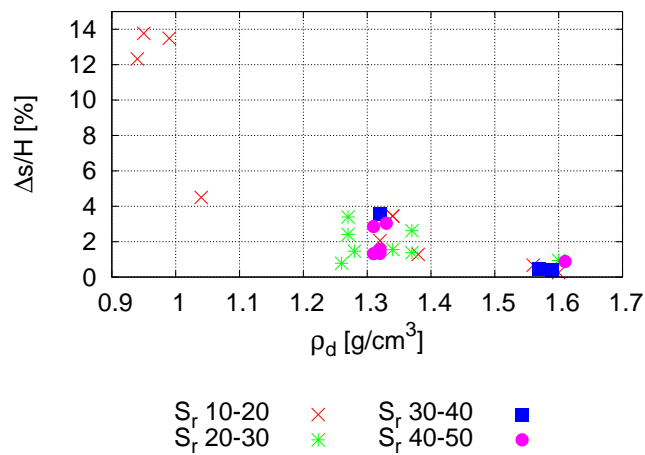


Figure 7.3: Dependence of collapse potential on dry unit weight at vertical stresses between 0 and 200 kPa and at different degrees of saturation for sands. Data from [[22], [60]].

7.2.2 Pressure limits

Threshold for closure of macrovoids

The higher initial stress level, the lower the amount of macrovoids in the soil structure. Consequently, the collapse potential vanishes at higher stresses when large open voids become closed. However, the threshold for closure of macrovoids is influenced by degree of saturation (matric suction). At low degrees of saturation, high suction acting at grain contacts increases the intergranular stresses, stabilizes grain arches and thus shifts the threshold for closure of macrovoids to higher stresses. Considering the result for low plasticity clays, the threshold for macrovoid closure is not reached even at the confining stresses of 1600 kPa, when degree of saturation is below 10 – 20 %. For degree of saturation between 40 and 70%, the threshold for macrovoids closure can be roughly recognized at stresses between 800 and 1500 kPa. In loose sands, the threshold for closure of macrovoids can be expected lower than in low plasticity clays, due to smaller matric suction within relatively large voids. A rough estimation yields the value between 300 and 400 kPa (see Fig. 7.5).

Threshold for volumetric compression

If loess or low plasticity clay is inundated under a very low stress, the soil would rather swell than collapse. Soil specimens flooded at 0 kPa did not collapse even under low degrees of saturation and low dry densities (Fig. 7.4). The above statement is not valid for loose sands. Inundating sand at low stresses produces always collapse (Fig. 7.5). Thus in case of sands the smaller the confining stress the higher the collapse potential can be observed without exception.

7.2.3 Degree of water saturation

A partial saturation is the essential prerequisite of collapse susceptible soils. At lower water contents greater matric suction acts and increases the effective stresses within the soil. Unsaturated soil is therefore less compressible since the soil stiffness depends on effective stress. The literature review has shown that for low plasticity silts and loess flooded at confining stresses up to 300 kPa the collapse effects vanish at the degree of saturation of about 70% (see Fig. 7.4). This is well in agreement with the observations of El-Ehwany and Houston [38], who observed the same collapse settlement when wetting the soil to the degrees of saturation of 65 – 70% as when flooding the same soil. Moderately severe (6.1 - 10%) to severe collapse potential (> 10%) was found for $S_r \leq 40\%$ and $\rho_d \leq 1.5 \text{ g/cm}^3$ (see Fig. 7.4).

7.2. THRESHOLDS FOR COLLAPSE OF TYPICAL COLLAPSIBLE SOILS

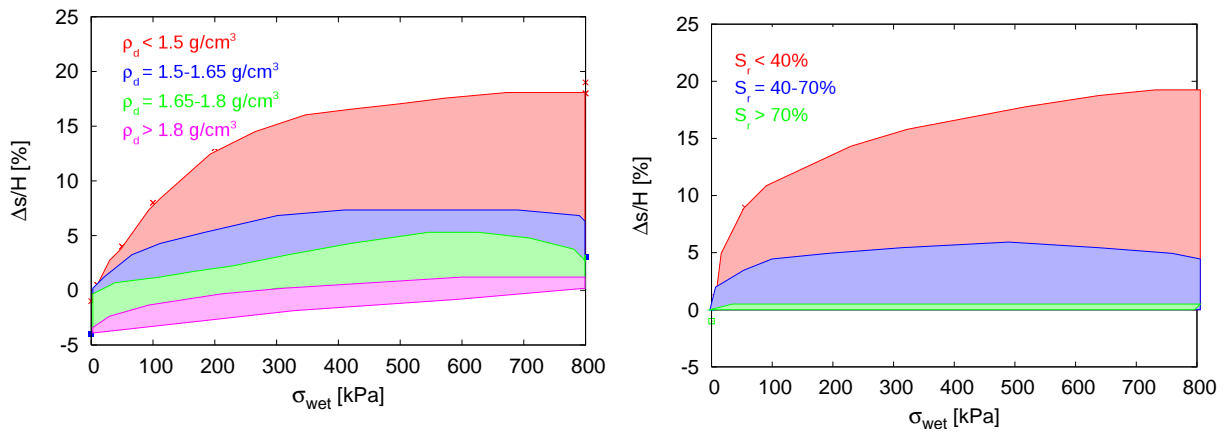


Figure 7.4: Typical collapse strains ($\Delta s/H$) in dependence on confining stress during wetting for different soil dry unit weights and degree of saturation $< 70\%$ (left), and for different degree of saturation and dry unit weight $< 1.5 \text{ g/cm}^3$ (right) for loess and low plasticity clays. (Compiled according to data from [[32], [103], [92], [121], [108]]).

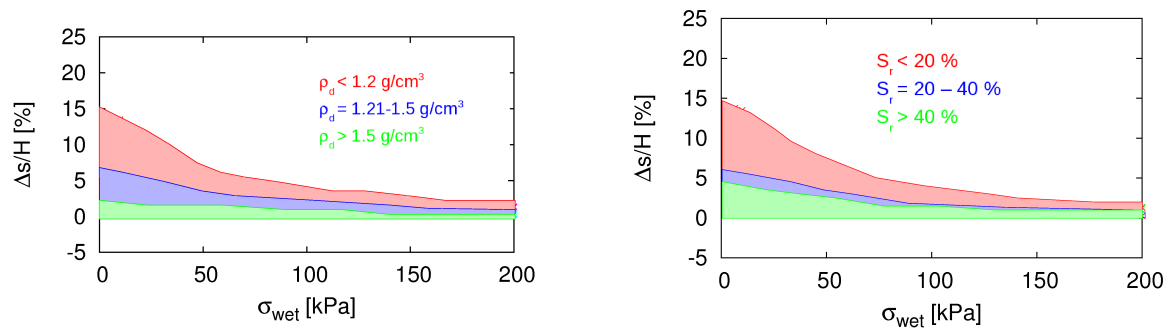


Figure 7.5: Typical collapse strain ($\Delta s/H$) in dependence on confining stress during wetting for different soil dry unit weights (left) and degree of saturations (right) for sands. Data from [[22], [60]].

The literature review and laboratory investigation of sands [[22], [60]] (Fig. 7.5) reveal that the collapse potential reaches high values at low degrees of saturation $S_r \leq 20\%$ and at low densities $\rho_d \leq 1.2 \text{ g/cm}^3$. Moderate collapse potential can be observed at degrees of saturation $S_r = 20 - 40\%$ and densities $\rho_d = 1.2 - 1.5 \text{ g/cm}^3$ if flooding takes place at low confining stresses (Fig. 7.5). Based on these results, it can be roughly predicted that the threshold saturation for the collapsible behaviour of sand lies between 60 and 70%.

7.3 Collapse behaviour of clay fills

The excavated overburden is typically filled in a dry way and therefore the intervoids are initially air-dry while the intravoids of lumps exhibit almost full saturation by its natural water content of about $w = 34\%$. Due to the granular nature of the freshly filled material there is no suction within the intergranular voids (at least at lower pressure ranges) and some suction within the intragranular voids in the lumps, where the negative pore water pressures develop first due to water-level sinking and subsequently due to unloading by excavation. The further development of suction within the lumps is affected by weather dependent moisture changes. The intergranular voids form the preferential paths for surface water, so the freshly filled clay fills are highly permeable until the initial structure degrades.

The initially lumpy structure of the soil degrades with the increasing load. The intervoids close up due to plastic straining of relatively weak lumps. In greater depths the interporosity vanishes (see Section 6). In lower depth wetting-drying suction cycles are the main agent, which causes the structure degradation (see Sections 4 and 5.2). Through the structure degradation clay fill material changes back into more or less homogeneous soil. The overall hydraulic conductivity decreases with increasing degree of structure degradation. Figure 5.8 right shows the aged spoilheap material of about 10 years after filling, where some lumps are already degraded into finer mass.

7.3.1 Testing method

To investigate the collapse potential of the clay fill, the technique of double oedometer tests was adopted (Tokar, 1937 in Abelev [6, 31, 69], see Sect. 7.1). Two "identical" specimens are placed into the oedometer and during the test one specimen is maintained at the original water content (referred to as "unsaturated" specimen, marked with *unsat* in Fig. 7.7), the second one is inundated at low total vertical pressure ("saturated" specimen, marked with *sat* in Fig. 7.7-top). Some curves needed to be transposed (marked with *trans* in Fig. 7.7) to the same initial void ratios as suggested by Jennings and Knight [69]. The divergence of the two stress-void ratio curves indicates the magnitude of collapse due to wetting.

The collapsibility index was determined using:

$$i = (e_{unsat} - e_{sat}) / (1 + e_{unsat}) \quad (7.2)$$

where e are void ratios of natural moisture and inundated specimens at each loading step.

A standard ASTM [10] method of flooding the samples at predetermined pressure was also used for verifying the test results.

7.3.2 Laboratory tests on undisturbed samples

The collapse potential was investigated at the site of motorway construction near the town Ústí nad Labem (see Fig. 3.4) at the former 5 květen mine. The particular spoilheap is about 40 years old and in a high degree of homogenization and interporosity decrease but the collapse potential could still be expected.

Sampling

The undisturbed samples were taken at the reclaimed clay fill of the former 5 květen mine. Three rotary cored boreholes were drilled down to the depths of 15 to 20 metres. The whole profile was formed by clay fill material, no underlying subsoil was reached. The samples were taken using a standard pushed-in sampler of about 110 mm diameter, with a plain cutting shoe and a plastic liner. Since the quality of the clay fill samples taken by the standard sampler was questioned, a comparison with a thin-walled steel pushed-in sampler according to DIN was made. The points in Fig. 7.6 represent averages of four oedometer laboratory moduli measured on specimens from the samples procured by both the samplers. The two scattered points resulted from a faulty procedure, when the samples were left to interact with the wall of the steel sampler for several weeks. The forces needed in extracting every sample were measured, and they were much higher for the two particular samples.

When the two faulty samples are disregarded, Fig. [54] shows that both samplers yielded a well defined single series of laboratory oedometer moduli. There was hardly any difference between the samples from the standard sampler and the samples from the DIN thin-walled sampler. The standard sampler used in the present study is therefore believed sufficient for taking samples from rotary cored boreholes in the clay fills [54].

The initial degrees of saturation varied from 0.83 to 1, when one scattered value of 0.62 was neglected. The initial degrees of saturation of the soils at the particular landfill were relatively high.

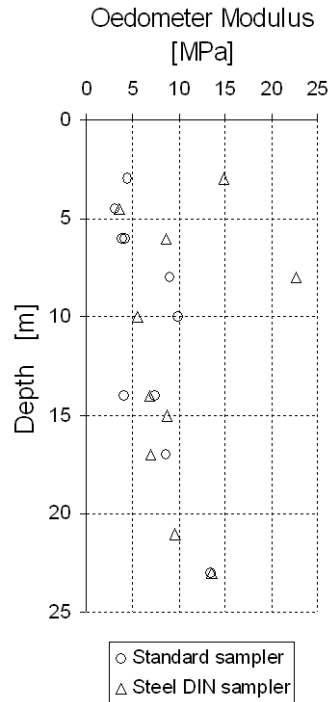


Figure 7.6: Comparison of data from two samplers [54]

Test results

The tests were carried out in standard fixed ring oedometers of the diameter of ca 100mm and height of ca 30mm. Changes in moisture content of the specimens without inundation were prevented by a plastic sheet, further to the standard laboratory air conditioning (temperature controlled room with $22^{\circ} \pm 1^{\circ}\text{C}$). In total 44 pairs of oedometer specimens were evaluated.

The test results are summarized in Table 7.2, i_m denotes maximal recorded collapse potential for the particular tests. In total 24 specimens exhibited some collapse, and only 2 of them were classified as moderately collapsible according to ASTM [10]. 20 specimens exhibited a negative value of collapse index which indicated swelling. The standard double oedometer tests did not confirmed any collapse potential.

7.3. COLLAPSE BEHAVIOUR OF CLAY FILLS

Table 7.2: Overview of the test results of 44 double oedometer tests. Minus denotes swelling.

Bore-hole Nr.	depth in m	S_r unsat init	S_r unsat σ_{vcol}	S_r sat init	S_r sat σ_{vcol}	ε_a after innun. [%]	Δe	ρ_d [g/cm ³]	σ_{vcol} [kPa]	i_m^* [%]
J801	1.0 - 1.3	0.94	1.00	0.94	1.00	-0.30	-0.015	1.64	500	0.9
	1.7 - 2.0	0.94	0.98	0.98	0.99	-0.02	0.034	1.61	1000	-2.1
	2.7 - 3.0	0.94	1.00	0.94	0.98	-0.06	0.014	1.58	900	-0.9
	4.3 - 4.6	0.94	0.99	0.95	0.98	-0.04	-0.025	1.57	900	1.6
	5.0 - 5.3	0.96	0.99	0.95	0.97	0.01	-0.009	1.65	800	0.5
	5.7 - 6.0	0.62	0.70	1.03	0.99	-0.02	0.030	1.85	600	-1.6
	6.2 - 6.5	0.96	1.00	0.96	1.00	-0.14	0.005	1.60	1000	-0.3
	8.1 - 8.4	0.91	0.99	0.93	0.98	-0.09	0.031	1.61	1000	-1.9
	9.2 - 9.5	0.93	1.00	0.91	0.98	-0.13	-0.010	1.66	600	0.6
	10.7 - 11.0	0.92	0.99	0.94	0.99	-0.22	0.027	1.61	1000	-1.7
	11.7 - 12.0	0.95	1.00	0.92	1.00	-0.04	-0.064	1.55	1000	4.1
13.0 - 13.3	0.94	1.00	0.94	1.00	-0.04	-0.003	1.49	1000	0.2	
J802	0.5 - 0.8	0.92	1.00	0.89	0.99	-0.74	-0.040	1.57	1000	2.5
	1.2 - 1.5	0.96	1.00	0.94	0.98	-0.30	-0.014	1.65	500	0.8
	2.0 - 2.3	0.94	1.00	0.93	0.98	-0.16	-0.018	1.64	500	1.1
	3.4 - 3.7	0.97	0.99	0.97	1.00	-0.01	0.040	1.62	900	-2.5
	4.6 - 4.9	0.93	1.00	0.93	0.96	0.00	0.058	1.62	1000	-3.6
	5.5 - 5.8	-	-	-	-	-	-	-	-	-
	7.0 - 7.3	0.95	1.00	0.91	0.99	0.00	-0.030	1.60	1000	1.9
	7.7 - 8.0	0.91	1.00	0.92	0.93	-0.02	0.022	1.71	500	-1.3
	9.2 - 9.5	0.93	1.00	0.91	0.98	0.01	-0.005	1.62	800	0.3
	10.2 - 10.5	0.91	1.00	0.89	0.98	-0.09	-0.010	1.63	1000	0.6
	11.5 - 11.8	0.95	0.99	0.96	0.98	0.02	-0.020	1.68	300	1.2
12.3 - 12.6	0.91	0.99	0.93	0.97	0.00	0.045	1.61	1000	-2.8	
J803	1.8 - 2.1	0.85	0.99	0.86	1.00	-1.61	-0.020	1.78	300	1.1
	3.5 - 3.8	0.89	0.98	0.89	0.99	-0.22	-0.007	1.83	200	0.4
	4.7 - 5.0	0.88	0.99	0.88	0.99	-0.26	-0.008	1.64	1000	0.5
	6.7 - 7.0	0.94	1.00	0.94	1.00	-0.30	-0.008	1.64	500	0.5
	7.6 - 7.9	0.91	1.00	0.91	0.98	-0.47	0.090	1.61	1000	-5.6
	9.6 - 9.9	0.92	0.99	0.91	0.98	-0.01	-0.016	1.66	1000	1.0

* minus denotes swelling

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Bore-hole Nr.	depth in m	S_r unsat init	S_r unsat σ_{vcol}	S_r sat init	S_r sat σ_{vcol}	ε_a after innun. [%]	Δe	ρ_d [g/cm ³]	σ_{vcol} [kPa]	i_m [%]
	10.7 – 11.0	0.89	0.97	0.91	1.00	-0.03	0.035	1.64	1000	-2.1
	12.5 – 12.8	0.94	0.98	0.94	1.00	0.00	-0.004	1.59	900	0.3
	13.6 – 13.9	0.95	0.98	0.95	1.00	-0.01	0.015	1.60	950	-0.9
	14.6 – 14.9	0.84	0.99	0.88	0.96	-0.06	0.018	1.59	1000	-1.1
79125	13.6	0.97	1.01	0.96	1.03	-0.11	0.023	1.60	500	-1.4
	17.6	0.89	0.83	0.89	0.99	-0.27	0.020	1.85	500	-1.1
	19.6	0.94	0.94	0.89	0.99	0.16	-0.005	1.83	500	0.3
79151	13.2	0.84	0.92	0.91	0.98	0.01	0.015	1.82	500	-0.8
	18.0	0.95	0.98	0.98	1.01	-0.18	0.020	1.87	500	-1.1
	23.0	0.94	0.94	0.91	0.98	0.33	0.008	1.81	500	-0.4
	24.5	0.95	0.95	0.91	0.97	-0.02	-0.001	1.87	500	0.1
	25.6	0.98	1.03	0.97	1.02	0.18	-0.002	1.66	500	0.1

Fig. 7.7 shows the typical test results of double oedometer tests.. There was a scatter in the initial void ratios of undisturbed samples, which could refer to some remaining nonhomogeneity of the aged spoilheap. The soil composes of the lower void ratio relicts of the overconsolidated lumps accompanied by the strained soft clayey mass of the higher void ratios (see also Sect. 6.5).

Matric suction of undisturbed specimens

In the laboratory suctions were measured during two oedometer tests by filter paper method. Following Leong and Rahardjo [85] who compared filter paper suction determination for different calibrations of retention curves, Whatman No. 42 filter paper was used with the ASTM [11] equation to obtain matric suctions via the contact method. Similarly to Leong and Rahardjo [85], from the four equations they quoted, the ASTM [11] equation gave the most consistent values of matric suction.

Matric suction was measured by placing the filter papers in the sandwich sample closed in the sealed jar for at least one week to reach equilibrium. Further, filter papers were placed under oedometer unsaturated specimens and matric suctions were determined after the tests. The measured suctions are summarized in Table 7.3. The matric suction of the specimen from the depth of 8.4 – 8.5 m is relatively low and was attributed to the sampling procedure. There was a slight collapse which

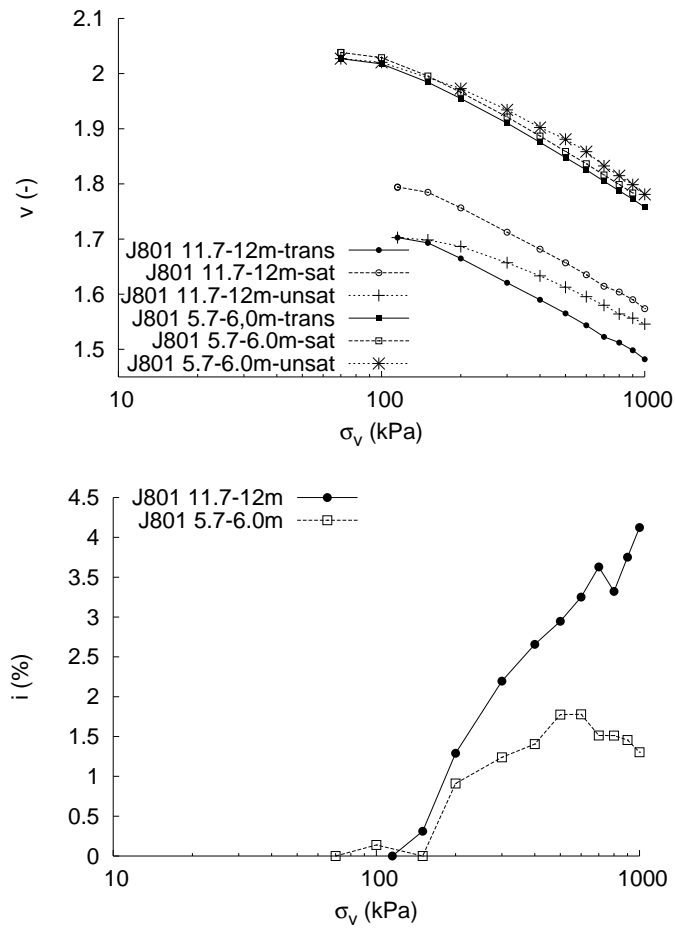


Figure 7.7: Compressibility curves (top) and collapsibility indexes of double oedometer tests on undisturbed specimens.

reached its maximum value $i = 1.6\%$ at the pressure of 400 kPa. The higher values of suction measured with the specimen from the depth of 11.5 – 11.7m are believed to be partly caused by loss of moisture during the test. The sample showed a moderate collapse index of maximum value of 2.9% at $\sigma_v = 700\text{kPa}$.

Table 7.3: Suctions measured by filter paper method in kPa.

Bore hole	Depth [m]	Degree of sat. S_r	Matric suction according to			
			Leong et al. [85]	van Genuch-ten [85]	Fredlund & Xing [85]	ASTM [11]
J803	8.4-8.6 sandwich	-	59	75	69	55
J803	8.4-8.6 before t.	0.92	67	97	89	60
J803	8.4-8.6 after t.	0.98	67	97	89	60
J803	11.3-11.6 sandw.	-	98	134	124	81
J803	11.3-11.6 b.t.	0.89	135	168	156	118
J803	11.3-11.6 a.t.	1.02	254	167	257	244

Evaluation in effective stresses

The evaluation of the collapse potential in total stresses, using the double oedometer test method, is the disadvantage of this method. The effective stresses are equal to total for the flooded specimen after finishing the consolidation, while the effective stress for the specimen compressed at its natural water content is not known.

When using the filter paper method, the matric suction is known at least before the test. The effective stresses using Bishop's relation can be evaluated, see Equation 5.1 in Sect. 5.1.1.

Fig. 7.8 displays the comparison of the compressibility curves of saturated - transposed (full circles), unsaturated in total stresses (empty boxes) and unsaturated in effective stresses (full boxes). For this particular case the χ was simply assumed equal to the degree of saturation $S_r = 0.94$. For the determined matric suction of $(u_a - u_w) = 60$ kPa (see Table 7.3, the collapse index rises from 2% (slight degree of collapse according to ASTM [10]) evaluating in total stresses to 4% (moderate degree after ASTM [10])). However it should be noted, that for this relatively high degree of saturation also any severe collapsible behaviour should not be detected.

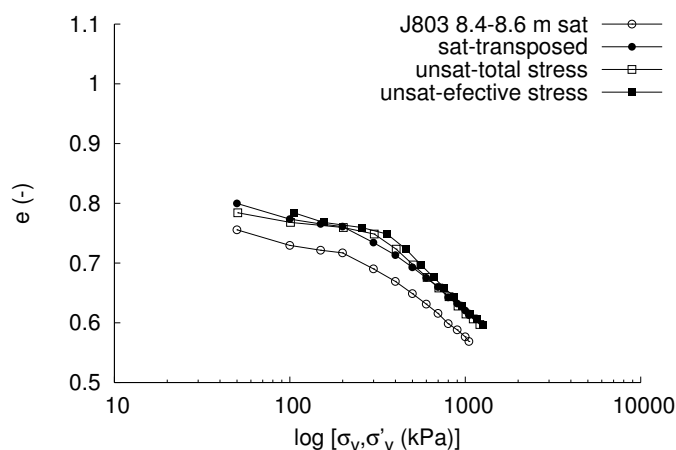


Figure 7.8: Evaluating of double oedometer test in total and effective stresses.

7.3.3 Laboratory tests on lumpy samples

The second set of tests was carried out on the compacted and loose lumpy specimens. The material used for the testing was taken at the Bílina mine fill (see Fig. 3.6) about 4–5 years after filling. In order to carry out the tests in standard types of oedometer rings ($d = 50\text{--}100\text{ mm}$, $h = 20\text{--}30\text{ mm}$) the clay fill material with reduced grain size was used see Sect. 3.2.1. The lumpy structure was maintained, but the differences between the void ratio of the of the natural and sieved material could

Figure 6.11 shows the results. The samples inundated at $\sigma_{ax} = 10\text{ kPa}$ are marked with s. Two sets of the tests with samples of natural water content ($w = 34\%$) were carried out on loose and compacted (marked with c in Fig. 6.11) soil.

Collapse potential vanishes at vertical confining pressures of about $\sigma_{ax} = 1500\text{ kPa}$, where saturated and natural moisture compression lines unite. Above this pressure the slope of NCL of natural moisture specimens changes and so the character of the soil. In the region of $\sigma_{ax} \geq 1500\text{ kPa}$ the interporosity vanishes and the compressibility is controlled just by the intragranular voids. After the sample dismantling at the end of test no obvious structure was visible. The material was homogenized in both natural moisture and saturated states (Fig. 7.10 right).

The degree of saturation and dry density over the applied stress are plotted in Fig. 7.9 for both loose and compacted specimens tested at their natural water content. No change of water content from the initial $w = 34\%$ was assumed for the computation of the degree of saturation. Both specimen reach the degree of saturation of $S_r = 100\%$ at the applied vertical stress of about $\sigma_v = 1500\text{ kPa}$. At this stress the collapse vulnerability and the open macroporosity also vanishes. From this stress, the water content of the specimens also starts changing due to the volumetric compression

of the lumps. The final water contents determined for the dismantled specimens reached $w = 31\text{-}33\%$.

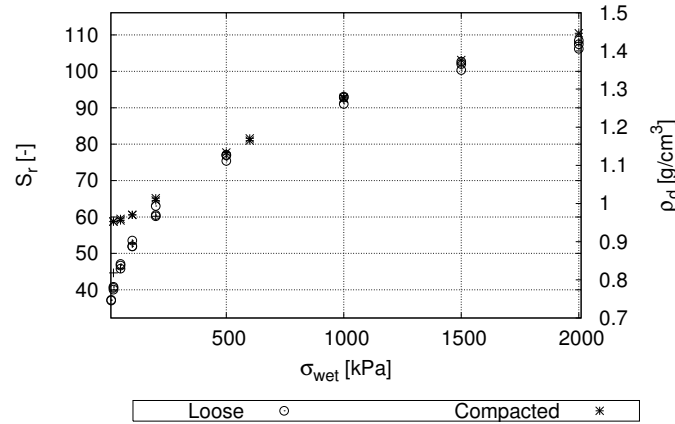


Figure 7.9: Degree of saturation and dry unit weight at different vertical stresses.

To confirm the collapse potential indicated by double oedometer tests the standard method for estimating collapse potential according to ASTM [10] was used. The samples prepared by compaction with natural water content were inundated at $\sigma_{ax} = 200$ kPa (tests no. P01 and P03 in Fig. 7.11). The collapse potential estimated through the standard method fits well to the double oedometer curves, which confirms the severe collapse vulnerability of initially lumpy specimens.

The matric suction which acts in the lumps is high. This was determined on bigger lumps with similar water content using filter paper method, see Sect. 5.1.1. This matric suction reached about 1.2 – 2.4 MPa by water contents at or close to saturation. Similar matric suction is expected also in the intragranular voids of smaller lumps of the same water contents. Matric suction of the lumpy specimens for double oedometer test however could not be determined using this method. The lumpy specimens have air dry macrovoids initially, which close during loading. The matric suction at the scale of macrovoids is low at smaller stress levels and grows on the contacts of lumps as the macrovoids get smaller. The application of the filter paper on this lumpy specimens would thus be influenced by suctions acting in the intragranular voids as well as in the intergranular voids. The possible poor contact between the macrovoids and filter paper would further make this measurement inaccurate.

7.3.4 Evaluation of the collapse potential of clay fills

Collapse potentials of lumpy and undisturbed specimens were evaluated using the equation 7.2 and plot against the applied normal loads in Fig. 7.12. Only the

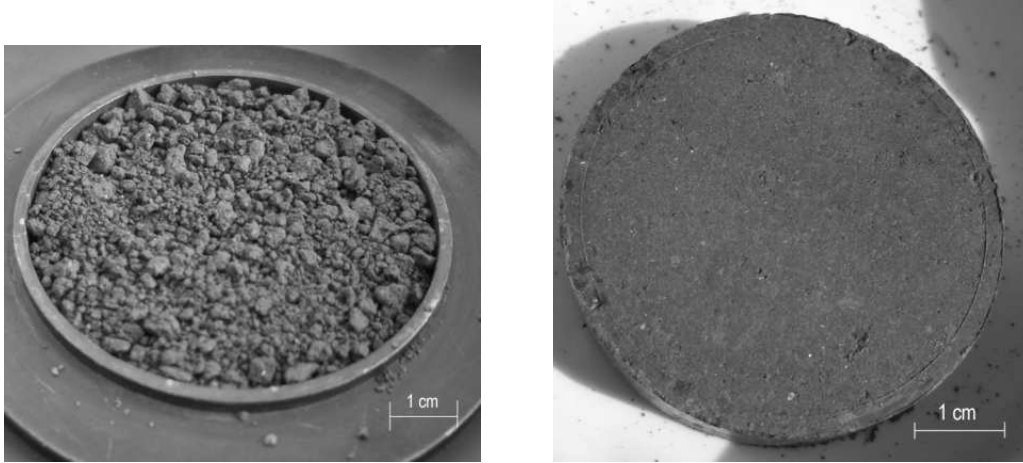


Figure 7.10: The lumpy material before the test (left) and after the test (right).

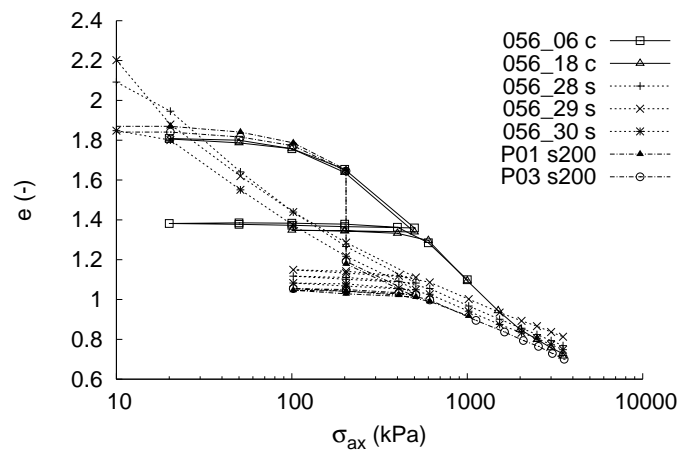


Figure 7.11: Double oedometer tests on compacted lumpy specimens and collapse under wetting at 200 kPa.

maximal collapse potentials at the applied stress were plotted for the undisturbed specimens (from Table 7.2).

The collapse potential was estimated with its maximum of $I_c = 20.2\%$ at $\sigma_{ax} = 50$ kPa for loose specimens and $I_c = 14.3\%$ at $\sigma_{ax} = 200$ kPa for initially compacted specimens, which indicates severe degree of collapse potential according to ASTM [10]. The collapse potential of compacted lumpy specimens grows up to the stresses of 200 kPa, then it starts decreasing, while collapse potential for loose specimen decreases with increasing load. From this stress the collapse potential of both loose and compacted specimen unites and has a decreasing tendency. The obvious scatter of the collapse potential for the undisturbed specimens from the aged fill of former 5. květen mine is attributed to the inhomogeneity of the sampled soil (discussed in previous Section).

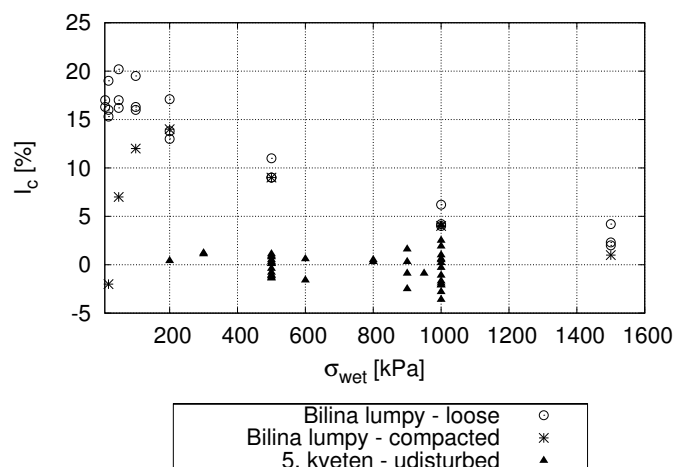


Figure 7.12: Collapse potential for the lumpy and undisturbed soil.

Table 7.4: Initial dry density and degree of saturation for the clay fill specimens.

Site	Specimen type	Init. dry Density ρ_d [g/cm ³]	Init. Degree of Saturation S_r [%]	Dry Density at max I_c $\rho_{dI_{cmax}}$ [g/cm ³]	Degree of Sat. at max I_c $S_{rI_{cmax}}$ [%]
Bílina	loose	0.73-0.75	34 – 36	0.73–0.74	34 – 35
Bílina	compacted	0.96 - 0.98	58	1.02	65
5. květen	undisturbed	1.57-1.87	83 - 96	1.63-1.95	90-100

The studied lumps consist of a high plasticity clay, which is not a typically collapse susceptible. But the dry-way filling of the lumps is responsible for the open

7.3. COLLAPSE BEHAVIOUR OF CLAY FILLS

metastable structure containing macrovoids. Their size is bigger than the size of macrovoids of a typical collapse susceptible soils. Therefore, the collapse potentials of the lumpy specimens were compared with the typical threshold values for sand and low plasticity soils determined in previous Section, see Figs. 7.13 and 7.14. The initial dry densities and degrees of saturation of the clay fill specimens are summarized in Table 7.4. The collapse potential for loose lumpy specimens surpasses the maximal collapse potentials of both soil types. Collapse potential of sands vanishes at much smaller stresses than of the lumpy specimens. The collapse potential of the lumpy specimens compacted to the initial dry density of $\rho_d = 0.96 - 0.98 \text{ g/cm}^3$ corresponds better to the collapse potential of low plasticity clay at the degree of saturation $S_r < 40\%$ and dry density of $\rho_d < 1.5 \text{ g/cm}^3$. This trend is more probable for the behaviour of the freshly filled lumpy clay in-situ. Due to the larger size of lumps and lightly compacted state resulting from the free fall during filling, the bulk densities of the freshly filled soil in the Bílina mine reaches approximately $\rho = 1.3 \text{ g/cm}^3$ (Dr. Mach personal communication). Assuming the initial water content of the filled lumps $w = 0.3-0.34$, the initial dry density reaches thus $\rho_d = 0.97 - 1.0 \text{ g/cm}^3$ and corresponds with the dry densities of the compacted specimens.

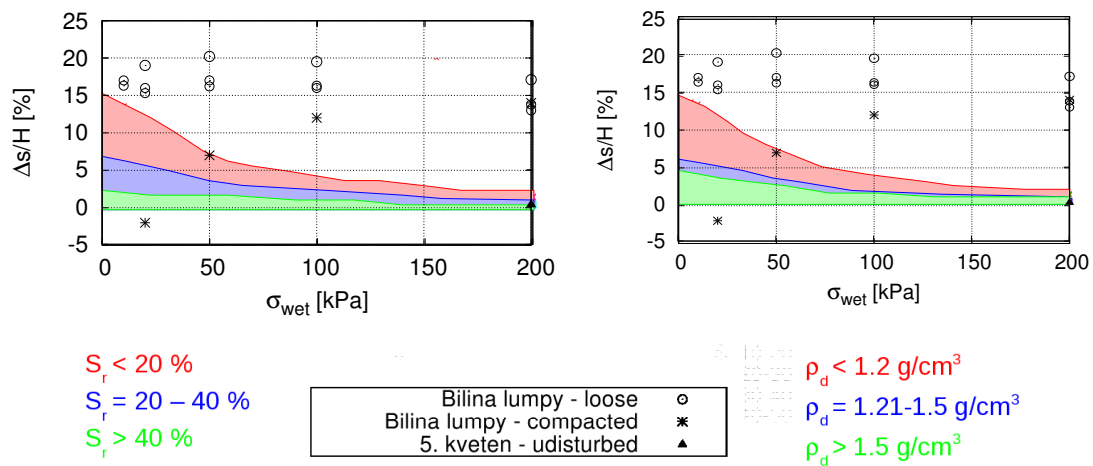


Figure 7.13: Collapse potential for the lumpy and undisturbed specimens in comparison with typical thresholds for S_r and ρ_d for sand.

A severe collapsibility was indicated by the tests on lumpy specimens contrary to the negligible collapse potential determined on undisturbed specimens. One possible explanation is that the fill of former 5. květen mine is in a high degree of homogenization so the intervoids are already closed up and the lumpy structure vanished in the whole profile due to suction cycles, overburden and possible past collapse under wetting. All undisturbed specimens have also high degrees of saturation indicating

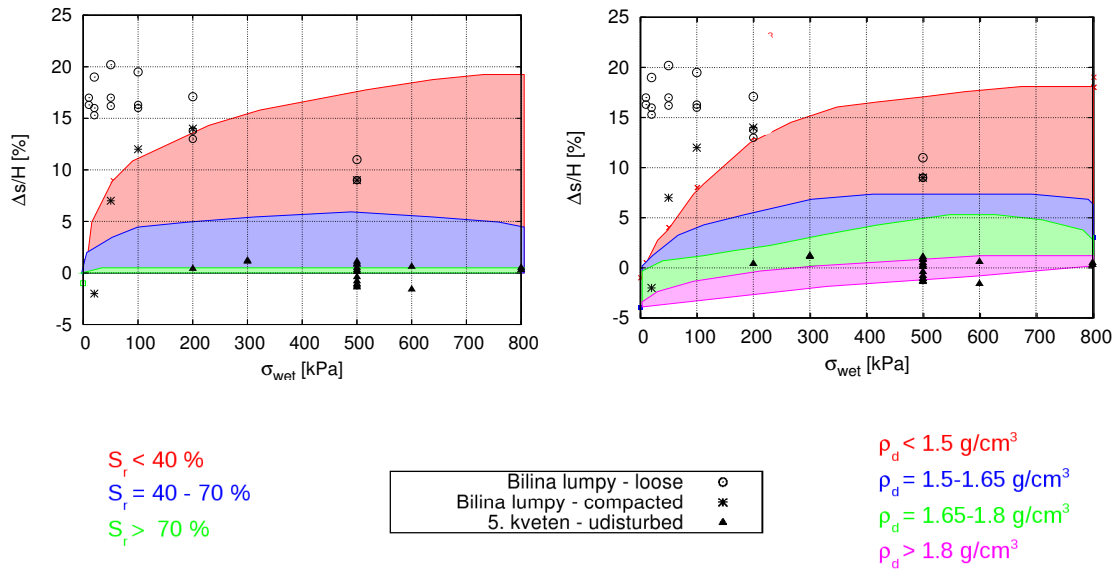


Figure 7.14: Collapse potential for the lumpy and undisturbed specimens in comparison with typical thresholds for S_r and ρ_d for low plasticity clay and loess.

low collapse risk. There was also no suction present in the field (see Sect. 4). On the other hand the significant scatter in initial void ratios of undisturbed samples indicates some remaining nonhomogeneity of the material. Therefore any extrapolation to other sites can be misleading, a careful re-consideration of collapse potential is recommended for any future site-investigation.

7.4 Summary

- The thresholds for the two types of typical collapse susceptible soils are based on review study of journal papers, national and international proceedings where a complete information about: soil type, initial dry unit weight (or equivalently void ratio) before flooding, degree of saturation deformation (or equivalently void ratio or dry unit weight) after flooding and stress level by flooding was required.
- The typical collapse potential depending on the applied load during flooding are defined for the different degrees of saturation and dry unit weights for two typical collapse susceptible soils.
- On the undisturbed samples from the aged clay fill of 5. kveten a negligible or no collapse potential was determined using double oedometer tests. There was also negligible matric suction recorded using filter paper method, which was

presumably caused by unloading during sampling. Almost all specimens were nearly or fully saturated at their natural state, which explains the negligible collapse potential for this site.

- Evaluating the collapse potential of the undisturbed specimen using the Bishop's equation for effective stresses, the determined collapse potential rises from 2% to 4%. Nevertheless no high collapse potentials should be expected, due to the high degree of saturation.
- For the material in question large scale experiments should be preferred. Standard laboratory testing may not capture the double porosity character of the landfill material. However high degrees of saturation in combination with no suctions monitored in situ and also the age of about 40 years of this clay fill minimizes the probability of the collapse under wetting for this site.
- A severe degree of collapse potential (according to ASTM [10]) was indicated for both loosely filled and compacted lumpy specimens, prepared from the lumps at natural water content sampled at the mine Bílina. The collapse potential reaches its maximum at 50 kPa or 200 kPa respectively.
- Comparing the collapse potential of lumpy specimens with the typical collapse potentials for sands and low plasticity clays, a more severe collapsibility was measured for both loose and dense lumpy specimens than for "standard" collapsible soils. The compacted specimens show increasing trend of collapse potential up to 200 kPa, then it drops in the same manner as for loose specimens. At very low stresses, up to 30 kPa, swelling was observed for compacted specimens. A similar trend can be expected in-situ, where the soil is also slightly compacted due to the free-fall energy only. However the influence of the bigger lumps should be taken into account.

Chapter 8

Conclusions and outlook

8.1 Conclusions

Structural transitions of clay fill soils have been studied in this thesis. There are two main agents that degrade the original open structure: overburden, and a drop in matric suction. Water infiltrating into the bodies of clay fills is responsible for a decrease in matric suction and contributes to changes in the structure and mechanical behaviour of clay fills.

Long-term monitoring of matric suction at two reclaimed clay fills of different age revealed two different trends. Seasonally dependent suction oscillations were monitored at and below a moderate slope on the site of Bílina mine, whereas no suction throughout the period was detected at the former 5. květen mine. This is attributed to different flow regimes. At the clay fill of the Bílina mine, the groundwater table is deep, and thus the pore water pressures at shallow depths reflect the seasonal moisture oscillations. An inverse analysis of the vertical flow in the unsaturated zone using the concept of dual permeability resulted in vertical profiles of matric suction depending on season. High matric suction and suction oscillations at shallow depths were successfully simulated.

The investigations of the mechanical behaviour of the lumpy clayey material show a complex nature involving various phenomena. The transition from the initially granular structure of the clay fill into compacted clay depends on stress, saturation and time. A comparison of standard and large scale tests showed that there is an important size effect that should be taken into account when characterizing this material. The low strength of the clay lumps in comparison with rock fills contributes significantly to the changes in the clay fill structure. The strength of the lumps is influenced by overconsolidation, matric suction and in some cases by diagenetic

8.1. CONCLUSIONS

bonding. About 3 - 4 months after filling, the matric suction of almost saturated or fully saturated lumps reached 1.2 - 2.4 MPa.

The shear strength of the lumpy material depends strongly on the stress level. In addition, water flooding decreases the intragranular suction and consequently the strength of the lumps. An open structure containing macrovoids is formed from loose and compacted lumps at their natural water content. The compression line of these specimens lies above the normal compression line of the reconstituted soil within a certain pressure range. The change in the slope of the normal compression line of the lumpy specimens at stresses higher than 1500 kPa indicates closure of the macrovoids and homogenization of the clay material. At these stresses, the compression lines of the lumpy and reconstituted specimens unite, which also shows that the lumpy structure vanishes. At greater depths, the overburden causes a decrease in the total porosity of the material. In addition, the creep behaviour of specimens with a natural water content shows a kind of secondary creep or collapse after some period of time.

The initially loose lumpy structure of clay fills remains open, particularly at small depths. At small depths, the clay fill soil is highly collapse-susceptible, mainly in the early stages after landfilling. On wetting, the high intravoids suction vanishes, the lumps become softer, and the structure collapses. The collapsibility of the lumpy soil was investigated by double oedometer tests. The collapse potential of the scaled-down lumps with a natural water content was higher than the typical collapse potentials of fine-grained and coarse-grained soils with a similar water content, and indicated severe collapsibility. At stresses about 1500 kPa, the collapse potential for both loose and compacted specimens vanished. The compacted lumpy soil showed a swelling response to wetting for low stresses, and collapses at loads of 50 kPa and more. The collapsible behaviour of the compacted specimens is expected to express more closely the behaviour of the soil in situ. This soil is also lightly compacted during the filling process due to free-fall of lumps. In general, collapse settlements close the open macrovoids, and this results in a clay fill structure with a denser state.

However, even under low surcharge pressures, the lumpy material changes back into more homogeneous soil, and the suction oscillations due to weather are considered responsible for this. Weather-induced wetting-drying cycles cause repeated changes in matric suction, which is responsible for the extra strength of the clayey lumps. A decrease in suction contributes to the destruction of the clayey lumps and to subsequent filling of the macrovoids with finer disintegrated lumps. It has been shown that lumps subjected to wetting-drying cycles disintegrate, while lumps flooded at their natural water content remain intact. A single air-drying and subsequent flood-

ing event was sufficient to break the lump structure. In this case, the lump was dried out to a water content below its air entry value. High hydraulic gradients resulting from the difference between the matric suction in the lump and the surrounding water under atmospheric conditions were responsible for the disintegration. Disintegration due to weathering effects can be hindered by diagenetic strengthening and overconsolidation of the original clay.

A comparison of the void ratios and compressibilities of saturated undisturbed, lumpy and reconstituted specimens from the aged clay fill of the former 5. květen mine shows that lower void ratios and a stiffer response were measured for the undisturbed specimens. This is attributed to the high degree of homogenization and intervoid closure for this clay fill, which had been in place for about 40 years when it was sampled. Two different trends in compressibility were further detected when comparing scaled-down saturated lumps of similar size distribution from the two sites. The compression lines of the lumps from Bílina mine, flooded at natural water content, lie below the NCL of reconstituted soil. However the compression lines of the lumpy soil from 5. květen mine, prepared by crushing from air-dried soil and consolidation in a mini-centrifuge, lie above the reconstituted soil in a certain pressure range. This difference is probably due to the lower void ratio and the higher strength of the lumps from Bílina mine, which did not experience any air-drying. The two soils also show a different initial void ratio and different compressibility of the reconstituted clay. This is attributed to the character of the original clay.

Lumps deposited at their natural water content at a sufficient depth inside the fill are unlikely to experience any drying below their air-entry value. Any flooding in the future will soften the lumps and lump contacts, resulting in collapsible behaviour and macrovoid closure. However, stronger and bigger lumps and lump centres can persist intact when saturated, and this contributes to the lower total porosity of the collapsed clay fill structure. The amount of macrovoids and also the original void ratio and the strength of the lumps are the factors influencing the total void ratio of the clay fill. Consequently, a clay fill structure which experiences a single saturation event, and thus collapses, can exist at a lower void ratio than the reconstituted soil, and can thus have a stiffer response to loading.

The mechanical behaviour of the filled lumps varies. A clay fill structure formed from stiffer lumps with natural water content requires a higher overburden pressure to close the macrovoids than a structure formed from weaker lumps. A record of the excavating depth, the void ratio and the water content of the filled lumps would provide useful information for modelling and for the future development of the fills.

8.2 Outlook

The inhomogeneity and the anisotropy of clay fill structures make their behaviour more difficult to describe. Heaps with a high angle of repose form structural elements in the fills. Denser centres of heaps are formed due to the slight compaction energy of the free fall of lumps, while looser slopes and toes develop, where bigger lumps roll down and are deposited. The anisotropy of the lumps is due to their sedimentary structure. Lumps tend to separate along the sedimentary clay layers. An investigation of these effects would enhance the investigation of uneven settlements of fills. For example Herštus et al. [66] developed a device for monitoring the anisotropy stress and the strain tensor in clay fills. This device could be installed in different positions of the heaps to record a possible different response to loading of the centres and the toes of the heaps, respectively.

Collapsibility is a severe problem when dealing with fills that have been filled in a dry-way. Double oedometer tests of standard size offer a useful tool for evaluating the collapse potential over a wide range of loads. However, large scale or field testing should supplement the laboratory investigation and evaluate the effects of scaling of the lumps and the clay fill inhomogeneity. A ponding test recommended by Ehwany and Houston [38] offers a method for the in-situ evaluation of collapsibility. A record of the position of the groundwater table and the flooding history are necessary for an assessment of the risk of collapse. Fills which have not experience any saturation in the past should be treated as collapse-susceptible. During ageing in a humid climate, clay fills experience wetting and this reduces the collapse risk. However, there can be some positions which were not saturated in the past even in aged clay fills. Charles and Watts [27] recommend controlled infiltration as a method for stabilizing fills.

Only two states of lumpy soil were considered in this thesis: at the natural water content, and saturated. However, the clay fill soil often experiences partial saturation. According to Větrovský [132], all precipitation either evaporates or infiltrates into the bodies of freshly dumped fills. A clay fill that is partially wetted experiences a drop in matric suction, and can partially collapse. As advised by Houston et al. [67], for soils where partial wetting due to infiltration is expected, it is necessary to study the collapse of the lumpy soil subjected to partial wetting resulting in different degrees of saturation.

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