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My thoughts about prof. Dr. Imre Szabó in his 70th birthday

In 1962 he enrolled to the Faculty of Mining Engineering at the Technical University for Heavy Industry, without having any miners among his ancestors. The reason of doing so could rather be the proximity of the University and the desire to become an engineer. This made me possible to come in contact and to work with him for 50 years. I could mind the unfoldment and brilliance of a wise and rather pessimistic man.

I got to know him as a calm and silent man, who, even during his student years, spoke only if needed. Nonetheless his remarks always suggested an individual and clever insight. While he faded into the class hardly noticeably, despite the lot of help given to his colleagues, he gained much attention on the exams owing to his thorough preparedness.

After graduating with excellent results, despite his "unfavorable" lineage, he had been employed to the Department, where he carried on his work silently. He did not push to excel, but with his dedicated and reliable work, he gained our respect. With his love-worthy personality we got to like him.

I had the pleasure, after a few attempts that Imre accepted to teach the practical classes for some of my subjects. He took the practical classes for hydrogeology, engineering geology and groundwater hydraulics. He assembled the topics of the practices mostly individually, based on the corresponding courses. This deviation from the method of my previous assistants had been almost worrying at first. I have missed the regular consultations and reports about the ongoing practical classes. After a while, based on the feedback from students, I realized that the topics of the practical classes fits very well with the courses and complement each other. Finally I had no reason to modify the resulting thematic; I had been satisfied with the work of Imre.

Later he began to teach subjects individually. His personality and individual mentality have also been reflected in these lectures. The courses previously taught in a conservative manner have been modified by introducing an individual thinking and by shifting the structure of the topics. Generally these modifications had positive effect on teaching.

As soon as he settled down at the Department, he started to look for research topics on which he could work individually, without interfering with his colleagues' field. After a few years his attention turned to soil mechanics and environmental protection. While the majority of constructions are done in overburden sediments, where erosion affects mostly these rocks, a separate discipline started to take shape. The new field, interwoven with constructions, evolved to geotechnics. Imre realized its importance and the new possibilities and in 1971 he enrolled to the foundation engineer post-graduate in Budapest. This brought a new color to our department. As every system may be subdivided in several ways, finally I accepted this new definition of fields of engineering geology, despite my different opinion. Imre set forth well on this path, in 1990 he became a board member in the Geotechnical Section of the Hungarian Chamber of Engineers.

Studying the possibilities of waste disposal from the point of view of environmental protection, have been another fortunate decision. He could work independently in this field and he managed to create its framework quickly. It became an important pillar of the training at the Department. The success in this field reflected in varied forms. Imre has been a board member in the Environmental Section of the Hungarian Chamber of Engineers since 2000. He has also demonstrated his experience in this subject in the book written together with his son Attila Szabó.

He compiled the complete syllabus and textbooks for the newly introduced subjects. He had been the Head of Department for several years as well as my principal. He had been a silent and calm head, who had instructed the teaching staff only when it was necessary, allowing everyone to administer their subjects considering the stated guidelines. Every issue had been arranged with a peaceful attitude; we could hardly notice his continuous guidance.

Imre is one of those scientists who place a particular emphasis on empirical research. He gained an outstanding experience of laboratory work over the years. He designed new laboratory equipments which made possible to acquire new knowledge and achieve more accurate results in the fields of geotechnics and soil mechanics.

When new methods had to be elaborated he usually based it on experimental studies. This fortunate approach helped him to solve many difficult tasks, which would be very hard to do on a theoretical basis, not to mention that analytical solutions also relies on parameters measured experimentally.

Our Department had a student exchange program with the Institute of Geology of the Gubkin Russian State University of Oil and Gas for several years. Every summer a group of 17 students and 3 faculty members had the possibility to visit the partner institute. While the Gubkin University was represented by prominent teachers, we could hardly manage to find three willing delegates. At one time I had been chosen, accompanied by Katalin Molnár and Imre Szabó. Since Imre had been there before, he only accepted the new delegation as a favor to me.

We spent the first week of our stay at the Lomonosov Moscow State University, by visiting the petrological and mineralogical museum. For the other two weeks we went out to the Caucasus for field trip. A prepared campsite had been waiting for us. In the first few days everything soaked in a huge rainfall, the water being pooled up in our tent. Thanks to Imre, we could accommodate in a school and dry our goods. One day we visited Tbilisi, where we benefited again the previous experiences of Imre. He remembered that the young Georgians are willing to do anything for a pair of denim trousers. We spent a pleasant rest day there.

Neither our travel back home went without troubles. Our hosts mistakenly booked ticket for a wrong train. We had to change train in Kiev, which went only to Chop, where we had to walk across the border and wait for another train.

The educational activity of Imre extends far beyond our university. He teaches waste disposal and waste management courses also at the University of Debrecen and the Eötvös Lóránd University. He is engaged in post-graduate engineering training and teaches at the post-graduate engineering specialist training organized together with the University of Pannonia (Veszprém).

His personality, humanity as well as his theoretical and practical knowledge attracts many students for thesis work.

He never aspired to maximalism, indeed he always offered the essence of the material in a manner that the students may remember for a lifetime. He is a particular, but outstanding teacher, with a general educational attitude towards the students. He is friendly and informal in the classes and can be sociable towards the less likeable students too. Thanks to his modesty he got many friends.

Beside his work at the university, Imre went in for sports in the last few decades. He started fencing at the secondary school. He did it successfully, participated on tournaments. When his work did not allow regular training and tournaments he started to play tennis at the court of the university. Skiing had been his third passion. He spent a few weeks at the ski resorts of Slovakia and Austria every year, but he often went out to the Bükk Mountains too. He particularly loved the weekends spent with his friends at the resorts around Kosice. He always took a great care not to overdo sports at the expense of his work.

Imre's love for his family is noteworthy. He keeps in touch with and visits his family, living in Germany on a regular basis. His son, following his footsteps, chose the profession of waste disposal design. Despite his independence, he always takes his father's advice. This kind of professional cooperation within a family is not common and reflects the outstanding educational attitude of Imre. Its success may be also being clear from the book written together about waste disposal.

I had the possibility to personally make certain that Imre is not only a good father but also was a good husband. He met and fell in love with his wife, Ágnes, at the university, where they had been classmates. He supported her through thick and thin, even when life raised difficulties. Ágnes worked at a design office, where bright future awaited her, but after the hopeful beginning she got ill. She became exhausted and her mobility got limited. She had to go hospital frequently and needed more and more care. Imre supported her in everything with devoted love, but the end could not be avoided.

It was hard to adapt to the new and painful situation. Imre slowly put himself together and buried himself in his work at the university. His work did not suffer much for his personal tragedy. We hope he can still support his students and the Department in the future many more years. Furthermore we count on his work in Hungarian Academy of Sciences, Hungarian Chamber of Engineers, Hungarian Standards Institution, International Society of Rock Mechanics, International Mine Water Association, ISSMGTE Hungarian National Committee, TC-5 Environmental Geotechnics.

According to his colleagues, Imre besides still playing sports regularly, travels a lot and visits museums and exhibitions. His wife embroidered beautiful pieces. I had the opportunity to see a few of them and still keep their photo. After Ágnes fell ill, their mobility had been reduced, now Imre tries to fulfill his need for arts.

On Behalf of myself and the colleagues of the department and his friends, I ask God's rich blessing on him, to grant him many happy busy working years, and peaceful elderly ages among his children and grandchildren.

Prof. em. Dr. József Juhász

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70th Anniversary of Imre Szabó

Dear Professor Szabó,
Dear Imre,

I am recalling pleasant memories while preparing for your 70-year-old birthday. 70 – really? Yes, 70 = 2x35 and you still work with the energy of two 35-year-old men. This energy comes from your sporty life: you fenced in the Hungarian Fencing Team and you still play tennis and also go skiing.

Your outdoor life reminds me of Géza Petrich, our descriptive geometry professor, who rode his bicycle surround the buildings of the University 50 years ago. On the other hand he was that person who told me off at the exam because I did not wear a tie. Your "gracefulness" is the other side. At your exams "it was the brain not the hair which mattered" – refer to our beloved *Illés* song. I also experienced at the state examinations that you ask and talk with the candidates so thoroughly as Professor Károly Széchy. He did not give mark 5 to many people at the Earthwork-Groundwork professional engineer exam, except for you. (I was proud of mark 4 as well.)

We have been working together since 25 years in the Presidency of the Environmental Department of the Hungarian Engineer's Chamber. At the foundation of the Society we wanted to reach engineering self-disposal and to strengthen the professional knowledge. Not the personal department of the ministries would decide, who what knows, who what kind of design-expert authority would get. Already at that time we have planned the compulsory 'postgraduate studies', what is very important for those who are dealing with environment protection, because among the new disciplines appeared very new scientific results.

I remember what excellent knowledge; experiences from abroad gave you to the students at conferences on environment protection, at graduate courses on construction, management, re-cultivation of waste-depots. You took part in the expert group invited by the Ministry at the cyanide waste at Tisza River and at the red-dross damage at Ajka.

As to transfer your knowledge you have published a lot of scientific articles, lecture notes, books. On behalf of the 'waste management experts' I would like to thank you especially for your book on 'Re-cultivation and Post-care of Waste-depots'.

The chapter on the 'Re-cultivation' in your book contains a lot of practical issue, such as: the measuring of slop-stability which needs geotechnical knowledge, the calculation of hydrogeological/chemical waste-spreading, the building structure knowledge of civil engineer, the organic process of decomposition for biologist's and chemist's knowledge. This represents the complexity and the required scientific knowledge of the environmental engineers.

The lawful background (authority 'thinking') is especially praiseworthy. We, environmental engineers – as 100 years ago the mechanical engineers did – have to acknowledge, that because for the sake of the public interest, the main elements of the safety have to be taken into laws, therefore the Authority will send back the plan-documentation – however it is technologically excellent – to the designing engineer, if the documentation doesn't suit to the formal requirements.

We entrusted you with the hydrogeological planning of the new part of the waste landfill of Aszód and the examination of the slope stability. Your theoretical knowledge and large practical experience, the conscientious soil-mechanical measurements of the

University of Miskolc will be very useful for the Saubermacher firm in the next 20 years; several hundred million HUF will be the reduction of its expenses. (In the former 20 years the waste-landfill was built and maintained by the plans of the well-known institution MÉLYÉPTERV).

Another new, very exciting, with large expenses, special field was (and is today too) the environmental damage-exemption of polluted territories. During the planning the common knowledge of the hydrogeologist, chemist, biologist and hydraulic engineer is required. During the execution it is necessary the sewage purification–civil engineer–waste management expert, contractor background.

I remember when 25 years ago we were surprised that the 'Darcy-type' coefficient is valid for water, but for TPH not, and moreover at chlorinated carbon-hydrogen another streaming model is necessary. After the finding of the illegal waste-depots of the LEHEL-Jászberény refrigerator factory – following the miners –we have discovered the geophysics.

Dear Imre,

On behalf of the practicing environmental engineers I would like to thank you very much for your exemplary life, for your profession, for your unselfish and conscientious teaching. Your ingenious research, engineering designing work carried out in the new engineering field will always serve as an example for the future generations. Trusting in those who will follow us – like your son, who you can be proud of – I wish you with an old salutation from "Selmecbánya":

"Glück auf!"

Elemér Gilyén

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ENVIRONMENTAL GEOTECHNICS ENERGIES – "ENERGIEWENDE" in Germany = Deep Geothermal Energy=

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We are delighted to serve with a contribution to the volume to honor

Prof. Dr. Imre Szabó

for his 70th birthday.

ABSTRACT

We congratulate Imre Szabó for his outstanding work in the broad and complex field of Geotechnics which is underlined by his published books, and we want to thank for the close and intensive cooperation between our institutes and universities over the years.

Knowing about his interest in environmental geotechnics – beside all the basics in geotechnical engineering – we have chosen the subject "deep geothermal energy" as part of the renewables.

This paper is based on a major research project funded by German Federal Ministry of Environment dealing with the stimulation in a Malm-layer in about 4000 m depth with the purpose to generate electrical power and heat. *"... energy is always exciting"* **-** Renewable Energies: - German concept – replacing fossil energy & closing down nuclear power stations.

The major focus of the present study is on hydraulic stimulation treatments where fluids are injected into a formation at high pressure such that a fracture initiates and propagates into the formation. The generated fracture may maintain a high permeability due to the additional injection of propping agents (e.g. sand or ceramics).

One can also distinguish between tensile fracturing and shear stimulation based on the stimulation mechanism. Tensile fractures are generated due to pressures above the fracturing pressure of the formation.

∗ wind ∗ photovoltaic ∗ hydro
∗ biomass ∗ GEOTHERMAL

SITUATION OF GEOTHERMAL ENERGY IN GERMANY

There are 290 000 shallow geothermal projects in Germany. Besides them there are 21 deep geothermal projects operational and 19 deep projects under construction. More than 70 deep projects are now in the planning phase. A huge engineering experience with power plants was gained in Germany. Nevertheless, the geological situation is rather disadvantageous.

We have gained great experience with project management, although, the lack of operational experience in GPP *(Figure 1)* has a certain effect.

In the debate about climate change, CO_2 footprint and measures to influence by the **BEE (German Renewable Energy Federation):** the energy issue is of high priority.

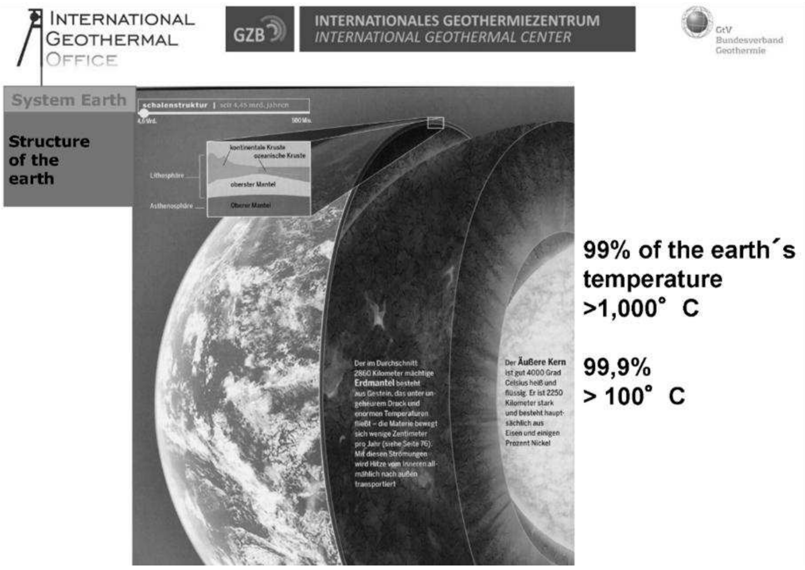

Figure 1. Earth Structure

KEY RECOMMENDATIONS

In the light of recently heightened energy security threats, the European Council and the European Parliament should reconsider the Commission's proposals for an Energy Security Strategy and the 2030 Climate and Energy Policy Framework:

- Europe needs ambitious and mutually reinforcing binding 2030 targets for renewables, energy efficiency and greenhouse gas reduction, underpinned by binding national targets.
- National and EU targets should be explicitly considered as minimum targets. Any one member state willing to accelerate renewables deployment beyond their targets should be encouraged to do so, as this will enhance energy security in the EU on the whole.
- Solidarity measures to support the energy security of particularly vulnerable member states are welcome and necessary. Furthermore, they should be linked to enhanced financial support for local renewables and efficiency measures with close monitoring of target achievements.
- The cost of short-term measures to help reduce the EU's vulnerability should be monitored and taken into consideration when evaluation future investments, particularly when investing in renewables and efficiency as the sole structural and sustainable solutions available.
- Europe needs to develop and agree upon a consistent strategy towards a sustainable energy system, based on domestic renewables and energy efficiency.

With an appropriate regulatory, financial, informational and policy framework behind it, no external power nor uncontrollable event can inhibit Europe from implementing its huge renewables and efficiency potential *(Figure 2)*.

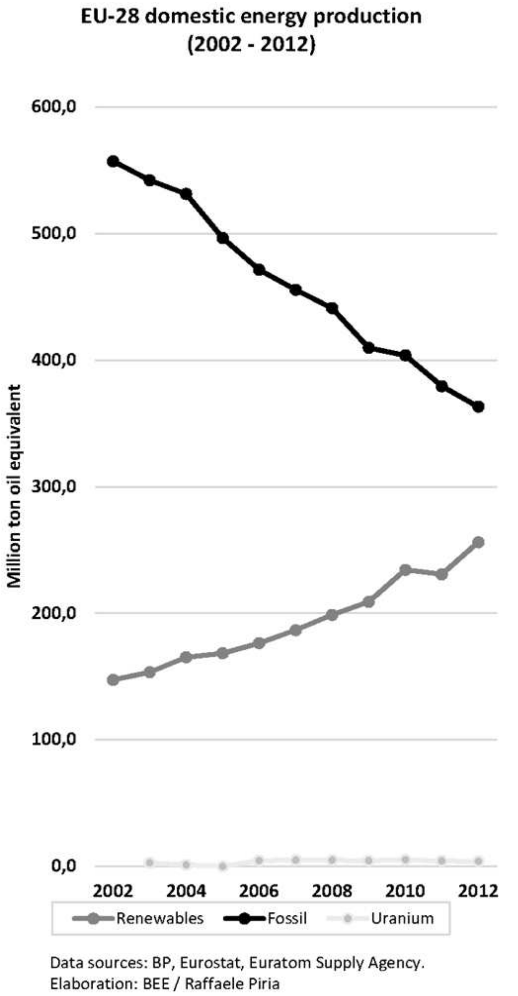

Figure 2. Sources of Energy

KEY ELEMENTS OF ENERGY TRANSITION IN GERMANY

- Nuclear phase should be shut down by 2022.
- Expand renewable energies more swiftly.
- Accelerate expansion of grids and storage facilities.
- Promote energy conservation.
- € 3 billion annually go to energy and climate funds.
- Renovation of old buildings.
- Collaborate on inter-regional and international level.
- GEOTHERMAL ENERGY to generate clean energy is a very potential tool to help to stop global warming.

- Exorka and Daldrup&Söhne AG are our industrial partners in the research project "Allgäu 2.0" – which make use of a deep drilling (4000 m) in Mauerstetten, Bavaria, Germany *(Figure 3)*.

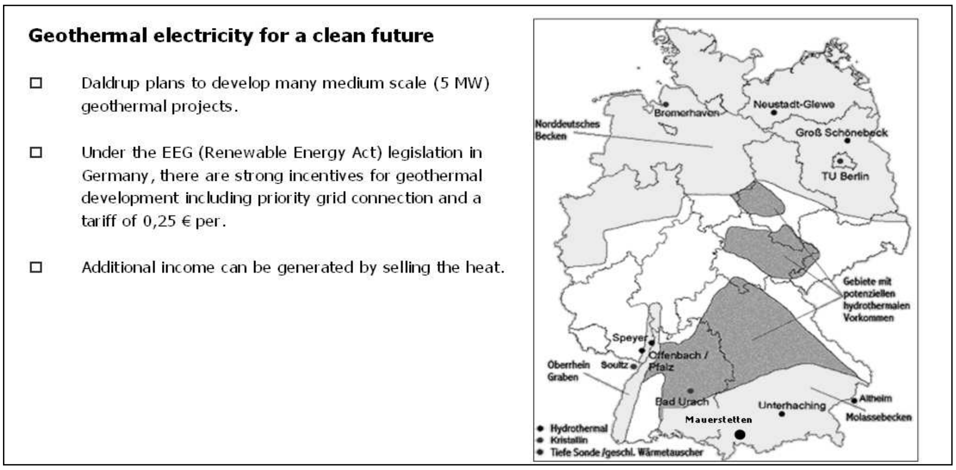

Figure 3. Geothermal Reservoirs in Germany

The structure of a deep geothermal project is outline in the *Figure 4*.

RESEARCH PROJECT "ALLGÄU 2.0"

A research group was built up to cover the interdisciplinary tasks around this first German project dealing with the stimulation in a Malm-layer:

- beside three institutes of TU Bergakademie Freiberg (geotechnics, hydrogeology, fluid mining)
- the GFZ (Deutsches Geoforschungszentrum, Potsdam)
- serveralengineeringcompaniesand
- Exorka, Grünwald andDaldrup& Söhne AG.

Exorka uses a five stage development plan:

- **Exploration:** Collection of all available geological and geophysical data to identify where suitable sites are for geothermal development. Implementation of seismic surveys.
- **Prefeasibility study and financing of drilling:** Planning and permitting. Geology is naturally critical, but of equal importance is ensuring that local regulations and permitting rules are complied with. This phase includes also the well design and procurement.
- **Drilling and testing:** Daldrup's engineers and planners start work many months before a drill rig appears on site. After the drilling extensive well testing takes place. And despite the best geo-science, not all wells are entirely successful.
- **Power plant design and construction:** Once wells have been successfully drilled and tested, Daldrup can start procurement and construction of the Kalina power plant. This phase takes about 18 months.
- **Power plant development and operation:** After completion and commissioning, the power plant and wells will continue to generate clean electricity for fifteen years or more. The electricity is exported to the national high voltage distribution network.

Figure 4. Geothermal Project

Hot water from the Malm limestone formation in the central part of the South German Molasse Basin is utilized by a number of geothermal power plants *(Schneider et al. 2009).* However, in the low permeability western part of the basin geothermal systems need to be engineered in order to become commercial prospects. This study investigates the potential productivity increase resulting from hydraulic stimulation treatments for reservoir conditions typically found in the western part of the basin using commercial hydraulic fracturing software which is routinely used in the hydrocarbon industry and in addition an open source reservoir simulator. The focus of this project is to evaluate the possible dimensions of generated tensile fractures for different reservoir and stimulation parameters and to show their potential to increase the productivity of an exemplary well in that region. One single stimulation treatment does not lead to an economic productivity increase. The most promising ways to stimulate the considered low permeability well sufficiently for geothermal power production are (1) to connect the well to a higher permeable damage zone around a major fault and (2) to perform multiple stimulation treatments. Water circulation through a second well (injection well) additionally increases the productivity of the production well. Under favorable conditions less than 10 parallel hydraulic fractures need to be developed *(Figure 5).*

Planning the utilization of geothermal systems requires the quantification of rock mechanical properties of reservoir rock to secure safe and sustainable reservoir operation. Engineering and geotechnical aspects have to be specialized in terms of reservoir enhanced production from Upper Jurassic carbonate rock in the southwest Bavarian Molasse Basin. Specifically, a well is drilled including sidetrack down to 4000 m depth at the research side Mauerstetten, close to Kaufbeuren. Cuttings were intensively investigated for biostratigraphy whereas drill cores are not taken from the reservoir *(Steiner et al. 2011).* Geomechanical tests were conducted on analogue rock material, sampled extensively on analogue outcrops in the Franconian Alp in the vicinity of Ingolstadt.

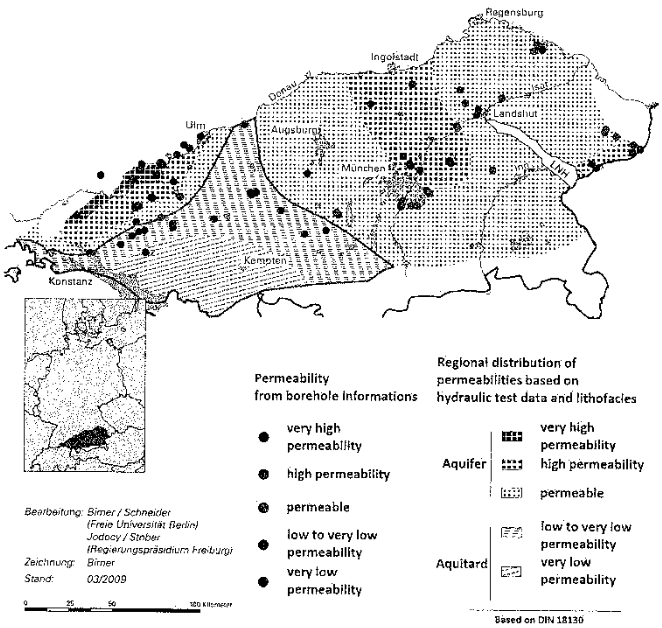

Figure 5. Permeability distribution of the Malm aquifer in the South German Molasse Basin (modified from Fritzer et al., 2010). The low to very low permeability western region is marked by a bold line.

Laboratory analysis on rock samples cover petrographical, petrophysical and geomechanical test. In particular, the reservoir rock is located in 3600 m TVD and has an average density of 2,4 g/cm³ causing vertical stress SV ~85 MPa. The methodological combination from hydraulic and structural geological characterization followed by 3D geological-hydraulically modeling of the reservoir helps to understand the impact of distinct fracture populations during reservoir operation. Results indicate that shear fractures significantly increase the rock permeability hence reservoir production *(Figure 6)*.

Our study is based on an existing geothermal well in the western part of the Molasse Basin. The exact location of the well cannot be given because of confidential reasons. In this well the Malm formation is reached within a depth of approximately 3430–3630 m below the surface. The temperatures between 110 and 130 at this depth are sufficient for electricity generation. An airlift test of the Malm formation conducted in this well showed only an initial productivity index of about 0,5 l/s/MPa which is well below the minimum desired value.

In order to enhance this natural productivity (1) thermal, (2) chemical and (3) hydraulic stimulation methods are the three main technologies.

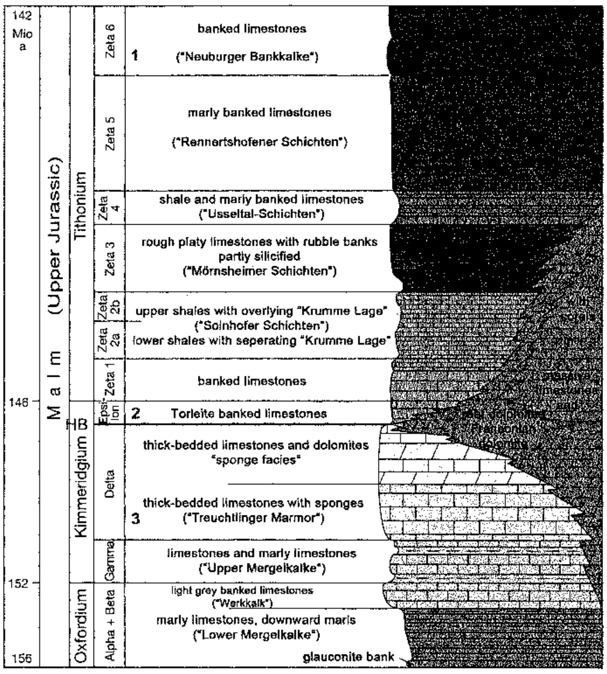

Figure 6. Stratigraphy of the Swabian Mountains of the Upper Jurassic from Malm alpha to Malm Zeta 6 with the Franconian Mountains lithostratigraphic units of the outcrop analogues form the Neuburger Bankkalke (1), Kelheimer Riffkalke (2), and the Treuchtlinger Marmor (3) (after state office for geology and mining Theinland-Pfalz). The hydraulic boundary (HB) divides the stratigraphic units in an Upper Malm from 3430 m TVD to 3631 m TVD (40%) and a Lower Malm from 3631 m TVD to 3932 m TVD (60%). The lithostratigraphic classification of the outcrop analogues of the Franconian Mountains is parallelized with the stratigraphy of the Swabian Alb after Meyer & Schmidt-Kaler (1984).

HYDRAULIC STIMULATION

The major focus of the present study is on hydraulic stimulation treatments where fluids are injected into a formation at high pressures such that a fracture initiates and propagates into the formation. The generated fracture may maintain a high permeability due to the additional injection of propping agents (e.g. sand or ceramics).

One can also distinguish between tensile fracturing and shear stimulation based on the stimulation mechanism. Tensile fractures are generated due to pressures above the fracturing pressure of the formation. These fractures need to be propped open by proppants to create long term permeability. In shear stimulation fluids are pumped at pressures below the fracturing pressure inducting shear slippage on critically oriented pre-existing fractures. Water fracturing and hybrid fracturing treatments are used to exploit the self-propping mechanism. This mechanism is typically assumed to be valid in EGS, but might not always be present.

The development of tensile or shear fractures depends on the state of stress and the orientation of natural fracture sets. For reproducibility reasons we focus on the development of tensile fractures in the low permeability Malm formation in the western part of the basin as one possible stimulation scenario. Tensile fractures could potentially act as hydraulic connections between a low productivity well and a high permeability fracture system. Three options a discussed: (1) the productivity improvement due to the generated tensile fractures alone, (2) the productivity improvement due to a hydraulic connection between an unproductive well and a permeable damage zone of a major fault and (3) the productivity improvement due to a hydraulic connection between the production well and an injection well placed within this higher permeable damage zone.

Finally, the questions will be answered whether hydraulic stimulation (tensile fracturing) is an appropriate method to enhance the productivity of a geothermal reservoir in the Malm karst and if there is a chance to stimulate the considered well sufficiently for geothermal power production.

GEOTECHNICAL INVESTIGATIONS ON MALM-SAMPLES

- Sampling

Figure 7. Sampling by Hilti-drilling and core boxes

- Uniaxial pressure tests

Figure 8. Form of failures after unaxial loading tests, Kelheimer Kalk B. 3-1-A, PK 79

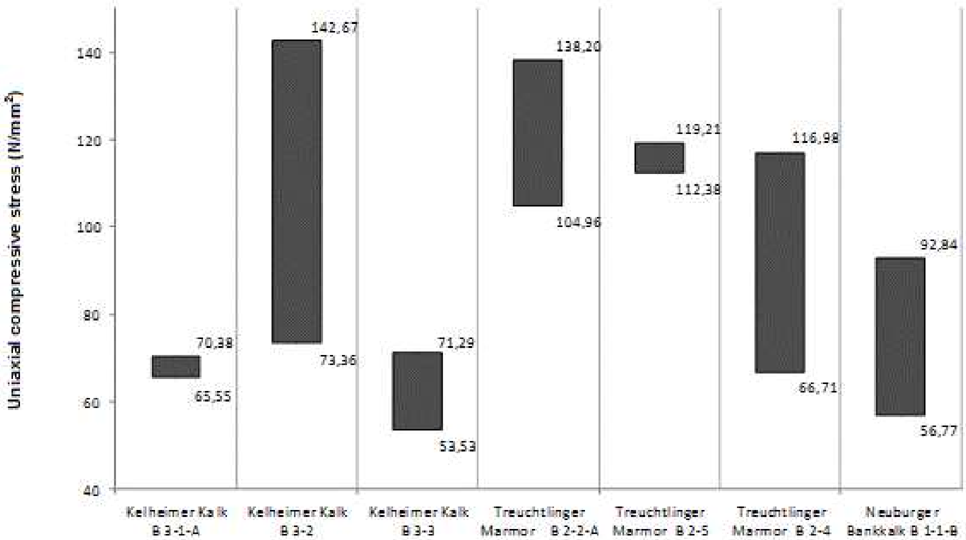

Figure 9. Overview compression strength values in different facies complexes

- Triaxial tests

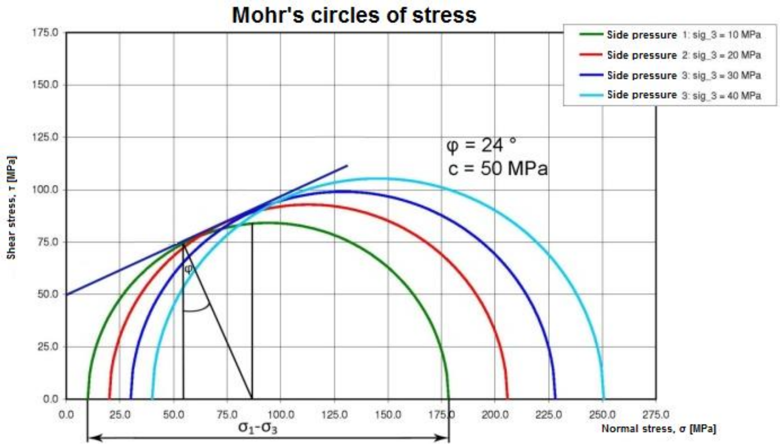

Figure 10. Mohr-Coloumb limit state sample 1 – Neuburger Bankkalk –

Figure 11. Shear failure PK 1 – Neuburger Bankkalk –

- Braziliantests

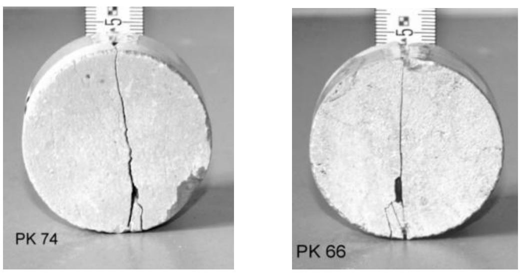

Figure 12. Weakly developed wedges of homogeneous test specimens with micritic structure PK 74 (B. 2–5) and PK 66 (B. 2–4)

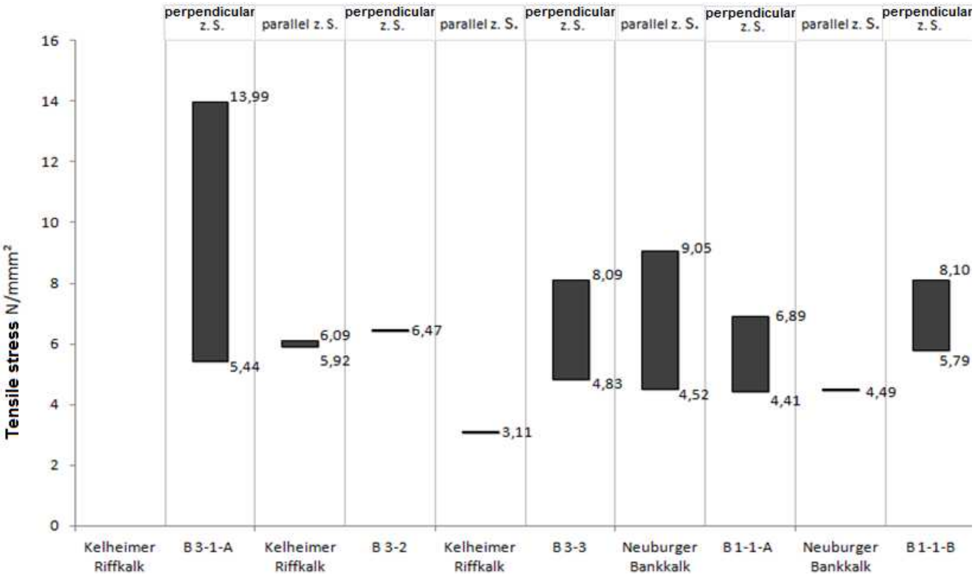

Figure 13. Overview of Brazilian test results of the Kelheimerand Neuburger Bankkalke (z. S.: to the stratification)

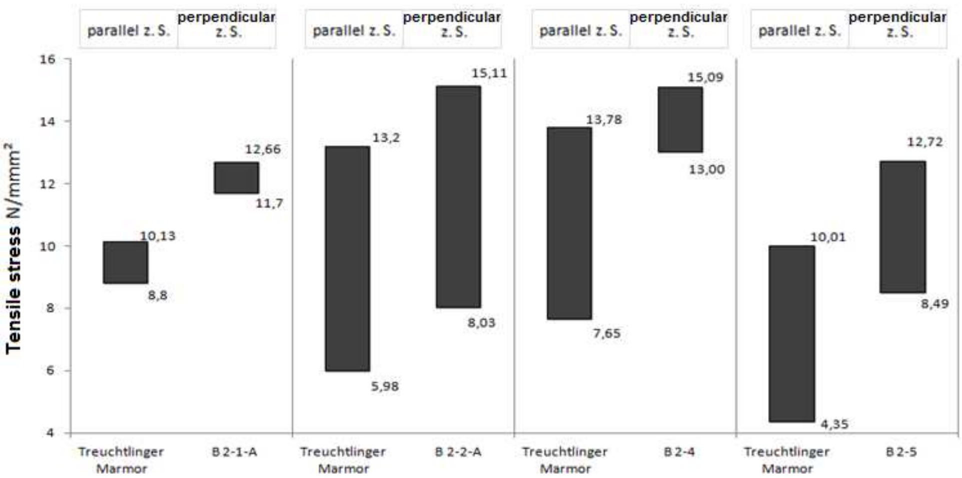

Figure 14. Overview of Brazilian test results of the Treuchtlinger Marmor (z. S.: to the stratification)

- Porosity

- Permeability

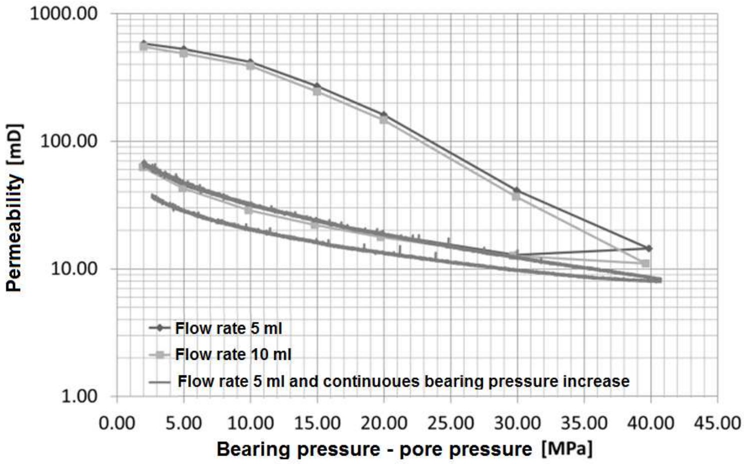

Figure 15. Fracture conductivity depending on confining pressure of Treuchtlinger Marmor

CONCLUSION

The research work is continuing with some delay in the field tests series after the lab-work as well as theoretical investigations.

It should be underlined that the proposed "stimulation-process" is not at all to be seen in the (political) debate in Germany about "fracturing" ...

To bring the geothermal-energy-potential more into the energy-mix, the role of geotechnical engineering is helpful in the interdisciplinary approach with other disciplines beside the dominant role of investors and "politics" by setting "boundary conditions" - like the EEG in Germany.

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ON THE RELATIONSHIP BETWEEN VARIATIONS IN PRECIPITATION AND GROUNDWATER LEVEL

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ABSTRACT

The introductory section of the paper discusses the significance of the relationship between precipitation and groundwater and changes in groundwater level. It also discusses the relationship between the so-called depression area (cone), formed during mining dewatering, and fluctuations in groundwater level under the effect of other influences.

During the research, the data on water level variations of 15 groundwater wells in 10 settlements of the Mátra-Bükkalja region have been analyzed in relation to specific precipitation.

It has been concluded that there is no direct relationship between monthly specific precipitation and the depth of groundwater level in the given month.

During the analysis of the year's monthly groundwater data, it has become clear that groundwater level reaches its maximum in the months of April–May within the year, as a result of the recharging of the ground during the winter and spring months while the monthly precipitation values increase within the year during May and June, and reach their maximum in June –July.

The long-term examination extended beyond a year has revealed that the groundwater level of some wells only reach the maximum after 0.6–1.2 years (8–10 months) depending primarily on the average precipitation level value of the wells. Greater average 'water depth' increases the follow-up (cycle) period; 1.5–3.0 m depth increases it to 0.7–0.9 year and 3–4 m depth to 1.0–1.4 years.

In the wells of Eger, Kerecsend and Kompolt (these three wells are situated within the catchment area of the Stream Eger), where in the 4.5–10.0 m depth range, the average water level of the three wells is approx. 7 meters, the absolute 'delay' period in groundwater level change is 0.7–1.2 years, which is essentially the same as with the other 8 wells.

INTRODUCTION

During the production of utilizable mineral resources, water lifting should be continuously performed during both the opening and preparation of mines (open cuts) and production with respect to cap-rock and dirt bands or wall rock of the coal beds or the coal beds to be exploited, for the purpose of the safety of mining activities, mostly in the course of open-cut and deep working in coal mining. As a result of water lifting from aquifers, the water level will decrease in the affected water table and the cap-rock as well as in the aquifer, depending on the geological, hydrogeological and hydrodynamic properties of the relevant area, the lifted amount of water, its intensity, etc., and a depression area will be formed. In the territory of the so-called cone of depression, in situ tension changes and as a result, movement, or to put it simply, depression occurs in the cap-rocks and the surface.

Naturally, independently from the impact of mining dewatering, the variation in the water content of surface and near-surface layers (or with respect to construction, ground working soils) and the fall or rise of groundwater level are decisively/primarily determined by the shorter or longer term (monthly, seasonal, annual or multi-annual) variation/change in natural precipitation besides the impacts of human activities/mining. The analysis/taking into consideration of the impact influencing/determining the variation ('fluctuation') of groundwater level of natural factors, and primarily that of the fluctuation in precipitation, is of particular significance so that in the investigation of damage caused by mining, the effect reducing groundwater of precipitation variation and that of water lifting in mining may be separated in the areas affected/bordered by mining activities.

With regard to this question, on the basis of the assessment in literature, the rule of thumb according to sources *Kézdi (1954)* and *Kézdi (1970)* is as follows:

'In the upper layer of natural soils, the distribution of ground moisture is influenced by precipitation, temperature and climate conditions, the quality of soil, the position of groundwater, the angle of the site, the vegetation covering the field and several other factors.'

Let us now highlight some practical statements and facts presented in the literature without analyzing such changes in the volume and tension of rocks and soils, in the cast sites and especially in the cap-rock sequence that are due to dewatering or change/decrease in water content, leading to deformation, depression, shrinking or swelling.

Dealing with the issues of groundwater fluctuation, József Juhász identifies the following as a general characteristic: if annual groundwater fluctuation only results from precipitation, then it approximately shows the regularity of temperature variation especially in lack of any considerable onflow or if the groundwater table is not too deep down. In the temperate climate zone of the northern hemisphere, groundwater gauge is highest in the months March to May. The minimum usually occurs in the months September to November *(p. 632. Juhász, 2002).*

The extent of groundwater level fluctuation is very different according to how deep groundwater level is below ground level, and what the permeability of upper soil is. Here, precipitation only influences groundwater level slowly and to a very small extent *(Figure 9, 42, p. 632, Juhász, 2002).*

Among the specialist sources, Zsolt Somosvári's book discusses the characteristics of precipitation, water content/saturation, and ground (clay) features also from the aspect of precipitation- and soil-related damage caused by mining. It states as an 'experiential fact' that water content in the surface and near-surface layers is affected by seasonal and weather changes only to the depths of 1.0–2.5 meters. On the surface, however, a variation of even Δw = 10–20% in water content can occur especially in rocks of fine loose structure, primarily in clays. If groundwater level is higher, the changes in the surface water content are smaller owing to capillary recharge, and with the changes of the seasons and weather, groundwater level also changes.

Considering the basic statements in literature, this paper examines the relationship between variation in time of specific precipitation and groundwater level/fluctuation *(Somosvári, 1990).*

THE GEOGRAPHICAL AREA INVESTIGATED IN THE RESEARCH

In this research, the relationship of precipitation and variations of groundwater level has been examined on the basis of data collected in the Mátra-Bükk, and Mátra-Bükkalja regions. Using the Hydrogeological Annals and the daily data of the National Meteorological Service, we have collected data of precipitation measurement stations in settlements where we have also had access to the water level data of groundwater wells. During the analysis, we have used the monthly and annual average data and examined the variation tendencies of longer periods (5–20 years) on the basis of annual data.

Figure 1 presents the names of settlements, the identification data of precipitation measurement stations and groundwater wells, and average precipitation and groundwater level depth (below ground level) during the studied years adopting the data of the interim research report *(Szűcs, 2012)*, made for Mátrai Erőmű Zrt ('Mátra Power Plant Plc').

THE INFLUENCE OF MONTHLY PRECIPITATION ON THE CURRENT VARIATION OF GROUNDWATER LEVEL

During the analysis of the relationship between the determining/influencing effect of precipitation and groundwater level, the following question arises: can the short term (e.g. monthly) influence of precipitation be realistically assessed, i.e. can it be revealed on the basis of observation/measurement data? While analyzing this short term relationship, the following question also arises, of course: depending on the average value of the groundwater level depth, does specific precipitation have a more direct effect on the variation of groundwater level in the case of lower groundwater depths?

The data of the precipitation measurement stations of the 10 settlements (10 stations) and the related 15 groundwater gauging wells respectively have yielded a total of 2,688 monthly data for 224 years. From this set of data, the data of four wells with significantly different groundwater depths have been selected, namely those of groundwater well No. 002146 in Heves, groundwater well No. 004171 in Eger, groundwater well No. 001841 in Mezőkövesd and groundwater well No. 001829 in Kompolt and the monthly precipitation data of the settlements where the average depth of groundwater level below the ground surface during the research period was 142 cm, 222 cm, 375 cm and 987 cm respectively. The four wells and the related precipitation data have yielded 432 monthly data pairs for 32 years.

On the basis of the 432 monthly data pairs, regarding the monthly precipitation as an independent (x) variable and the monthly groundwater depths as a dependent variable (y), we have analyzed the relationship using conventional statistical methods, and then we have defined regression functions to describe the relationship between the 'variables'. This paper presents four of the figures containing the defined functions and calculated properties, which four have been chosen randomly from among the annual data *(Figures 1–4)*.

To characterize the 'closeness' of relationship between the monthly precipitation and the groundwater level depth measured in the given month, we have used the value the correlation index (coefficient), as is customary. The data and results of the analysis are summarized as follows:

Settlement, well	**Period year**	**Number of data**	**Average groundwater level cm**	**Correlation constant (average)**
Heves well No. 002146	11	132	142	$r^2 = 0.25$
Eger well No. 004171	4	48	222	$r^2 = 0.18$
Mezőkövesd well No. 001841	11	132	375	$r^2 = 0.20$
Kompolt well No. 001829	10	120	987	$r^2 = 0.26$

On the basis of the results of the analysis, and based also on the figures presented, the overall conclusion can be made that no reliably verifiable (r^2>0.7-0.8) relationship can be detected between monthly specific precipitation and monthly groundwater level (depth) (for the same month). With different average groundwater values (142-222-375-987 cm), the regression method has yielded nearly the same results (r^2 = 0.25–0.18–0.20–0.26)

VARIATION OF MONTHLY AVERAGE PRECIPITATION AND GROUNDWATER LEVEL VALUES WITHIN A YEAR

On the basis of the available set of data, the variation within a year of average monthly specific precipitation (mm/year) and average groundwater level (depth below surface) has also been analyzed by settlements and by wells. The process within the year has been demonstrated in figures. The upper part of the figures shows the annual variation of the two properties in the regular calendar pattern, in monthly order from January to December, while the lower part shows the variation in line through the year starting with the winter semester, from November to April.

During the research, the annual data of wells in Füzesabony (2 wells), Heves (2), Ludas (1), Kerecsend (1), Kompolt (1), Eger (2), Egerfarmos (1), Mezőcsát (2), Mezőkeresztes (1) and Mezőkövesd (2) have been displayed. The figures have yielded 'similar' variations in the two parameters within the year, which similarity can be very simply characterized as follows: during the late winter-spring months, the level of the groundwater 'precedes' the increase in precipitation while in (and after) the summer months, groundwater level is reduced and groundwater 'decreases' despite the higher specific precipitation.

Considering the space restrictions here, the changes/variation within a year in average monthly precipitation and groundwater level are presented on the basis of the data of 15 wells, gathered in the period of 1973–2006 and presented in *Figure 5*. Based on *Figure 5*, it can stated that the winter-spring rise of groundwater level and the 'steady' maximum values of it both in individual wells and on the average during April and May precede the increase in monthly precipitation and the occurrence of its maximum in June and July.

The maximum monthly values of precipitation variation and groundwater level within a year are presented in *Table 2.* The first two rows of the table show the absolute maximum monthly precipitation value and its frequency in each month for each of the 13 analyzed wells. (In the case of two wells, the data set is given for a different period.) The aggregate frequency for May–July is 92.3% while it is 100% for June–August.

The absolute maximum values of groundwater level appear in the months of April–August. The individual (monthly) frequency is the highest in April while the maximum monthly frequency of precipitation appears with a three-month delay, in July. The three-month frequency aggregate in groundwater level in April-June is 73.3%, which shows a monthly advance compared to the 92.3% total frequency of precipitation and a two months' advance in relation to the 100% total relative frequency.

About the variation in groundwater level (depth below the surface), or the so-called groundwater fluctuation, professor József Juhász gives the following explanation in his book *(Juhász, 2002)*, referred to above: groundwater level is the highest in the months of March-May while the minimum level sets in during September–November.

Elaborating on the 'summary' data of the 15 wells of the Mátra-Bükk region, analyzed above, the monthly groundwater level variation is illustratively presented in two tables.

In *Table 3*, the first three months of maximum water level (▲) are provided for each well, and the monthly total figures (items) of the 15 wells, and, in addition, the values of relative frequency within 100% are given. The three-month total frequency is 57.8% (> 50%) for March–May, and the maximum three-month total in the area investigated is 73.3%, which appeared in April-June. However, even the total May-July frequency is higher than 50% (51.1%). In 22.2% of the wells examined, even the month of June gives one of the three highest groundwater levels in the year.

According to the frequency data and illustration of *Table 4*, the three-month total frequency of minimum groundwater level, similarly to the data in literature, appears in September–November (51.1%), and according to the data of the area investigated, the groundwater 'shortage' during October–December is of identical frequency (51.1%), as well.

ANALYSIS OF THE RELATIONSHIP BETWEEN PRECIPITATION AND GROUNDWATER VARIATION OVER A LONGER PERIOD (BEYOND A YEAR)

Naturally, the amount of precipitation (during heavy rainfall) and its influence on the changes in groundwater – in years of drought its reducing effect – also means a relationship that extends beyond a year. During the analysis, we have plotted the precipitation and water level data over a longer period of the 12 wells on a time axis, and analyzed the temporal variations of the two parameters. The rising, the constant and the falling sections of the two 'functions' have been drawn 'by hand', and then the same type of sections (points and periods in time) of groundwater level have been paired with the 'time function' of precipitation with a time shift. On the basis of the time difference between the same type of sections of the two 'time functions', less frequently, we have been able to define monthly or several months of delay, and generally a 0.5–1.0–1.5–2.0 year lag or periodic delay both in the rising period (how much later water level changes after increasing precipitation) and in the falling period.

The time functions have enabled us to analyze the data of 11 wells. We have defined the period of variation in precipitation and groundwater level, essentially the delay between the change of groundwater level and the increase and decrease of precipitation.

The data included in *Table 5* are presented in *Figure 6.* Depending on the average groundwater level data of the wells, we have calculated the functions describing the variation in time delay and the indices characterizing the closeness of function relations (D_{dy}, r^2) with a regression method. Considering the data set (11 data) together, it can be concluded that after the occurrence of the maximum precipitation of a given cycle, groundwater recharge occurs with a greater delay (in general) with greater average groundwater depth, and naturally, the decrease in groundwater level also occurs with similar lag following a smaller amount of annual (seasonal) precipitation.

In the case of the 8 wells in the first group, following a period of greater amount of precipitation (more precipitation over 1-2 years), with an average groundwater level of 1.5–2.5 meters (depth), the maximum of groundwater level sets in after 0.6–0.8 years (7–9 months) while with an average groundwater level of 3.5–4.5 meters (depth), the maximum of groundwater level appears with a lag of 1.2–1.4 years (14–17 months). During/after a period of drought, a similar tendency (lag) occurs so the lower (cycle-minimum) water level sets in with delay. In the case of this data set (8 data), the regression characteristics are

$r^2 = 0.64$, the corrected empirical deviation being $D_{dy}/Y_{mean} = 0.17/0.997 = 0.17 = 17\%$, and these indices indicate acceptable reliability.

The 'delay' in groundwater level recharge following precipitation is smaller in the settlements of the second data set (3 data): with an average depth of 4.5 meters, it is 0.5 years (6 months) while with an about 10 meter depth of groundwater level, it is still 'only' 1.0–1.2 years (12–14 months). (The regression figures are not indicative in the case of a small amount of data.)

Referring back to sources *(Kézdi, 1954, 1970)*, in the area of the two data sets, in addition to the groundwater level depth from the ground, the soil properties (e.g. permeability) can be significantly different since the settlements (wells) of the second data set are situated differently from the rest of the wells as they are in the catchment area of the stream Eger.

ACKNOWLEDGEMENT

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Table 1

Identification data of settlements, precipitation measurement stations and groundwater level measurement wells

	Settlement	**Period investigated**	**Precipitation measurement station**	**Average precipitation in years investigated [mm/year]**	**Number of ground-water well**	**Average ground-water level depth below surface [cm]**
1	Füzes-abony	1973–1991	53604 (609)	511	575	339
		1996–2006	53604 (609)	571	003663	406
2	Heves	1996–2006	54200 (588)	573	002146	142
		1996–2006	54200 (588)	573	002144	145
3	Ludas	1973–2001	53502 (553)	523	001827 (638)	349
4	Kerecsend	1996–2006	53603 (596)	539	001808	458
5	Kompolt	1996–2006	53511 (574)	628	001829	987
6	Eger	1996–2006	53207 (598)	650	001809	669
		1996–2001	53207 (598)	676	004171	222
7	Egerfarmos	1997–2006	53700 (633)	569	003949	260
8	Mezőcsát	1973–2006	53802 (690)	533	001817 (585)	238
		1996–2006	53802 (690)	612	001820	253
9	Mező-keresztes	1973–2006	53705 (660)	546	001814 (579)	345
10	Mező-kövesd	1996–2006	53703 (634)	570	001841	375
		1996–2006	53703 (634)	570	001812	345

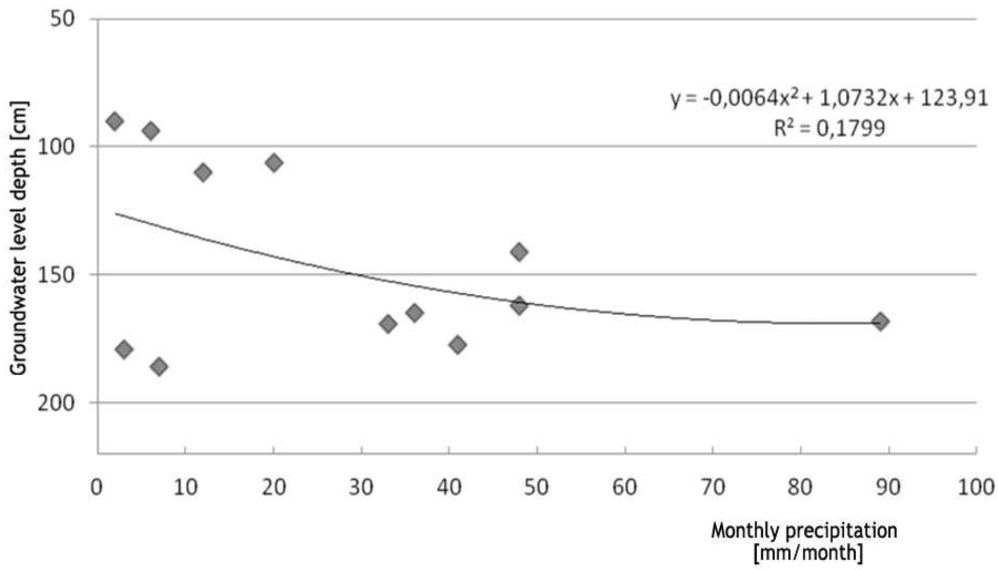

Figure 1. Variation in groundwater level depth as the function of monthly specific precipitation for well No. 002146 in Heves (1997)

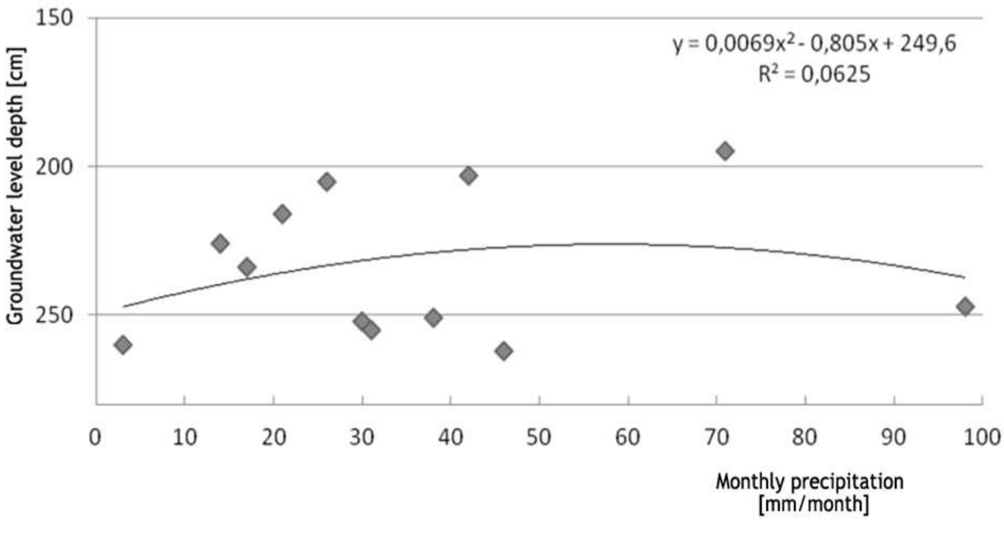

Figure 2. Variation in groundwater level depth as the function of monthly specific precipitation for well No. 004171 in Eger (2000)

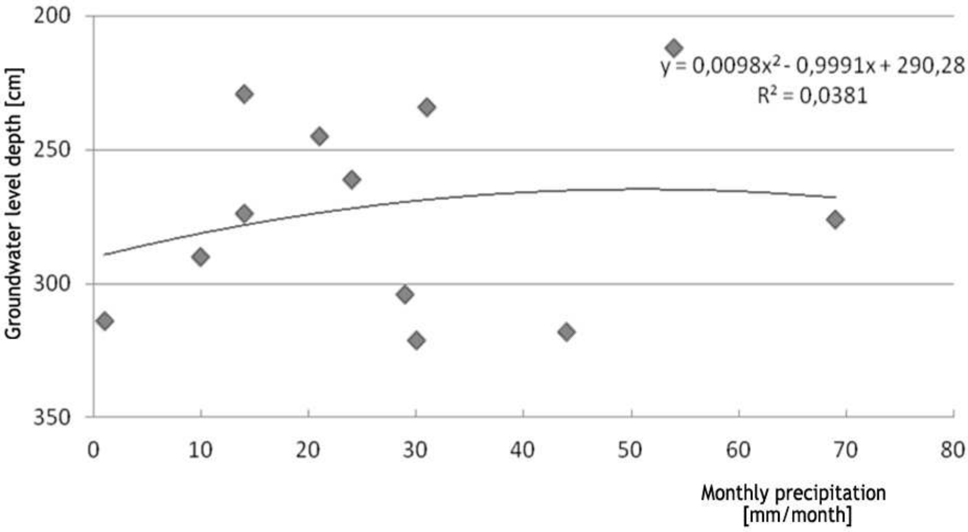

Figure 3. Variation in groundwater level depth as the function of monthly specific precipitation for well No. 001841 in Mezőkövesd (2000)

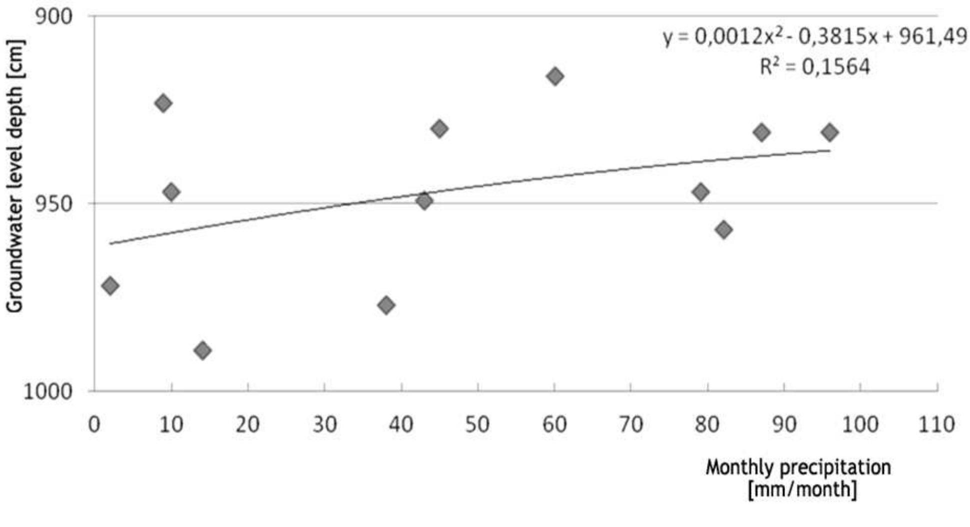

Figure 4. Variation in groundwater level depth as the function of monthly specific precipitation for well No. 001829 in Kompolt (2001)

Table 2

Occurrence of maximum monthly precipitation average and maximum groundwater level (minimum depth from surface) in the months of the year (within a year)

<table>
<tr><td>Months</td><td>I.</td><td>II.</td><td>III.</td><td>IV.</td><td>V.</td><td>VI.</td><td>VII.</td><td>VIII.</td><td>IX.</td><td>X.</td><td>XI.</td><td>XII.</td></tr>
<tr><td>The month of maximum precipitation [item]</td><td></td><td></td><td></td><td></td><td></td><td>4</td><td>8</td><td>1</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Relative frequency</td><td></td><td></td><td></td><td></td><td></td><td>30.8</td><td>61.5</td><td>7.7</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Relative frequency in 3 months [%]</td><td colspan="4"></td><td></td><td colspan="2">92.3</td><td colspan="5"></td></tr>
<tr><td colspan="5"></td><td colspan="3">100.0</td><td colspan="4"></td></tr>
<tr><td>Occurrence of maximum groundwater level [item]</td><td></td><td></td><td></td><td>6</td><td>2</td><td>3</td><td>2</td><td>2</td><td></td><td></td><td></td><td></td></tr>
<tr><td>Relative frequency [%]</td><td></td><td></td><td></td><td>40.0</td><td>13.3</td><td>20.0</td><td>13.3</td><td>13.3</td><td></td><td></td><td></td><td></td></tr>
<tr><td rowspan="3">Relative frequency in 3 months [%]</td><td colspan="3"></td><td></td><td colspan="2">73.3</td><td colspan="6"></td></tr>
<tr><td colspan="4"></td><td></td><td colspan="2">46.6</td><td colspan="5"></td></tr>
<tr><td colspan="5"></td><td colspan="3">46.6</td><td colspan="4"></td></tr>
</table>

Table 5

Average time lag (period delay) between precipitation and groundwater variation (delay)

	Settlement	**Number of wells**	**Average groundwater depth [cm]**	**Average time delay of period lag [year]**
1	Füzesabony	575	339	0.875
2	Füzesabony	003663	406	1.375
3	Heves	002144	145	0.667
4	Ludas	001827	349	0.929
5	Kerecsend	001808	458	0.500
6	Kompolt	001829	987	1.167
7	Eger	001809	669	0.800
8	Egerfarmos	003949	260	1.000
9	Mezőcsát	001817	238	0.663
10	Mezőkeresztes	001814	345	1.300
11	Mezőkövesd	001812	345	1.167

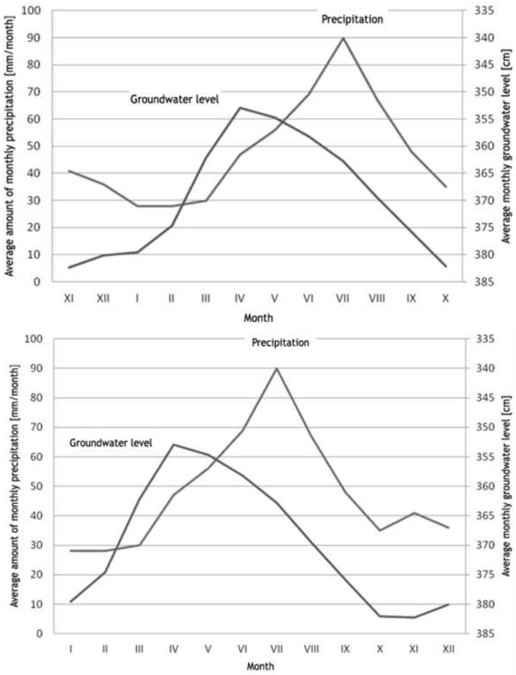

Figure 5. Average amount of monthly precipitation and average depth from surface of groundwater (12 precipitation measurement stations, 15 groundwater wells in the Mátra and Bükk regions, data from 1973–2006)

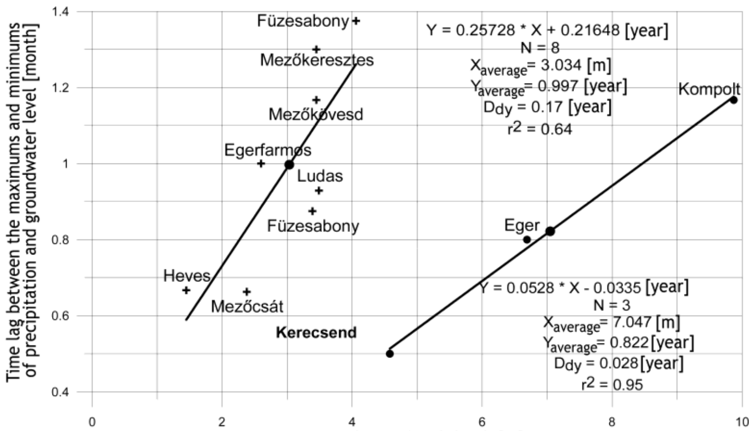

Figure 6. Time lag between the maximums and minimums of precipitation and groundwater level as the function of average groundwater level depth below surface

Table 3

Monthly and 3 months' relative frequency of the occurrence of maximum groundwater level (minimum depth from surface)

Settlement	Well No.	Months of the year											
		I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.	XI.	XII.
Eger	001809						▲	▲	▲				
	004171			▲	▲	▲							
Egerfarmos	003949			▲	▲	▲							
Füzesabony	575				▲	▲	▲						
	003663			▲	▲	▲							
Heves	002144			▲	▲	▲							
	002146		▲	▲	▲								
Kerecsend	001808				▲	▲	▲						
Kompolt	001829						▲	▲	▲				
Ludas	001827				▲	▲	▲						
Mezőcsát	001817				▲	▲	▲						
	001820				▲	▲	▲						
Mezőkeresztes	001814				▲	▲	▲						
Mezőkövesd	001812						▲		▲	▲			
	001841						▲	▲	▲				
Frequency [item]			1	5	11	10	10	3	4	1			
Relative frequency [%]			2.2	11.1	24.4	22.2	22.2	6.7	8.9	2.2			
Total relative frequency for three months [%]					57.8								
						73.3							
							51.1						

Table 4

Monthly and 3 months' relative frequency of the occurrence of minimum groundwater level (maximum depth from surface)

Settlement	Well No.	Months of the year											
		I.	II.	III.	IV.	V.	VI.	VII.	VIII.	IX.	X.	XI.	XII.
Eger	001809	▲	▲	▲									
	004171									▲	▲	▲	
Egerfarmos	003949									▲	▲	▲	
Füzesabony	575										▲	▲	▲
	003663	▲									▲	▲	
Heves	002144									▲	▲	▲	
	002146								▲	▲	▲		
Kerecsend	001808	▲	▲										▲
Kompolt	001829	▲	▲										▲
Ludas	001827										▲	▲	▲
Mezőcsát	001817									▲	▲	▲	
	001820	▲									▲	▲	
Mezőkeresztes	001814	▲										▲	▲
Mezőkövesd	001812	▲	▲	▲									
	001841	▲	▲	▲									
Frequency [item]		8	5	3					1	5	9	9	5
Relative frequency [%]		17.8	11.1	6.7					2.2	11.1	20.0	20.0	11.1
Total relative frequency for three months [%]		35.6											
										51.1			
											51.1		

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Refuse Derived Fuels from Municipal Solid Wastes

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ABSTRACT

Traditional waste treatment of solid municipal waste in Europe consisted of selective collection and sorting of packs; production of compost and/or biogas from the selectively collected biologically degradable fraction; as well as disposal and/or incineration of the residue. However, this traditional system does not sufficiently reduce the waste quantity to be landfilled; a significant portion of the waste is landfilled without being treated and the incineration of the residue is economically unfavorable because of its low caloric value and high moisture content. The above facts made it necessary to introduce the preparation technologies for producing refuse derived fuels from municipal solid waste; the biological stabilizing treatment of the residue of municipal solid waste to reduce its moisture content and to produce an alternative fuel of a higher quality. The paper gives a summary of the experience on mechanical preparation and mechanical-biological stabilization experiments of a great number carried out in Hungary.

INTRODUCTION

The most urgent problem in Hungary's waste management is the treatment and recycling of municipal solid waste. In Hungary, about 4.5-5 million tones of household waste and industrial-commercial waste like municipal one are annually generated. In the developed countries, a significant proportion of this waste is UTILIZED as valuable secondary raw materials, as well as secondary energy source.

In beginnings in Europe, a more or less uniform system of municipal solid waste treatment has been developed (Figure 1).

According to this, municipal solid waste can be divided in three main groups from the point of view of uniform treatment:

1) packs: 30…40% of the solid municipal waste; *glass:* white, brown, green; *paper:* printing products, cardboard, corrugated paper; *light packs*: metals: iron and aluminum beverage cans and, plastic packs;
2) bio-waste: 40…50%; biologically decomposable materials (vegetable waste, food debris, etc.);
3) residue: 10...30%.

Accordingly, the main elements of the system are as follows: selective collection and sorting of packs; selective collection and treatment of the biologically decomposable proportion (composting and biogas production); incineration and/or landfilling the residue.

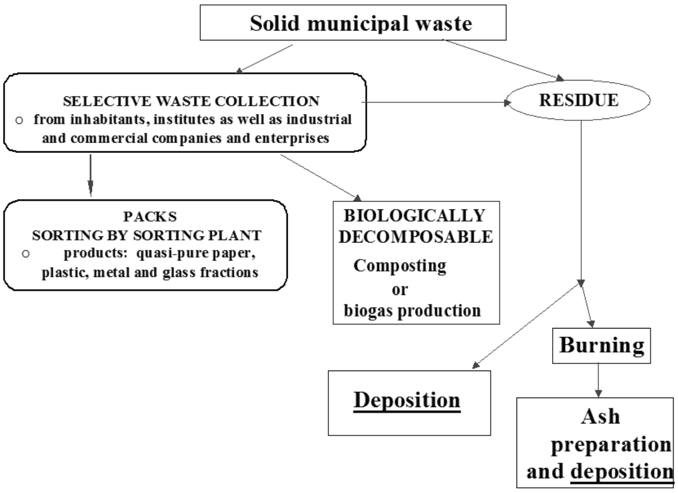

Figure 1. System of traditional MSW treatment

This traditional system has several disadvantages:

- It does not sufficiently reduce the waste quantity to be landfilled (if there is no burning).
- A significant proportion of the waste (40…60%) is landfilled without treatment (residue).
- Because of the above facts, the landfilling costs are high.
- Forcing the separative collection (of packs and biologically degradable materials) and manual/mechanical sorting could make this system uneconomical too.
- At the same time, the residue of municipal solid waste consists of agriculturally and energetically useful components, namely biologically decomposable and non- or hardly decomposable organic ones (the latter with a higher caloric value).
- Because of the unfavorable burning properties of the untreated municipal waste residue (low caloric value and high moisture content), it is disadvantageous to incinerate it from both technical and economical points of view.

The above facts lead to the new complex solutions: to introduce mechanical and mechanical-/biological treatment of the residue of municipal solid wastes. So that the MSW (remained after separative collection) can be treated through:

- mechanical,
- biological,
- or thermal processes,
- or (most frequently) the combination of the above.

Mechanical treatment (preparation) is usually the preceding (and succeeding) step of the biological or thermal treatment of the wastes. Mechanical processes are already used during waste collection. Already in this process, it is reasonable to separate the different material types as much as possible. During the treatment of residual MSW waste, it is reasonable to separate first the metals from the material flow, the fraction with a high calorific value and the less valuable fractions hampering the treatment, such as rock pieces, ceramics and glass. Mechanical treatment is needed for the recycling of each raw material.

The main target of biological treatment is to reduce the quantity of degradable biowaste, moisture content and the content of different volatiles. Another target is to minimize or fully eliminate the environment polluting emissions of possibly generating in the landfill leachate and methane. It can be achieved by the utilization of the activity of available aerobic micro-organisms and the enhancement of their function in such a way as to minimize the biologically degradable content of the initial material (waste) via aerobic microbial degradation as much as possible. This process is called the biological stabilization.

The main purpose of aerobe biological treatment complementing mechanical processes, when CO_2 and water vapor are generated from the organic materials due to their aerobic biodegradation, the full quantity of residue decreases by 40%, the moisture content decreases to one half/one third, and the calorific value of the treated material increases more than twofold.

The combination of the mechanical and process is called mechanical-biological stabilization *(Niederdränk, 2003; Thomé-Kozmiensky, 2002, Csőke et al., 2006)*.

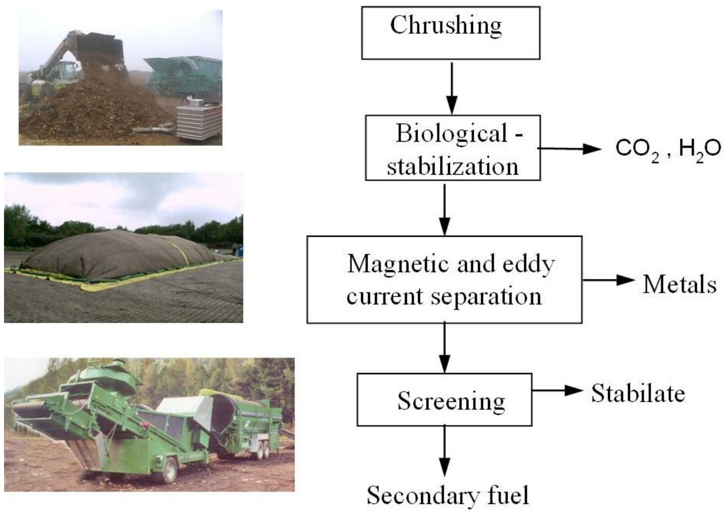

Figure 2. General (simplified) technological flow sheet of mechanical-biological stabilization

General aim of *mechanical-biological treatment* (Figure 2): Producing a fraction rich in components with high calorific value and a fraction poor in components with high calorific value, and landfilling this product: producing a product to be landfilled that meets the requirements of landfilling.

Usually, the material is stabilized biologically (composting) and the fraction rich in components with high caloric value is won from the stabilized material by screening and separation.

LARGE-SCALE EXPERIMENTAL TESTS

The main purpose of the mechanical-biological stabilization experiments carried out in Hungary was to elaborate the technology to produce secondary fuel from the municipal residue after bio-stabilization, in such a way as to obtain deeper knowledge on process in order to control it. During the large-scale mechanical-biological stabilization experiments carried out at the Polgárdi Landfill, the proper methodology of sampling and determining the material composition was elaborated. To determine the calorific value, a measurement procedure based on an experimental boiler, which enables one to determine the combustion parameters, as well as the chemical composition of ash and flue gas, was developed too *(Csőke et al., 2003)*. We explored the relationship between grain size and calorific value concerning domestic solid municipal waste.

Experimental mechanical-biological stabilization

During mechanical-biological stabilization the necessary operations were carried out as follows:

Pre-treatment: As the first step of mechanical-biological treatment, the comminution of the MSW using a hammer crusher was carried out.

Building aerated pile: Then the comminuted and homogenized MSW was piled into a pile of 300–400 m^3. Intensive aerobic degradation and stabilization took place here for 3…4 weeks. The forced aeration was applied using aerating ducts.

Placing the probes: After making the bed, the probes measuring the temperature and oxygen content necessary for controlling the aeration were placed.

Covering the pile: The bed built and provided with probes was covered using a three-layer GORE-Cover® semi-permeable membrane cover.

Aeration: After covering the prism, we started the aeration system operated by feeding back the data of the probes measuring the temperate and oxygen content. Aeration is of vital importance for a quick and odor-free decomposition and recycling of organic wastes. We applied a pressure aeration system which sucks the ambient air and blows it into the maturing material through in-floor aeration ducts running under the material.

Operation: During the two/four-week maturing period, aeration was operated according to the limit values for temperature and oxygen content. During the treatment period, it is not necessary to control the moisture content of the prism and to rotate the material. Because of the material loss during maturation, the membrane cover has to be re-stretched several times.

Demolishing the pile: The pile was demolished after the degradation period of 2–4 weeks.

Removal of the stabilate (CLO) compost like material

First, the material of the demolished pile was screened. The fine end-product of the drum screen is compost like material, the bio stabilate which can be landfilled and used for agricultural and re-cultivation purposes (mainly for growing energy plants; currently, experiments in this direction are in progress). After removing the ferrous metals from the coarse product of the screen by magnetic separation, the RDF secondary fuel rich in components with high calorific value was obtained. The composition and properties of the products were tested.

RESULTS OF MECHANICAL-BIOLOGICAL WITH SELECTIVE COMMINUTION EXPERIMENTS

According to the data of the mechanical-biological experiments, it can be stated:

- During aerobic biological decomposition, the waste quantity decreases by 20…40 % and water vapor and carbon dioxide exhaust.
- The largest mass proportion – 45…50% – of the stabilate is formed by a compost-like fines (< 20 mm).
- The other considerable proportion (about 45%) is formed by a biologically non-degradable or hardly decomposable coarse (> 20 mm) organic product rich in combustibles.
- In this product, the moisture content decreased from 27…30% to 8…10%, while the calorific value increased from 3.5…6 MJ/kg up to 12…13 MJ/kg (with the indicated moisture content).
- The metals represent 4–5% of the material balance.

In terms of process technology, the details, namely the properties of the grain fractions, are very important. It can be stated *(Table 1)* that

- the calorific value increases significantly with the grain size, while the ash and moisture content behave in the opposite way: they increase in the finer grain fractions;
- concerning material composition, it is worth highlighting that plastic and textile is concentrated in the coarsest size fraction (> 100 mm), paper in the medium-size fraction (20…100 mm) and CLO in the finest size fraction (< 20 mm).

Results of the selective comminution of coarse stabilate for advancing the quality of Refuse Derived Fuel by hammer crusher.

Examining of the selective comminution of coarse stabilate

During mechanical-biological stabilisation, the coarse (> 20 mm) secondary fuel product obtained at the drum screen was further size-reduced using a hammer crusher in order to produce, after crushing, a product rich in materials with high calorific value (plastic, textile, rubber etc.) by separation (selective comminution) for cement works.

Table 1

Process technology characteristics of grain fractions

BIO-STABILISED WASTE				
Grain size x, [mm]	**Mass yield [%]**	**Calorific value MJ/kg**	**Moisture content [%]**	**Ash content [%]**
< 50	54.2	6.3	10.6	41.2
50–150	34.7	12.9	6.1	25.7
> 150	11.2	20.4	3.3	27.5
$\sum$	100.00	10.2	8.2	34.3
$\sum_{measured}$		11.2		32.9
RAW WASTE				
		Calorificvalue [MJ/kg]	**Moisturecontent [%]**	**Ash content [%]**
$\sum_{measured}$		5.8	26.9	29.8

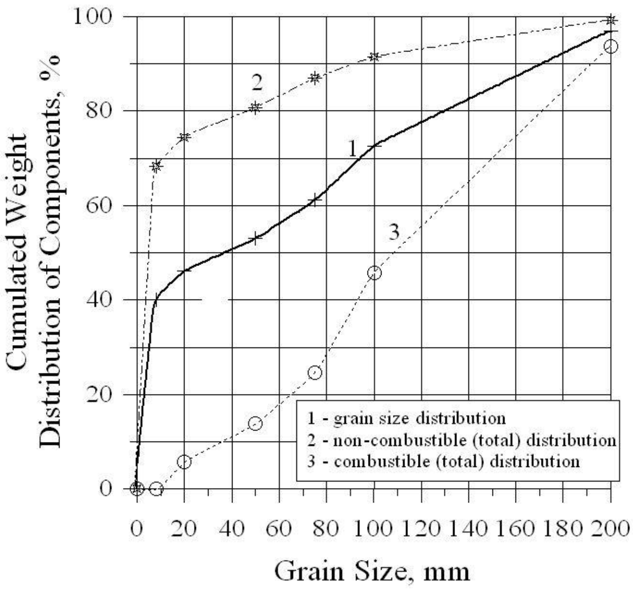

Figure 3. The grain size distribution of coarse stabilate along with the distribution of the combustible and non-combustible components

Figure 3 and *Table 2* visualizes the grain size distribution of coarse stabilate as well as the totalized distribution of the combustible and non-combustible individual components in the screen fractions, after additional comminution at the hammer crusher *Doppstadt* .

Table 2

Calorific values of coarse stabilate grain fractions comminuted additionally

Grain size, mm	**Calorific value, MJ/kg**	
	Comminution once	**Comminution twice**
> 200	16.3	19.4
100–200	20.4	22.8
75–100	12.8	18.2
50–75	11.7	10.4
20–50	10.7	11.5
8–20	4.7	9.2
< 8	0.6	5.8

Figures 4 and *Table 2* show that after crushing the combustible components (especially plastic and textile) become richer in the coarser fractions, and by separation at 60…80 mm one can obtain, with a yield of 40–50%, a product that is rich in combustible components and has high calorific value.

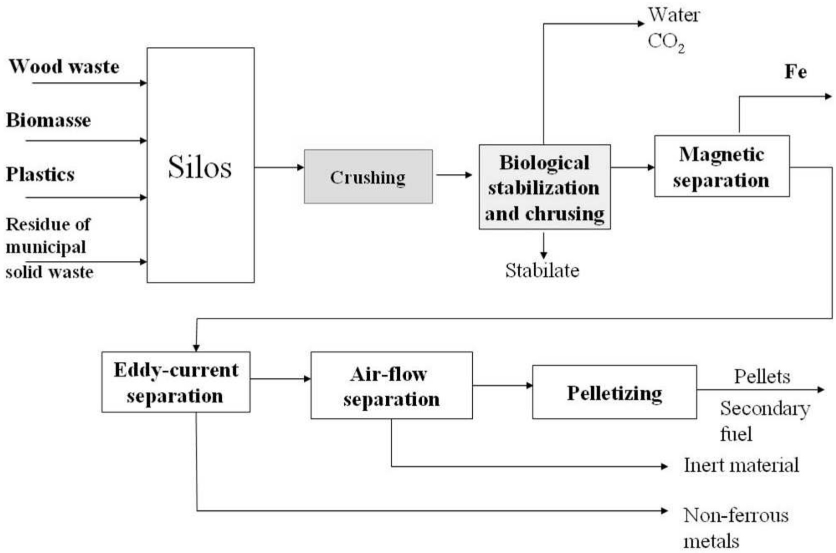

Figure 4. Waste preparation technology for RDF production

The MSW residue is not collected separatively and the residue of the sorting plants are bio-stabilized together. From the stabilized material by means of preparation processes (screening, as well as magnetic, eddy-current and air-flow separation, selective comminution) stabilized bio-waste, metal products, a combustible fraction and – from the components undesirable in the above products like non-combustible inert *(Figure 4)* can be produced.

CONCLUSIONS

According to the above results, it can be stated:

- One must strive for a full utilization of solid municipal wastes.
- A favorable solution to this is offered by separative collection and further sorting, as well as the combined technological system of mechanical-biological stabilization.
- The quality of RDF, the secondary fuel produced from municipal waste can be consciously controlled by means of process technology.
- The combustible fraction extracted in this way can be thermally utilized in several industries (cement works, power stations, metallurgy, etc.), especially if it is formed – along with other industrial wastes – into a real, always well-homogenized fuel with prescribed composition.

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EFFECTS OF THE DANUBE WATER LEVEL TO THE SURROUNDING THERITORY

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ABSTRACT

The water fluctuation of Danube's is significant; the difference between the minimum and maximum water levels reaches 8.0 m. What extent and range of effects can this significant fluctuation have on ground water levels of the surrounding areas? What fundamental tendencies contribute to the water fluctuation of the river and to what extent can a change in ground water levels be predicted on the basis of the water level fluctuation of large rivers?

This paper presents correlations and consequences drawn from data gathered on long-term observation of section of the Danube.

1. INTRODUCTION

In solving engineering tasks and planning the insulation and/or drainage-water-flow control systems of buildings one have to determine the levels and extreme values of ground water,as well as to be aware of how the fluctuation influences the water pressure on the built structures.

Underground constructionsare being now performed over larger areas, and at greater depths. In urban environment deep excavations sometimes have to be excavated in direct neighborhood of buildings. Ground water is one of the main concerns when performing deep open excavations.

With the building up the territory the original water flow system is changing, man-made blockade cross the water flow in aquifers. High water pressure and water velocity areremarkable both during and after construction. An important question is how the deep construction is protected from the water? What shall we do: to isolate or to regulate the water flow? If the traditional damp-proof system is used then the large isolated area makes always failures.

It is necessary to consider carefully the possible random fluctuations of groundwater pressure and flow by long and extensive investigations. Then it should be verified which would be the most probable seepage paths, considering realistic and prudent assumptions *(Mecsi, 1999.)*. A detailed analysis of groundwater seepage and provision for its control will certainly be less costly than the repair works related to an inappropriate assessment of the infiltration.

On the basis of the above facts it can be concluded that in areas lying along large watercourses the investigation of water table fluctuation, a thorough analysis of the laws of water flow and an analyzing assessment of observations are of primary importance.

2. INVESTIGATION METHOD

2.1 Data series of observations

The investigations present the correlations between the high resolution data series of the water level logger at 1645.5 river kilometer segment of the Danube at Vígadó tér, Budapest, on one hand, and the two ground water observation wells at a distance of 300-500 m from Vígadó tér, on the other. Water level 0 is located at 94.98 MASL(MASL ≡ meters above Baltic Sea level).

The primary aim of the investigation was to gain information about how the water level of the Danube influences the construction of a multi-story underground car park in the surrounding area and to assess the effects of the changed water table conditions after the completion of the construction.

Starting in 1971, FŐMTERV collected some 900 data from the two ground water observation wells on a weekly basis. In addition, a detailed data series of the Danube s water level, recorded at Vígadó tér on a daily basis was also available. This water level graph was accessible at *www.hydroinfo.hu/vituki*.

2.2 Soilmechanics– geological layers of the investigated area

Soil explorations and in situ investigations revealed that the subsoil stratification of the area concerned is almost uniform.

The thin grey clay layer, which is solid and can be considered practically waterproof, starts at the depth of 14– 14.5 m below the natural ground level – it is a compact layer with stone-like behavior and high resistivity. This type of clay can be found at the bottom of the Danube river-bed.

The Danube terrace, which sits on the previously mentioned bed, is of porous material: at the bottom there is a rubble-stone layer, while above it, there are gravels, sandy gravels.

There is no clay or silt fraction in the sandy gravel layer above the solid and almost waterproof clay layer. The layer is 7.5–8.0 m in thickness with good water supply capacity.The gravel and sandy gravel layers are good water suppliers with good load and water bearing capacity; they directly supply water to the Danube. The sandy gravel layer stores ground water and, in the case of high ground water level, its water table appears in the soil layers.

The alluvial deposit is of varying composition – it mainly consists of sandy-gravel inclusions, although in some places there can be smaller tracks with silt and clay layers due to the unpredictable changes of the shoreline.

Above the gravel layer there is silty sand, above which there is sand and then the sequence of layers is closed with a 1.8– 2.6 m thick embankment.

3. THE FLUCTUATION OF THE DANUBE'S WATER LEVEL

Figure 1 shows the diagram of the Danube's water levels, drawn on the basis of observations recorded through 12600 days: the average water levels of the individual months as well as the scattering range of the significant number of figures. The average water level ranges from 97.3 MASL to 98.4 MASL with the lowest level recorded in October-November (96.9 MASL) and the highest between April and June. It is interesting to note that the maximum water levels of the Danube are the highest in July and August.

Figure 2 shows the particular trend of the 3 month average water levels of Danube, based on daily measurements.

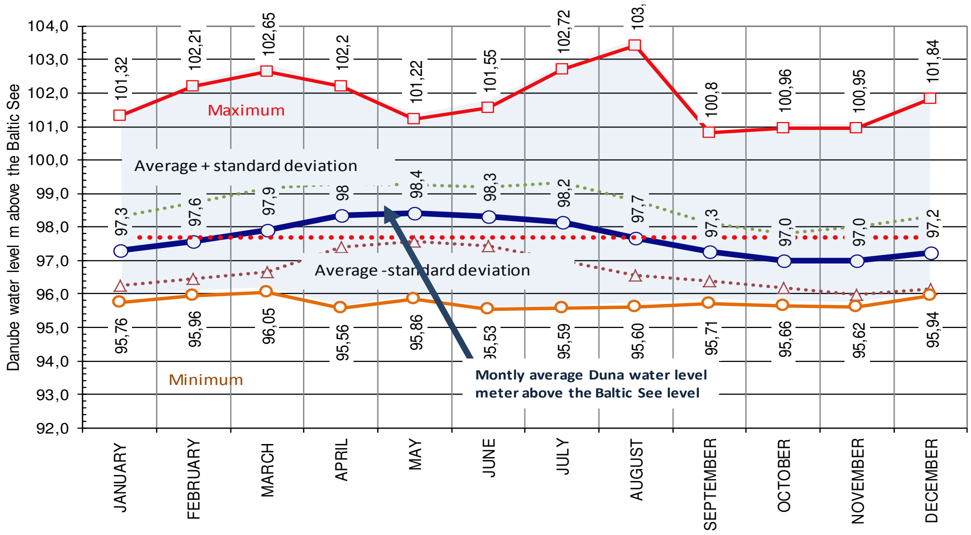

Figure 1. Monthly average of the Danube's water level between January 1971 and April 2006 at 1645.5 river km segment on based on 12600 observed data

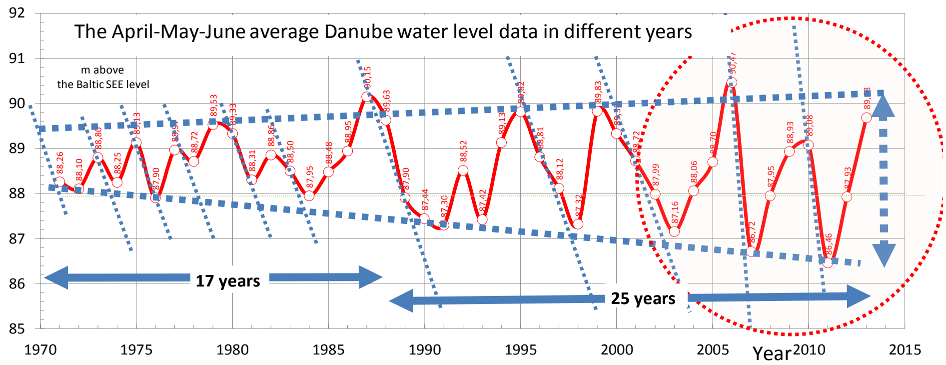

Figure 2. The 3 month average water level of Danube based on daily measurements

The distance between three-month average minimum and maximum water levels, starting from the 1970s, gradually increased in the range of up to 1.2 meters to 4 meters.

After the investigations, it became clear that the effect of the Danube on the water balance of the neighboring area is significant and requires a comprehensive analysis *(Mecsi, 2007)*.

4. THE EFFECT OF THE DANUBE'S WATER LEVEL FLUCTUATION ON THE WATER LEVEL OF THE NEARBY MONITORING WELLS

After dividing the water levels of the Danube into 1 m intervals we investigated the levels of ground water in monitoring wells V1 and V2 for the selected water level groups of the Danube.

On the basis of the large number of measured data we defined the average of these water level groups and the deviation of the measured data *(Figure 3)*.

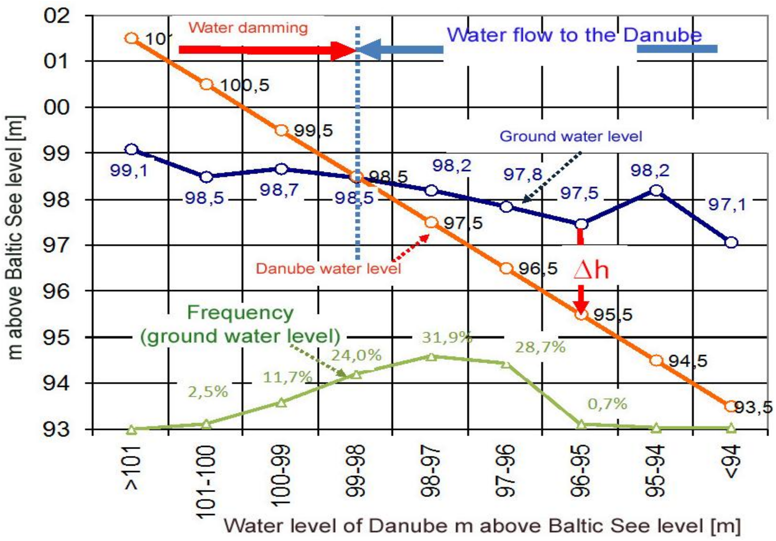

Figure 3. The relationship between monitoring well V2 and the water level of the Danube. The figure also shows the average of the observed data and the range of deviation

This data processing, which in fact was the creation of a rough system containing the Danube's water level ranges, proved to be useful because this way our database became less sensitive and at the same time the focus of our observation was shifted to a wider scale of monitoring.

Provided that the water level of the Danube exceeds the height of 98.5 MASL, the level of the ground water is lower than the water level of the Danube; however, below the water level of 98.5 MASL, the ground water level of the area is higher than the water level of the Danube, thus a water flow into the Danube is created. *Csoma R. and Gálos M.(2012)* showed similar results at the INFOPARK territory. While the water level of the Danube ranges within an interval of 8 m, this significant difference in the water levels in the monitoring wells at a distance of 300 and 900 meters is moderated.

5. THE FREQUENCY DISTRIBUTION OF THE WATER LEVEL OF THE DANUBE

The frequency investigations of the water level of the Danube are also providinguseful data, because we can find out how often low and high water levels occur. *Figure 4* shows the frequency distribution of the water levels of the Danube based on 126,000 observation data using 0.5 m height intervals *(Mecsi, 2010)*.

The distribution of water level data is not symmetrical; towards the high water levels – as typical of the Danube – we find an elongated diagram. It means that typically the lower water levels are the dominant, and the high water levels occur less frequently.

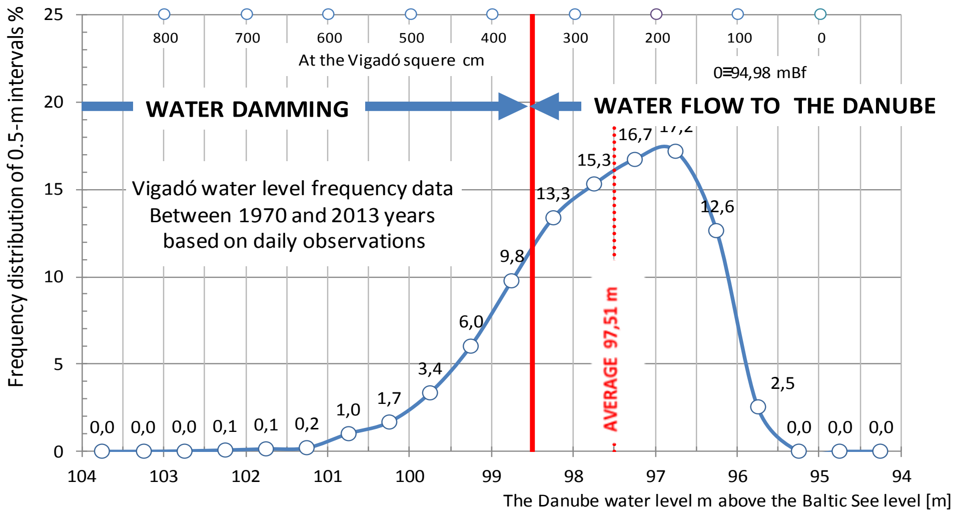

Figure 4. The frequency of Danube water level based on 0.5 m water level height intervals

6. SUMMARY

Investigations and analyses have shown that the Danube has a significant effect on the changes of ground water levels in the neighboring areas.

Not only water table fluctuation itself, but its nature can also be an important factor. The significant decrease of the water table can notably raise the effective stress from its own weight inside the soil-mass due to the ceasing upheave force; that is the soil-mass below the former water table becomes heavier and thus passes extreme loads in a big extent onto the soil-mass lying deeper; as a consequence, it induces a more and more compact soil.

The installation of civil engineering structures, especially with the construction of underground car parks, concrete structures prevent free ground water flow and change the former conditions. Thus,the effects of these changed conditions must also be investigated on the basis of the laws of water flow available up to now.

On the basis of the above facts it can be concluded that in areas lying along large watercourses the investigation of water table fluctuation, a thorough analysis of the laws of water flow and the assessment of observations are of primary importance.

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PHYSICAL MODELLING ON THE ENHANCEMENT OF AEROBIC STABILIZATION OF MUNICIPAL WASTE LANDFILL

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ABSTRACT

The stabilization period of old closed landfills is generally 13– 35 years, while the soil polluting leachate and green-house effecting methane has been producing for decades after the closure. According to the studies, 40– 60 million tons of methane generate from these old landfills, which is about 11– 12% of the global anthropogenic methane emission (Heyer et al., 2005). To reduce the harmful emissions of the closed landfills the in situ enhanced aerobic stabilization can be applied. The Department of Bioprocessing and Reaction Techniques has been dealing with the aerobic degradation and other biotechniques for two decades, it was Professor Imre Szabó who advised our team to look at the possibility to apply forced aeration to enhance the aerobic degradation in the landfills providing opportunity to shorten the stabilization period after the closure.

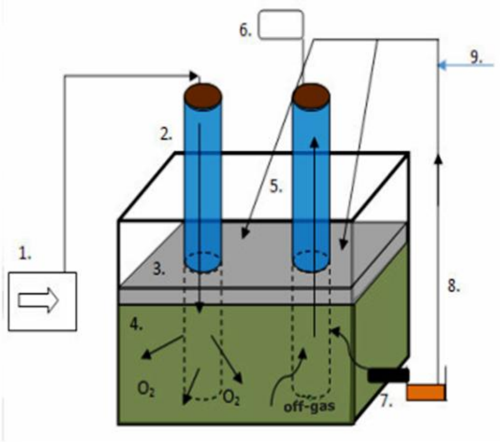
INTRODUCTION

The least desirable solution in the waste management is the deposition, however in Hungary there was no comprehensive legislation for landfilling until 2000 (Szabó I. et al., 2011). Before the 2000s, the most commonly applied process for the disposal of solid wastes was the uncontrolled dumping on the surface or in the hollows of the ground. The stabilization period of these old landfills is generally 13– 35 years, so the polluting leachate and methane has been producing for decades after closure (Vermes, 2005). According to the studies, 40– 60 million tons of methane generate from these old landfills, which is about 11– 12% of the global anthropogenic methane emission (Heyer et al., 2005). To reduce the harmful emissions of the closed landfills the in situ aerobic stabilization can be applied, which has been successfully implemented in many countries. The aerobic stabilization is recommended to apply, when there is no recoverable amount of methane in the landfill, but it still carries a significant risk to the environment (Szabó I. et al., Ritzkowski et al., 2012). Due to the aerobic stabilization the degradable organic materials decompose aerobically faster, the degree of methane production reduces significantly, the sedimentation process of landfills takes place in a shorter time, and the polluting impact of the leachate also decreases, as well as the carbon-dioxide production increases (Ritzkowski et al., 2006, 2012). The aim of the experiment, which was performed at the Bioprocessing Laboratory of Institute of Raw Material Preparation and Environmental Processing, University of Miskolc, was the investigation of the possibility to enhance the landfill stabilization process by forced aeration using physical modeling of aerated and non-aerated municipal solid landfill.

MATERIALS AND METHODS

For the experiments two 115 cm long, 75 cm wide and 66.5 cm high plastic tanks, as the reactors were installed. The aeration was carried out through PVC tubes using a compressor and the sampling of leachate was performed at the bottom of the tanks *(Figure 1)*. The fresh

MSW were supplied by AVE Company, it was sampled and analyzed. After loading of the homogenized municipal solid waste portion into the reactors, they were covered by plastic bags, on which small halls were cut to simulate the failures on the external insulation of the old landfills. Finally, the bags were covered with 3cm layer of gravel. One of the reactors was aerated by a compressor for 4 hours a day. Another one was without forced aeration.



1. Compressor
2. Perforated tube for air inlet
3. Covering with plastic bags and gravel
4. Municipal solid waste
5. Tube for gas collecting
6. Gas analyzer
7. Drain tube for leachate
8. Leachate recirculation
9. Water addition

Figure 1. The schematic illustration of the aerated reactor, physical model of landfill (Bokányi et al., 2012)

In the first part of the experiments one of the reactors was aerated using a Pneumair 85/15-A.P. type compressor and 470.4 m^3 of air was introduced into the waste mass during 196 h. Another reactor was operated without aeration. At this part of the experiments the temperature, the composition of the generated gas (methane and carbon-dioxide), as well as the leachate production were measured.

The temperature was measured below the surface in 15cm depth. It can be seen in *Figure 2* that it was higher by 5− 6°C within each reactor, than the ambient one. Difference in the measured values between the two reactors was not observed. Nevertheless, the aerobic degradation can be obviously recognized based on the tendency of temperature profile.

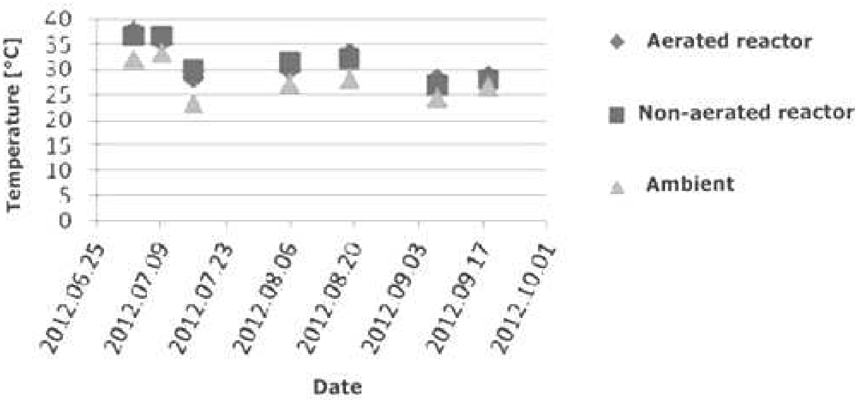


Figure 2. Variation of the temperature in the first part of the experiments

The composition of the generated gas was determined using a Dräger X-am 7000 type gas-analyzer device. According to the results, it can be noticed that no methane and only small amount of carbon-dioxide was produced *(Figure 3)*. Significant difference between the results obtained from the two reactors was not observed. The reason might be the low bio fraction rate of the feed-waste (appr. 10%), or in case of the aerated reactor the produced gas was blown through the perforations of the tube by the compressor.

Separable amount of leachate was not produced in the aerated reactor, only in the non-aerated one (20 cm^3) during 3 months.

After the first four months there were no significant changes in the process, therefore the reactors were inoculated with food residues in order to increase the biodegradable amount in the waste. Furthermore, the waste was sprinkled with water to promote the leachate-production, as well as to ensure the optimal environment for the microorganisms. The aeration was carried out using Pneumair 85/15-A.P. and an Elektra Beckum type compressor, and 391.44 m^3 air was introduced into the waste mass during 185 h. Hereinafter, there was still no any changes in the temperature.

In the second part of the experiments methane was obviously not generated. Only a small amount of carbon-dioxide was detected in the gas generated in the aerated reactor. This amount was a little bit higher, than in case of the gas obtained from the reactor operated without aeration. It can be concluded that the inoculation with kitchen residue resulted in an additional degradation peak in respect to the gas composition.

According to the evidence of *Figure 3*, it can be noticed that in the first part of the experiments the amount of the carbon-dioxide produced in the aerated reactor reaches a maximum value at 27.04.2012, while in case of the another, non-aerated reactor a shift in the time can be seen. The maximal value appears one month later. In the second period the maximum value is higher, which can be resulted by the addition of kitchen residue and the delay can also be observed: the aerobic degradation is more intensive. Summarizing the results, especially based on the time delay, it can be postulated that the aeration can intensify the degradation, thus the stabilization will occur faster.

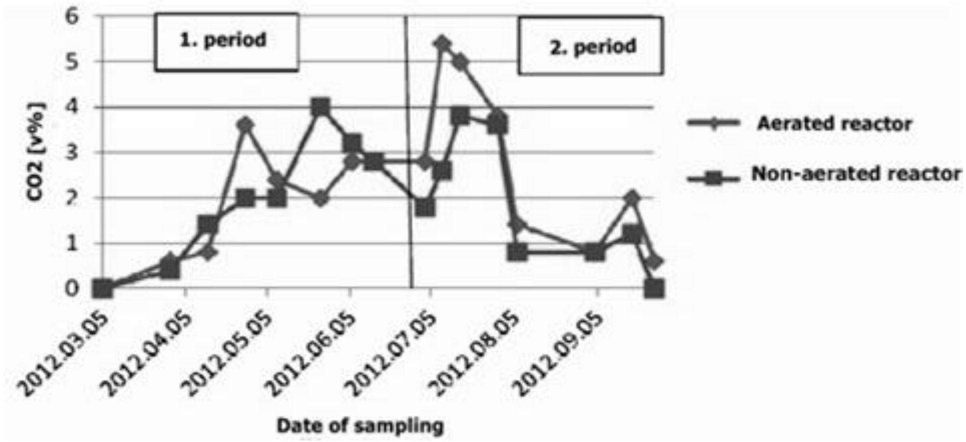


Figure 3. Produced CO_2 amount as function of time

In the second part of the experiments the waste was sprinkled initially with 10 L water, later with 8.5 L water and 2.5 L of leachate. The obtained leachate was analyzed, and the

Figure 4 and 5 represent the COD (chemical oxygen demand) and BOD (biological oxygen demand) values respectively.

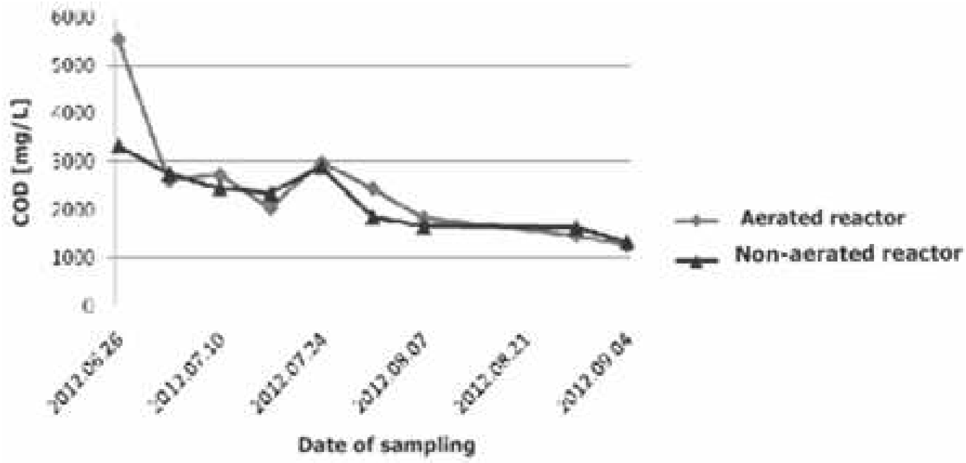


Figure 4 Chemical oxygen demand values of leachate sampled from the reactors

As it can be seen in *Figure 4* the reduction of the COD values in the aerated process is more prominent, thus the aeration has a positive effect to the bio waste degradation. In terms of the BOD_5 values, the degree of the reduction is quite similar in both reactors.

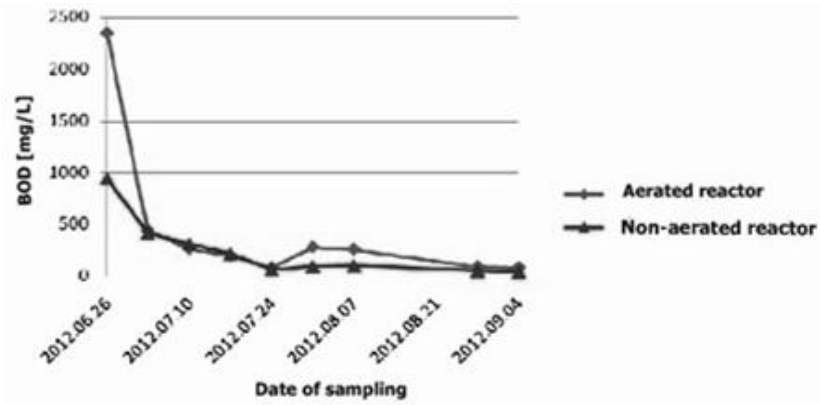


Figure 5. BOD_5 (biological oxygen demand) values of leachate separated from the reactors

CONCLUSIONS

A very first experimental attempt on the physical modeling of the degradation processes in MSW landfill with and without forced aeration was done in the Bioprocessing Laboratory of our Institute. Based on the gained experience the following consequences can be drawn. The size of the reactor was not chosen properly: it should be larger than used in our physical model. The volumetric air distribution in the forced aeration should be more homogeneous. The composition of the applied MSW feed next time should be chosen more carefully as well. Moisture content of the MSW feed is also of a great importance. Nevertheless the experiments should be continued, the obtained in this phase results in general are promising. Analyzing the composition of the generated gas it was found that the amount of carbon-dioxide was higher in case of the forced aerated degradation process, methane was not detected. The time delay in the CO_2 production of the two reactors indicates that the forced aeration does intensify the aerobic degradation, thus the stabilization process will occur sooner. The COD values of leachate samples show a decreasing tendency, which reduction was more prominent (appr. 77%) due to the forced aeration, so this aerobic degradation process was faster. In respect of BOD_5 values the degree of the reduction was around 96% at both reactor types. Finally, the surface subsidence was found to take place relatively quickly, only after 6 months. In this respect there was no difference between the forced aerated and the non-aerated reactors.

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PILE DESIGN BASED ON CPT ACCORDING TO THE REQUIREMENTS OF EUROCODE 7

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ABSTRACT

Pile foundations now make up a greater percentage of foundations for new construction. That is why, an accurate calculation of pile resistance and the knowledge of its reliability has a great economic importance. A more rigorous approach has become necessary since the introduction of Eurocodes. The pile resistances are more often calculated based on CPT results. This paper suggests a new method based on comparison of several approaches and recent design experience. The method takes into account soil type and piling technology. It was verified by 63 pile load tests and its reliability was evaluated as well. This paper contains the first part of the author's PhD thesis which was written at University of Miskolc under the supervision of Professor Imre Szabó.

RESEARCH TOPIC

Pile foundations have now become a very common alternative for new construction. In the last 15 years, the continuous flight auger (CFA) method has been the dominant method for pile installation. Alternatively, bored piles made with slurry or casing are used only for diameters in the range of 120÷150 cm. Traditional precast concrete driven piles are declining and the modern types of these piles (cylindrical, conical, prestressed and jointed) have not been readily adapted by contractors. Driven cast-in-place piles are also rarely used and the Franki-method; once very popular in Hungary, has almost disappeared. On the other hand, a new displacement pile type, the screwed pile (Screwpile, Omega, Atlas, TSD, CMC etc.) has gained acceptance and is expected to be more common than the CFA-pile within 10 years. It can be installed as fast as the CFA-pile, it does not disturb the environment, nor does it produce soil to transfer and sometimes to handle as a hazardous material. Its specific bearing capacity can be higher due to the compactive effects of soil displacement.

Piling technology has an important influence on the pile resistance. So, the appearance of new types and changes in the proportion of different types' utilization cause new design challenges for the geotechnical engineer. The increase in piling volume enhances the claim for a more economical design. In this situation, engineers dealing with these questions on a scientific level should find answers for these challenges. They should develop methods which can allow for the design of different pile types in a unified, fast, reliable, and economical way.

Beginning in 2011, structures and the piles that support them must be designed to meet the requirements of Eurocode *(Szepesházi, 2008)*. For pile design, the engineer should verify that the GEO ultimate limit state will not occur *(Eurocode, 2004)*. This means there should be no strength failure in the soil. For the determination of the compressive resistance failure of a single pile, EC 7 accepts three methods: 1. static load tests on similar piles; 2. dynamic load tests verified by static load results 3. empirical or analytical calculation methods validated by static load tests. The calculation method based on ground test results is not detailed in the EC 7. We may think that sounding tests (e.g. CPT) and laboratory shear strength tests can provide a basis for the calculations, but it is not forbidden to assume pile resistances based on simpler index tests such as soil classification. In Hungarian practice, the calculation based on CPT results is becoming more common. For a 10÷20 m long pile, soils can be explored and characterized often only by this method in compliance with requirements and traditions of practice. CPT data for pile design is supported by some software (M-Pile, UniPile 5.0, Geo5) used in Hungarian practice as well.

An important requirement of EC 7 is that calculation methods based on ground test results may only be used when verified by static load tests on piles having similar soil condition, technology and geometry. In the annex of the EC 7-2 the methods of German and Dutch codes *(De Cock at al. 2003)* are suggested. These fulfil perhaps the mentioned requirement, but this is not declared clearly in the EC 7-2. That is why it was necessary to accept a pile calculation method calibrated on load test data in typical Hungarian soils at the usual level of construction quality. Eurocode 7 requires that the reliability of the accepted calculation method be established and considered when applying partial factors and other safety elemets given in the national annex.

The author has dealt with the issue outlined above for more than 20 years. A team guided by him at the Széchenyi István University executed about 160 pile load tests in the last 20 years. The idea of pile resistance calculation based on CPT data came from study in the Netherlands in 1995. Shortly thereafter it was first used in Hungary and later refined through projects mainly for bridge construction. The author leads the implementation of the European geotechnical standards as the Head of the Technical Committee for Special Geotechnical Works at the Hungarian Standardization Institution. Through all of these activities it was not difficult to recognize the needs outlined above and apply special knowledge and performance data to solve the tasks formulated. From these prerequisites pile design based on CPT according to Eurocode 7 was the focus of his research work in 2010 and the results were accepted as PhD thesis in University of Miskolc. Professor Imre Szabó was his scientific supervisor.

This paper presents the first part of the research results, other details can be studied in other papers *(Szepesházi, 2011/a, /b and /c)*.

TRADITIONAL PILE RESISTANCE CALCULATION METHODS

The pile resistance based on calculation can be determined with the following equation

$$R_{c;cal} = R_{b;cal} + R_{s;cal} = A_b \cdot q_{b;cal} + \sum A_{s;i} \cdot q_{s;cal;i} \qquad (1)$$

in which

- $R_{c;cal}$ ultimate total compressive resistance of the ground against a pile calculated from ground test results in kN,
- $R_{b;cal}$ ultimate pile base resistance calculated from ground test results in kN,
- $R_{s;cal}$ ultimate pile shaft resistance calculated from ground test results in kN;
- A_b pile base area in m^2,
- $A_{s;i}$ pile shaft surface area in layer i in m^2,
- $q_{b;cal}$ specific vaue of ultimate pile base resistance calculated from ground test results in kPa,
- $q_{s;cal;i}$ specific vaue of ultimate pile shaft resistance in layer i in kPa,

A lot of suggestions for the determination of resistances are given in the literature and in "old" national standards *(Tumay, Fakhroo, 1982; ME 15005/2:1989; Briaud, 1991; Titi, Abu Farsakh, 1999; De Cock et al., 2003)*. The resistances in national standards may be considered as values which were proved and verified by numerous pile load tests, in accordance with use of a calculation method based on ground test results according to the EC 7. By the comparative analysis and synthesis of the methods found, I could adapt such parameters, considering mainly the influence of the piling technology, into the new design method. This analysis gives a support to create new equations, as well.

The specific value of the ultimate base resistances in fine-grained soils can be calculated by the equation

$$q_{b;cal} = N_c \cdot c_{u;k} \tag{2}$$

where

- N_c bearing capacity factor,
- $c_{u;k}$ characteristic undrained shear strength.

N_c = 9.0 is generally accepted based on the suggestion by *Skempton (1959)*, but some researchers suggested smaller value (7.0– 8.0) for bored piles. There is a consensus among the authors, that the average of the measured c_u values of the zone from 1.5·D above to 3.0·D below the pile base should be used as characteristic value to calculate the ultimate base resistance.

The ultimate shaft resistance for layer i is to calculate from the measured $c_{u;i}$ by the equation

$$q_{s;cal;i} = \alpha_u \cdot c_{u;i} \tag{3}$$

where

- α_u adhesion factor,
- $c_{u;i}$ undrained shear strength in layer i.

The α_u values depend on the piling technology and the soil strength and many suggestions can be found in the literature. A comparison of values for bored piles is shown in *Figure 1.* (A similar diagram was constructed for driven piles which is not presented here to save space.)

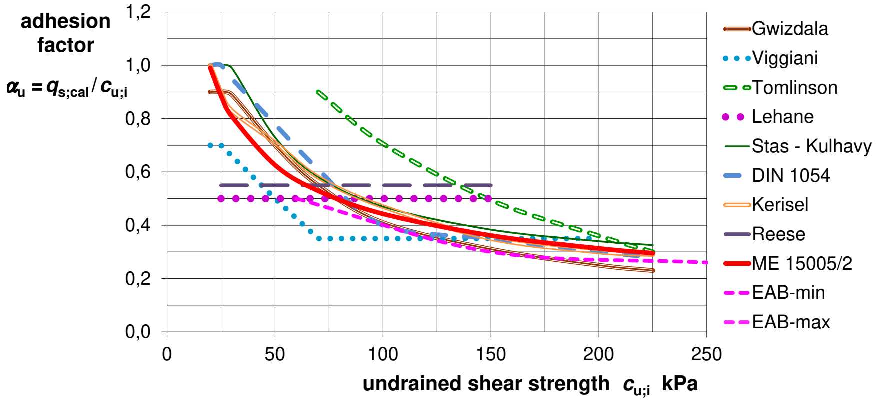

Figure 1. Comparison of the adhesion factors for bored piles in fine-grained soils

Figure 1 shows clearly that equation (3) is strictly speaking not true, $q_{s;cal;i}$ is not linearly proportional to $c_{u;i}$. That is why, at the introduction of the ME 15005/2 in 1989 the $q_{s;cal;i}$ – $c_{u;i}$ relation was described by the equation

$$q_{b;cal;i} = q_{so} \cdot \sqrt{\frac{c_{u;i}}{c_1}} \tag{4}$$

where

- q_{s0} 140 kPa for bored and 180 kPa for driven piles were suggested,
- c_1 1000 kPa.

The α_u values of *ME 15005/2* and *ME15005/2mod* were calculated for *Figure 1* from eq. (4).

To calculate the specific value of the ultimate pile base resistance in coarse-grained soils the CPT point resistance q_c is generally used with the equation

$$q_{b;cal} = \alpha_b \cdot \frac{1}{2} \cdot \left(\frac{q_{cI} + q_{cII}}{2} + q_{cIII} \right) \tag{5}$$

where

- α_b base resistance factor (1.0 for driven, 0.8 for CFA- and 0.6 for bored piles),
- t the critical depth between $0.7 \cdot D$ and $4.0 \cdot D$ under the pile base which gives the minimum q_b value,
- q_{cI} average of the q_c values in the zone between the pile base and the depth t under the pile base,
- q_{cII} average of the minimum q_c values in the zone between the pile base and the depth t under the pile base which minimum values are to be determined going upwards from the depth t to the pile base taking account allways only the smaller values,
- q_{cIII} average of the minimum q_c values in the zone between the pile base and $8{,}0 \cdot D$ height above the pile base which minimum values are to be determined going upwards from the pile base starting from the last value determined at the calculation of q_{cIIm} and taking account only the smaller values.

Equation (5) is given by EC 7-2 and it is from the Dutch Code *(Van Tol, 1993)* which is a synthesis of several suggestions found in the literature. Its main feature is that weak layers have a strong influence on reducing the base resistance which I have found as well.

Ultimate pile shaft resistance in a coarse-grained soil layer can be calculated based on the CPT point resistance as well, using the equation

$$q_{s;cal;i} = \alpha_s \cdot q_{c;i} \tag{6}$$

- α_s shaft resistance factor,
- $q_{c;i}$ the CPT point resistance in layer i is.

Factors (α_s) from different codes and authors are shown in *Figure 2.*

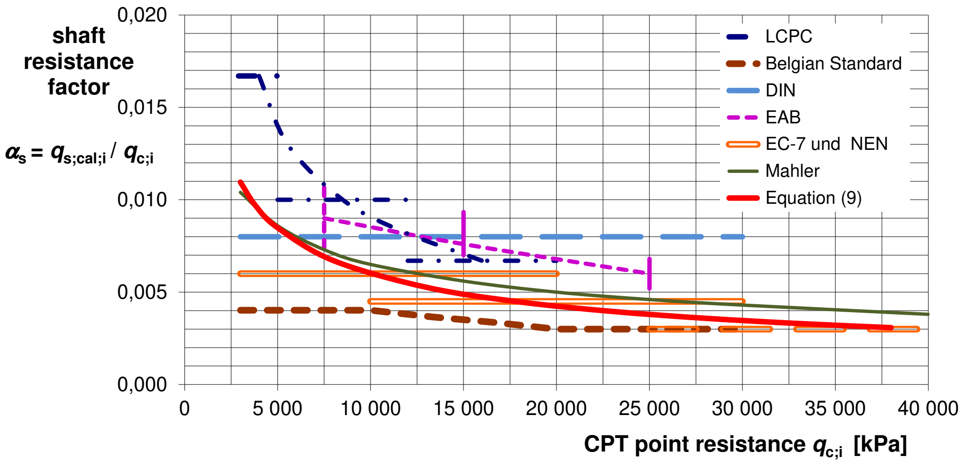

Figure 2. Comparison of shaft resistance factors for CFA-piles in coarse-grained soils

Some researchers believe that α_s values do not depend on the piling technology but rather on q_c, as shown in *Figure 2.* The first idea is surely not true as it is stated in EC 7-2. The
$q_s - q_c$ relation shows that the equation (6) is not true with constant α_s. Instead, it has a square root form as compared to a linear one for fine-grained soils.

NEW CALCULATION METHODS FOR PILE RESISTANCES BASED ON CPT

Using the observation of the comparison of the traditional methods and based on the experience of calculating capacities for a number of pile load tests, new formulas were developed.

For fine-grained soils, the undrained shear strength cab be calculated from CPT according to *Lunne et al., (1997)*

$$c_{u;I} = q_{c;i} / N_k \tag{7}$$

formula was introduced, where

- N_k cone factor and N_k=15.5 is an acceptable average value.

The ultimate pile base resistance in fine-grained soils can be calculated using equation (7) and the Skempton formula with N_c = 9.0 but taking into account the piling technology, data from pile load tests, and suggestions of national standards with the equation

$$q_{b;cal} = 0.6 \cdot \mu_b \cdot q_{c;k} \tag{8}$$

where

- $q_{c;k}$ characteristic CPT point resistance which can be taken as the average of the CPT point resistances of the zone from 1.5·D above the pile base to 3.0·D under the base,
- μ_b base resistance factor depending on the technology given in Table 2.

For the ultimate pile shaft resistance in fine-grained soils equation (9) was developed using the square root formula based on Figure 1 and introducing equation (7) and taking into account the national standards allowing for different piling technologies:

$$q_{s;cal;i} = 1.2 \cdot \mu_s \cdot \sqrt{q_{c;i}} \tag{9}$$

where

- μ_s shaft resistance factor depending on the technology given in *Table 1*.

The following limits are to be used:

- the extreme high $q_{c;i}$ values in sections smaller than 1.0 m should be filtered,
- $c_{u,I}$ > 500 kPa undrained shear strength may be taken into account only after verifying through investigation tests,
- $q_{b;cal}$ > 2.5 MPa can be used only by verifying by pile load test,
- $q_{b;max} \leq 4$ MPa for hard clays and $q_{b;max} \leq 8$ MPa for highly overconsolidated clays or claystones shall be considered.

The new calculation method for coarse-grained soils was developed based on former methods and more recent pile load test evaluation.

Table 1

Pile resistance factors and limit values for fine-grained soils

Pile type		Base resistance factor μ_b	Shaft resistance factor μ_s	Maximum pile shaft resistance q_{smax} [kPa]
Displacement piles	driven precast concrete	1.00	1.05	85
	steel tube driving with closed end	1.00	0.80	70
	cast in place made by a steel tube driving with closed end and reclaiming during concreting	1.00	1.10	90
	screwed cast in place	0.90	1.25	100
Replacement piles	Continous Flight Auger pile	0.90	1.00	80
	bored piles with drilling mud	0.80	1.00	80
	bored piles with casing	0.80	1.00	80

The base resistance calculation uses the method suggested in EC 7-2, with a new reduction factor

$$q_{b;cal} = \lambda_b \cdot \alpha_b \cdot \frac{1}{2} \cdot \left(\frac{q_{c\,\mathrm{Im}} + q_{cI\,\mathrm{Im}ean}}{2} + q_{cII\,\mathrm{Im}} \right) \tag{10}$$

where beside the symbols given at eq. (5)

- α_b base resistance factor *(see Table 2)*,
- λ_b reduction factor to take into account the suggestion of De Ruiter and Beringen *(in Van Tol, 1993 and Fellenius, 2006)* which was overlooked in EC 7-2 and/or other effects analysed later.

For shaft resistance in coarse-grained soils eq. (11) is used with the square root formula for this soil type too. Based on analysis of Figure 2, considering piling technology according to different national standards:

$$q_{s;cal;i} = \alpha_{sq} \cdot \sqrt{q_{c;i}} \tag{11}$$

where

- α_{sq} shaft resistance factor depending on the technology given in *Table 2*,
- q_c and q_s is in kPa.
- For this case the following limits are to be used:
- the extreme high q_c values in sections thinner than 1.0 m should be filtered,
- q_b > 5.0 MPa can be used only if verified by pile load test,
- $q_{bmax} \leq$ 15 MPa for all cases,
- An extremely cautious approach is needed to estimate the base resistances of CFA-piles made in sands having calcareous bonds or liquefaction potential, because the soil condition can deteriorate suddenly throughout the formation.

Table 2

Pile resistance factors and limit values for coarse-grained soils

Pile type		Base resistance factor α_b	Shaft resistance factor α_{sq}	Maximum shaft resistance q_{smax} [kPa]
Displacement piles	driven precast concrete	1.00	0.90	150
	steel tube driving with closed end	1.00	0.75	120
	cast in place made by driving a steel tube with closed end and reclaiming during concreting	1.00	1.10	160
	screwed cast in place	0.80	0.75	160
Replacement piles	Continous Flight Auger pile	0.70	0.55	120
	bored piles with drilling mud	0.50	0.55	100
	bored piles with casing	0.50	0.45	80

4. THE RELIABILITY OF THE NEW PILE CALCULATION METHOD

The new pile calculation method was checked on a database of 63 CFA-pile load test results supervised by the author using an Excel spreadsheet for his calculations. The reliability of the method was quantified by statistical estimates.

The measured total resistances were separated into shaft and base resistance components to make possible the separate evaluation of the above new equations. Based on the shape of the load vs. settlement curves, the most probable value of the shaft resistance was established by drawing a curve from the settlement value s = (0.02-0.03)·D to the load axis (*Figure 3*, auxiliary line) bearing in mind the different mobilisation of the two components. The resulting components are shown as base and shaft mobilization in Figure 3.

Furthermore, the comparative analysis of time vs. settlement curves of the stages of a pile load test allow for making more accurate the separation of resistance components by noting changes in pile behaviour, namely,

- a strong increase of the time needed for the pile consolidation, accurately, for the achievement a settlement rate of 0.025mm/5min;
- an obvious change in the shape of the time vs. settlement curves with both data normalized by their final value;
- the load at which the final settlement is about 1.0 mm higher than the initial one.

The statistical parameters of the ratio of the pile resistances measured by pile load tests and calculated by the new method are shown in *Table 3*. It should be emphasized that good agreement, e.g. the $\kappa_I \approx 1.0$ values were achieved by the introduction of the λ_b = 0.6 reduction factor for the base resistances in coarse-grained soils. As it was mentioned such a reduction with a similar value is suggested by De Ruiter and Beringham, and it is cited in other papers and books *(e.g. Tomlison, 2001, Lunne et al., 1997)* as well as inthe Dutch Code, but it was not put in into EC 7-2.

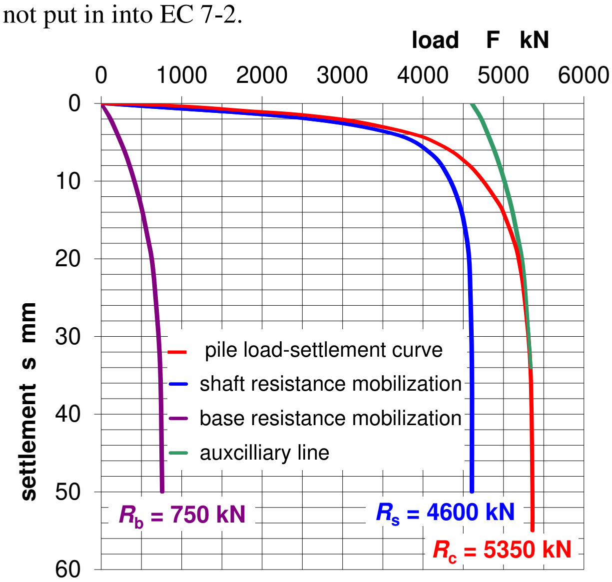

Figure 3. Separation of the pile resistance components

On the other hand, in most of the standards, e.g. in German Standard the suggested characteristic values for pile base resistances in coarse-grained soils (q_b) are quite small related to the average q_c values around the pile base, multipliers of about 0.2–0.3 can be observed. The multiplication ($\lambda_b \cdot \alpha_b$) in the equation (10) for CFA-piles with λ_b = 0.6 and α_b = 0.7 *(from Table 3)* is 0.42 and it uses a q_c value which can be considered more cautious than the average value around the pile. That means the use of λ_b = 0.6 reduction factor is by and large in accordance with the Geman and Dutch codes and with other suggestions. It gives quite smaller pile base resistance than EC 7-2, but the reason for that is the error in the citation of Dutch Code in EC 7-2.

From Table 3 one can establish that the new method gives reliable results:

- the mean values of measured/calculated resistance(κ_i) differ very little from 1.0 and the coefficients of variation (~0.15) are also favourable,
- the reliability of specific shaft resistance is slightly better than that of base resistance; in particular, coarse-grained soils show very good agreement. This indicates that it was appropriate to calculate them by the squareroot function,
- the base resistance in coarse-grained soils is the most uncertain result, which is certainly caused by the strong effect of the CFA-technology.

Table 3

Parameters of the reliability of the new calculation method for CFA-piles

Pile resistance	Soil type	Number	Ratio of the measured and calculated pile resistances		Correlation parameters of the measured and calculated pile resistances		
			mean value	coefficient of variation	regression line gradient	regression coefficient	5 % lower limit line gradient
		N	κ_i	v_i	a_1	R^2	a_2
R_c	all	63	1.01	0.17	1.00	0.77	0.80
ultimate compressive	coarse-grained	10	1.01	0.11	0.98	0.91	0.85
total pile resistance	fine-grained	14	1.01	0.15	1.05	0.85	0.85
q_s	all	63	0.98	0.13	0.99	0.67	0.80
specific value of pile shaft	coarse-grained	12	0.97	0.08	0.96	0.85	0.85
resistance	fine-grained	18	1.02	0.13	1.03	0.72	0.85
q_b	all	63	1.07	0.32	0.96	0.71	0.70
specific value of pile base	coarse-grained	40	1.09	0.34	0.95	0.71	0.70
resistance	fine-grained	23	1.03	0.27	0.99	0.63	0.75

Figure 4 shows the measured and the calculated total pile failure resistances and the 90 % confidence intervals for prediction. Practically, no point is below the lower confidence line which means that the total pile resistance was never overestimated to a critical extent. For further use it may be better to draw lines in an intuitive manner, starting from the origin, having an a_2 gradient and ensuring that not more than 5% of the points fall below it. Thus, the $1/a_2$ value may be considered a safety factor which assures that the expected resistance may be smaller than the calculated one in only 5% of the cases. From the *Table 3* $1/a_2 = 1.18 - 1.43$ safety factors can be determined as necessary on the resistance side. The a_2 values show again that base resistance in coarse-grained soils is quite uncertain.

The reliability of the pile design was also investigated using the reliability method suggested by *EN 1990* and *Szalai (1998)*. The uncertainty of the the parameters influencing the pile resistances were assessed and the necessary global factor of safety were calculated. The results of these calculations were taken into account as well as data from Table 3 to determine the safety parameters presented in the Hungarian National Annex of EC 7 (see *Szepesházi, 2011/b and /c*).

5. SUMMARY

The EUROCODE 7 gives clear and forward directives for pile design. The acceptable methods for determining the pile resistances and the frames of the safety are well established. But, no method for the calculation of the pile compression resistance is proposed, and there are only suggested values for the safety elements.

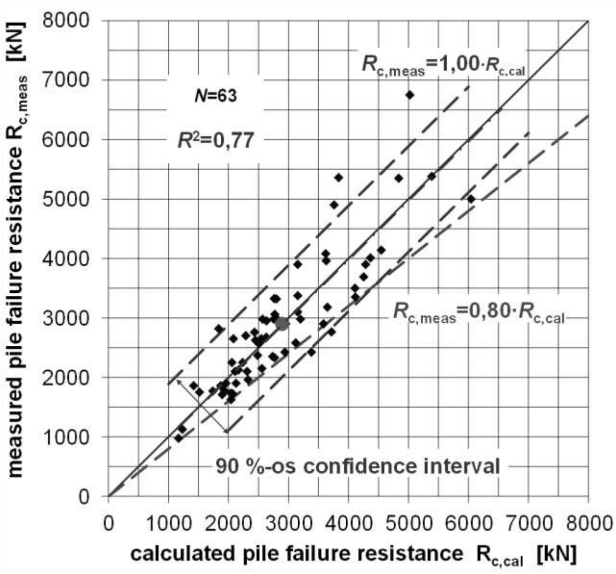

Figure 4. The correlation of the total pile resistances

However, the requirement is formulated that calculation method based on ground tests results may be only applied if they are verified by static load tests. Furthermore, it is clear, that the values of safety system should be defined related to the reliability of the calculation method chosen.

In this paper a calculation method is presented which is based on CPT results, which is surely the most obvious possibility in the practice. The method is a synthesis of the methods found in various national codes and recent design experiences. The main special feature of the method is the square root formula for the calculation of shaft resistances and the introduction of a reduction factor for the base resistance in course-grained soils. The method was verified by static load tests of 63 CFA-piles. The method was extended for other pile types by using factors to take into account the technology and these were defined by comparison of national code suggestions. The reliability of the new method was assessed on comparison of the calculated and measured pile resistances and by using reliability method analyzing all the uncertainties of the pile resistance elements. Based on these findings a safety system for the pile design was elaborated and involved into the Hungarian National Annex of EC 7. These system and the values applied fit the requirements of the EC7, perhaps much more than the most of the national annexes, and so it can be recommended for an international use.

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COMPARISON OF LANDFILL STABILITY ANALYSIS RESULTS BASED ON LITERATURE RECCOMMENDATIONS

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ABSTRACT

Determination of MSW shear strength parameters is a difficult task because of its heterogeneity. It has been widely researched using laboratory, in situ test results, or back analysis calculations. The aim of my research is to define the stability of a landfill based on different literature recommendations and compare it to the results of our laboratory tests. In our laboratory tests the degree of the municipal solid waste degradation was also considered.

1. INTRODUCTION

Rapid growth of population, urbanization, economic growth, and the increase in the standard of living have all contributed to the fast increase of municipal solid waste. More waste coupled with increasing prices of sites forced engineers to design higher and steeper landfills for better utilization. Changes in the size of landfills result in increased shear strength. In the analysis of long term behavior of landfills, slope stability analysis plays a major role because the gas and leachate pipes, the monitoring and lining system can easily get damaged *(Szabó I., 2009)*.

Calculating slope stability of landfills is a difficult task given the heterogeneity of MSW. The material, the size of the parts may vary by magnitudes making it very difficult to characterize the MSW body with a single pair of average shear strength parameters *(Szabó I., Faur K. B., 2011.)*. Several values have been proposed in the literature but none of those can be economically applied to all landfills *(Dixon, N., Jones, D. R .V., Whittle, R. W., 1999)*. In this paper we determine the safety factor of a given landfill using different methods proposed in Hungary and internationally and then we compare the results.

2. MATERIAL AND METHOD

2.1 The phases of waste decomposition

Previously I carried out a research on the effect of decomposition to the shear strength of MSW *(Varga, G., 2011)*.

In our computer based simulation we divided the waste body into five layers, corresponding to the degree of decomposition. We varied the parameters of waste layers according to the degradation phases.

Pohland et. al. describe five distinct phases of waste decomposition *(Pohland, F. G., Harper, S. R., 1986)*:

1. Phase I (lag phase). It is an acclimation period in which moisture starts accumulating and aerobic micro organisms begin to consume the oxygen trapped in freshly deposited solid waste.

2. Phase II (transition phase). This is the first anaerobic phase where total volatile acid (TVA) reaches a detectable level and chemical oxygen demand (COD) increases.
3. Phase III (acid formation phase). The second anaerobic phase is characterized by microbial conversion of biodegradable organic content and the activity of acidogenic micro organisms increases.
4. Phase IV (methane fermentation phase). In this phase intermediate acids are consumed by methanogenic bacteria and converted into methane and carbon dioxide.
5. Phase V (maturation phase). A marked drop in landfill gas production, stable concentrations of leachate constituents, and the continued relatively slow degradation of recalcitrant organic matter all characterize this phase.

2.2 The effect of decomposition on the safety factor

As discussed before, shear strength parameters of MSW vary widely. In previous studies we defined the safety factor of MSW as a function of decomposition both for layered and staggered layering. We used Plaxis and GEOSLOPE software and results are discussed below.

2.3. Static slope stability analysis with PLAXIS 8.0

The stability of a landfill as a function of decomposition was analyzed.

Finite element modeling

Figure 1 depicts the cross-section of the landfill under examination. Five layers corresponding to the five degradation phases were defined. Characteristics of each layer change with degradation, thus their parameters continuously need to be adjusted. The landfill is modeled as a two dimensional plane strain model with a 3H:1V slope *(Varga, G., 2011).*

Material Model and Type of Analysis

The analysis used the Mohr– Coulomb model. This model involves five parameters, namely Young's modulus, E, Poisson's ratio,υ, the cohesion, c, the friction angle, ϕ, and the dilatancy angle, ψ *(Varga and Czap, 2004).*

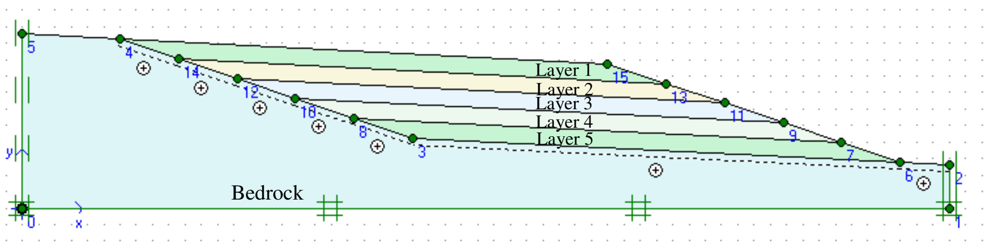

Figure 1. Cross- section of the examined landfill

The dilatancy angle was assumed to be zero. *Table 1* summarizes the parameters used in the FEM analysis of different phases of decomposition.

I used the parameters that resulted from my laboratory tests.

Table 1

Parameters used in finite element calculations

	Unit weight γ(kN/m^3)	**Poisson ratio**	**Cohesion c(kPa)**	**Internal friction angle φ(°)**
Phase 1	8.13	0.25	26.15	35.42
Phase 2	9.11	0.4	22.13	22.92
Phase 3	9.44	0.41	12.68	23.08
Phase 4	10.11	0.42	9.87	21.74
Phase 5	10.70	0.45	3.44	19.77

Unit weight of solid waste has a major effect on estimating the stability of landfills. The type of waste, the degree of decomposition and compaction, and the depth from which the sample is taken all influence unit weight *(Zekkos et al. 2006)*. The higher the degree of decomposition, the larger the unit weight is. With degradation, larger particles in MSW break into smaller pieces, which reduce the voids and increase the density of solid waste. In Plaxis one may use multiple stiffness moduli, such as shear modulus, G, and the odometer modulus, E_{oed}. According to Hooke's law of isotropic elasticity, which involves Poisson's ratio, these moduli are related to Young's modulus. Poisson's ratio is higher for waste with advanced stage of decomposition than for freshly deposited waste. The factor of safety was determined using the φ-c reduction method of Plaxis. It was computed for each stage respectively. This is in line with the EUROCODE stability analysis.

3. RESULTS

Decomposition was found to have a significant impact on stability. The factor of safety decreased with advanced decomposition. As we mentioned above, with time, every layer of waste advanced to the next phase of decomposition, which degraded their stability properties *(Varga, G., 2011)*.

With time, failure surfaces have moved up along the slope risking the stability of a larger and larger waste body. As the decomposition of MSW in landfills advances the extent of the collapsed waste body increases, which can result in a catastrophic failure and may compromise the utilization of the landfill.

Table 2

Factors of safety from PLAXIS at different stages of decomposition

	Stage 1	**Stage 2**	**Stage 3**	**Stage 4**	**Stage 5**
PLAXIS	1.232	1.138	1.086	1.056	1.026

Decrease in shear strength results in a decreased safety factor. Based on these result we suggest taking time and the phase of decomposition into account when analyzing the stability of landfills rather than using average values or values determined at deposition.

When reviewing the stability of landfills it is recommended to use degradation dependent soil mechanical parameters. Resulting factors of safety are smaller than factors of safety coming from freshly deposited waste parameters. In order to achieve safe operation degree of decomposition is not to be neglected.

4. STATIC SLOPE STABILITY ANALYSIS WITH GEOSLOPE PROGRAM

The examined landfill stability problems have also been solved with the help of the Geoslope computer simulation. We used the same solid waste parameters, dimensions, and procedure as those of the FEM analysis. The aim of the calculations is to determine the critical failure surface that belongs to the lowest factor of safety. Our calculations were based on the Morgenstern-Price method *(Varga, G., 2011)*.

4.1. Results of the GEOSLOPE simulation

Similarly to the results of FEM analysis factors of safety decreased with the advancement of degradation, while critical failure surface extended. Table 3 shows factors of safety from PLAXIS and GEOSLOPE simulations at the 5 stages of decomposition analyzed. The results of the two simulations are very close, with GEOSLOPE having slightly lower values that can be explained with the differences in their respective models.

Table 3

Comparison of safety factor as a function of degradation using Plaxis and GEOSLOPE program

	Stage 1	**Stage 2**		**Stage 3**	**Stage 4**	**Stage 5**
PLAXIS	1.232	1.138		1.086	1.056	1.026
GEOSLOPE	1.226	1.117		1.060	1.034	1.019

4.2. Stability analysis based on literature data

In order to compare the results of my previous studies to literature data, I have performed the above simulations based on literature data with the help of the GEOSLOPE program both for layered and staggered landfills. I have used the following literature data:

- Recommendation of the Austrian ÖNORM 2073, which generally uses $\varphi = 25°$ $c = 5kPa$. It treats the whole MSW as a homogenous body *(Faur K. B., 2012)*.
- Recommendation of Sanchez-Alcituri et al.: $\varphi = 20°$ $c = 10kPa$ and $\varphi = 20°$ $c = 20kPa$. This method proposes to use only parameters that are supported by both in situ and laboratory tests. It also calculates with a homogenous waste body.
- A. Szabó uses probability variables, where the randomly changing shear strength parameters of MSW are used in a way that each layer is assigned a random shear strength parameter but with the same probability based on literature data. Shear strength is analyzed repeatedly for each layer with multiple parameters. As a result of this, a distribution function of the safety factor is generated. It then shows the probability of a given or expected safety factor parameter *(Szabó A., 2008)*.

In case of determining shear strength parameters as probability variables I used the results from my own laboratory tests. Average results is shown in following figure.

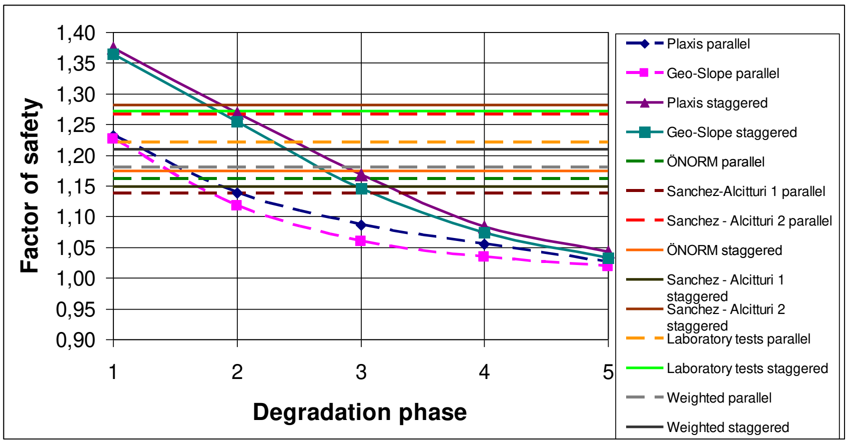

Figure 2. Comparison of safety factors of literature data and laboratory results

5. CONCLUSIONS

Numerical tests were used to confirm that the degree of decomposition affects the stability of landfills. I have compared literature recommendations with results coming from laboratory tests performed on Hungarian solid waste. My model divides the waste body into five layers according to the degree of decomposition.

I used PLAXIS and GEOSLOPE program in my simulations then I compared their results. I have found that the factor of safety decreased significantly with the advancement of degradation *(Varga, G., 2011)*.

The results show that the factors of safety computed from literature recommendation parameters are close to the 2nd or 3rd degree decomposition parameters determined by my simulations. With the advancement of decomposition, the factor of safety may be smaller than the results coming from literature recommended parameters, which may compromise the stability of landfill.

Accordingly, I propose that the stability of landfills should be determined with shear strength parameters defined as a function of degradation (and time). Commonly applied fresh waste- or average-based parameters may generate unjustifiably high safety factors, which may result in unexpected stability problems. Literature recommendations should be treated uniquely in every landfill with careful considerations.

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GENERAL CHARACTERIZATION OF MINERAL AND THERMAL WATER RESOURCES IN THE TOKAJ MOUNTAINS

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ABSTRACT

The Tokaj Mountains are one of the most well-known volcanic mountains in Hungary. They are located in the north-eastern part of the country. Their geology and mineralogy have been studied thoroughly, however we have just little knowledge about the hydrogeological features. Research on the hydraulics of groundwater and water-rock interactions have been performed earlier, but a detailed understanding of the flow system was lacking. A goal of the WELL aHEAD project (launched in 2012 at the Institute of Environmental Management, University of Miskolc) is to study the groundwater flow system of the Tokaj Mountains in details. This work includes the investigation of water chemistry, for which samples from 37 wells and springs were collected on the site, and the temperature measurements of outflow water were carried out. A database of water chemistry and geothermal parameters have been created for the management of our measurement data, which can be completed arbitrarily with other available data.

INTRODUCTION

The Tokaj Mountain range, located in the north-eastern part of Hungary as a part of the Carpathian Basin, is one of the most famous volcanic mountains of Hungary. It strikes north– south, and it is approximately 100– 120 km long. The mountains run between Tokaj and Eperjes (Slovakia) *(Kiss, 2007)*. The thermal and geothermal research in the Tokaj Mountains has always been of secondary importance from the point of view of hydrogeology, due to its complex geological structure and the insufficient number of deep exploration wells. During our research we experienced that several potential thermal aquifers may be found in the investigated area despite the complex geological structure. Many wells in the mountains produce good quality outflow water with high amount of total dissolved solids. We collected data about various parameters, then a hydrogeological – hydrochemical database was created, which serves as the base of the regional hydrogeological research.

GEOLOGICAL BACKGROUND

The investigated area was defined mainly by natural boundaries. The boundary in the North is the Hungarian-Slovakian border, while the western boundary runs along the Hernád Valley, which is a north-northeast south-southwest structural line, known also as the ALCAPA-Tisza tectonic line, a part of the Central Hungarian lineament. The southern boundaries of the area are the Tisza and the Sajó Rivers. Finally, the eastern borders are the Tisza and the Bodrog Rivers. The geological structure of the Tokaj Mountains is rather complicated. The depth and the rock material of the basement are still in question. Some basement maps show that it is approximately at 1500– 2000 m deep and the material of the basement is probably metamorphic mica. Concerning the geological structure of the investigated area, it has been proved in the thermal karst of Sárospatak and Bükk that shallow marine carbonate sediments were also deposited in the Triassic. Neogene vulcanite formations settled on the basement with hundreds or – in some places thousands – of meters in thickness. The reason for the extraordinary thickness is that the volcanism has started in the Miocene and ended in the Lower-Pannonian. The Pannonian Lake gradually lost its salinity, and it was filled by river drift. The Upper Pannonian clastic sediments pinched out on the southern part of the research area, but toward the Great Hungarian Plain their thickness increases. In several areas above the Pannonian layers Pleistocene fluvial sediments have settled in large thickness, where the Holocene formations are negligible.

MATERIALS AND METHODS

Temperature values of water samples taken from the area of the mountains show that some subsurface structures situated in the eastern and western boundaries emerge from their environment. Higher temperatures coincide to high structural lines running on either side of the range. This indicates a high flow pathway, and the outflowing waters have higher TDS. From the data collected at the Eperjes-Tokaj Mountains an overall water chemistry database was created. This database contains 9 different sub-databases including more than 800 water samples. The water chemistry analysis are unfortunately rather incomplete. Only 66 springs and 54 wells had the same parameters required for further analysis (Na, K, Ca, Mg, SO_4, H_2CO_3, Cl). The Piper diagram is a graphical representation of the water chemistry values. The major cations and anions dissolved in the water can be plotted in a triangle diagram as a function of the concentration ratio. Subsequently the diamond diagram was constructed, in which the composition of the material is shown to demonstrate the aquifer system. The points of the triangular diagram are not indicated on the map, but they can be easily added with a color code. The vertices of the triangle diagram are assigned to the HSV color coordinates of a system parameter. The HSV color scheme turns the colors to the hue (H), saturation (S), and brightness (V-value) *(Luk, 2013)*.

RESULTS

The water chemistry data of the Tokaj Mountains are plotted in the Piper-diagram *(Figures 1 and 2).* In the figures the points of springs and wells were plotted separately.

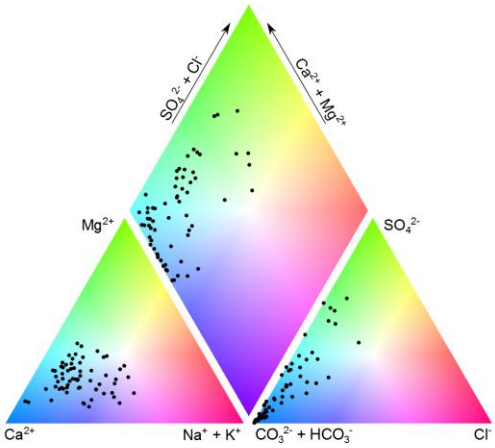

Figure 1. Chemistry of the water samples from springs in the Tokaj Mountains

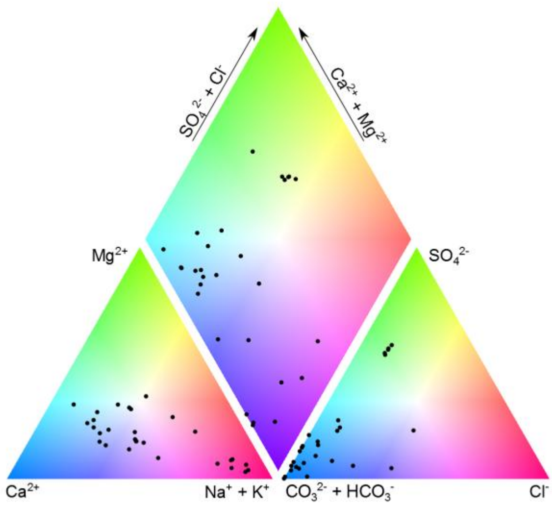

Figure 2. Chemistry of the water samples from wells in the Tokaj Mountains

The data points of *Figure 1 and 2* are also presented in the form of maps *(Figure 3 and 4).*

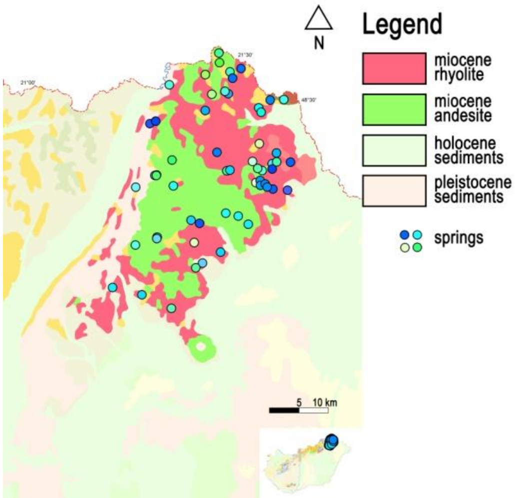

Figure 3. Springs in the Tokaj Mountains presented on geological map

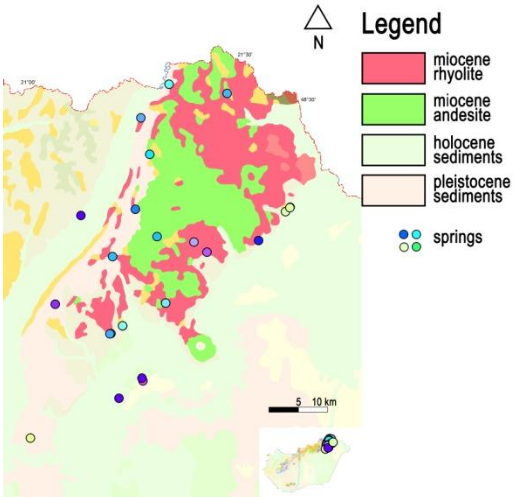

Figure 4. Wells in the Tokaj Mountains presented on geological map

Flow systems of the volcanic areas can be connected to local tectonics. The main structural lines of the mountains and the wells are presented in *Figure 5*.

We can present the different spring and well water parameters on the geological maps. The main reasons of the difference are the tectonics in the mountain and the types of the lava rock (andesite, rhyolite). In the northern and eastern part of the mountains there is much more springs, than on the other parts. So, the distribution of the water samples is not homogeneous *(Figure 3)*.

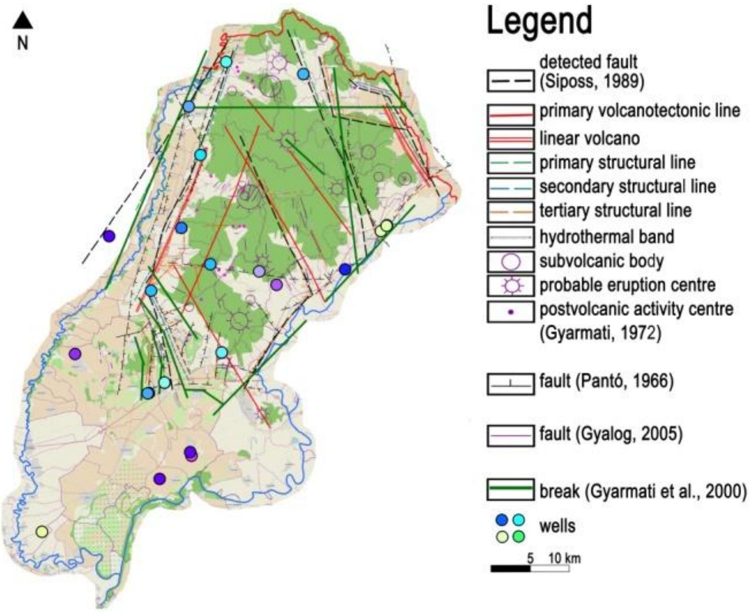

Figure 5. The main structural lines of the mountains and the wells

Lot of wells in the mountain do not have complete water chemical analyses results. Only 21 wells have got the main water chamical parameters for the Piper diagram. These wells were placed on the tectonical map, and marked the main tectonical and structural lines *(Figure 4 and 5) (Fekete et al., 2014)*.

CONCLUSIONS

In the studied area, the spring water has relatively low bicarbonate content. The chloride and sulfate content is varying. The most characteristic cation of the spring waters is calcium. Due to the low flow pathway, the sodium and magnesium concentrations are both low. The most characteristic anion of spring is the sulfate. The groundwater pumped from the wells have more hydrogen carbonate and sodium content, due to higher flow path. In particular, calcium and potassium rich sodium-water type is common in the mountains. From the point of view of anions the water samples from wells are of sulfate and bicarbonate types. The andesitic areas (green shaded area in *Figure 5*) is mainly dominated by pale green and pale blue circles indicate small or medium amount of sulfate and bicarbonate content of the material. The rhyolitic areas in turn have crucially higher bicarbonate components *(Figure 3). Figure 5* shows the effect of the structural lines on the water chemistry. The groundwater of aquifer in the volcanic area is mainly bicarbonate type (dark blue points in *Figure 5*). In the southern part of the mountains the Pannonian layers are thicker than in the north (purple points in *Figure 5*).

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THE PERMEABILITY OF LASER DRILLED LATERALS IN SANDSTONE RESERVOIRS

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ABSTRACT

The aim of our research is prove the performance of a recently developed economical and environmentally safe technology using subsurface laser drilling. Instead of the already tested and known laser spallation drilling technology another method is under development which based on the melting of the subsurface geological formations and the removal of debris. This technology may help both the well completion and rework tasks in fluid mining especially in geothermal energy and hydrocarbon production but also in special remediation works at contaminated sites.

The drilling procedure was performed using sandstone samples originated from the Upper Pannonian reservoirs of the Great Hungarian Plain in high pressure chamber.

Current tests were performed to determine the permeability of the system surround the laser completed holes and to compare them with permeability of the original „virgin" rock samples. There was a testing and drilling methodology developed in Hungary that was applied on hundreds of samples by the laser engineering company in Malta using their own laser device. The evaluation of test results may help the developers of the laser drilling technology to fine tune their developments on design and manufacturing laser driven boring heads.

Laboratory measurements indicate that this innovative new technology will help save reservoir integrity and will increase drainage area resulting in low pressure gradient both production and injection wells.

INTRODUCTION

The extraction of all fluids from the pore volume of any structure has a special importance in fluid mining, especially drinking water supply, geothermics and even in remediation *(Szanyi & Kovács, 2010)*. At all applications the need to produce the fluids using the smallest gradient is essential, therefore all developments that makes the reduction of well or production unit losses possible are of high importance. The laser drilling technology offers us a new possibility on forming subsurface structures since during the drilling the melted material forms the possible in situ "screen" of the tube *(Bajcsi et al., 2014)*. The drilling procedure is now on technical readiness level 2–3, therefore in situ tests are not possible now, but the drilling procedure can be performed on small samples in high pressure chamber or on real size samples in normal pressure conditions. The paper deals with the laboratory permeability tests performed.

PRINCIPLES OF LASER DRILLING TECHNOLOGY

Recent developments on the fields of laser technology enables us to use low energy loss high power laser devices (HPLD) even in large depths via the new standard high carrying capacity optical fibers.

The HPLD will utilize cutting-edge, underbalanced laser well completion and rework technology in fluid mining, including oil and gas as well as the geothermal industry. The system is comprised of a high power laser generator and a specially designed directional laser drilling head. The laser head is attached to coiled tubing or an umbilical system to maximize production and to carry out special jobs.

The laser tool will superheat the subsurface formation, melt the target material and will remove the molten debris while the borehole is being drilled *(Figure 1*, *Bajcsi et al. 2015 in press*).

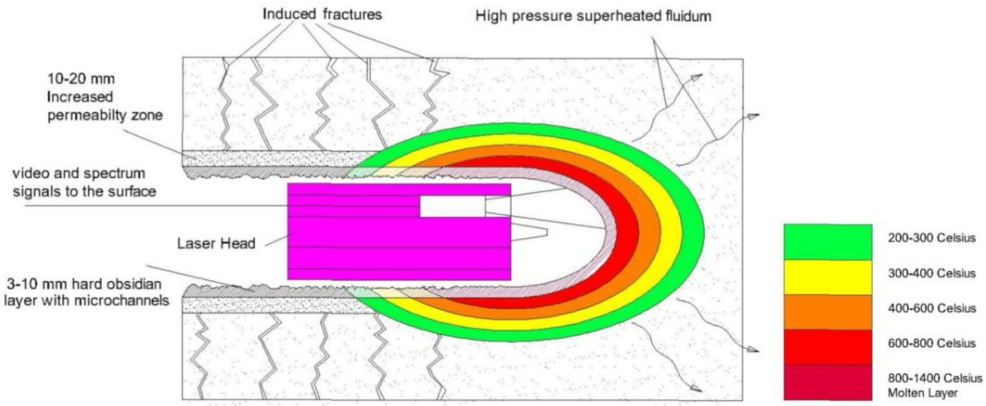

Figure1. Scheme of laser drilling procedure

The technology allows the operator to adjust the permeability of the borehole wall. The result of this process is a highly permeable approx. 1 – 2 inch diameter lateral pointing into any desired direction *(Figure 2)*, with large active surface to increase either water production yields or to reduce the pressure gradient nearby the opened section of the well.

Laser drilling complies with standard industry safety standards and rules and fits into existing drilling equipment and offers an in-situ, real-time fully controlled procedure with video and spectroscopy feedback to the operator, with no wearing parts, no chemicals and low maintenance while maintaining formation integrity and environmental safety.

In this phase of the development effort the laser technology is especially well suited for economically drilling short laterals from existing wells in a single work phase, drilling through the casing, cement and the formation as well, with one tool in and out. Elements of the technology are patent pending.

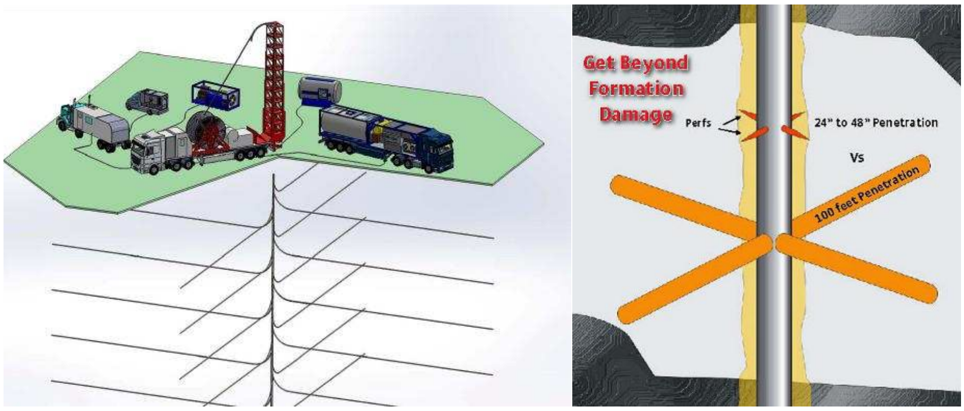

Figure 2. Laterals in any desired direction can be drilled using the heads (Bajcsi et al, 2015 in press)

PERMEABILITY TESTING METHODOLOGY

The permeability tests were performed on cylindrical samples using radial infiltration. The sample size was 54 mm diameter with 40 mm cylinder height that fits the standards 150 bar high pressure chamber of the laser testing company. There is a possibility to make pressure difference between the two sides of the sample that is to simulate underbalanced or over-pressurized drilling conditions.

At the beginning the samples were drilled out from large diameter oil industrial samples and cut to the predefined size. Using a special tool a small diameter hole of 8– 10 mm was drilled into the centerline (axis) using the traditional drilling technology. The permeability of the sample was tested using air, organic fluid and water. The organic fluid consisted of short carbon chain aliphatic hydrocarbons of a constant mixing ratio. Before, between and after the tests the pore fluid was pressed out or vaporized form the pore volume that was proven by scaling of the samples.

After the test of the “virgin” samples the laser treatment was done by the laser engineering company. A laser device with 1.5 kW laser beam power was applied until the drilling hole of the required size was formed. Due to technical reasons to avoid the damages of high pressure chamber damages and to keep ± 5 bar pressure difference on the samples during the drilling the mechanically drilled hole was filled up using the drilling powder produced during the mechanical drilling procedure.

After the laser treatment the permeability of the sample was once more measured using air, organic liquid and water.

To make the permeability tests a self developed pressure chamber was used, which contained seven sample holders. The tests were completed at constant hydraulic gradient (constant head) for the seven parallel measurements using 50– 200 kPa overpressure in the pressure container. The applied overpressure was reverse to the average permeability of the samples; the pressure range was set to highest measurement error reduction. The circle shaped walls of the samples were clogged using high deformable silicon plates and plastic silicone gel therefore the infiltration from the chamber could happen only on the side-walls.

The mechanically or laser drilled holes formed the drainage path from where the liquid or the air was led out and its volume was continuously measured.

Based on the well formula the yield (Q) of radial flow of v Darcy velocity through an area A, in a medium of K hydraulic conductivity can be calculated using the r radial distance as follows *(Figure 3)*:

$$Q = A \cdot v = 2\pi \cdot r \cdot l \cdot K \frac{dh}{dr}$$

or after integration:

$$Q = 2\pi \cdot l \cdot K \cdot \frac{H - h_0}{\ln \frac{R}{r_0}} .$$

This makes possible to calculate the K hydraulic conductivity or k permeability of the sample based on ν kinematic viscosity of the liquid and ρ_v liquid density and the g gravitational acceleration:

$$K = \frac{Q \cdot \ln \frac{R}{r_0}}{2\pi \cdot l \cdot (H - h_0)} \quad \text{and}$$

$$k = \frac{K \cdot \nu}{\rho_v \cdot g} = \frac{\nu \cdot Q \cdot \ln \frac{R}{r_0}}{2\pi \cdot l \cdot (H - h_0) \cdot \rho_v \cdot g} ,$$

respectively. The meaning of the letters is shown in *Figure 3*.

After the laser drilling a rigid wall is formed from the melted material that has an approximately constant thickness. As previous investigation proved that the permeability changes of the non-melted region is irrelevant, therefore the total hydraulic head using the R total and R_0 melted range radius can be written into the form:

$$H = H_0 + \frac{Q}{2\pi \cdot l \cdot K_1} \cdot \ln \frac{R}{R_0} = h_0 + \frac{Q}{2\pi \cdot l \cdot K_2} \cdot \ln \frac{R_0}{r_0} + \frac{Q}{2\pi \cdot l \cdot K_1} \cdot \ln \frac{R}{R_0} .$$

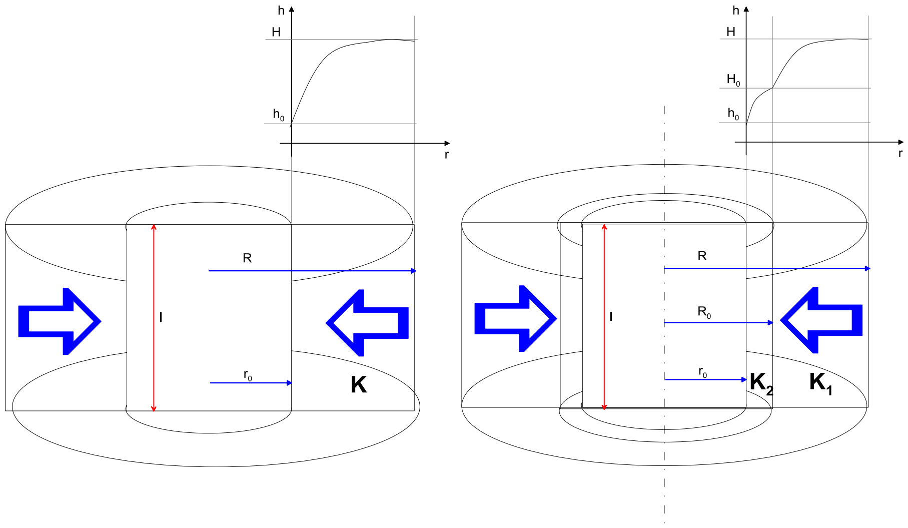

Figure 3. Calculation scheme of permeability in homogeneous and axially symmetric inhomogeneous system

The hydraulic conductivity and the permeability of the melted zone can be calculated as follows:

$$K_2 = \frac{\ln\frac{R_0}{r_0}}{2\pi \cdot l \cdot \frac{H-h_0}{Q} - \frac{\ln\frac{R}{R_0}}{K_1}} \quad \text{and}$$

$$k_2 = \frac{K_2 \cdot \nu}{\rho_v \cdot g} = \frac{\nu}{\rho_v g} \cdot \frac{\ln\frac{R_0}{r_0}}{2\pi \cdot l \cdot \frac{H-h_0}{Q} - \frac{\ln\frac{R}{R_0}}{K_1}}.$$

The above described testing methodology has several approximations: First of all the melted surface may have irregular surface *(Figure 4)*, moreover the interface between the melted and undisturbed zone has a given extent *(Figure 5)*. The biggest measurement problem is that in the reality the heat shock creates numerous micro and macro-fissures in the sample *(Figure 6)* that are prescribed flow paths that are included into the calculations. Previously some plug samples were investigated where the plate perpendicular to the axis was melted and laser treated *(Bódi, 2014)*. In this case a smaller heat shock was applied and therefore the forming of heat induced fractures was almost negligible. Based on the permeability measurements it was stated obviously that the extent of the permeability change remains within 25 percent in most of the cases, which has no harmful effect on hydraulic well performance *(Bajcsi et al., 2015 in press)*.

In our case the investigation of over two hundred samples was performed to analyze the probabilities of different phenomena.

Figure 4. Laser drilled holes with melted smooth and bubble containing surfaces with and without heat shock induced fractures

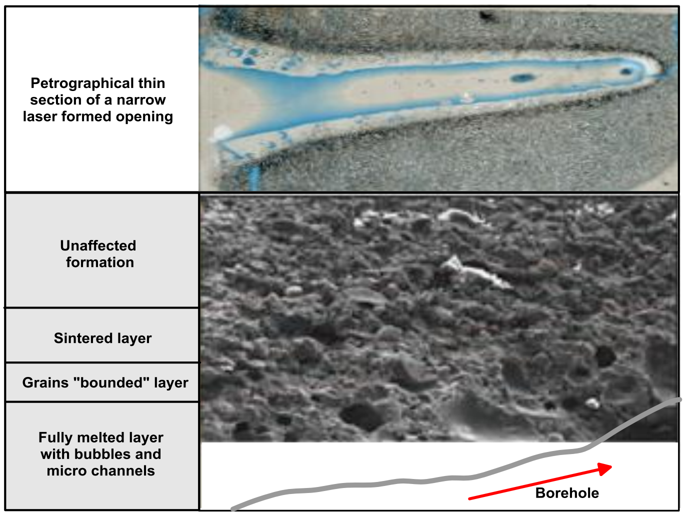

Figure 5. The optical microscopic (above) electron microscopic (below) image of the laser drilled hole's surroundings

PERMEABILITY MEASUREMENT RESULTS

During the test phase the permeability of several hundreds of samples were measured. Besides air, water and aliphatic hydrocarbons were used as test liquids. The permeability of "virgin" or original samples that were collected from the lowest permeability sections of the sandstone reservoir was measured in the range of 0.01–200 mD *(Figure 7 and 8)*.

Surprisingly after the laser treatment the average permeability of the samples was increasing by a factor 1.5– 4. In some cases discrete fractures could be observed in radial directions but there were several samples where only invisible micro-fissures were formed due to the treatment. The increase of permeability was both due to the matrix and both to the fractures therefore this was called later as apparent permeability.

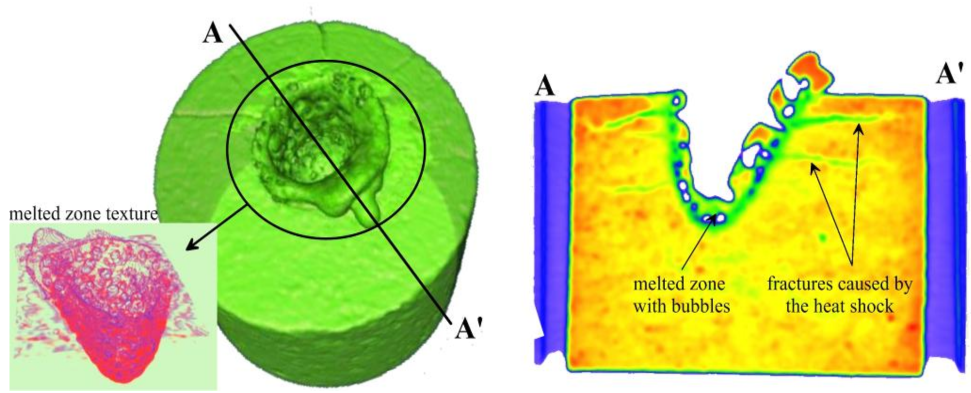

Figure 6. The 3D structure and the cross section of the laser treated sandstone sample on computed tomography images

Only in less than 5% of the cases the fractures did not reached the sidewall of the samples, in these cases a slight reduction of the permeability was observed. In a few cases the increase of permeability reached the 2– 3 order of magnitudes, these are generally not plotted due to technical reasons.

Based on the experiences the range of heat shock induced fractures is larger than 2 centimeters from the wall, which is coherent to the previous studies and CT investigations. Based on the test results it was proven that a 1 inch diameter laser drilled borehole acts as an approx. 3 inch diameter drainage system which has large side wall stability. During the tests no sand infiltration or leakage was experienced even at large hydraulic gradients (I > 600– 800 m/m). The tests showed that the laser drilled lateral may function even at unfavorable geological and geotechnical conditions as wet silty formations or soils liable to liquefaction.

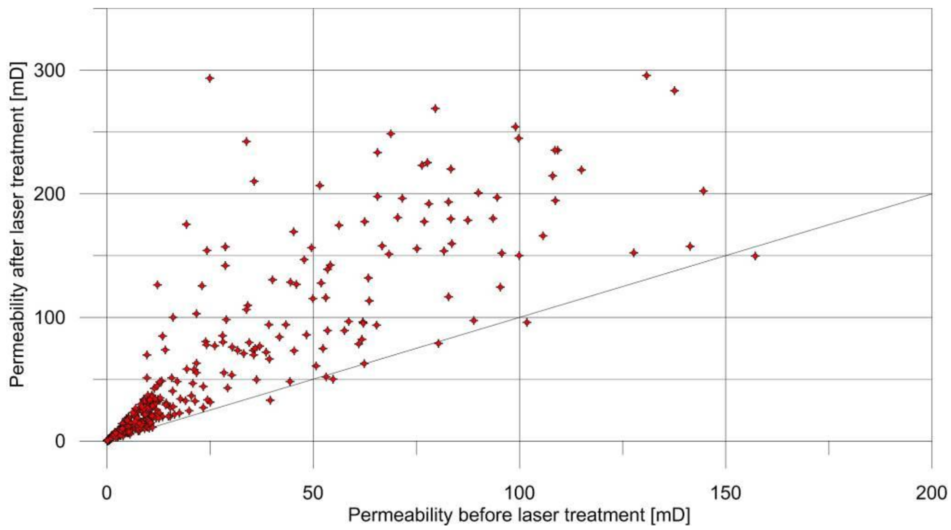

Figure 7. The apparent permeability increase due to heat shock induced fracturing caused by laser drilling

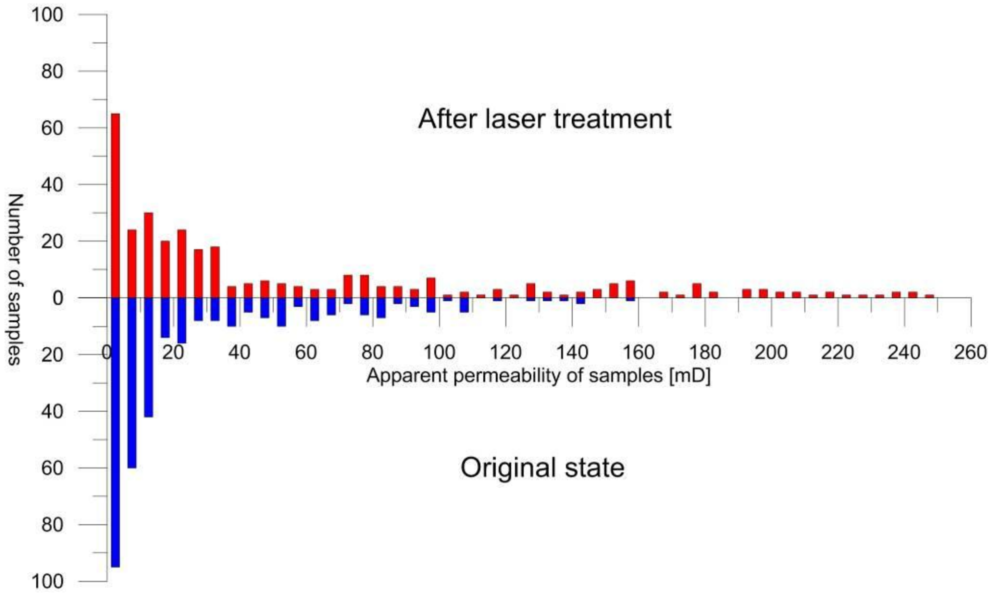

Figure 8. Histogram of apparent permeability of samples before and after the treatment

CONCLUSIONS

A series of permeability tests have been completed in order to determine the effect of laser drillings to the aquifer. Measurements indicated that the laterals drilled using this new technology increases the permeability round the lateral due to heat shock induced fracturing but it also keeps the reservoir integrity. During the tests no sand intrusions or leaking was detected.

The technology may help to complete injection wells in geothermal industry with higher productivity (Szanyi et al. 2011, 2012), to build remedial wells to inject nanoliquids right on the spot or to reach hot spots on highly covered industrial sites. Since the tests showed only small permeability reductions due to laser treatment, we may assume that the melted tube principle may hydraulically work in underground systems.

In hydrocarbon mining well completion laser offers some distinct advantages as easy steering and control. Since the laser head used nitrogen, all fluids are displaced during the drilling process, which prevents the formation from being contaminated by drilling mud.

ACKNOWLEDGEMENT

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THE ARTIFICIAL RECHARGES OF GROUNDWATER – A CASE STUDY

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ABSTRACT

Nowadays, the importance of artificial recharge of groundwater – used for short or long term underground storage – is increasing due to the growing demand for groundwater and to the continuous assurance of advantageous or at least acceptable agricultural conditions *(Scanlon et al., 2010)*. In this study we focused on performance of the only Hungarian groundwater recharge system that is continuously working to reduce the effects of groundwater discharge in the surroundings of Borsodszirák village. At the site we analyzed the hydrodynamic effects on groundwater level of the artificial recharge of groundwater using finite difference method based numerical simulations, highlighted the potential advantages and disadvantages what are hidden in the methods. Four models - systems were built with the same geological structure. In the simulations the aim of the hydrodynamic calculations was to determine the effects of the recharge to the water levels in each model – variants and compare the results, which scenario has the less harmful effect for the vegetation *(Nagy et al., 2007)*.

INTRODUCTION

In Hungary the main part of civil water supply is ensured from waterworks settled to subsurface water resources. The two–thirds of resources are in vulnerable geological environment, where the aquifers are not protected against surface contaminations. At sites of overexploitation the hydraulic heads in the aquifer are sinking endangering groundwater-related ecosystems. In this study we focused on the area of the Borsodszirák waterworks, which is using artificial groundwater recharge to improve the quality and quantity of existing groundwater through the modification of the groundwater flow potential field to modify the subsurface path lines of groundwater of high manganese, chloride and sulphate concentration and to enhance the groundwater resources. Four different scenarios of pumping/recharge rate were calculated and their effect to the groundwater-level was compared. Due to the fact that the agriculture needs balanced water budget to assure the acceptable conditions for the growing of the plants it can be possible to use artificial recharge in the future not only in groundwater production but also in plant or crop production *(Baser, 2004)*. We believe that all hydrogeological experiences gained from artificial groundwater recharge provide valuable information for agricultural engineers.

THE INVESTIGATED AREA AND THE APPLIED NUMERICAL SIMULATIONS

The investigated area is at the North-East side of Hungary, in the valley of the rivers Sajó and Bódva. In the region the surface is flat, surrounding agricultural areas. The cover is Holocene flood plain clayey formation with 3 m average thickness. The waterworks was settled to the second layer – 4–5 m thick Pleistocene gravely aquifer – with 40 shallow depth pumping wells and 25 infiltration basins. Unfortunately the hydraulic connection between the riverbed sediments and the gravelly aquifer is very poor; therefore water resources are supplied by the artificial recharge-system based on the surface water of the Bódva– river. The river drift is sedimented in a horseshoe-shaped basin before utilization, and in case of necessity an artificial filtering and cleaning is performed before the water is stored into two 400 m^3 clear water tanks. The pretreated water is pumped into the artificial recharge system where the water is infiltrated into the gravelly aquifer and after a natural purification the groundwater is exploited using the production wells.

For the numerical three-dimensional groundwater-flow simulations through porous medium we applied Processing MODFLOW software, which is based on finite difference method. The extent of model-grid is 3km x 5km *(Figure 1)*.

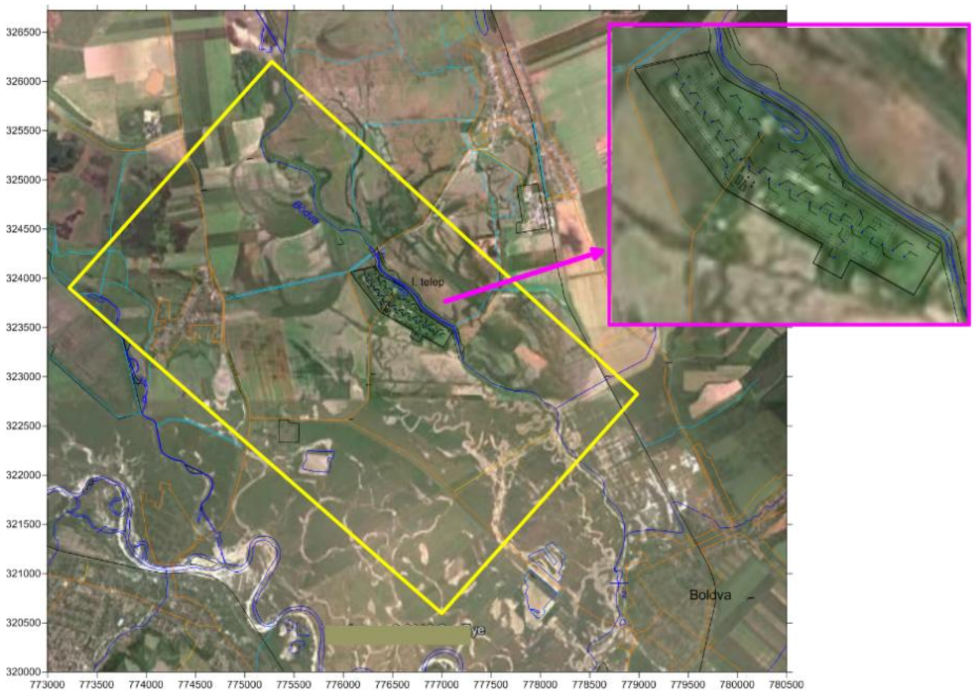

Figure 1. The mesh and the investigated area

The clayey cover formation's horizontal hydraulic conductivity is $8.5 \cdot 10^{-4}$ m/d *(Kovács et al., 2010)*. The aquifer is homogeneous and isotropic with a measured horizontal hydraulic conductivity of 90 m/d and an effective porosity of 0.2. The 25 infiltration basins are in direct hydraulic connection with the aquifer and treated as recharge cells *(Pedretti et al., 2012)*. The pumping wells and infiltration basins are shown in *Figure 2*.

Figure 2. The infiltration basins and wells

At all model variants the production of the pumping wells is 8730 m^3/d. In the 1st case there are no infiltration basins, in the 2nd case 50% of the production assured by artificial recharge. In the 3rd variant artificial recharge reaches the 120 % of the production (over-recharge). To determine the effects of the system a base model (4th variant) was calculated without any pumping wells.

RESULTS AND DISCUSSION

The results of the numerical simulations - the calculated hydraulic heads – are plotted in *Figure 3.* The analysis of the different scenarios shows that the increase of artificial recharge reduces the drawdown on the well system and reduces the area of influence as well. Analyzing the water budget *(Table 1)* one can see that the river leakage is weak to compensate GW production, therefore the waterworks may only operate simultaneously to the artificial recharge system, otherwise regional sinking of the GW table will happen. Without artificial recharge the operation of the waterworks at the recent production rate became impossible. In case of the 3rd and 4th scenario the drainage effect of the river is dominant. The artificial recharge of 2280 m^3/d is possible even when no pumping is done, that causes maximum 2.7 m rise of the water table at the site center.

Table 1

Water budget of the model varinats

Model variants	**Primer state**		**1. scenario**		**2. scenario**		**3. scenario**		**4. scenario**	
Flow term	In	Out	In	Out	In	Out	In	Out	In	Out
	m^3/d		m^3/d		m^3/d		m^3/d		m^3/d	
Wells	No pumping		-	8733	-	8733	-	8733	No pumping	
Recharge	No infiltration		No infiltration		4282	-	10277	-	2282	-
River leakage	27	230	661	29	477	56	5	432	-	618
Head dep. Boud.	1598	1395	8519	418	4719	689	1156	2273	1016	2680
Sum	1625	1625	9180	9180	9478	9478	11438	11438	3298	3298

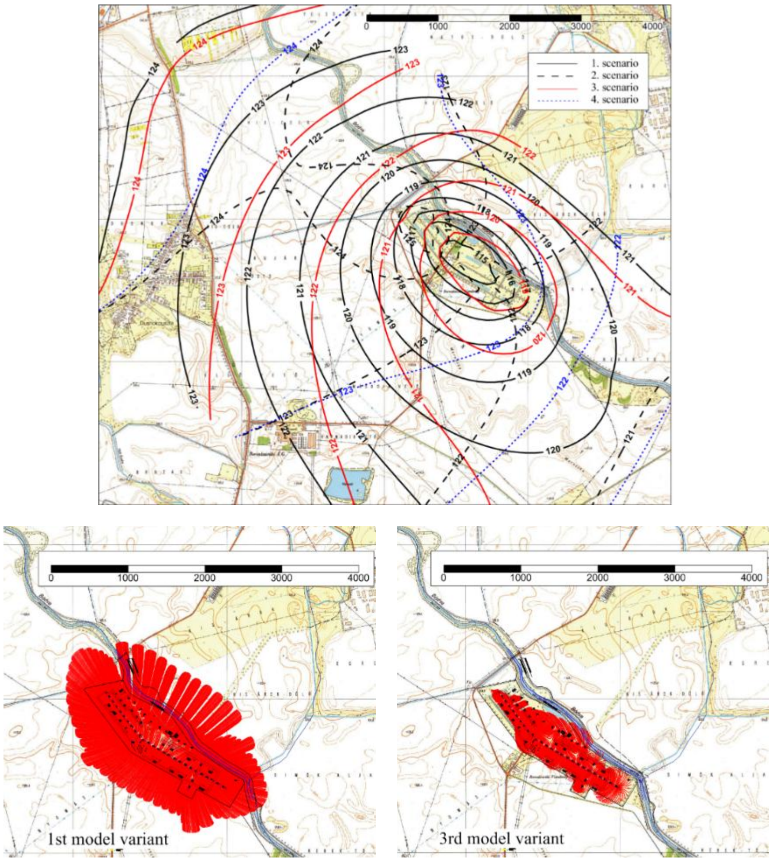

Figure 3. The calculated hydraulic heads (flow potential field) and 90 d path lines to the wells (case 1 and 3)

CONCLUSIONS

The study proved that artificial recharge can be an important factor in keeping the groundwater budget even at large precipitation deficits. The groundwater model of the Borsodszirák area showed that there is a potential of rising GW table locally up to 2–3 meters and 0,2–1 m in a wider area even in gravelly aquifers. Considering the flow equation at sandy groundwater aquifers smaller artificial infiltration rates may lead to the same effects, therefore at dry periods due to climate changes the artificial recharge may compensate the overexploitation of subsurface aquifers. There is a need to test and apply artificial recharge on more sites to get more theoretical and experimental experiences *(Várallyay et al, 1989)* on the investigated topics in the future to assure the stability of groundwater related ecosystems in Hungary.

ACKNOWLEDGEMENTS

The research was carried out in the framework of the Sustainable Resource Management Center of Excellence at the University of Miskolc as a part of the TÁMOP-4.2.2/A-11/1-KONV-2012-0049

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APPLICATION OF A MODIFIED ANALYTICAL SOLUTION OF TRANSPORT EQUATION TO DETERMINE LONGITUDINAL DISPERSIVITY

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ABSTRACT

The column tests are suitable tools to determine the transport parameters of the investigated porous media. The measurement protocol is simple and the originated data can improve the accuracy of mass transport calculations. In our work we performed labor experiments to determine the dispersion coefficient of the waste dump material of Rudabánya. In these experiments a unit concentration of pollutant was injected into the column and the spread of this pollutant was observed in time. The Dirac-function describes the concentration change in the function of time and distance. By minor conversion the independent variable of this function (time) could be changed to volume. In this paper we would like to introduce the application of this modified function. We used curve fitting method on the data of experiments to determine the longitudinal dispersivity of waste dump material which was found 7 cm. In the following transport modeling work we could integrate the result of this experiment into the models and we could start to investigate the adsorption process of heavy metals in the waste dump matter.

INTRODUCTION

In Rudabánya several ores of zinc, copper, lead and cadmium were originated near iron ores by hydrothermal processes. During decades of mining activity a lot of waste dumps were created with high heavy metal content. In our previous research works the significant heavy metal content of these waste dumps have already been proven by sequential extraction investigations *(Tóth & Kovács, 2013)*. Not only the total heavy metal content but also the mobile element content is high in the waste dumps that means the elements can mobilize by effect of weak acids. In our present work column tests were performed to determine the transport parameters of the waste dumps material to prepare the contaminant transport simulations of mobile heavy metal compounds.

The column tests are applicable tools to calculate the dispersion coefficient *(Czinkota et al., 2006)*.

The basis of calculation is the one-dimensional form of transport equation *(Kovács, 2004, Kinzelbach, 1986)*:

$$\frac{\partial C}{\partial t} = \frac{\alpha_L \bar{v}_x}{R} \frac{\partial^2 C}{\partial x^2} - \frac{\bar{v}_x}{R} \frac{\partial C}{\partial x} - \lambda C \, . \tag{1}$$

In case of an instantaneous injection of pollutant mass M in the spatial point x = 0 and at time t = 0. This initial condition could be described by Dirac-function:

$$C_\delta(x,0) = \frac{\Delta M}{n_0 m w R}\delta(x)\,. \tag{2}$$

The Dirac-function $\delta(x)$ is defined by

$$\delta(x) = 0 \text{ for } x \neq 0\,,\ \int_{-\infty}^{\infty}\delta(x)dx = 1 \text{ for } x = 0\,. \tag{3}$$

Further demand that the solution is bounded, this means

$$C_\delta(\pm\infty,t) = 0\,. \tag{4}$$

In this case the solution of equation 1. that satisfied equation 2 and 4 is the Gauss-function.

$$C(x,t) = \frac{M}{2wmn_0R\sqrt{\dfrac{\pi\alpha_L \bar{v}_x t}{R}}}\exp\left(-\frac{\left(x-\dfrac{\bar{v}_x t}{R}\right)^2}{\dfrac{4\alpha_L \bar{v}_x t}{R}}\right)\exp(-\lambda t) \tag{5}$$

where: *M* is the mass of pollutant, *w* is the infinitesimally small width of aquifer, m is the thickness of aquifer, n_0 is the porosity of aquifer, *R* is the reduction factor about the adsorption of pollutant, $\alpha_L$is the longitudinal dispersivity, $\bar{U}_x$ is the average velocity of seepage, *t* is the time, *x* is the distance and λ is the decay constant.

MATERIALS AND METHODS

In Equation 5. the concentration is the function of distance (x) and time (t). By a

conversion we could change the second independent variable from time (t) to volume of effluent fluid (q).This change is based on the Darcy-function.

$$Q = K \cdot A \cdot I \tag{6}$$

where *K* is the hydraulic conductivity, *A* is the flow cross-section, *I* is the hydraulic gradient and *Q* is the discharge where $Q = \frac{q}{t}$.

The average velocity of seepage is determined by the following expression.

$$\bar{v}_x = K \cdot I \; . \tag{7}$$

To substitute Equation 7 into Equation 6 :

$$\bar{v}_x \cdot t = \frac{q}{A} \; . \tag{8}$$

This conversion is practical because the measuring of volume is comfortable and it could be more accurate than measuring of time.

After substitution of Equation 8 into Equation 5 the Gauss-function could be modified:

$$c(x,t) = \frac{M}{2wmn_0 R\sqrt{\dfrac{\pi\alpha_L \dfrac{q}{A}}{R}}} \exp\left(-\frac{\left(x - \dfrac{\left(\dfrac{q}{A}\right)}{R} \right)^2}{\dfrac{4\alpha_L \dfrac{q}{A}}{R}} \right) \exp(-\lambda \frac{q}{A\bar{v}_x}) \tag{9}$$

where q is the volume of effluent fluid.

Column test was performed to determine longitudinal dispersivity. This transport parameter could be determined in this indirect measurement by curve fitting method on measured data. The best fitting of function is found by iteration of non-measurable parameters.

Some parameters of function could be measured. A 53 cm high plastic column was filled up with waste dump sample. The cross-section of column was A = 10.17 cm^2. In time t = 0 a high concentration of manganese solution was injected in the system. The total mass of manganese was M = 274 mg. The flow of water inside the column was continuous before injection. The manganese solution was seeped thought the column with a constant velocity (v = 0.002 m/s). The effluent fluid was collected on the outflow side of column. The time and the volume of collected sample were registered, simultaneously. The outflow solution was collected in approximately 25 cm^3 units. The effective volume of each unit was determined after mass measuring and dividing by density of solution. After filling up of each unit the related time was registered.

RESULTS

First of all the original Gauss-function was fitted on measured data. The measured concentration was graphed in function of time (*Figure 1*). The correlation coefficient of fitted curve was r^2=0.91. The values of fitted parameters are presented in *Table 1*.

The fitting of modified Gauss-function is displayed in *Figure 2*. By application of this function a better fitting was experienced. The correlation coefficient of curve is r^2 = 0.93. The values of fitted parameters are presented in *Table 2*.

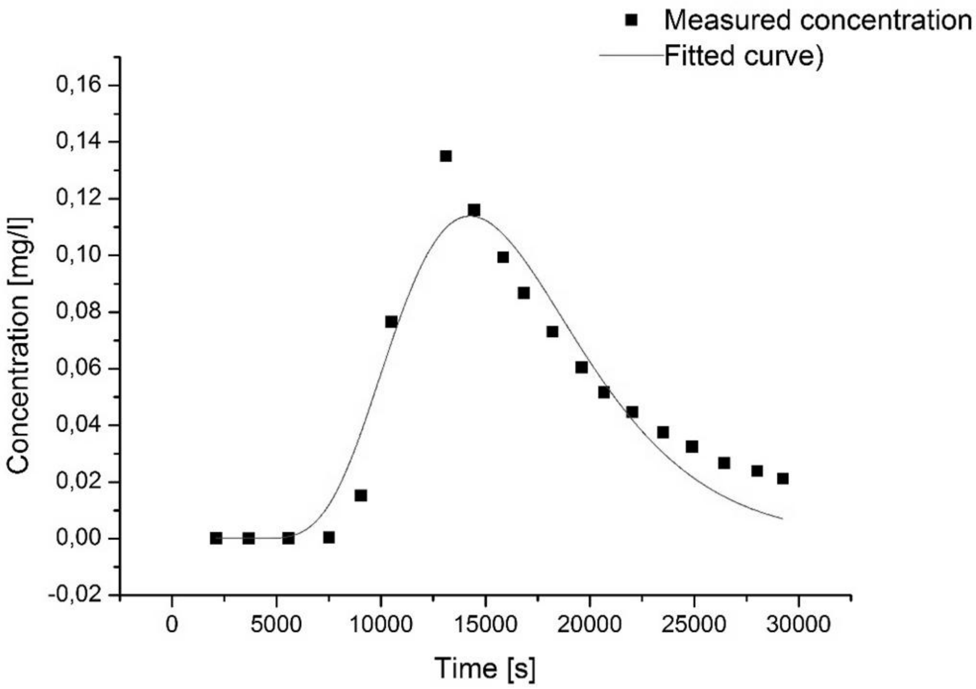

Figure 1. The measured concentration in function of time

Table 1

Values of fitted and measured parameters when the concentration is graphed in function of time

	Mass of pollutant	**Cross-section**	**Porosity**	**Retardation factor**	**Longitudinal dispersivity**	**Average velocity of seepage**	**Distance**	**Decay constant**
	M	A	n_0	R	α_L	v	x	λ
	mg	cm^2	–	–	cm	cm/s	cm	1/d
Value	274	10.17	0.35	0.80061	3.5521	0.002	52.7	0.0001737
Error	No fit	No fit	No fit	0.03769	0.60274	No fit	No fit	$8.538 \cdot 10^{-6}$

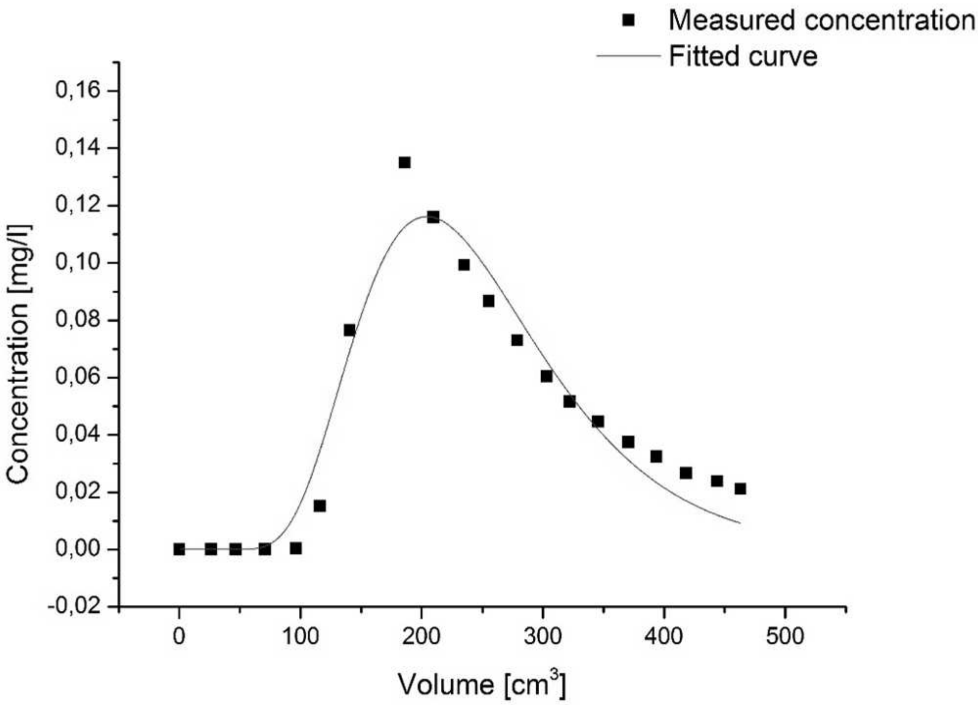

Figure 2. The measured concentration in function of volume of effluent fluid

Table 2

Values of fitted and measured parameters when the concentration is graphed in function of volume of effluent fluid

	Mass of pollutant	**Cross-section**	**Porosity**	**Retardation factor**	**Longitudinal dispersivity**	**Average velocity of seepage**	**Distance**	**Decay constant**
	M	A	n_0	R	α_L	v	x	λ
	mg	cm^2	–	–	cm	cm/s	cm	1/d
Value	274	10.17	0.35	0.69316	6.16084	0.002	52.7	$2.21 \cdot 10^{-4}$
Error	No fit	No fit	No fit	0.0423	1.01664	No fit	No fit	$1.22 \cdot 10^{-5}$

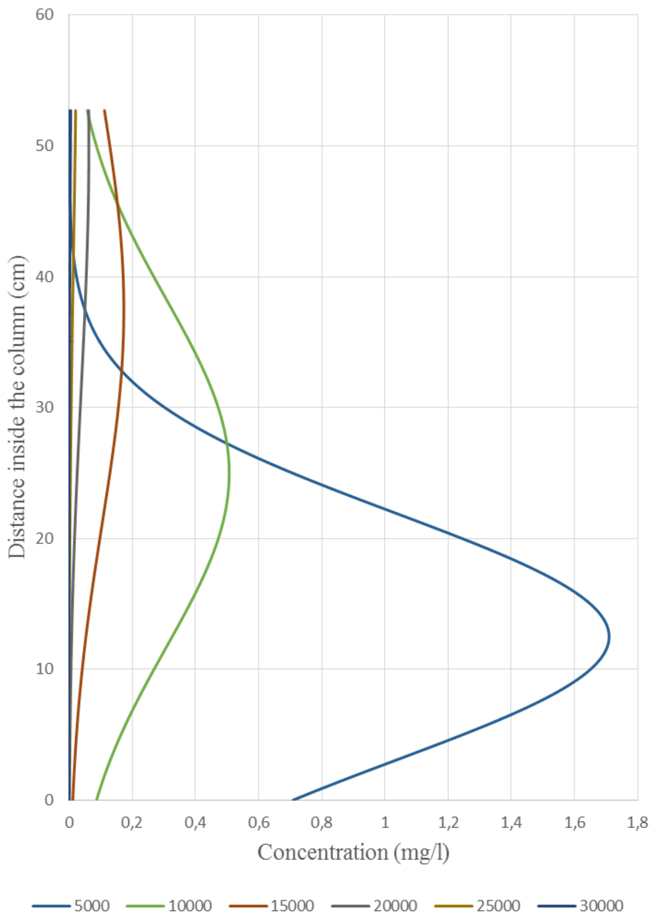

Figure 3. The concentration change of manganese solution along the column in different moments (unit of time: second)

CONCLUSIONS

In our previous works the longitudinal dispersivity of waste dump samples of Rudabánya have already been defined in breakthrough curve experiments *(Tóth et al., 2014)*. The breakthrough curve could be described by another one-dimensional solution of transport equation so the longitudinal dispersion could be determined with the same curve fitting method which method was introduced in this article. The calculated value of longitudinal dispersivity was 7 cm in the breakthrough curve experiment. In this investigation we could see the values of fitted parameters are largely depends on accuracy of measured parameters. In case of original function the correlation coefficient of best fitting is 0.91. The iterated parameter of longitudinal dispersivity is 3.5 cm.During application of modified Gauss-function the fitting of curve is better than in case of the original form of the function. Furthermore the determined value of longitudinal dispersivity is nearly 7 cm (6.16 cm) by fitting of the modified Gauss-function as like as in case of breakthrough curve experiment *(Tóth et al., 2014)*.

The concentration of effluent fluid could be graphed as a function of time or distance (Equation 9). After determination of fitted parameters of Gauss-function we could determine the concentration distribution of manganese solution along the column in different moments. In this case the concentration is the function of distance. (*Figure 3*) The graph shows the nonconservative case of pollutant distribution in different location and time because the value of decay constant $\lambda$is greater than 0 so the area under the curve is decreased more than the run out amount of pollution.

During measurement the decreasing of pollutant was observed because of irreversible adsorption of manganese. This process could be approximated by application of decay but it needs a better formula to describe the irreversible adsorption which process could be a kind of mineralization in case of waste dumps. In our following work we would like to investigate the irreversible adsorption and we would like to give a formula on it.

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THE IMPORTANCE OF BÜKK KARST WATER MONITORING SYSTEM (BKWMS) IN RESEARCHING THE RELATIONS OF COLD AND WARM KARST WATERS IN THE AREA

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ABSTRACT

Hydrogeological research of the whole Bükk region requires measurements to be carried out at many different locations and for extended times. Monitoring systems satisfy this requirement. Such systems are usually built on an ongoing basis, and at times the existing measurement sites are integrated into the expanded system *(Juhász, 1984, Lénárt, 2005, Schréter, 195)*.

The Bükk Mountain area is located in Northern Hungary. The size of the open (cold water) karst of the Bükk is 207 km^2 and the size of the water catchment area is 230 km^2. The thermal karst water system of the Bükk region is an estimated 1000 km^2 and its boundaries cannot be delineated exactly. The entire Bükk thermal karst is 4300 km^2.

Today, the Bükk Karst Water Monitoring System (BKWMS) together with other, hydrogeologically relevant measurement sites in the Bükk region is one of Hungary's determined largest karst water quantitative monitoring systems.

73 measurement sites have been created in caves, springs, karst-water monitoring wells, and in cold and warm karst-water wells. 15 to 17 million data sets have been collected so far. (The frequency of the measurement taking is mostly 5 to 60 minutes, recently 15 minutes.) Most of the data are water levels (water pressure), but almost as many are water temperature data sets. A significant amount of water conductivity data have been collected as well, and sporadically radon measurements were also taken. Most of the precipitation data was taken at daily intervals, but there are also hourly measurements. Over a 100 publications have been written about the results of the research *(Lénárt, 2002, Lénárt et al. 2013a, Lénárt et al. 2013b, Lénárt et al. 2013c).*

INTRODUCTION

We would like to discuss briefly our monitoring results obtained for 22 years in this paper, mainly for the sake of Professor Dr. Imre Szabó Head of Department, who has always been supporting our research as university leader.

Our potential and seal measurement sites and data collecting locations are marked on various professional maps. Our research supported by EU is devoted to the area shown in Figure 1 *(Karsztvíztestek, 2014)*.

Present paper is also a means to pay our respect to the professional work of Dr Tivadar Böcker and Márton Stéfán. In *1983, Tivadar Böcker* drilled the first monitoring wells necessary for the delineation of the cold and thermal karst water protected zones and thus he played a very significant role in the hydrological research of the Bükk. *In 1992, Márton Stéfán*, based on his professional leading role, was able to force the start up of the continuous device measured Bükk Karst Water Monitoring System *(Mező et al. 1995)*. The measurements have been carried out by the University of Miskolc, with the financial support of water works for whom the Bükk is a relevant area. Unfortunately, in recent years the maintenance of the monitoring system is becoming more and more difficult, partly due to lack of funds, and partly due to lack of professional approach.

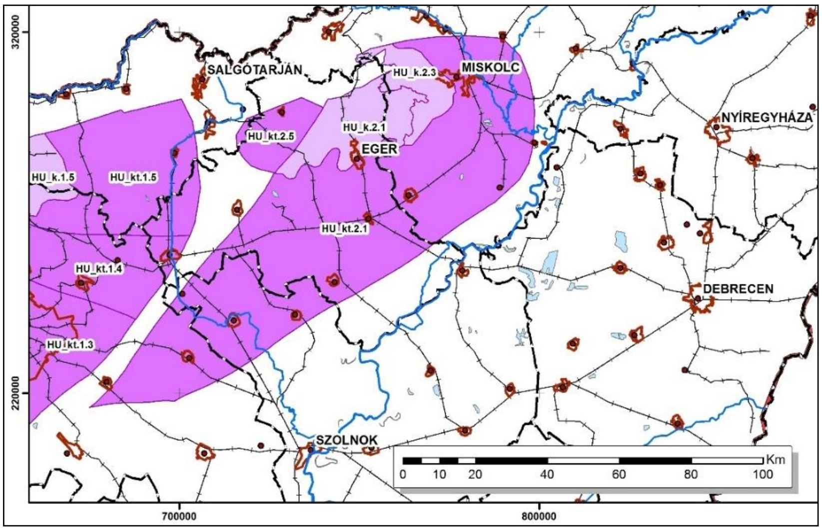

Figure 1. Karst water bodies of the Bükk mountain and its region [Karst water bodies] (HU_k.2.1, HU_k.2.3: cold karst water body; HU_kt.2.1, HU_kt.2.5: warm karst water body)

MATERIALS AND METHODS

The first step of the research was the data collection from various sources (hydrogeological logs, well cadastre, well examination documentations, in-cave measurement logs etc.), and to compile a simple Excel database. There were many data sets completely opposing each other. Only our own field knowledge helped us in these cases to actually define the real values.

Next, the following categories were defined:

Name	Piece
In-cave measurement site	18
Important karst spring	6
Karst spring used for water supply	31
Karst water monitoring well	54
Cold and tepid karst water productive well	7
Warm and hot karst water productive well (thermal well)	36
Water return thermal well	3
Weather station	11
Hydrological measurement object	8
Total	174
BKWMS measurement sites at present	73

Important to know that we receive measurement data regularly from 5 wells of the Egerszalók– Demjén area, and it is planned that we are going to receive further data from 6 wells in Bogács and Berva. In the framework of the research Kács– Sály, Miskolc and Eger protected zones, about 24 monitoring wells were drilled, but their data reaches us only sporadically. However, regular data exchange is underway.

There have been many changes in the locations of BKWMS measurement sites during these 22 years but there are 10 measurement locations from where at least 13 years worth of data is aviable, and also there are 6 measurement sites where data collection have been carried out continuously since 1992 *(Darabos & Lénárt, 2012, Szegediné et al. 2014, Szegediné et al. 2014)*.

Over 100 publications have been published regarding the evaluation results of the cold karst water of the Bükk and warm karst water of the Bükk region, and their relations *(Lénárt, 2010, Lénárt, 2011, Lénárt & Hernádi, 2012)*. Please, see the List of References for some of the most recent ones.

It is important to emphasize that our monitoring system have been an important element in both the GVOP-VIMORE, and the Kútfő university projects. Furthermore, we can provide data and interpretation for diagnostics evaluations in the Bükk, for the monitoring system of the Mályi-Kistokaj productive thermal well and water return system, and many other various researches.

RESULTS

The most important result of the paper is collecting most data relevant for the hydrogeological evaluation of the Bükk and its region, working out a database and preparing it for use.

The data collected is in a database, but to use it for other publications is difficult. Therefore we separated the data, and marked it on different maps. The accuracy of the information is enhanced by small part-maps on the main map.

The maps and the data shown are the following:

Springs joined to the drinking water supply system are shown on a topographical map, see *Figure 2.* These are mostly located around the brim of the Bükk. Significant karst springs are only present in the large valleys (Garadna Valley, Szinva Valley, Tárkányi Vallay, Szalajka Valley) inside the mountains. Besides the springs joined to the drinking water supply system, some large springs, important from ecological point of view, are also shown.12 of the springs on the map were or are still part of the BKWMS.

Figure 3 shows those caves that were part of the BKWMS for a period of time. These measurements yielded great results in the framework of the GVOP-VIMORE project and Miskolc diagnostics evaluations. Before that time, there was limited resource to carry out the measurements; today, it is not possible to continue them. Except for 2 caves, all measurement were carried out in caves situated in the anisus limestone zone since the most significant sinkhole caves of the Bükk are located there. Only a few dozen caves, out of the 1050 caves of the Bükk, are suitable for water level measurements from hydrological point of view, but only those are shown on the map where measurements actually did take place. The map provides good information regarding the situation of the karstified and non-karstified rocks, and regarding the karst water topography, based on the water level measurements taken in karst water monitoring wells and in caves.

Figure 4 shows the karst water level monitoring wells. Most of them follow the water level fluctuations of the cold or cold-tepid waters in open karsts. It is important that there is warm water monitoring well in Miskolctapolca, and a hot water monitoring well in the Zsóry Spa in Mezőkövesd. Since these wells are currently not productive, the water in them is significantly cooled off, so at the moment we are not measuring the temperature of the thermal karst water; but, these are still very important. There is or there was measurement carried out within the framework of the BKWMS in 27 of the monitoring wells shown on the map.

Figure 5 shows the thermal karst water wells, on Bouguer anomaly map *(MÁELGI, 2004)*. It seems that this geophysical map provides the best help in order to interpret the cold and warm karst water system relations; therefore we prefer to use it. The map shows the warm and hot karst water productive wells, water return wells, and barren CH wells (drillings) *(Lénárt & Darabos, 2013a, Lénárt & Darabos, 2013b)*. There were BKWMS measurement in 9 thermal wells. Also, the data of Egerszalók and Demjén wells are available continuously.

Figure 6 shows all measurement sites where we carried out evaluations. It shows that the measurement sites of the BKWMS are situated in many different locations and there are many of them. The various hydrogeological measurement sites are shown on an overview map where settlements and hydrological data are also marked. In order to easily identify objects in close proximity with one another, 7 small maps have been prepared as well.

CONCLUSIONS

It was necessary and practical to collect the measurement sites providing hydrogeological data of the Bükk and its region and render them into one single database. Data deviations were successfully corrected without taking new measurements so a good quality database was compiled.

The most important characteristics of our data is shown on various maps, and these clearly define the possibilites of creating further maps or enhancing the accuracy of existing ones (heat flow, depth of 30 °C isotherm, reciprocal geothermic gradient, base mountain, etc.).

The summarized map also provides information regarding the harmonization of the measurement system and the expansion of BKWMS.

Due to lack of space, only a few publications will be listed of the 100 publications published regarding this topic, but we also list some other important literature. The list of manuscripts, also as many as the publications themselves, won’t be listed due to lack of space.

ACKNOWLEDGEMENT

The research was carried out in the framework of the Sustainable Resource Management Center of Excellence at the University of Miskolc, as part of the TÁMOP-4.2.2/A-11/1-KONV-2012-0049 “WELL aHEAD” project in the framework of the New Széchenyi Plan, funded by the European Union, co-financed by the European Social Fund.

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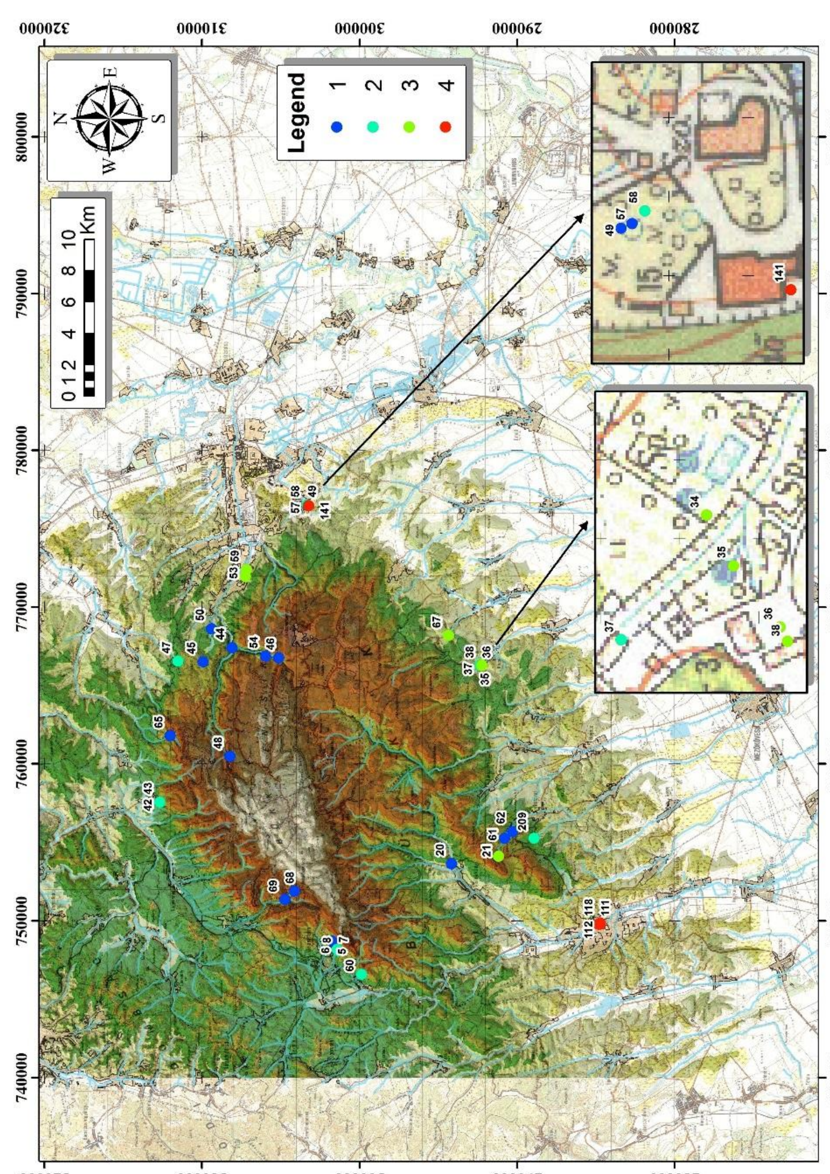

Figure.2. The temperature of karst springs joined to the drinking water work, and some other larger springs (1: cold karst water; 2: cold tepid karst water, 10 – 16 °C; 3: warm tepid karst water, 16 – 25 °C; 4: warm karst water, 25 – 37 °C)

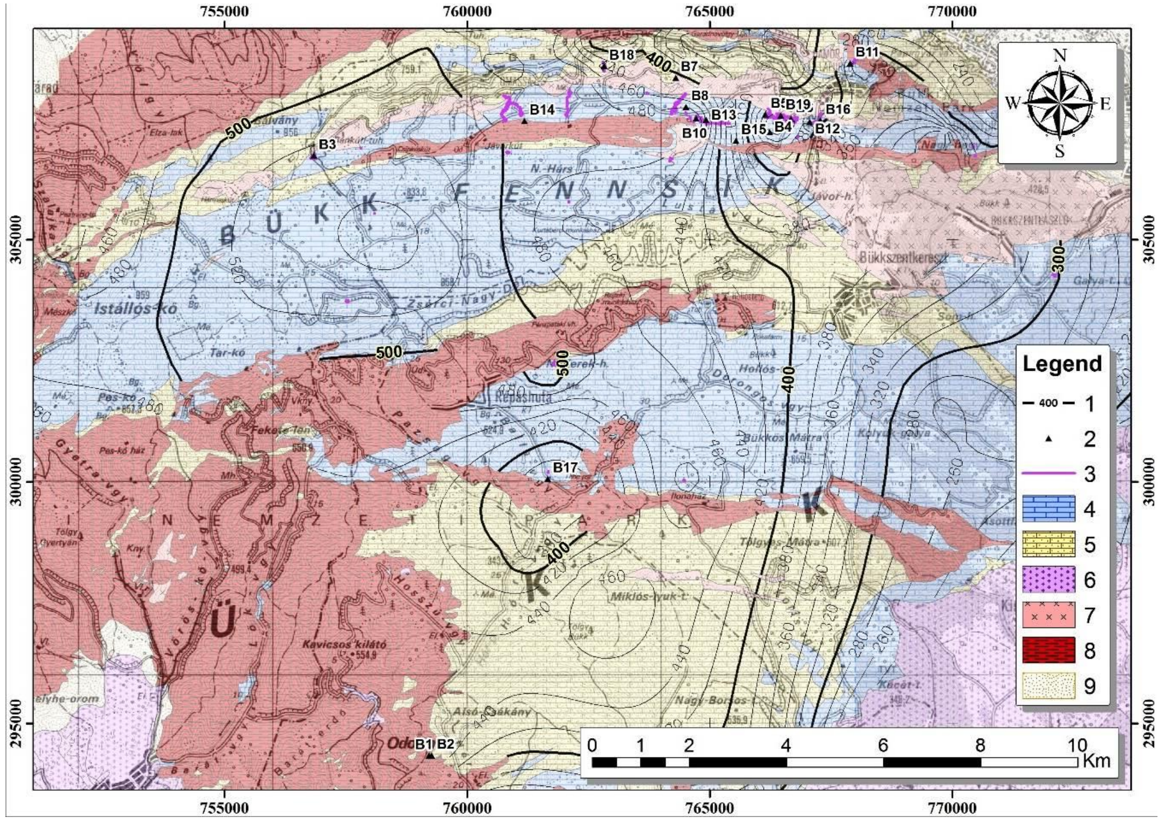

Figure.3. In-cave measurement sites (1: maximum karst water level [above Baltic Sea level]; 2: measurement sites; 3: layout map of caves; 4: well-karstified rocks; 5: poorly karstified rocks; 6:Non-karstified tuff; 7:Non-karstified other fissure rock; 8:Rocks with low hydraulic water conductivity; 9:Debris sediments)

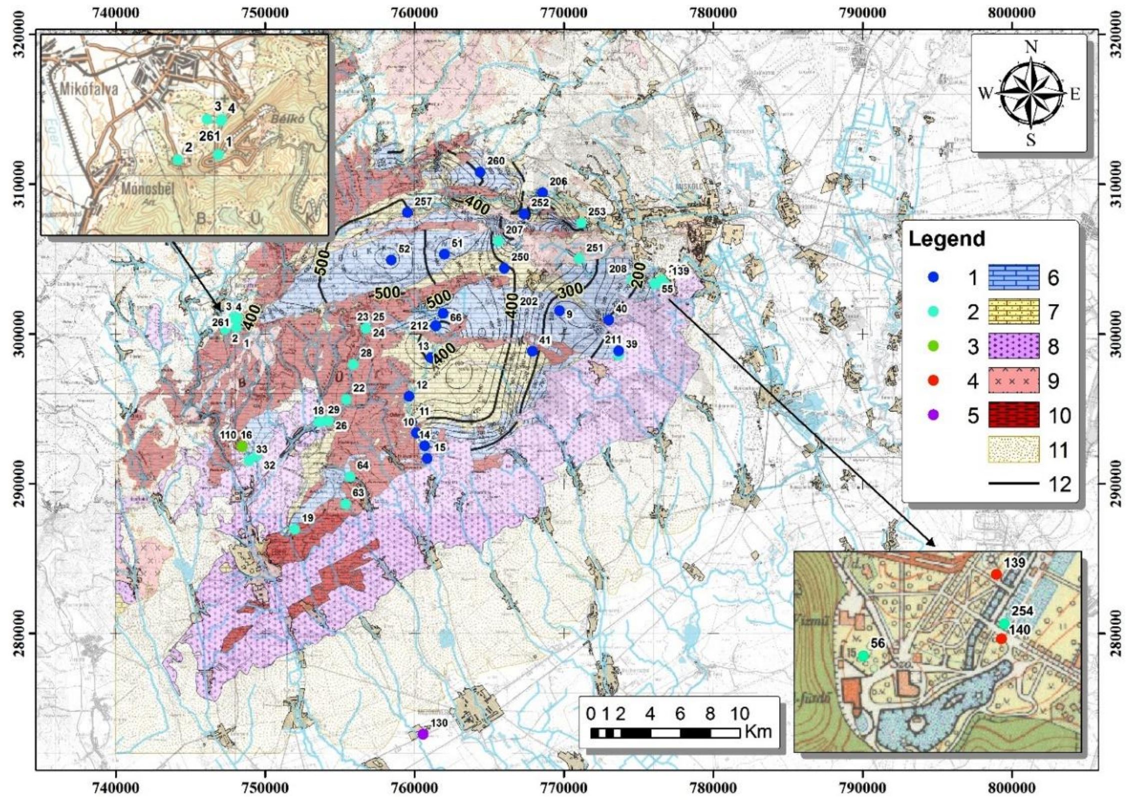

Figure.4. Karst water monitoring wells with karst water levels (1: cold karst water; 2: cold tepid karst water, 10 – 16 °C; 3: warm tepid karst water, 16 – 25 °C; 4: warm karst water, 25 – 37 °C; 5: hot karst water, above 37 °C; 6: well-karstified rocks; 7: poorly karstified rocks; 8:Non-karstified tuff; 9:Non-karstified other fissure rock; 10:Rocks with low water conductivity; 11:Debris sediments ; 12: maximum karst water level [above Baltic Sea level])

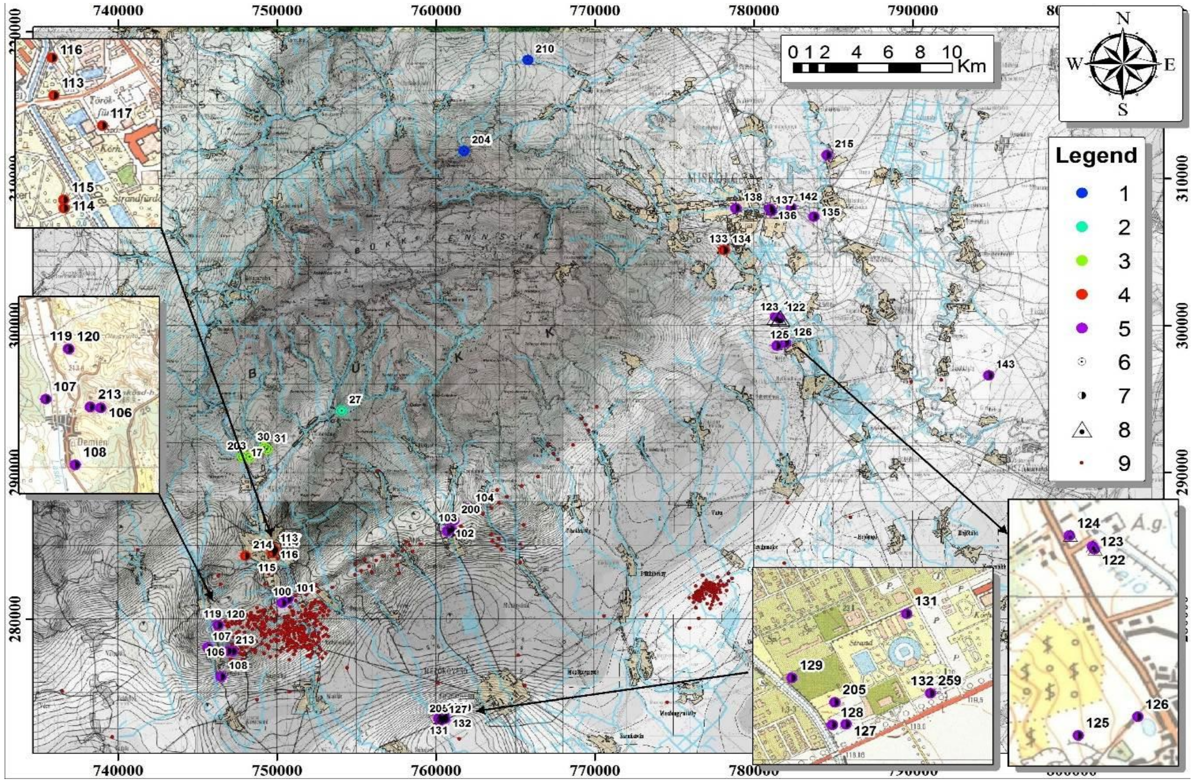

Figure.5. Thermal karst water wells on Bouguer-anomaly map [base map: 17: MÁELGI] (1: cold karst water; 2: cold tepid karst water, 10 – 16 °C; 3: warm tepid karst water, 16 – 25 °C; 4: warm karst water, 25 – 37 °C; 5: hot karst water, above 37 °C; 6: cold and tepid karst water productive wells; 7: warm and hot karst water productive wells ; 8: thermal karst water return wells ; 9: barren CH drillings)

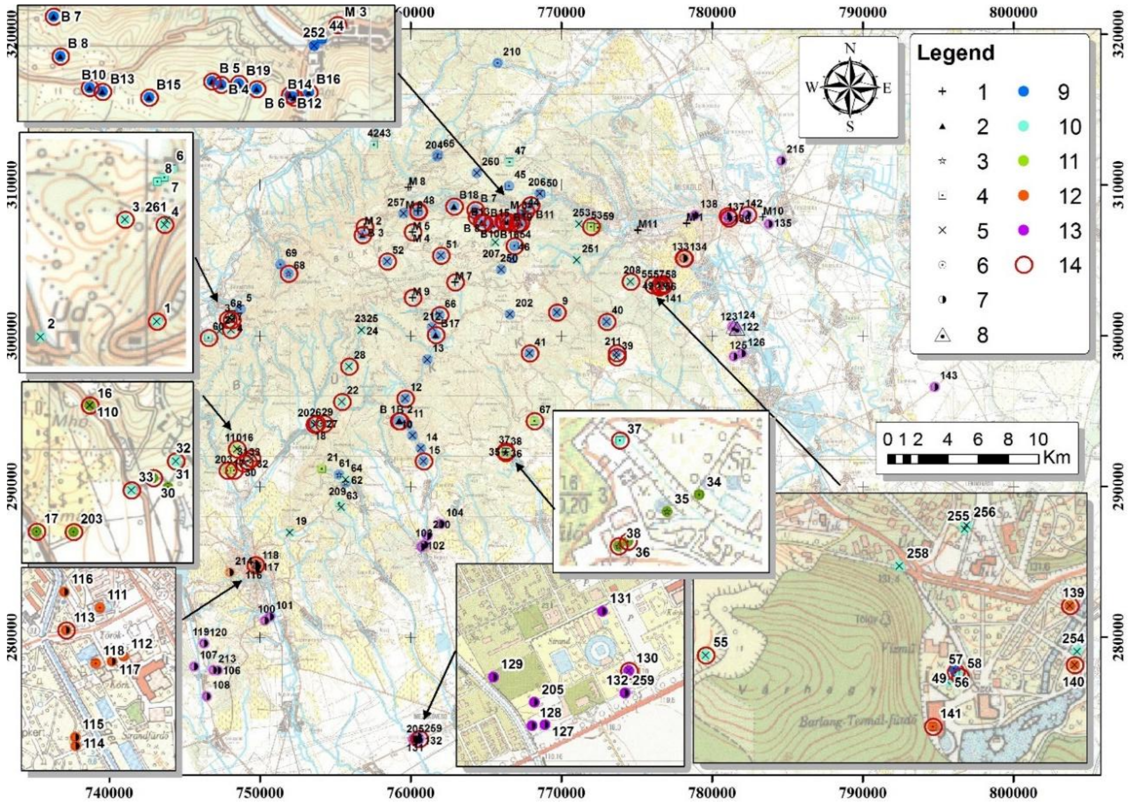

Fig.6 Measurement sites in Bükk and Bükk region (1: weather station; 2: in-cave measurement site; 3: spring; 4: spring joined to drinking water system; 5: karst water monitoring well; 6: cold and tepid karst water productive well; 7: warm and hot karst water productive well ; 8: thermal water return wells; 9: cold karst water; 10: cold tepid karst water, 10-16 °C; 11: warm tepid karst water, 16-25 °C; 12: warm karst water, 25-37 °C; 13: hot karst water, above 37 °C; 14:BKWMS measurement sites)

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HYDROGEOLOGICAL INVESTIGATION OF THE GARADNA CATCHMENT AREA

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ABSTRACT

Generally, karstic aquifers are good drinking water resources providing high quality water. But these types of aquifers are sensitive for contamination and continuous monitoring is necessary. Monitoring provides a better knowledge of karst systems. Since 1992 there has been a continuous water level observing system in the Bükk. The elements of this system are water production wells, observation wells, springs and caves. This system also works as a quantity monitoring system and it can help the activities of waterworks companies.

In 2013 a hydro chemical investigation began in the Garadna spring where calcium, magnesium, sodium, potassium were determined from the collected water samples. There is continuous discharge and conductivity measurement during the spring from the end of July and continuous measurement from November 2013. During this summer we could develop a field and a laboratory measuring protocol. In our research we will compare the new hydrochemical data with water level, conductivity, temperature and discharge data.

INTRODUCTION

The investigated Garadna Spring is located in the center of the Bükk Mountain at the NE part of Hungary. The springis the main recharge source of the Garadna Stream running through in the Garadna Valley. The spring discharges from the Hámor Dolomite Formation which is a Triassic medium karstified dolomite *(MÁFI, 2005)*. The water catchment area of the spring is located on the Bükk Plateau. The two ponor caves of the spring are the Jávorkút and the Bolhás cave which were developed in the well-karstifiableFehérkő Limestone Formation *(Figure 1)*. Also water tracing tests prove that the most part of water entering the spring originates from the Fehérkő Limestone Formation. The rock of this formation is a bright grey, massive, banked, platform facies limestone *(MÁFI, 2005)*.

In May of 1996 a continuous water level and water temperature register equipment was installed in the Garadna Spring. This monitoring point serves data aboutthemechanism of water level recession from the last 16 years and it is one of the main elements of the Bükk Karst Water Level Observing System. The basic measuring points of this system were installed in 1983 and digital pressure logging were added to the system in 1992 *(Lénárt, 2006)*. Nowadays, the water level and temperature data are registered in production wells, observing wells, in caves and in springs. The measuring points cover the whole area of the Bükk Mountain which is one of the main benefits of the Bükk Karst Water Level Observing System because we can compare data from different measuring points *(Darabos & Lénárt, 2008, Szűcs & Horne, 2009)*.

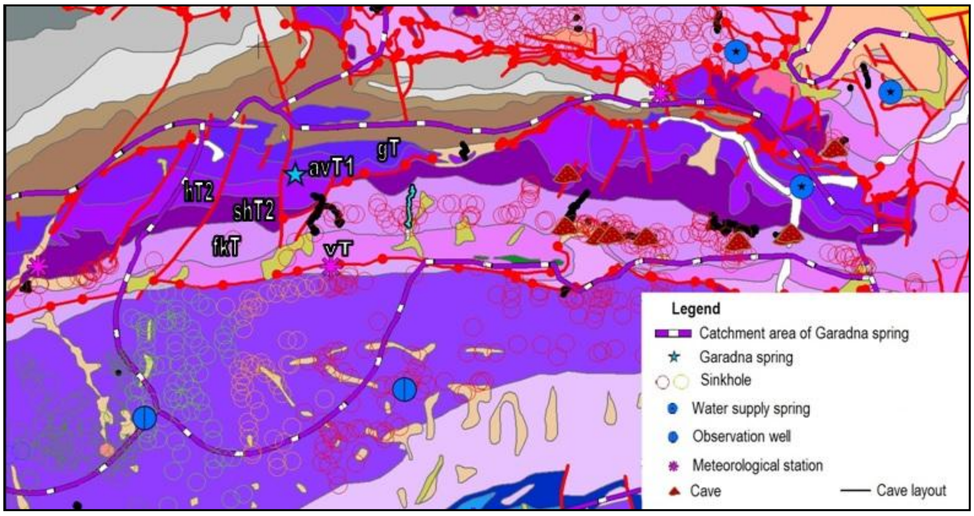

Figure 1. Geological build-up of region of Garadna-spring (${}^{g}T$ – Gerennavár Limestone Formation, ${}^{h}T_2$ – Hámor Dolomite Formation, ${}^{fk}T$ – Fehérkő Limestone Formation, ${}^{sh}T_2$ – SzentistvánhegyMetaandesite Formation, ${}^{v}T$ – Vesszős Formation) (MÁFI, 2005, Lénárt&Hernádi, 2012)

MATERIALS AND METHODS

The construction of measurements was based on previous results of investigations. The reaction time of the Garadna Spring is some hours which means that the raise of water level is experienced 4–5 hours after rain event. The pressure transfer always happens in the interval of 24 hours in every investigated case. The flood crest is generally 2 days after rain (it changes between 1–3 days) *(Darabos&Lénárt, 2008)*. About the relatively quick reaction for rain the spring was sampled in every 3 and half hours into the twenty-four 1 liter bottles of water sampler equipment. The collected samples were treated with nitric acid and they were transported into the laboratory at almost the same time occurring every week.

The most common ions in karst water are usually analyzed by titration, ion chromatography (IC), UV/VIS spectrophotometry and less frequently by atomic emission or absorption spectrophotometry (AES or AAS) *(Hunkeler&Mudry, 2006).*

Because of huge number of sample the atomic adsorption spectrophotometer was applied for analyzing. Generally the analysis of alkali and the alkali-earth metals are simple with AAS because they have strong sign and the equipment is sensitive enough for that. We had problems with analysis of calcium because it had interferences with phosphate, sulfate, aluminum, and silicon. In our samples the presence of sulfate caused interferences. But it could be largely controlled by adding 10 g/L La as chloride *(Welz & Sperling, 1999).*

Another new equipment in the Garadna Spring is the ultrasonic flowmeter which measures the mean velocity of flow every 15 minutes. Unusually the instrument has only one sensor which uses continuous wave Doppler technology. For calculations water level data was also needed so in July of 2013 a new DATAQUA sensor was installed in the spring, which measures water level, water temperature and conductivity. In order to calculate the discharge we determined the relationship of water level and the cross-section flow where the flowmeter sensor was installed. After determination of function of water level and cross-section flow area the calculations were simple.

RESULTS

Results of chemical and correlation analysis

The chemical data was analyzed and also compared with water level, temperature, conductivity and precipitation data. In this article only the main correlations and relationships are presented. First of all the Ca^{2+} concentration, Ca/Mg ratio and the precipitation data were plotted on a diagram. Ca-concentration has a lot of outlier data but overall the data line shows a slow decreasing trend. The mean value of the whole dataset is 92 mg/L *(Figure 2).*

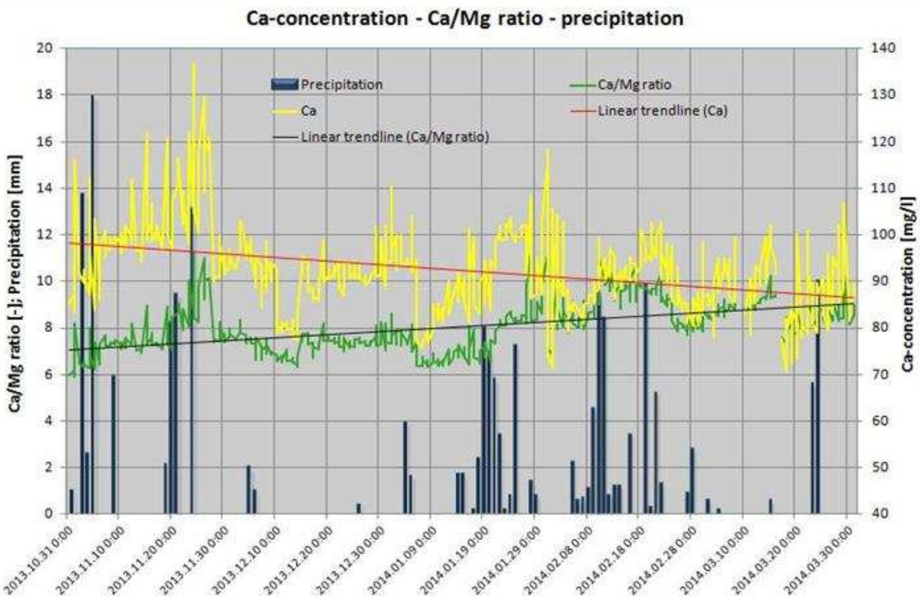

Figure 2. – Diagram of Ca-concentration – Ca/Mg ratio – precipitation and trend lines

In case of precipitation the Ca concentration rises at first then decreases and returns to the average value. A possible explanation of increasing of concentration is the pushing out of the well mineralized water from the karst system. During rainless period the karst water has a lower seepage velocity which results more contact time to solve minerals. After rain event the infiltrated water increases the water level and it causes higher seepage velocity and higher hydraulic gradient. This kind of water spends less time in the system and it contains less Ca so the concentration decreases. The length and rate of changes depend on several factors. These factors are the geometry and size of the karst system, the distribution of rainfall, the time when rainwater appears in the spring, the ratio of dilution and the chemical characteristics of the water held in the system.

The shape of the graph also gives information about the karst system. A graph with sharp positive and negative peaks represents a well karstified system, which system is able to deliver diluted and infiltrated components to the spring *(Hunkeler&Mudry, 2006)*.

Rainfall causes small decrease in Mg concentration, and in long term rainless period Mg shows a slow decreasing trend *(Figure 3)*. It can be also seen, when Mg concentration decreases, the Ca/Mg ratio increases ($R^2 = -0.7$).

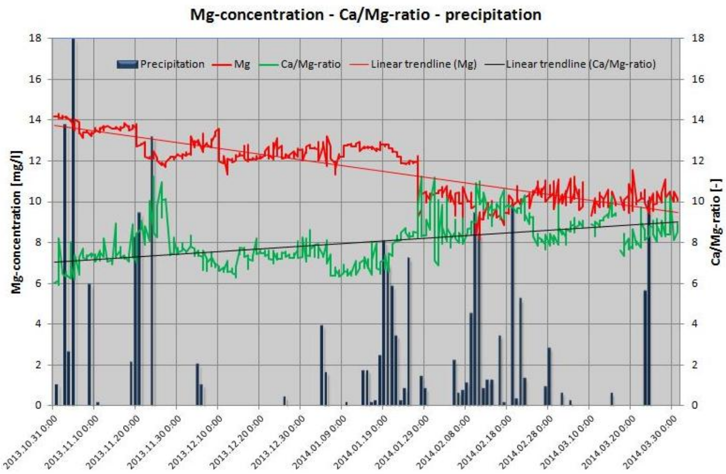

Figure 3. – Diagram of Mg concentration, Ca/Mg ratio – precipitation and trend lines

Water level – discharge

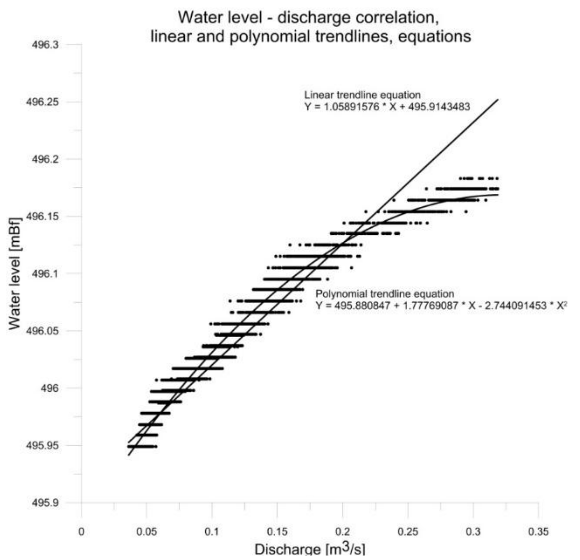

Figure 4. – Relationship between discharge and water level, equations of linear and polynomial trend lines, $R^2=1$

While chemical analysis of water samples stopped at the end of March 2014, water level, temperature, conductivity measurements and discharge data calculations continued.

First of all we determined the mathematical function between water level and discharge data. For this purpose we plotted the two data sets and fit linear and polynomial trend lines and determined their equations (Figure 4).

Y = z – water level [maBSl]
X = Q – discharge [m^3/s]
Equation of linear trend line :$z = 1.05891576 \cdot X + 495.9143483$.
Reordered to discharge:

$$Q = \frac{z - 495.9143483}{1.05891576}.$$

Equation of polynomial trend line: $z = 495.880847 + 1.77769087 \cdot X - 2.744091453 \cdot X^2$.
Reordered to discharge:

$$Q = 730243.6717 - 2945.230605 \cdot z + 2.969688075 \cdot z^2.$$

After reordering the equations of the trend lines we calculated discharges from them in both cases. Then we compared the calculated and the real discharge data as *Figure 5* shows. Between two calculated data sets and real discharge data correlation coefficient is R^2=0.97, which means very good correlation.

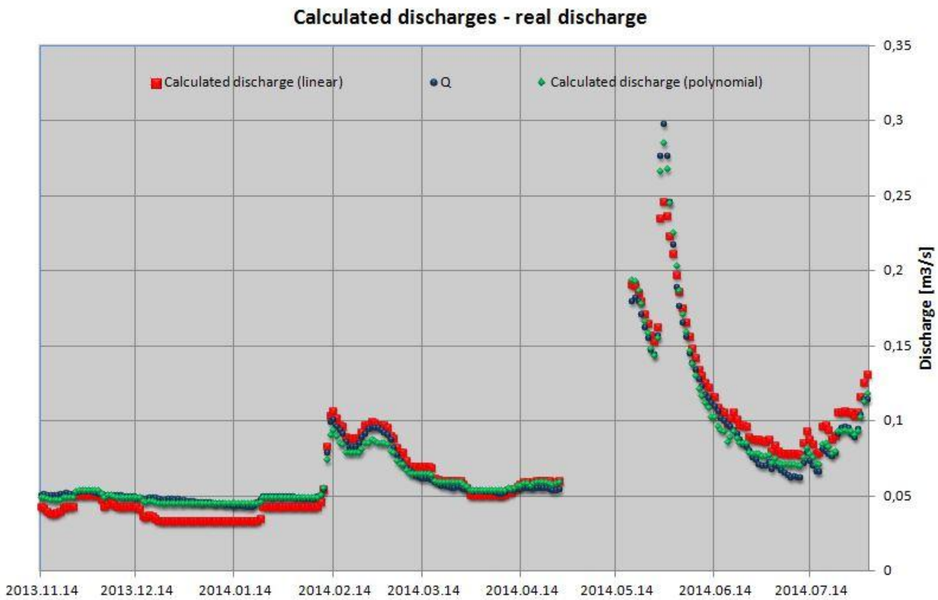

Figure 5. – Calculated discharges compared to the real discharge (R^2 = 0.97)

Calculated and real graphs correlate well and correlation coefficient is also excellent *(Figure 5)*. Based on this information we think – although precise discharge data cannot be calculated –the trend can be determined from water level data.

Determination of base discharge of Garadna Spring, calculation of discharge data from 2001

We have daily water level data provided from 2001 until today, which were measured and registered within the framework of the BükkKarstwater Level Observing System. From these data we calculated discharge data as it was written in the previous chapter.

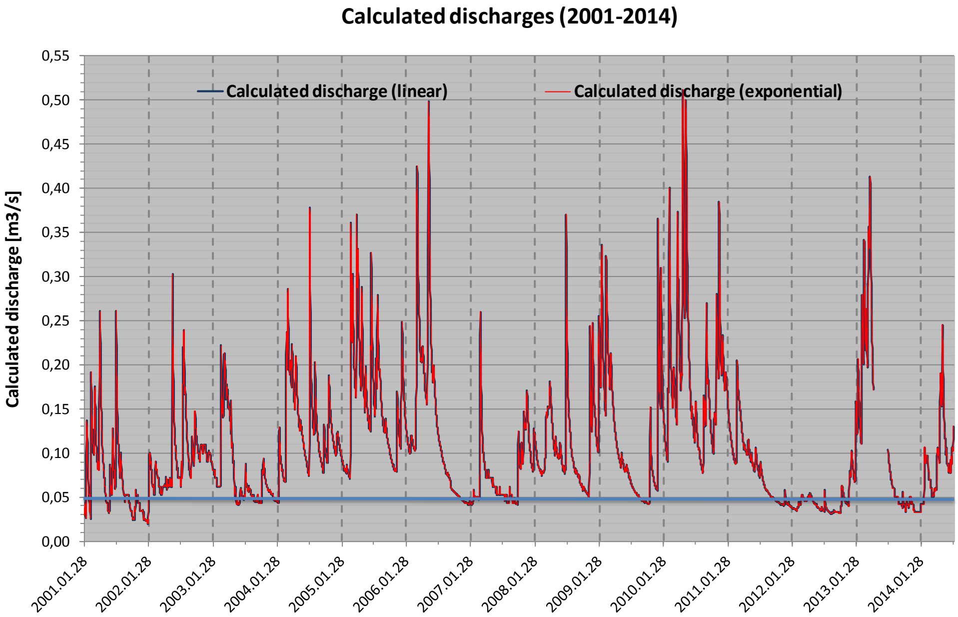

Figure 6. – Linearly and exponentially calculated discharges from 2001 to 2014 ($R^2 = 1$) and base discharge (0.05 m^3/s)

Linearly and exponentially calculated discharge graphs fully cover each other, there is no difference between them, their correlation coefficient is $R^2 = 1$ *(Figure 6)*.

According to water level data of years the Garadna spring has a constant water level which is 495.96 meter above Baltic Sea level (maBSl) but it could change between 495.9 maBSl and 496 maBSl. After rain event the precipitation increases the water level of spring so the precipitation is superimposed on this water level but it decreases to the mentioned constant water level in the rainless periods. The discharge of spring on this characteristic water level (495.96ma BSl) is 0.05 m^3/s.

So it can be established that the base discharge of the Garadna Spring is about 0.05m^3/s, which marked in *Figure 6* with a blue line. In long rainless periods – as in 2011 and 2012, when the annual precipitation was 56 and 645 mm, respectively – for longer or shorter periods discharge was even lower, because recharge from the micro fissure system was worse.

Determination of water type by using Piper-diagrams

A piper-diagram was used to visualizechemical data. By using water chemistry data the chemical type of the water can be determined.

For this method we collected water samples at four different times, which samples were not treated by acid and all the necessary chemical measurements were performed within 24 hours.

The results are presented and summarized on *Figure 7.*

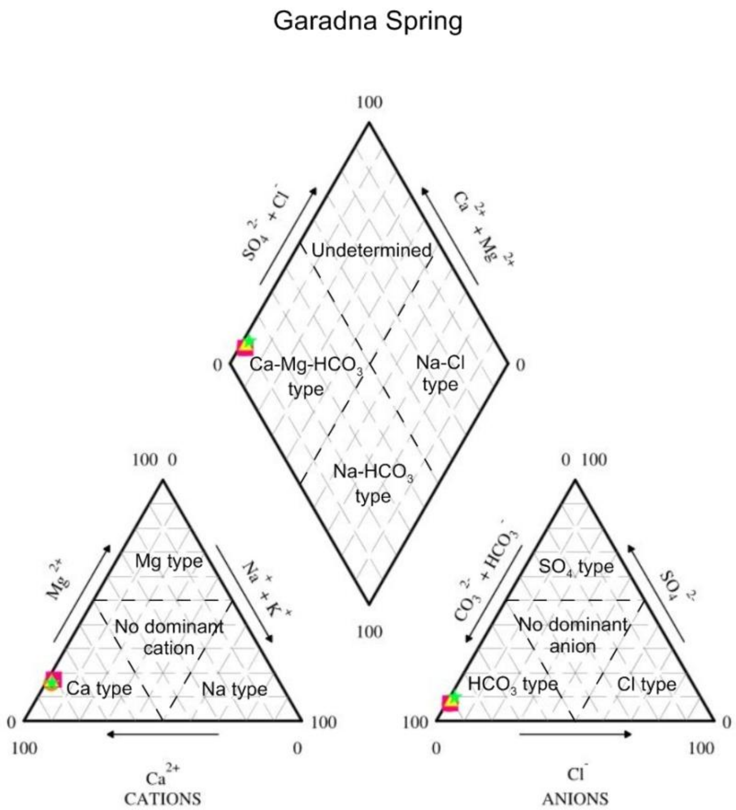

Figure 7.Results represented on Piper-diagram

From aspect of cations the type of water is ’Ca^{2+} type’ because of the low Na^+ and Mg^{2+} concentration. From aspect of anions water belongs to the ’HCO_3^- type’ because of low concentration of Cl^- and SO_4^{2-} *(Figure 7)*.

Based on overall results the chemical type of the spring water is obviously’Ca^{2+}-Mg^{2+}-HCO_3^- type’.

The results of this method are important because the spring discharges from dolomite but the main part of it is water catchment spreads on limestone. Based on the diagram the rate of Mg is less than 5 % so the type of water is 'Ca type'. The chemical type of water originated by the Fehérkői Limestone Formation.

Additionally we calculated the saturation index of calcite and aragonite by PHREEQC modeling program. In case of SI=0 means equilibrium, SI>0 water is oversaturated SI<0 water is undersaturatedfrom aspect of given phase.

Table 1

Saturation index of calcite and aragonite and water conductivity

Date	$SI_{calcite}$	$SI_{aragonite}$	Conductivity[mS/cm]
17/03.	0.17	0.02	0.422
07/04.	0.14	-0.02	0.359
14/04.	0.14	-0.02	0.339

Table 1 shows that saturation index of calcite over zero is in all cases, it means, that spring water is oversaturated and precipitation can be expected what is supported by literature and we also experienced it during our measurements.

CONCLUSIONS

The last few months were a very rainless period in the Bükk Mountains. So the chemical analysis of these months only serves data about general chemical composition of spring. The hypothesis is that in our work the change of chemical composition of karst water after rain. In this period we did not have a significant rain event which could confirm our hypothesis.

The continuous monitoring of spring will be completed with mixed investigations. Now we can prove that the Garadna spring has inflow from other water body. We will collect water sample from lower point of the ponor cave and we will try to sample other ponor caves of the spring. After analysis of water samples data will be used for geochemical modeling. Additionally it is planned to analyze the strontium content of water samples of caves to identify the occurred underground water inflow.

Due to the high correlation of discharge and water level we can calculate the discharge data from the water level data of last 17 years according to water level-discharge function.

ACKNOWLEDGEMENT

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Physical and chemical characterization of red mud in terms of its environmental effects

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ABSTRACT

Red mud coming from different sampling points of an abandoned disposal site was investigated in this research. We examined the physical and chemical properties of red mud in terms of environmental effects especially for the subsurface environment. Their particle size, consistency, compressibility, shear strength and hydraulic characteristics were measured. The compositions, pH and cation exchange capacity also had been analyzed and special leaching tests were performed to characterize its behavior in the subsurface environment. Horizontal and vertical inhomogenity in the material of the disposal site and the evidence of secondary mineral precipitation was found based on the main properties of the samples. Differences between the rates of basic minerals were found in some samples, but overall the red mud has a lot of hazardous elements such as As, Pb, Cr and Hg. The red mud has very fine particles, high water content, and it is hard to compact. Some samples showed thixotropic behavior during the compression and shear tests. At the same time it has low hydraulic conductivity, and high cation exchange capacity, but even with these favorable properties there is the potential environmental risk in long-term storage of red mud.

INTRODUCTION

Red mud is the largest amount of industrial waste generated during the production of alumina. Depending on the amount of bauxite and the type of process, during the production of each 1 t alumina 1.2–1.3 t dry red mud is generated *(MTA, 2006).*

In the investigated site during operation of alumina industry more than 10 Mm^3 total amount of red mud was accumulated in the disposals. The recultivation process is in progress with covering the surface of the disposals, which has primary importance not only because of the dusting of red mud, but also to decrease the infiltration of precipitation and the amount of leachate. As there is a great amount of alkali, sulfate, fluoride, heavy metals and other potential pollutants in red mud, its storage can pollute the subsurface environment. In addition, in case of operating alumina plants the continuous increasing of storaged amount may lead to potential human and environmental disasters. Studies on the physical and chemical properties of red mud have become a focus within science and engineering fields *(Loa Pez et al. 1998, Yalxin & Sevic, 2000, Sglavo et al. 2000, Paramguru et al. 2005, Gray et al. 2006, Felix et al. 2006, Wang & Liu, 2012)*, but the main goal of these studies usually theutilizationof related materials. But it does matter what purpose we examine the properties of a material: we want to use it or to estimate the environmental risks. In our research we plan a series of measurements that is aimed to exactly know the behavior of red mud body in the subsurface environment, the inhomogenity of its properties in the disposal site, and the effect on the soil and groundwater.

SAMPLING

The red mud samples were taken from an abandoned disposal site, where the process of alumina refining was finished for almost 20 years. The samples were taken totally from 30 different points in horizontal and vertical direction. The largest sampling depth was 2 m from the 10 m total depth of the disposal. In each point a mass of 5–30 liter was collected to be enough material for every laboratory tests. In the laboratory before taking the portion for testing a mixturing of the material (if it was possible) was realized.

In the site we found the red mud in several different states, with different particle sizes, and water contents. The very dry and fine material on the surface is shown in the left part of *Figure 1*, while the red mud with high water content in silty, jelly consistency is in the centre of *Figure 1*. In the middle of the disposal site a smaller or larger pond is always present, which only disappears after long dry periods. In some parts of the disposal we found a very hard, thin material, like tiles (right part of *Figure 1*). In these areas green vegetation was observed and there were much higher water content of the red mud under this thin layer than elsewhere.

Figure 1. The red mud in different points of the disposal

MINERAL AND CHEMICAL COMPOSITION

The composition of red mud is varying with the type of the bauxite and the alumina production process. At the investigated site the Bayer's process was used for production. We measured the mineral and chemical constituents of such samples, which showed different physical characteristics. The mineral composition determined by XRD analyzer. The results show that except some samples – which obviously created both due to some secondary mineral precipitation – the red mud contains non-crystalline materials in about 70 percent. In addition, the presence of cancrinite and hematite minerals is the most typical. The white precipitates were identified in 100% to calcite, and the hard, thin plates from the surface of the disposal are almost entirely of alumina - hydroxide minerals.

The chemical constituents measured by XRF analyzer are summarized in *Table 1*.

Table 1

The main chemical constituents (%) and trace elements (ppm) of red mud

SiO_2	12,2	**Cu**	39
Al_2O_3	10,5	**Zn**	55
MgO	0,53	**Pb**	128
CaO	11,66	**Sr**	585
Na_2O	6,09	**As**	85
Fe_2O_3	31,63	**Cr**	506
TiO_2	5,9	**Co**	38
P_2O_5	0,55	**Ni**	122
MnO	0,34	**Ga**	22
K_2O	0,05	**Sc**	48
–	–	**Y**	28

It can be seen in *Table 1*, that the main chemical compositions of red mud are Fe_2O_3, CaO, Al_2O_3, SiO_2, Na_2O, TiO_2. In addition to the major elements the strontium, chrome, lead and nickel are present in the greatest quantity, and other hazardous trace elements including As, Hg, Cu, and rare earth elements were also detected by the XRF analysis.

SOIL MECHANICAL MEASUREMENTS

The long-term disposal of alkaline red mud easily leads to pollution of the surrounding soil, air and groundwater. Therefore, the physical and mechanical characteristics of red mud - which determine the hydrodynamic behavior – are needed to know exactly. We determined water content, particle size distributions, plasticity index, hydraulic conductivity, shear strength parameters, and other parameters (water suction for example) which are not presented in this paper.

The particle size distribution of samples were measured by laser diffraction because of difficulties occurred during hydrometer analysis. It was interesting to see that although samples contain very fine particles, in the results the ratio of them was too small. The phenomena caused by the behavior of particles to create aggregates very easy. In this way we cannot see the "clay" fraction in the samples if laser diffraction is used without any kind of modification. First we added dispersant, and the resulted finest particle size distribution is shown in *Figure 2*, with solid line. After that we applied UV-ray during the measurement to prevent aggregation. The result is presented in *Figure 2* with dashed line.

In addition to most of the samples, some samples contain coarser particles, mainly which coming from the upper part of the disposal or next to the dam. This inhomogenity of particle size is very well correlated with water content: finer the particle size more water content of the samples.

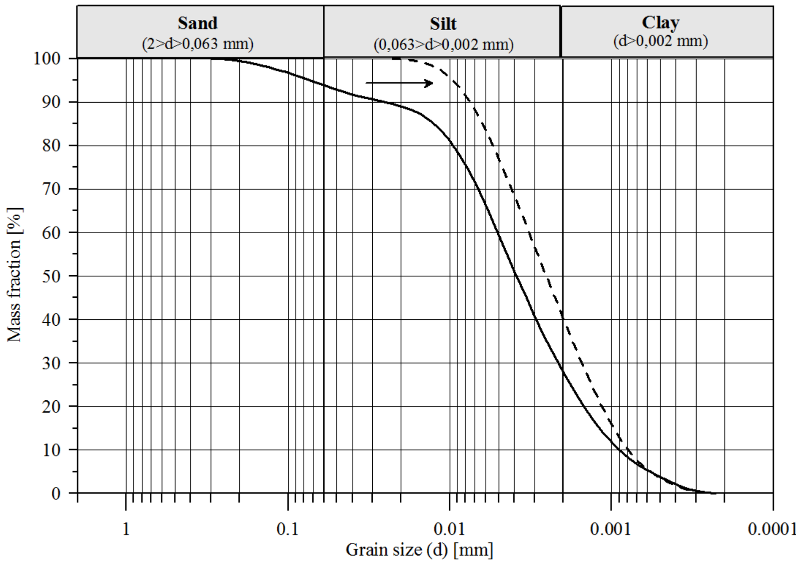

Figure 2. The typical particle size distribution of red mud

We measured water content of almost 30 samples taking from different points of the disposal site. The water content of red mud increases with depth, but horizontal changing are also recognized. At the same depth the red mud contain much more water at centre of the disposal than along the dam. The vertical and horizontal change in particle size and water content can be explained by the way of spreading the material into the disposal. As it is abandoned from more than 10 years, sedimentation was also happened. The left part of *Figure 3* shows the water content in depth of the sampling.

The hydraulic conductivity of samples was measured by flexible-wall permeameter with changing the pressure of testing. The consolidation pressure of the sample preparation was also varied from 1 to 3 bars, and the measurements were repeated. The results of 2 samples are presented in the right part of *Figure 3*. The red mud shows very low hydraulic conductivity, which can be compared to the permeability of clay soils. Based on the results of more than 10 samples, the minimum value is $1{\cdot}10^{-9}$m/s, while the maximum is $2{\cdot}10^{-8}$ m/s. At the same time we also performed in-situ infiltration tests, which results are in good agreement with the lab results. In some points of the disposal very rapid water filtration was experienced, which caused by presence of fissures in the red mud body.

We determined the plasticity index based on the liquid and plastic limits to provide information regarding the effect of water content on the consistency of red mud. Certain clay minerals have the ability to hold water molecules and electrically bind them to their surface. Therefore, they can exist in a plastic state over a relatively wide range of water content, and have high swell potential. The plasticity index of red mud samples are in the range of 4 to 27.The smaller values are related to the coarser, while larger values are resulted for finer samples. It shows the water sensitivity of red mud means, that relatively small changes in water content results higher changes in consistency.

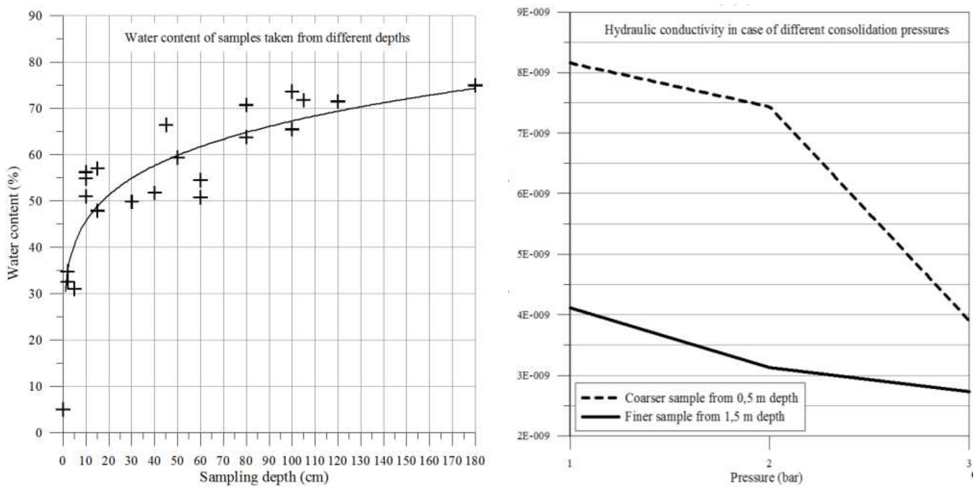

Figure 3. Water content and hydraulic conductivity results

Before shear testing one-dimensional consolidation of red mud samples were realized. Certain samples showed thixotropic behavior and started to act as liquid during consolidation. So the material was pushed out from gaps of the consolidation cell. Because we cannot consolidate all samples for direct shear strength tests, instrument for torsion shear measurements (TORSHEAR 27-WF2202) were applied. During the torsion shear, the failure occurs by rotating the shear boxes on each other. In this way the shear distance is increased to quasi infinite long, and you have the possibility to determine not only the maximal, but also the residual shear strength parameters. The resulted graph of two samples is presented in *Figure 4*. The samples with higher water content have lower cohesion.

SOIL CHEMICAL MEASUREMENTS

In chemical analysis first the cation exchange capacity and the pH was determined. The cation exchange capacity gives information about the total amount of cations can be adsorb by a soil (for 100 g of soil under conventional pH) *(Stefanovics, 1992)*. Several methods exist to determine, after a series of testing finally sodium - acetate, as exchange solution was applied. The range of resulted values is summarized in Table 2, where the typical values of some soils are also presented. The red mud showed very good adsorbent properties, its cation exchange capacity is in the range of clayey soils.

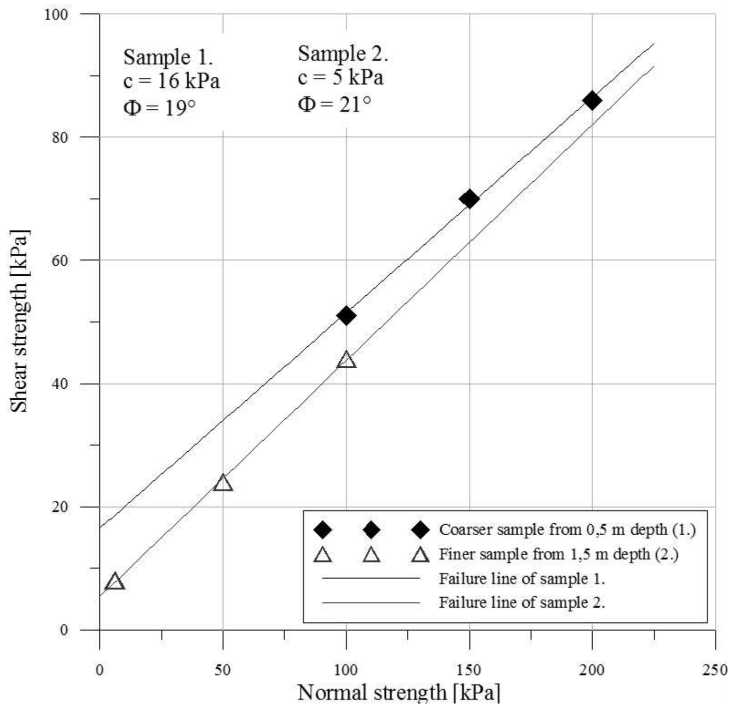

Figure 4. The results of shear tests for red mud samples

Table 2

The cation exchange capacity of red mud and different soils (Stefanovics, 1992)

Soil type	Cation exchange capacity (T) [mgeqv/100 g]
Red mud	43.2–75
Kaolinite	5–15
Illite	15–40
Smektite	60–120
Vermikulite	120–200
Chlorite	10–40
Humus	150–300

The measured pH of red mud is in the range of 9.5–10, which means that it is a very alkaline material. The high pH is caused by composition of the red mud and as a result it is a very good neutralizing material. Because of this capability, researches are aimed to use red mud in the treatment of acid mine drainage, and contaminated soil as well *(Gray et al. 2005, Felix et al. 2006)*, where its main task is rising up the pH. It is interesting for our research, because most of the toxic heavy metals are not mobile in neutral and in slightly alkaline environment, so metal content of the leachate and its human risk maybe lower.

During the permeameter tests we continuously measured the pH of water filtrated through the samples, and also collected the leachate to analyze for some cations and anions. The left part of *Figure 5.* shows the changing pH of the distilled water filtrated through a sample. The pH does not drop below 9 even after 10 days measurement, that's why in case of heavy metals concentrations we didn't recognize any changing in time.

In another laboratory testing to see more intensive changing in shorter time, we tried to increase the flow velocity, means to decrease the measurement's time, and also wanted to lower pH of the initial solution. To reach these circumstances, a mixture of red mud and wood chips was filled into a 1 m long plexiglass tube, and a strongly acidic (pH = 2.9) solution was applied for leaching. The changing of pH during the test is shown in the right part of *Figure 5.* In the end of testing the pH was near neutral without clogging of the column.

Finally we perform a multi-stage acidic leaching of heavy metals from red mud samples according to the Hungarian standard of waste mobilization process. During the test a small amount of the material is added to the different, stronger and stronger extracting solution.

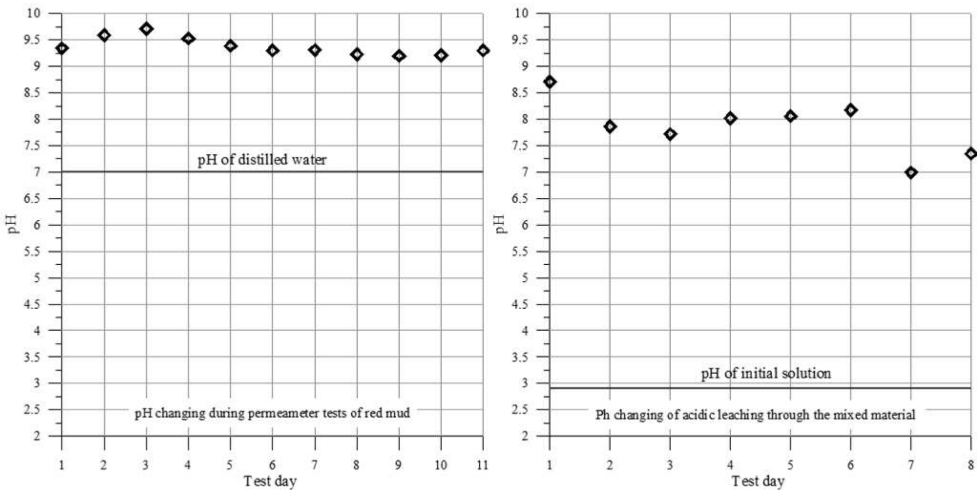

Figure 5. pH changing of leachate filtrated through the material

After 6 or 10 days extraction time heavy metals concentration of the solution was analyzed by AAS. In each step the stronger acid can solute the more strongly bounded metals, so we can distinguish the mobile and less mobile phases. *Figure 6* shows for some metals after 10 days the percentage of the amount extracted by distilled water related to that by the strongest acid ($KClO_3$). It can be seen that for Pb, and Cu just the distilled water can mobilize 35–50% of the total amount, despite high buffer capacity of red mud. Other metals, like arsenic and molybdenum also can be mobilized in a higher amount already among alkaline conditions.

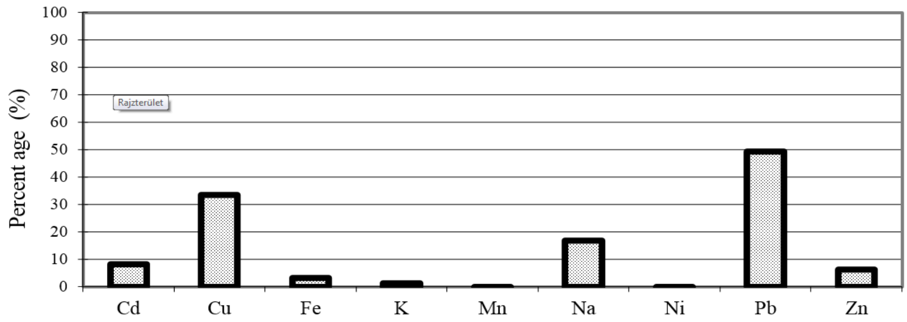

Figure 6. Percentage of the metal amount extracted by distilled water related to that by the strongest acid ($KClO_3$)

CONCLUSIONS

With a series of mechanical and chemical laboratory testing of large number of samples the characterization of red mud was performed related to a research project. The main goal of the project is the exact determination of the environmental risks of an abandoned disposal site in Hungary.

In shear testing the consolidation of red mud was very hard and it was only possible with active dewatering. The contact between the particles is terminated because of the presence of solvation shell around them. During shear the particles can slide over each other easily. So the only way of increasing the stability with compaction is coupled dewatering. Huge amount of red mud found at the disposal site have high water content and is in the state near its liquid limit.

Based on the chemical composition there is a high amount of hazardous elements in the red mud, but at the same time it contains a lot of such minerals, like hematite and aluminum- hydroxide, goethite etc, that have high active surface and high affinity to adsorb these elements. The low permeability of the material indicates that the phenomenon of leaching out is very slow, which is also a good property in the view of its environmental impact. It has high pH, as a result high buffer capacity, and the cation exchange capacity is comparable with some clay mineral's capacity. But based on results of the different leaching tests we applied, it was realized that already distilled water can mobilize the heavy metals from red mud in more or less quantity according to pH dependence of the metal's solubility. The high alkali content also gives the potential environmental risk of long-term accumulation of red mud.

ACKNOWLEDGEMENT

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Application of relative permeability measurements to Improve remediation process modeling accuracy

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ABSTRACT

The paper deals with the differences caused by the use of measured relative permeability data instead of those in the literature. To evaluate the input data sensitivity of results, two transport models were created to compare the spread of TCE (trichloroethylene) in saturated media. The model runs were performed both using the relative permeability data published by *Grant et al. (2004)*, and the ones measured in the lab, based on the method developed by *Tóth et al. (1998a, 1998b, 2002)* using the UTCHEM code. The calculations proved the sensitivity of transport models on relative permeability data that highlights the need of using measured relative permeability curves to improve the accuracy of the models.

INTRODUCTION

Several methods are known for gaining information about the contamination plume of chlorinated hydrocarbons like geophysical measurements *(Plank et al. 2011)* and detected concentration in monitoring wells, but these methods only serve data for a given moment. On the other hand, transport models give prognosis about the change of pollutant concentrations in a period *(Kool et al. 1987)*. Although the results of models are only approximations of the real case, they support the planning of the remediation process. The efficiency of remediation technologies depends on the geology of the aquifer, the geochemical properties of porous media, the chemical properties of groundwater and the characteristics of the pollutant *(Puzder et al. 2001)*. In the following sections a couple of effective and economical technologies will be introduced like biodegradation, in situ chemical reduction and in situ flushing.

Degradation of organic pollutants in case of bioremediation is intensified by increasing concentrations of electron acceptors and nutrients *(Puzder et al. 2001)* but the halogenated hydrocarbons do not serve as carbon source for bacteria and fungi. During remediation a nutrient needs to be added to the system which includes the carbon in organic bond. This nutrient could be methanol, methane, molasses, butane or vegetable oils. After the degradation of these nutrients, hydrogen is released ($H_2 \rightarrow 2H^+ + 2e^-$) which could be added to the chlorinated organic compounds by means of chlorine-respiring microorganisms. At the end of this progress the halogenated organics aredehalogenated. If sufficient quantity of the auxiliary substrate is available, the reductive dechlorination could proceed to the chlorine-free final products (ethylene and ethane) that could easily be degradated to CO_2 in aerobe conditions *(Anton, 2010)*. The dehalogenization efficiency is influenced by the chemical characteristics of the aquifer because in presence of alternative electron acceptors (for example nitrate), transformation is limited. In case of volatile chlorinated organic compounds the degradability of these compounds in aerobe and anaerobe conditions is different. For example the 1,1 DCA is non degradable in aerobe conditions *(Van Agteren et al. 1998),* but in anaerobe conditions it is degradable *(Ferris et al. 2006, Best, 1999, Grostern & Edwards, 2006)*. The 1,2 DCA is degradable both in aerobe *(Dyer et al. 2000, De Wildeman et al. 2002, Grostern & Edwards, 2009)* and anaerobe *(Michiels & Breugelmans, 1994, Davis et al. 2009)* conditions. The chlorinated ethenes are more degradable in aerobe conditions than in anaerobe ones and the efficiency of anaerobe degradation depends on significant presence of other DNAPLs like TCE and PCE.

The most effective chemical reduction technology is the PRB (permeable reactive barrier) with zero valent iron (ZVI). In the barriers the chlorinated hydrocarbons like TCE are reduced in the process which is called beta-elimination. During this process two electrons are added to TCE which loses chlorine and transforms to cis-1,2-DCE. The chlorinated hydrocarbons lose the chlorines step by step while at the end of process the neutral etheneis formed *(Lai et al. 2006)*. This method is effective in the center of contamination to reduce the concentration of the pollutant.

In-situ flushing is also an effective technology for the remediation of chlorinated hydrocarbons, but it needs contaminant specific planning, because acidic or alkaline solution or surfactants are injected into the contaminated layer. The solution mobilizes the contamination which is pumped and treated on the surface *(Puzder et al. 2001)*.

MATERIALS AND METHODS

In the laboratory of Research Institute of Applied Earth Science at the University of Miskolc a measuring and calculation methodology was developed to investigate and describe the process of transient radial water displacement in core samples *(Tóth et al. 1998a, 1998b, 2002, 2005)*. The calculation of relative permeability curves are derived from labor measurements performed using a simple apparatus *(Figure 1)* that could be utilized in the UTCHEM transport module instead of applying data from the literature.

To investigate the model performance two parallel models were built, one using relative permeability curves from the literature *(Grant et al. 2004)* and the other one with parameters from the above mentioned labor measurements *(Table 1)*.

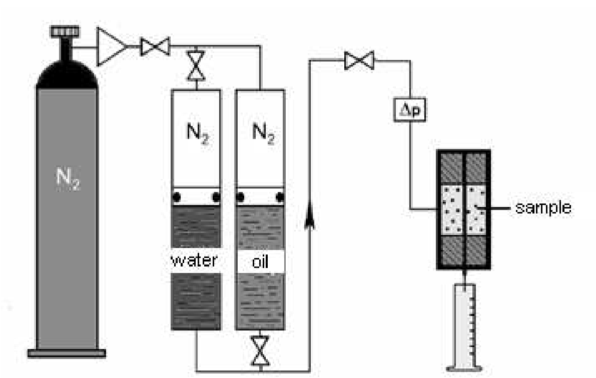

Figure 1.Schematic drawing of apparatus for measurement of radial water displacement

The models were created in UTCHEM with the following parameters:

- initial hydraulic head: 6 m,
- hydraulic conductivity: k_{aq} = 0.00001 m/s,
- anisotropy: 1.2,

- flow direction: from the left side to right, hydraulic gradient 0.001,
- porosity: $n_{aq} = 0.3$.

Table 1

Parameters of relative permeability curves

Origin of data for UTCHEM	**Aqueous phase**		**TCE phase**	
	Residual saturation	**K_r endpoint**	**Residual saturation**	**K_r endpoint**
Literature	0.24	0.17	0.15	0.45
Measurement	0.1	0.4	0.2	0.65

The created mesh consisted of 32 rows and 120 columns *(Figure 2)*. After the simulation the transit time of contamination to the aquiclude, the peak concentration of the plume and the extension of plume were investigated. The results of the two transport models are presented in *Table 2*.

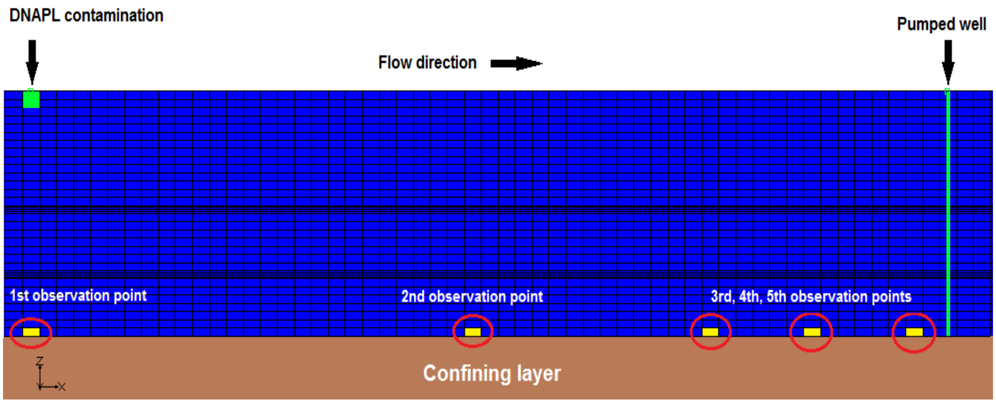

Figure 2. Mesh of UTCHEM transport model and observation points

Table 2

Results of models based on different relative permeability curves

	Transit time to aquiclude (d)	**Highest observed concentration (µg/l)**	**Length of plume (m)**
Relative permeability curve in the literature	15	582	25.5
Relative permeability curve from measurements	20	663	24.5

After that a remediation process was simulated. A pumped, fully screened well of ∅0.11 m was installed at 27.5 m distance from the source of contamination with 100 m^3/d discharge. In order to compare the two model runs, observation points were defined in the model (Figure 2). The TCE concentration was investigated in these points after 60, 120, 180, 240, 300 and 360 days.

RESULTS

The spread of contamination is quicker, the highest observed concentration is lower, and the longitudinal extension of plume is higher in the model which is based on the data from the literature (henceforth Case 1) than in the model which is based on the measured data (henceforth Case 2). The difference between the extensions of plumes is 1 m *(Figure 3).*

The concentration changes in time are shown in *Figure 4* and in *Figure 5*. At the first observation point beneath the source of contamination, the concentration of pollutant is higher in Case 2 than in Case 1 at the start of remediation modeling, and stays higher until the end of the simulated 360 day-long period. Significant difference is only experienced in case of initial concentrations *(Figure 4).*

The differences between concentrations are more relevant at the fifth observation point than at the first one. At the start of remediation, contamination was not yet detected at this point because of the slow transport of pollutants, but after 60 days contamination appeared. The calculated concentrations of the two models differ significantly at the beginning of the process (after 60 and 120 days) but later they start to decrease. At the end of the simulation period (after 360 days) the concentration in Case 1 is almost the double of the concentration in Case 2 *(Figure 5).*

CONCLUSIONS

In this paper the sensitivity of transport modeling based on data of relative permeability curves was investigated. The transport models are integrant elements in the planning process of remediation, so the confidence of these models is a relevant aspect. In the application of the measuring method developed by J. Tóth and his colleagues, the accuracy of transport modeling of chlorinated hydrocarbons and also the modeling of the remediation progress could be improved.

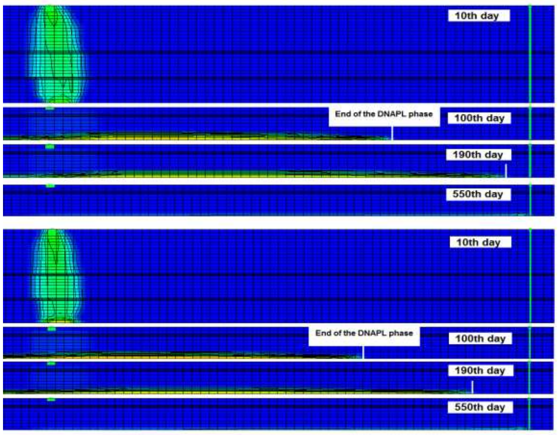

Figure 3. Spread of contamination using data of relative permeability curve, from literature (above) and from measurements (below)

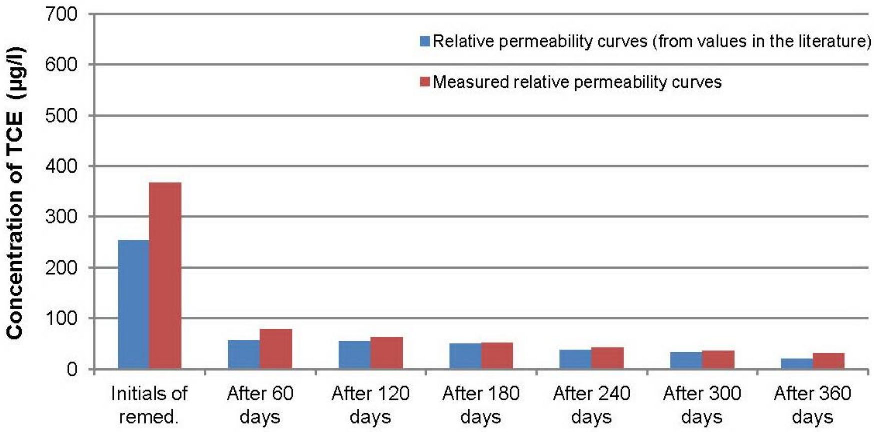

Figure 4. Concentration of TCE at the 1st observation point

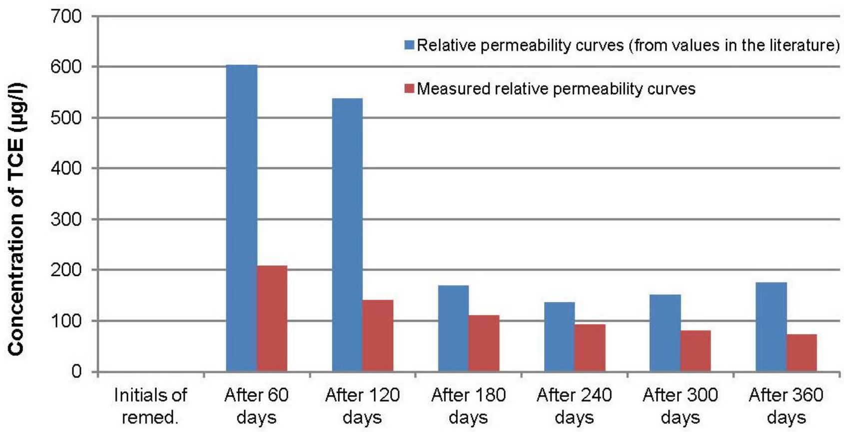

Figure 5. Concentration of TCE at the 5th observation point

A significant difference was experienced between the results of modeled scenarios based on data from the literature and those from the measurements. The simulation isomer accurate in case of applying the data from the measurements than from the literature but the data of more measurements could further increase the accuracy of models. The most correct model could be built on the data of field experiments. Putatively, the observed concentrations are pollutant-specific and they also depend on the velocity of dehalogenization.

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Statistical analysis of SHear strength parameters of municipal solid wastes by slope stability analysis

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ABSTRACT

The determination of the shear strength parameters of municipal solid waste material is one of the most difficult problems of the slope stability analysis of landfills. The aim of this study was to examine the frequencies and the correlation of the cohesion and the internal friction angle of wastes based on the collected results of previous international laboratory measurements, site investigations or back analysis.

INTRODUCTION

Geotechnical performance of a structure is strongly dependent on the properties of the constituent materials. These properties can be described using deterministic and/or probabilistic models. Deterministic models typically use a single discrete descriptor for the parameter of interest. Probabilistic models describe parameters by using discrete statistical descriptors or probability distribution (density) functions. The spatial variation of the properties can be described by using stochastic interpolation methods *(Jones et. al., 2002)*.

Uncertainty is usually our first thought, when trying to determine physical parameters for slope stability of landfills. Considering that the variability of the properties of wastes is much higher than of soils, we cannot solve geotechnical problems using the deterministic approach, because it cannot deal with this uncertainty. If we force the use of traditional methods our landfills will be either over-designed and not really cost effective, or unsafe and dangerous. Thus the importance of the application of probabilistic methods cannot be questioned in order to get more reliable results.

Variability of the properties of wastes is caused by different reasons, such as the circumstances and the technology of landfill construction, the age, degradation and composition of municipal waste, etc.

Several new geotechnical software packages provide the applicability of probabilistic methods in slope stability analysis, but for this we have to determine the frequencies and the correlation of the two shear strength parameters, of the cohesion and the internal friction angle of the waste material.

UNCERTAINTY IN GEOTECHNICS

Geotechnical variability results from different sources of uncertainties. The three primary sources are inherent variability, measurement error and model uncertainty, as described in *Figure 1.*

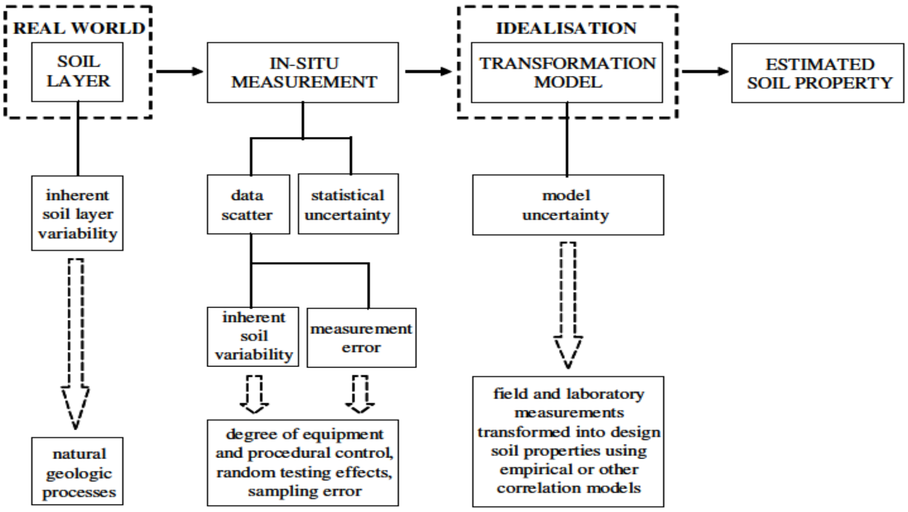


Figure 1. Uncertainty in soil property estimates (Kulhawy, 1992 in [RUSSELLI, 2008])

Inherent variability results primarily from natural geologic processes that created in-situ soil layers (or the variability of the collected waste material by landfilling). Measurement error is caused by sampling and laboratory testing.

This error is increased by statistical uncertainty that arises from limited amount of information. Finally the model uncertainty is introduced when field or laboratory measurements are transformed into input parameters for design models involving simplifications and idealizations *(Russelli, 2008)*.

The uncertainty in geotechnical properties of soils (and also waste materials) can be formally grouped into aleatory and epistemic uncertainty *(Lacasse et al., 1996 in [Jones et. al., 2002])*. Aleatory uncertainty represents the natural randomness of a property and, as such, is a function of the spatial variability of the property. Recognizing spatial variability is important because it can help distinguish the distances over which it occurs compared to the scale of the data of interest *(Whitman, 1996 in [Jones et. al., 2002])*. Epistemic uncertainty results from a lack of information and shortcomings in measurement and/or calculation. Epistemic uncertainty includes the systematic error resulting from factors such as the method of property measurement, the quantity of available data, and modeling errors.

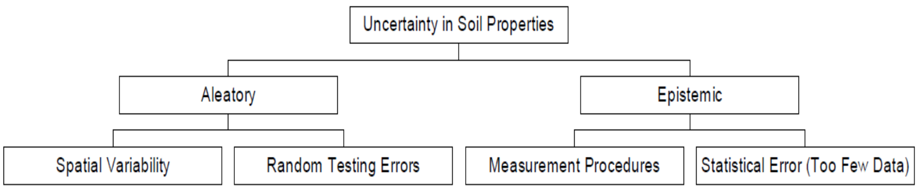


Figure 2. Sources of uncertainty in geotechnical soil properties (adapted from Whitman, 1996 in [JONES et. al., 2002])

Figure 2 illustrates the types of uncertainty in geotechnical soil properties. Human error would be considered a third source of uncertainty, however it is not considered in this overview because it is difficult to isolate and its effects on probability are usually included in compilations of statistics on aleatory uncertainty *(Jones et. al., 2002)*.

In a probabilistic analysis the geotechnical parameters, which represent the major sources of uncertainties, are treated as random variables. A random variable is a mathematical function defined on a sample space that assigns a probability to each possible event within the sample space.

In practical terms, it is a variable for which the precise value (or range of values) cannot be predicted with certainty, but only with an associated probability, which describes the possible outcome of a particular experiment in terms of real numbers.

In this study the waste shears strength parameters: the cohesion and the internal friction angle are considered as random variables for the probabilistic analysis of the slope stability problem.

The most important statistical parameters related to the waste material properties variability are the mean value (μ_x), the standard deviation (σ_x), the skewness coefficient (ν_x) and the correlation coefficients (ρ_{xy}) between the two shear strength parameters. The variability of these data can be plotted graphically as histograms, or frequency diagrams *(Russelli, 2008)*.

The normal Gaussian distribution is the probability distribution most frequently used because of its symmetry and mathematical simplicity. It is commonly assumed to characterize many random variables where the coefficient of variation (COV = σ_x /μ_x)) is less than about 30%, as seen in *Figure 3 (a)* for the effective friction angle of the Frankfurt clay *(Russelli, 2008)*.

The lognormal distribution is generally accepted to reasonably model many soil properties, because it is strictly non-negative. It often provides a reasonable shape in cases where the coefficient of variation is larger than 30%, as for the effective cohesion of the Frankfurt clay in *Figure 3 (b)*. It can be concluded that the lognormal distribution may well represent the natural distribution for many spatially varying soil properties *(Russelli, 2008)*.

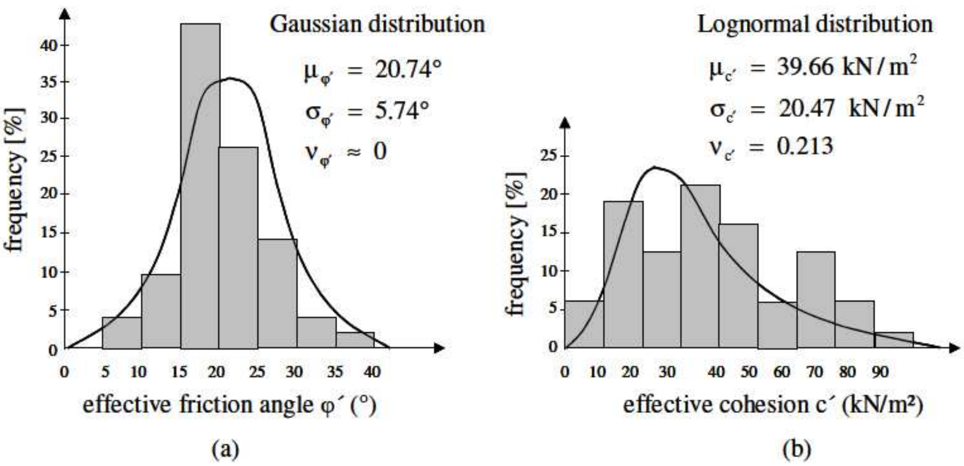


Figure 3. Probability density functions of soil strength parameters for the Frankfurt clay (MOORMANN and KATZENBACH, 2000 in (RUSSELLI, 2008))

In geotechnical practice the normal Gaussian distribution was considered to characterize the shear strength parameters of soils where their correlation was supposed be not perfectly, but negative correlated and we presumed in previous works, that we can base our landfill slope stability research on experiences associated with soils. But the lack of exact knowledge can lead to fatal errors, thus the exact determination of the distribution of the shear strength parameters of wastes and their coefficient of correlation is essential.

SHEAR STRENGTH PARAMETERS OF WASTES

Shear strength parameters of different types of municipal solid wastes were measured, determined and discussed by several authors in geotechnical literature, but there are only some suggestions, how to take them into account by the slope stability analysis of landfills, such as *Manassero et al., (1996), König-Jessberger (1997), Sanchez-Alciturri et al. (1993)* and the ÖNORM (Austrian Standard) in *(Szabó, 2008)* and *(Szabó and Szabó, 2012)*.

Based on the collection of data in *(Szabó, 2008), (Varga, 2010, 2011a and 2011b)* and *(Szabó and Szabó, 2012)* we summarized the shear strength parameters in several tables and plotted graphically, see *Figure 4*.

As it can be seen in Figure 4a and b this collection was grouped based on the method of determination. Normal Gaussian fits of the relative frequencies of these data groups were graphed in Figure 5. and statistical parameters were determined (mostly by SPSS) in Table 1.

Both the graphs and the descriptive data in the table show, that the distribution of the internal friction angle of wastes is a little bit negatively skewed (compare with Figure 3 a, where the skewness of the density function of friction angle is 0, thus considered normal Gaussian distribution), with higher than 40% coefficient of variation. The distribution of the cohesion of wastes is a positively skewed, with higher than 80% coefficient of variation.

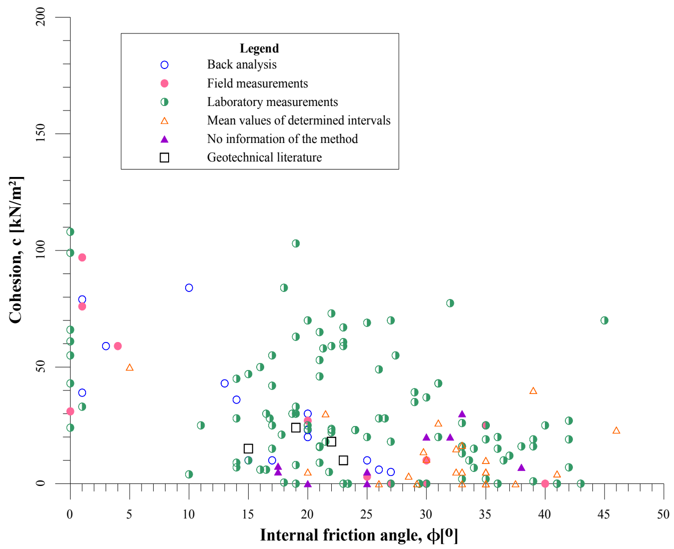
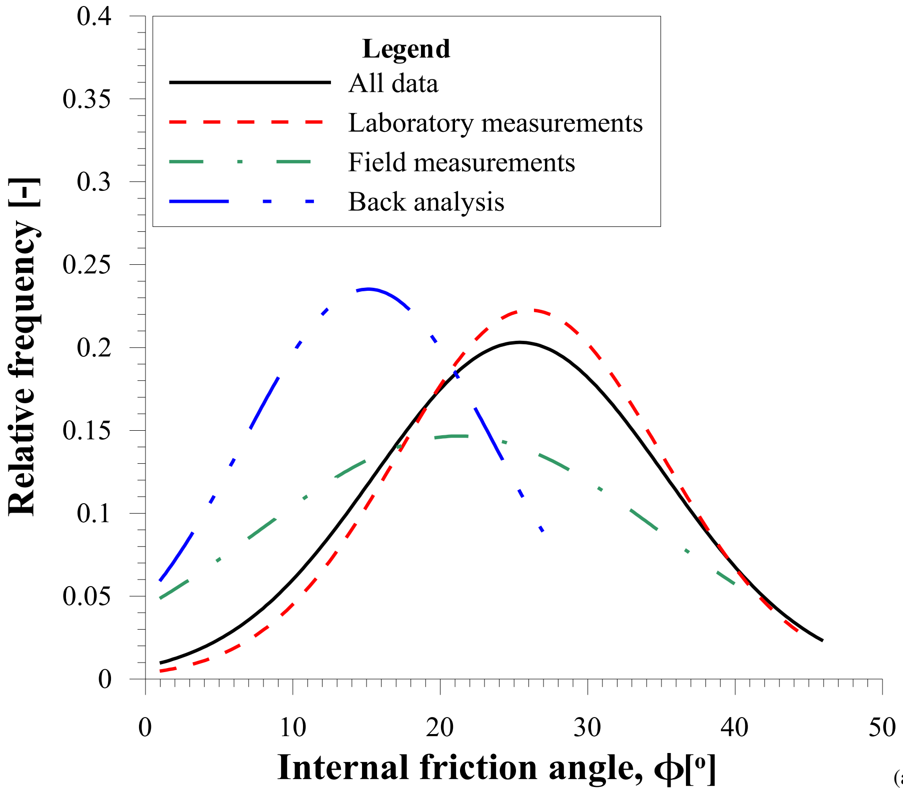
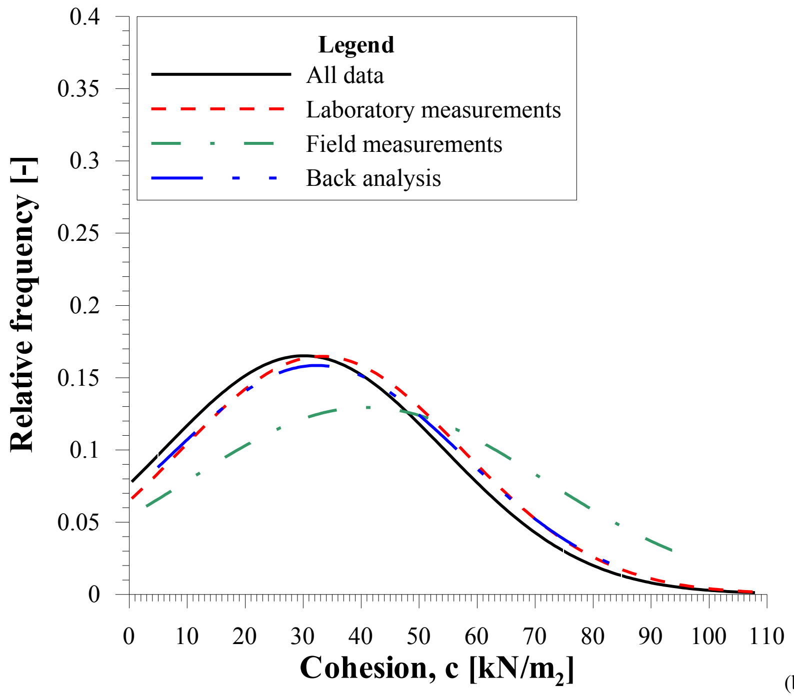


Figure 4. Shear strength parameters of wastes based on the results of back analysis, field and laboratory measurements and other geotechnical literature sources (Faur, 2012)

Some authors have collected information on the ranges of variation coefficient values for spatial variability of different soil properties, as derived from in-situ soil investigation and for the variability due to measurement errors. A well-known study on the COV values is that of *Phoon and Kulhawy (1999)*, which represents a good indication of the order of magnitude for the COV values of soil variability. In this study *Phoon and Kulhawy* presented data for sand and a clay layer and they found COV values between 5–15% for the effective friction angle. This range was also put forward by *Harr (1989)* and *Cherubini (1997)*. Occasionally higher COV values can be found for the friction angle, as in the report of Moormann and *Katzenbach (2000)*, where a value of about 30% is indicated for the Frankfurt clay. But this high COV value of the friction angle is not for a particular site, but for the entire Frankfurt area.



(a)



(b)

Figure 5. Normal Gaussian fits of the relative frequencies of these data groups (a) Internal friction angle (b) Cohesion

Table 1.a

Descriptives of the shear strength parameter data sets

	N [Number]	Minimum	Maximum	Mean (μ)	Standard Deviation (σ)
Valid N (listwise)	169	**All data [Unit in lines]**			
Cohesion, c [kN/m^2]	169	0.000	108.000	25.922	24.848
Internal friction angle, ϕ [°]	169	0.000	46.000	24.222	11.049
Valid N (listwise)	109	**Laboratory measurements [Unit in lines]**			
Cohesion, c [kN/m^2]	109	0.000	108.000	29.313	25.311
Internal friction angle, ϕ [°]	109	0.000	45.000	24.432	10.838
Valid N (listwise)	11	**Field measurements [Unit in lines]**			
Cohesion, c [kN/m^2]	11	0.000	97.000	29.818	33.630
Internal friction angle, ϕ [°]	11	0.000	40.000	19.364	15.075
Valid N (listwise)	14	**Back analysis [Unit in lines]**			
Cohesion, c [kN/m^2]	14	5.000	84.000	32.429	26.182
Internal friction angle, ϕ [°]	14	1.000	27.000	15.143	8.813

Table 1.b

Descriptives of the shear strength parameter data sets

	Coefficient of variation (COV) [%]	**Variance**	**Skewness**		**Kurtosis**	
		Statistic	**Statistic**	**Std. Error**	**Statistic**	**Std. Error**
Valid N (listwise)	**All data**					
Cohesion, c [kN/m^2]	95.855	617.401	1.164	0.187	0.811	0.371
Internal friction angle, ϕ [°]	45.615	122.081	–0.451	0.187	–0.193	0.371
Valid N (listwise)	**Laboratory measurements**					
Cohesion, c [kN/m^2]	86.348	640.653	0.979	0.231	0.469	0.459
Internal friction angle, ϕ [°]	44.359	117.461	–0.374	0.231	–0.069	0.459
Valid N (listwise)	**Field measurements**					
Cohesion, c [kN/m^2]	112.783	1130.964	1.028	0.661	–0.055	1.279
Internal friction angle, ϕ [°]	77.852	227.255	–0.272	0.661	–1.700	1.279
Valid N (listwise)	**Back analysis**					
Cohesion, c [kN/m^2]	80.737	685.495	0.914	0.597	–0.160	1.154
Internal friction angle, ϕ [°]	58.200	77.670	–0.438	0.597	–0.854	1.154

Considering the effective cohesion, *Harr* suggests in his work a value of about 20%. *Cherubini* presents values between 20–30%, *Li & Lumb (1987)* report a particular clay layer with a COV of 40% and *Moormann & Katzenbach* quote 50% for the Frankfurt clay. It can be seen that the available data on the effective cohesion show much more variation than for the effective internal friction angle (Russelli, 2008).

Table 2 shows the calculated correlations between the shear strength parameters.

Table 2

Correlations of the shear strength parameter data sets

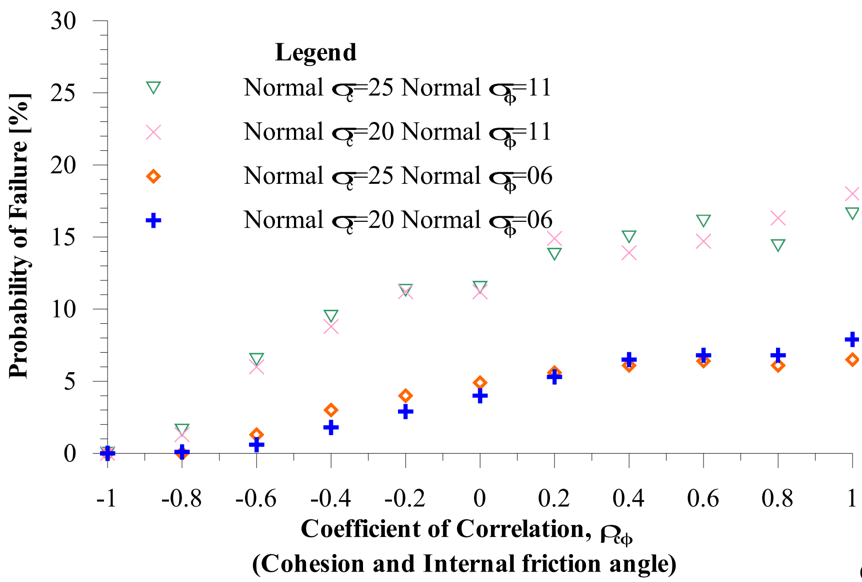
		All data		**Laboratory meas.**	
		Cohesion, c [kN/m^2]	**Internal friction angle, ϕ [°]**	**Cohesion, c [kN/m^2]**	**Internal friction angle, ϕ [°]**
Cohesion, c [kN/m^2]	Pearson Correlation	1	–.487**	1	–.400**
	Sig. (2-tailed)		.000		.000
	N	169	169	109	109
Internal friction angle, ϕ [°]	Pearson Correlation	–.487**	1	–.400**	1
	Sig. (2-tailed)	.000		.000	
	N	169	169	109	109
		Field measurements		**Back analysis**	
		Cohesion c [kN/m^2]	**Internal friction angle, ϕ [°]**	**Cohesion c [kN/m^2]**	**Internal friction angle, ϕ [°]**

Cohesion, c [kN/m^2]	Pearson Correlation	1	$-.819^{**}$	1	$-.785^{**}$
	Sig. (2-tailed)		.002		.001
	N	11	11	14	14
Internal friction angle, ϕ [°]	Pearson Correlation	$-.819^{**}$	1	$-.785^{**}$	1
	Sig. (2-tailed)	.002		.001	
	N	11	11	14	14
**. Correlation is significant at the 0.01 level (2-tailed).					

The correlation of the two shear strength parameters measured in laboratory is about –0.4, but the correlation of the results of field measurements and back analysis is about –0.8. Results of several running of slope stability analysis of a Hungarian landfill by Soilvision's SVSlope module are graphed on *Figure 6.*

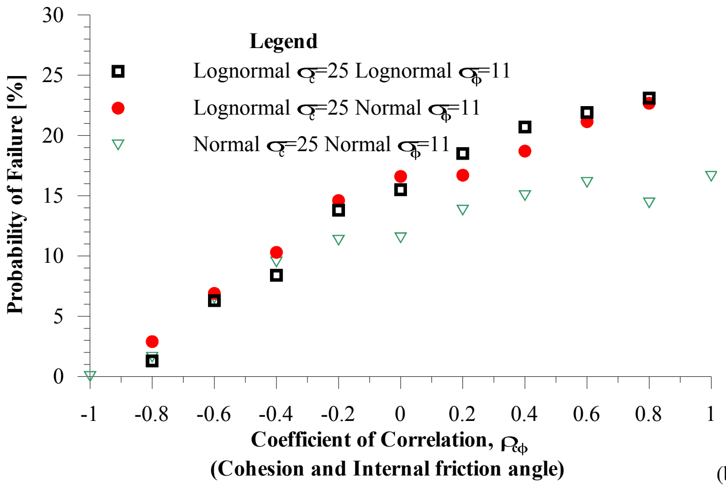
CONCLUSIONS

The reduction of the standard deviation (SD) of the cohesion of caused only insignificant changes in the probability of failure, but the reduction of the SD of the internal friction angle caused remarkable changes in the probability of failure (see Figure 6 a). Thus if we decide to base our slope stability analysis on some own results instead of all the available data, it can lead to false conclusions.



(a)

Figure 6 b shows, that by negative correlations the effect of the application of the lognormal or the normal distributions for the shear strength parameters is insignificant, but if we decide to use the normal Gaussian distribution for the internal friction angle, and the lognormal distribution for the cohesion by slope stability analysis, we can stay on the safe side. As our results show, the correct determination of the statistical parameters of the shear strength parameters of municipal solid waste is essential to achieve reliable results of slope stability analysis of landfills.



(b)

Figure 6.
Probability of failure by slope stability analysis as a function of the coefficient of correlation; (a) Effect of the reduction of the standard deviation
(b) Effect of the application of the normal or lognormal distributions

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Effect of precipitation on the slope stability of landfills

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ABSTRACT

Precipitation and pore water pressure increase can cause landslides not only on tropical climate, but also in Hungary. The determination of the effect of different amounts of precipitation is the main goal of this paper. The slope stability calculations were run by Soilvision's coupled SVFlux and SVSlope modules. Due to the lack of knowledge of the input parameters several problem occurred by the evaluation of the results. Further investigation of the hydraulic and the geotechnical parameters of municipal solid wastes are suggested for more reliable calculations.

INTRODUCTION

Several large, sometimes fatal landfill failures in the world prove the effect of pore water pressure increase on slope stability. Since we have the opportunity to use the Soilvision software package, we tried to couple the SV Flux and the SV Slope modules to determine the effect of precipitation on landfills without cover systems (e.g. landfills under construction).

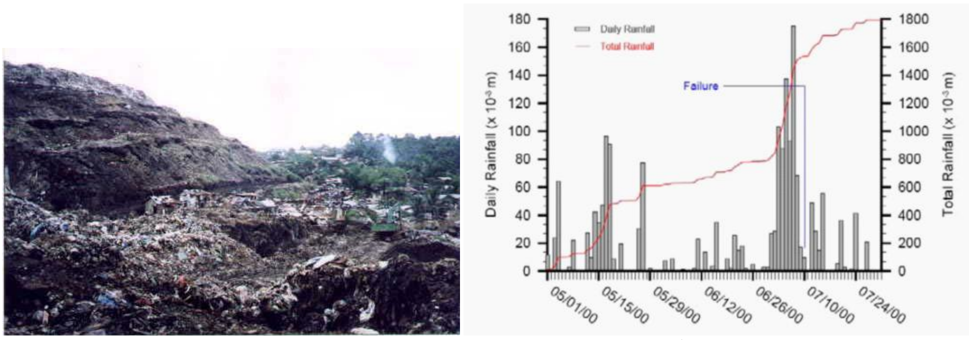
The stability analysis of the failure (in February 2005) of the Leuwigajah dumpsite near Bandung, Indonesia proved that the failure most likely have been triggered by water pressure in the soft subsoil in combination with a severe damage of reinforcement particles due to a smoldering landfill fire. This result addresses a specific stability problem in tropical countries. *Since precipitation is high (1500–2000 m per year), but non-uniform, both events may happen.* On one hand there are acute drainage problem coming along with pore water pressure in soft soils or high water tables inside landfills *(Kölsch et. al. 2005)*.

On July 10th 2000around 5 A.M., a slope of the landfill failed and 1.2 Mio m^3 waste slid down on the Payatas dumpsite (located in the North-East of Metro Manila within in the boundaries of Quezon City on Luzon, the major island of the Philippines). *Figure 1a* shows a part of this area at the day after the landslide in front of a remaining slope, where still huts of waste pickers can be seen. Rescue actions were hindered by landfill gas, which created several fires. More than 220 people were found dead in the plastered area *(Kölsch, 2001)*.

Basically, a landslide happens in case a slope is constructed steeper than the shear strength of the material is sufficient to bear. Since the strength of waste is not well-known and is usually not being investigated in the laboratory, the waste is placed due to common experiences. It seems that the geometry of the Payatas dumpsite has met these experiences showing slopes of about 1:3 with a height of 25 m. However, some specific conditions, which show up in tropical developing countries, apparently have not been considered, thoroughly.

- *Low density of waste* due to a high portion of light plastic materials and an insufficient waste compaction
- *Unlimited water percolation* The water percolation is usually adjusted by paper. The observation has shown a lack of paper due to an outstanding high recycling ratio

- *Extremely high rates and non-uniform distribution of rainfall (see Figure 1b) (Kölsch, 2001)*



(a) *(b)*

Figure 1. (a) Collapsed slope at the day after the failure (Kölsch, 2001)
(b) Precipitation in Manila, Payatas (Philippines), 2000

Anyways, stability problems are not restricted to the tropics; a troubled water balance jeopardizes the landfill stability, even in temperate climate areas, as long as the waste composition leads to a low waste density at the landfill. The Rumpke landslide, which occurred 1996 in Cincinnati (USA), demonstrates that US-landfills are more jeopardized than European ones. The reason is the extraordinary large portion of light waste (plastics) combined with disturbed water balances due to leachate circulation *(Kölsch, 2001)*.

2. SVFLUX-SVSLOPE COUPLED SLOPE STABILITY ANALYSIS OF LANDFILLS

Based on the experiences of these large landfill failures there is a need to prove the slope stability of landfills also in extreme climatic circumstances.

The most common trigger for slope stability failures world-wide is rainfall events. Such events destroy the negative water pressures (suctions) in the upper zone of the soil and cause failures. As such, it is important that slope stability software be able to accommodate this type of failure mechanism. SVSlope implements three types of unsaturated soil strength models in order to provide increased capabilities to the user. In addition, the SVSlope software may import steady-state or transient pore-water pressure distributions from the SVFlux software *(Feng–Lu, 2012)*.

Although the annual precipitation in Hungary is between 500 and 800 mm, sometimes there were extreme values, for example the maximum estimated daily precipitation was 260 mm in Dad on the 9th of June 1953, the maximum measured daily precipitation was 203 mm in Gyömrő on the 8th of September 1963 and the maximum precipitation in 60 minutes was 120 mm in Heves on 23^{d} of August 1988 *(Bihari et. al. 2009)* and our experiences show that the frequencies of such extreme precipitation events can be higher due to supposed climatic changes.

In our slope stability calculations several different landfill geometry was analyzed (such as 10–20–30–40 meters high, and 1:1.5, 1:2, 1:2.5 slope). One selected geometry is shown on *Figure 2* (40 meters high, with a slope of 1:2.5).

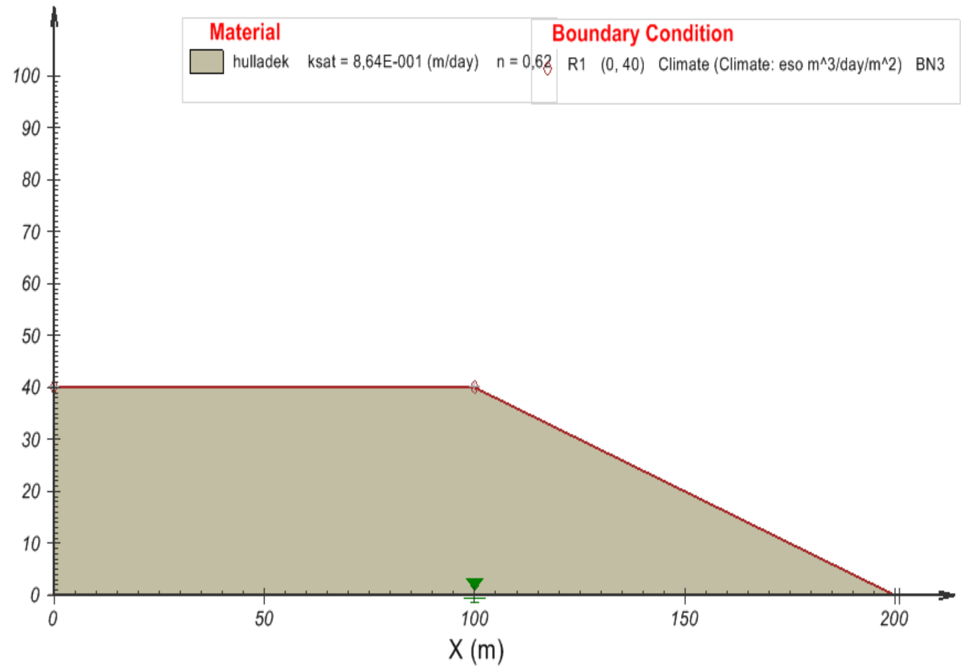


Figure 2. Geometry of the landfill

Two soil strength models were applied by our calculations: the most common *Mohr–Coulomb soil model*, and the *unsaturated linear (Phi-b) model*.

The most commonly used shear strength designation for saturated soils encountered geotechnical engineering is *the Mohr–Coulomb equation*:

$\tau = c' + (\sigma_n - u_w) \cdot \tan\phi'$ where τ is the effective shear strength, c' is the effective cohesion, σ_n is the normal stress on the shear plane, u_w is the pore-water pressure and ϕ' is the effective angle of internal friction.

The Mohr–Coulomb equation can also be written in terms of total stress condition. For a total stress analysis the pore-water pressures are not considered and the cohesion and friction angle are defined in terms of total stress conditions. The Mohr–Coulomb equation can be written: $\tau = c + \sigma_n \cdot \tan\phi$ *(Feng – Lu,2012)*.

The theory of *the unsaturated linear model (Phi-b)* is based on the following: Matrix suction (or negative pore-water pressures) in an unsaturated soil affects the shear strength. The negative pore-water pressure increases the shear strength of an unsaturated soil. The shear strength of an unsaturated soil has been shown to be nonlinear in form when matric suction is varied over a considerable range. Consequently, there are nonlinear forms for the unsaturated shear strength equation as well as a linear approximation of the unsaturated shear strength. The linear approximation of the unsaturated soil shear strength can be written as follows:

$\tau = c' + (\sigma_n - u_a) \cdot \tan\phi' + (u_a - u_w) \cdot \tan\phi^b$ where c' is the effective cohesion, ϕ' is the effective angle of internal friction, ϕ^b is the angle defining the increase in shear strength due to matric suction (or negative pore-water pressure), σ_n is the total normal stress, u_a is the pore-air pressure, and u_w is the pore-water pressure.

The term (u_a–u_w) is called matric suction and is a positive representation of the negative pore-water pressure. The angle ϕ^b is a material property. The angle ϕ^b can range from zero to the ϕ' value depending on the material type and the magnitude of matric suction. The default value for ϕ^b is set to $\phi'/2$.

In SVSlope, the effect of negative pore-water pressures is ignored when ϕ^b is not specified (i.e., zero). For other values of ϕ^b an increase in shear strength due to matric suction (u_a–u_w) will be included in accordance with the above equation *(Feng–Lu, 2012)*.

The geotechnical properties of the waste body were based on values from literature *Varga & Czap (2004)*, *Szabó (2008,) Szabó I. & Szabó A. (2012), Varga (2011)* and can be seen in Table 1. The cohesion and the internal friction angle of the waste were calculated as the mean value of 169 collected data *(Faur, 2012)*. The unsaturated internal friction angle is not known for municipal solid waste material, thus it was set as a little bit less than half of the internal friction angle.

Table 1

Applied physical properties of the waste body by the slope stability calculations

Unit weight [kN/m^3]	13
Cohesion, c [kN/m^2]	25
Internal friction angle, ϕ [°]	24
Unsaturated internal friction angle, ϕ^b [°]	10

The hydraulic conductivity of the waste body were based on the laboratory measurements of Heyer in *(Szabó I. & Szabó A., 2012)*, on municipal solid waste material of different age and treatment (such as fresh municipal solid waste, 20 years old deposited municipal solid waste, shredded municipal solid waste), see in *Table 2.*

Table 2

Hydraulic conductivity of municipal solid waste material
(Heyer in [SzabóI. – Szabó A. 2012])

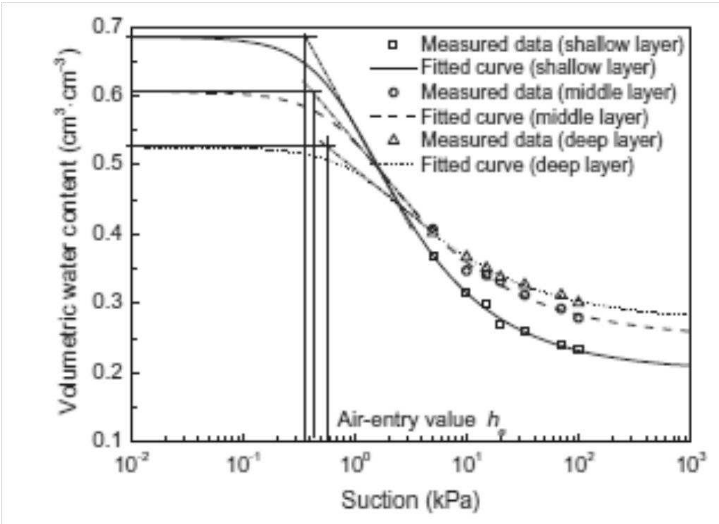
Vertical load [kPa]	**Modeled thickness of the waste body [m]**	**Wet**	**Dry**	**Hydraulic conductivity, k_{sat} [m/s]**
		Bulk density, ρ [t/m^3]		
0–100	0–10	0.7–1.15	0.25–0.7	5×10^{-3}–10^{-5}
100–200	10–20	0.8–1.25	0.35–0.8	10^{-4}–10^{-6}
~ 320	~ 34	0.9–1.3	0.5–0.9	10^{-6}–10^{-7}
~ 600	~ 63	1.2–1.4	0.6–0.95	10^{-7}–10^{-9}

The height of the landfill in our calculations vary between 10-40 meters, thus two different the hydraulic conductivities were chosen: For Case 1: $k_{case\_1} = 10^{-5}$ m/s = 0.864 m/d and for Case 2: $k_{case\_2} = 10^{-6}$ m/s = 0,0864 m/d.

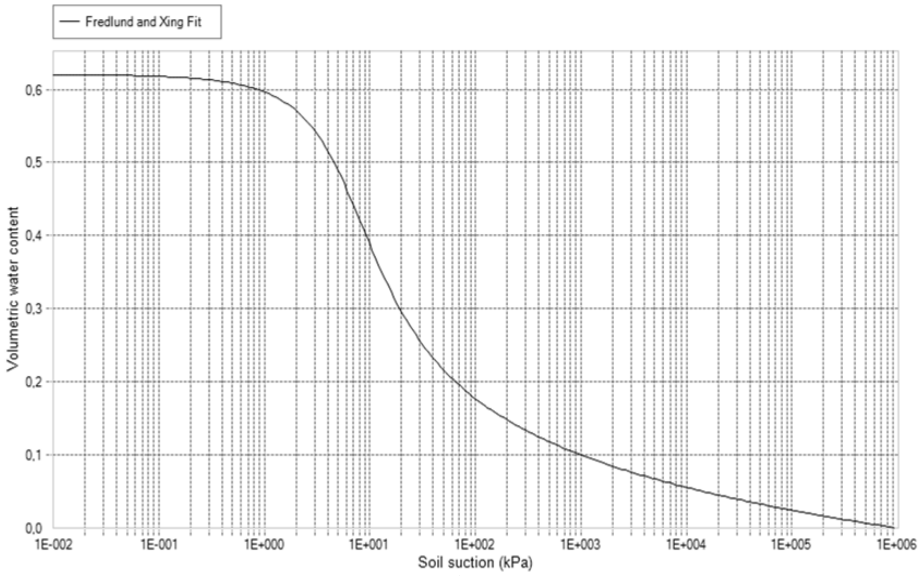
The moisture retention curve (see *Figure 3 a*) of municipal solid waste was determined by Wu et. al. based on some measurements on a landfill in Beijing, China (Wu et. al. 2012). Figure 3.b shows the SWCC curve applied in SVFlux.

In SV Flux we defined several different precipitation values, such as 10, 20, 30, 40, 50, 100 mm in 6, 2 and 1 hours time (Rainfall start was set at 08:00 first day, and the calculations were set to run for 3 days. After running all the cases in SV Flux the results can be opened in the SV Slope module. The results of the slope stability analysis were graphed in function of time for both hydraulic conductivities. The Bishop method was selected for our calculations in order to be comparable with previous results. Figure 4 shows, that the smaller the hydraulic conductivity of the waste, the larger the effect of precipitation on the factor of safety. The cases of Mohr-Coulomb soil models lead to smaller values of the factor of safety, according to the increase in shear strength due to matric suction ($u_a–u_w$) by the linear unsaturated (Phi-b) soil models. The value of $\phi^b = 10°$ of the waste material increases the factor of safety by 0,42 at the starting value (long before the precipitation starts), and 0,35 at the minimum values (but only by this geometry and soil properties).

If we apply the value of $\phi^b = 0°$, the results are the same, as the ones of the Mohr-Coulomb soil models.

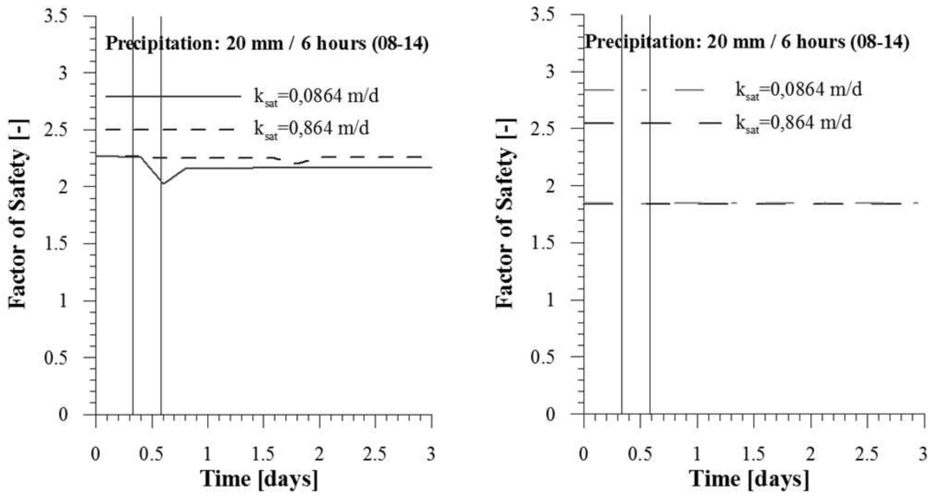


(a)

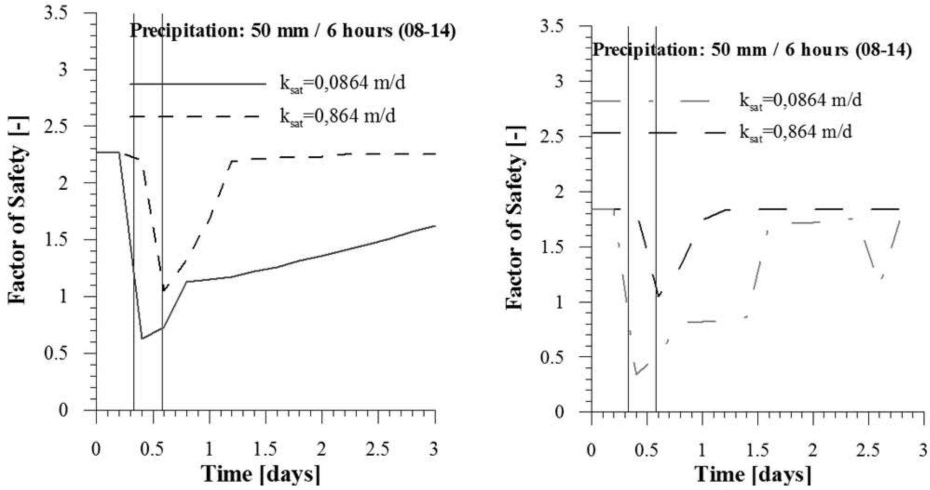


(b)

Figure 3. (a) Moisture retention curves of landfilled MSW: data from pressure plate tests, together with fitted van Genuchten-Mualem model curves using RETC (Wu et al. 2012) (b) SWCC curve applied in SVFlux



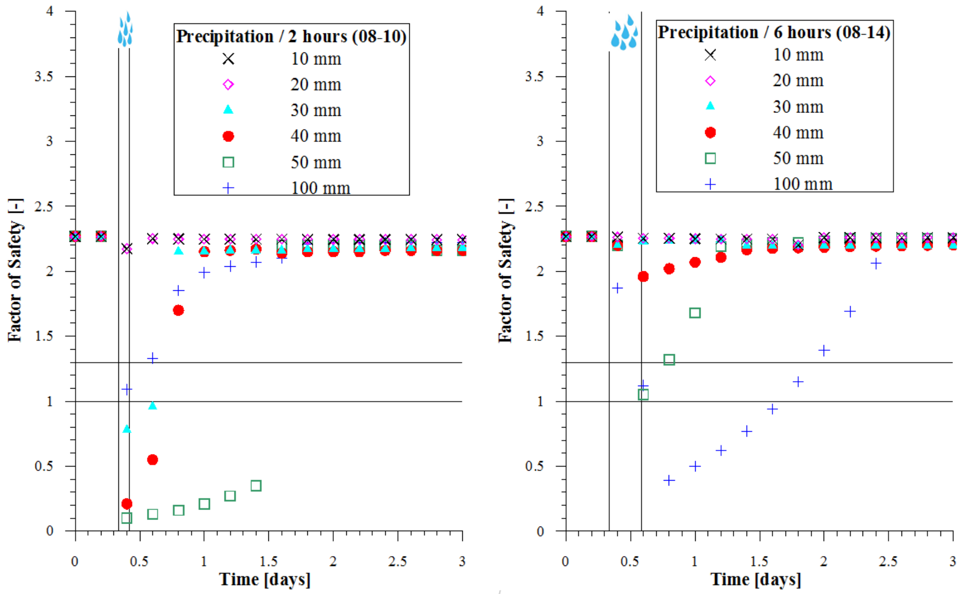
(a) (b)



(c) (d)

Figure 4. Effect of the hydraulic conductivity of waste on the factor of safety by slope stability analysis (a) & (b) 20 mm/6 hours (c) & (d) 50 mm/6 hours precipitation (a)&(c) Unsaturated (Phi-b) soil models (b) & (d) Mohr–Coulomb soil models

Figure 5 shows the change of the factor of safety in function of time (in Case 2:k_{case_2} = 10^{-6} m/s = 0.0864 m/d) for the linear unsaturated (Phi-b) soil model. If precipitation is falling for 2 hours 30 mm of rainfall is enough to cause failure, but if it is falling for 6 hours only 50 mm of rainfall causes failure (but only by this geometry and soil properties).



(a) (b)

Figure 5. Results of slope stability analysis by (a) 2 hours (b) 6 hours precipitation

In our calculations we met a severe problem of the evaluation of the results. Several very small factors of safety were calculated for very slip surfaces (by the application of Slope Search method) with small amounts (smaller than 1 m^3) of moving waste materials. There is a need to find a slip surface search method, which allows us to exclude these too small slip surfaces, but we can find all the critical slip surfaces otherwise. *Figure 6.* shows the results of the slope stability analysis of the landfill (by 20 mm precipitation in 6 hours, and Mohr-Coulomb soil model). On Figure 6.a the results of the slope stability analysis are drawn before the precipitation starts (0,0 day = 0 hours starting time), and the slip surfaces of the factors of safety 1 < F_{Bishop}< 2 are drawn, at *Figure 6.b* slip surfaces of the factors of safety 0 < F_{Bishop}< 1 are drawn after the precipitation started (at 0.4 day~9,6 hours time).

CONCLUSIONS

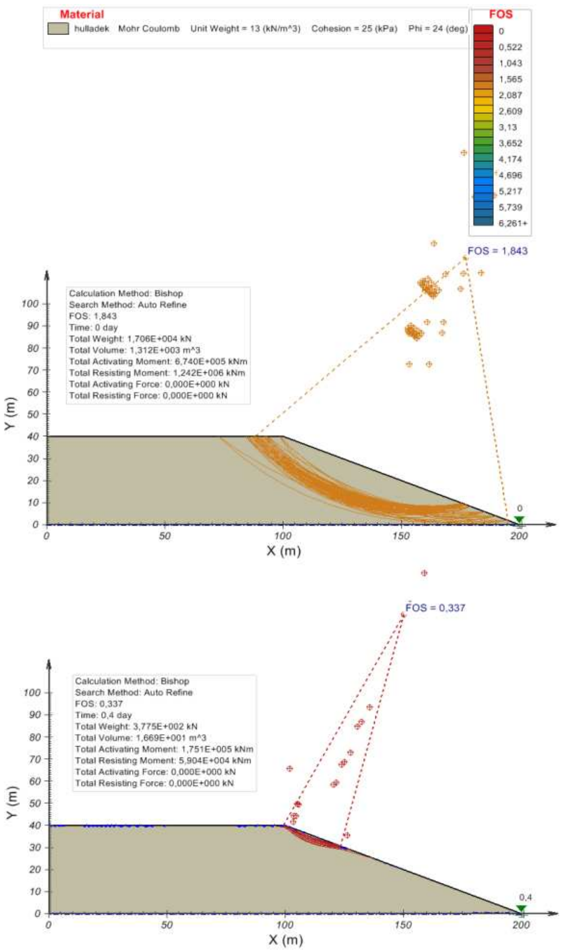


Figure 6. Results of slope stability analysis at 0,0 day = 0 hours starting time and the slip surfaces of the factors of safety 1 < F_{Bishop}< 2 and at 0.4 day~9.6 hours, slip surfaces of the factors of safety 0 < F_{Bishop}< 1

The effect precipitation and the increase of pore water pressure on the slope stability of a landfill were discussed in this paper. Our results show, that the smaller the hydraulic conductivity of the waste, the larger the effect of precipitation on the factor of safety. The period of time of the precipitation is also important. If 30 mm rain falls in 6 hours, the landfill remains stable, but if it falls only in 2 hours time, than a failure can occur.

A more important conclusion is, that the thorough knowledge of the waste properties is essential to achieve correct results of the slope stability of landfills. Further investigation of the hydraulic and the geotechnical parameters of municipal solid wastes are suggested for more reliable calculations.

These calculations were initial attempts to raise attention that by the slope stability analysis of either landfills or natural slopes the investigation of extreme climatic conditions is absolutely unavoidable. Further research must be focused to find the analysis method between the maximal safety and the optimum economic expenses - but within acceptable risks.

ACKNOWLEDGEMENTS

The authors would like to express their gratitude to Dr. Imre Szabó for his support in this research and wish himmany years of happiness.

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