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LABORATORY ASSESSMENT OF SOME GEOTECHNICAL PROPERTIES
OF A BIOPOLYMER TREATED SAND

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IN
CIVIL ENGINEERING

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Supervisor

Assoc. Prof. Dr. Ali Fırat ÇABALAR

by

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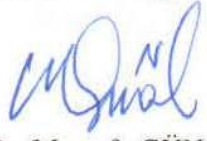
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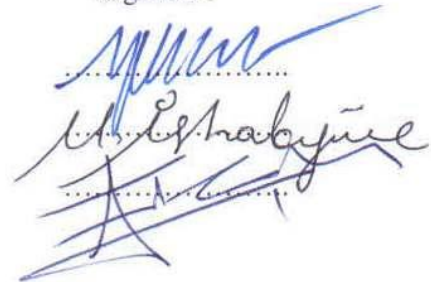
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Mateusz WISZNIEWSKI

ABSTRACT

LABORATORY ASSESSMENT OF SOME GEOTECHNICAL PROPERTIES OF A BIOPOLYMER TREATED SAND

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M.Sc. in Civil Engineering

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This thesis presents results of laboratory assessment of creating seepage barriers in a sandy soil and improving its geotechnical parameters using biopolymer additives, which consist of polysaccharides and water. Polysaccharides strongly interact with water to produce a viscous suspension. The research analyzes the potential use of biopolymer, namely xanthan gum to reduce hydraulic conductivity of coarse and medium sandy soils, and to strengthen their engineering parameters. For that reasons hydraulic conductivity, consolidation, triaxial, and unconfined compressive strength tests were carried out. Amount of the biopolymer used in a sample fall between 0.1 and 1.5%, by dry weight. Permeability tests were carried out on two different specimens, which were coarse sand (1.18 - 2.00 mm) and medium sand (0.075 - 1.00 mm), and other tests were conducted on medium quartz sand only. The results indicate that geotechnical parameters of the soil change significantly based on the amount of bio substance, gradation of the sand and curing time. It is concluded that biopolymers application could be used as a seepage barrier installation or general soil improvement technique, required to protect some geotechnical works including foundation, underground structures and waste disposals.

Key Words: biopolymers, permeability, seepage barriers

ÖZET

BIYOPOLİMER İLE İYİLEŞTİRİLMİŞ KUMLARIN BAZI MÜHENDİSLİK PARAMETRELERİNİN İNCELENMESİ

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Bu çalışmada, polisakkarit orijinli bir biyopolimer olan Xanthan Gum ile iyileştirilmiş kumların bazı mühendislik özellikleri incelenmektedir. İri ve orta büyüklükteki kum örnekleri kullanılarak yapılan deneysel çalışmalarda; geçirimsizlik, konsolidasyon, üç eksenli basınç, ve serbest basınç testleri yapılmıştır. Deneysel sonuçlarda, ağırlıkça %0.1 ile %1.5 arasında değişen oranlarda Xanthan Gum kullanılmıştır. Geçirimsizlik testleri iri (1.18 mm- 2.00 mm) ve orta (0.075 mm- 1.00 mm) büyüklükteki dane çapına sahip kum örnekleri kullanılarak yapılırken, diğer testler orta büyüklükteki dane çapına sahip kum örnekleri kullanılarak yapılmıştır. Tüm deney sonuçlarının, Xanthan Gum miktarı, kür süresi, efektif gerilme gibi değişkenlerden etkilendiği tespit edilmiştir. Yapılan deneysel çalışmalar ve taranan literatür bilgileri ışığında, bir biyopolimer olan Xanthan Gum'ın bir zemin iyileştirme tekniği olarak düşünülebileceği sonucuna ulaşılmıştır.

Anahtar Kelimeler: Biyopolimer, kum, geçirimsizlik, konsolidasyon, üç eksenli basınç

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LIST OF SYMBOLS

e_{\max}	Maximum void ratio
e_{\min}	Minimum void ratio
R_D	Relative density
ϕ°	Internal friction angle
c	Cohesion
C_c	Coefficient of curvature
C_u	Coefficient of uniformity
G_s	Specific gravity
k	Coefficient of permeability
i	Hydraulic gradient
m_v	Coefficient of volume compressibility
ϵ_a	Axial strain
σ_c	Confining stress
σ_1	Major principal stress
σ_3	Minor principal stress

CHAPTER 1

INTRODUCTION

1.1. General background

Over the past years, many types of chemicals have been used for a geotechnical application. Chemical grouts are mostly toxic and hazardous for the environment. This constantly pushes researchers to find alternative eco-friendly techniques of ground improvement. It comes to light, that several natural biopolymers have important characteristics, such as excellent viscosifying in high-salinity waters, pseudo plasticity, stability at large ranges of temperature and pH, jellifying agents and resistance to shear degradation.

The use of biopolymers for soil improvement is already described in readily available literature. Several studies (Li et al., 1993; Martin et al., 1996; Stewart and Fogler, 2001) have investigated the application of biopolymers as plugging agents in construction of various impervious barriers. Some researchers (Khachatoorian et al., 2003; Momeni et al., 1999) demonstrated the capacity of certain biopolymers, including xanthan gum and sodium alginate to decrease the permeability and increase the shear strength, thus reduce the leaching of contaminants.

Biopolymers are polymeric mixtures produced by many living organisms (plants, microorganisms). They contain repeating functional groups such as hydroxyl, carboxyl, amino. That gives an ability to react easily go through some cross-linking process. Therefore, biopolymers have great opportunity for chemo-physical interaction with other compounds. They can bind soil particles, metals, or organic contaminants and create cross-linking networks with other polymers (Knox et al., 2010).

Moreover, the application of biopolymers as a plugging agent is well known. They can be easily injected into the soil under required pressure using some drilling equipment described in Chapter 2. In Yen et al. (1996) and Khachatoorian et al. (2004) works a biopolymer application for impervious barriers construction was discussed.

This technology allows site improvement using stabilizing agent without a need of removing the bulk soil. Those methods were found to be applied into shallow foundations, deep foundations and contaminated sites. For correct design of mixing and appropriate technique choice, the selection and assessment of engineering properties of stabilized soil are necessary (Keller Inc.).

Studies showed that the strength parameters of soil can be improved by either inclusion of slime-forming bacteria to the ground to produce a biopolymer inside it, or by direct application of biopolymer from slime-forming bacteria or other commercial products such as guar gum, agar, and sodium alginates to the soil structure. In the case of this research, only a direct application of biopolymer, namely xanthan gum will be considered.

The ground improvement process can be also accomplished by injection into soils materials such cement and lime in dry or wet forms. They will undergo a cementations process, which is introduced more precisely in the following chapter. The way to apply either dry or wet deep mixing methods depends on many other factors; the in-situ soil conditions, effectiveness of binders to be used, soil moisture contents, and the type of construction to be founded.

1.2. Scope of study (aims and objectives)

The purpose of this work is to investigate geotechnical behavior of a biopolymer treated sand by conducting several laboratory tests. This study shall improve the understanding of precisely how the biopolymers affect soil and their possible applications.

While most of the investigations focus on fine and cohesive soils, the present research analyzes the potential use of biopolymer, namely xanthan gum to reduce hydraulic conductivity of coarse and medium sandy soils, and to strengthen their engineering parameters. For that reasons hydraulic conductivity, consolidation,

triaxial, and unconfined compressive strength tests were carried out. Applied procedures and test results are explained in details. Some recent research findings on the biopolymer and microorganism inclusion into the soil were presented. The other available ground improvement techniques also are discussed.

1.3. Organization of thesis

The whole thesis works are divided into 6 chapters. The first one presents some basic information about the outline of conducted studies. Chapter 2 gives a background related with general ground improvement techniques and recent researches on biopolymer treated soils. Also other stabilizing agents are discussed. In Chapter 3 properties of materials used for testing and applied procedures are explained. The results are presented and debated in Chapter 4. Next one Chapter 5 gives conclusions drawn from the previous chapters. Some ideas and inspirations for future works related to biopolymers can be found in the last part, Chapter 6.

CHAPTER 2

LITERATURE REVIEW

2.1. Ground improvement

Site investigation and precise determination of its geotechnical parameters are very crucial before any construction project can take off. It is required to understand the soil characteristics upon which the decision on location of the project can be made. During the site selection, engineers must consider the following criteria (Makusa, 2012):

- ✓ Function of the structure
- ✓ Design load
- ✓ Type of the foundation
- ✓ Bearing capacity of the soil

In the past years, bearing capacity of the soil was a major factor on decision making, when it was poor, engineers usually change the design to be suitable for site conditions, or remove and replace the soil, or just abandon the site and choose a new location. Number of rejected site due to undesirable soil conditions dramatically growth, and the result of that was a deficiency of land and boosted demand for natural resources. Affected areas were those where liquefaction or a landslide could occur, and those filled with clay and organic soils. However, generally for a geotechnical projects, it is very hard, nearly impossible to find a construction site, where no ground modification will be necessary. Nowadays the trend is to improve engineering properties of problematic soils in order to meet the design specifications. Currently, weak soils, such as soft clays and organic soils can be upgraded to the civil engineering requirements.

As population on the world increases at every year a need of civil engineering structures continues to grow. New construction sites but also a renovation of already built structures on weak soils very often require some soil modifications. Worldwide, more than 40,000 soil improvement projects, worth US\$ 6 billion, is being done every year (DeJong et al., 2010). Ground improvement is usually carried out to:

- ✓ Increase bearing capacity, that is shear or frictional strength of the soil,
- ✓ Increase density,
- ✓ Decrease compressibility of foundation soil,
- ✓ Control deformations,
- ✓ Accelerate consolidation,
- ✓ Reduce the applied load on the foundation soil,
- ✓ Form seepage cutoffs or fill voids,
- ✓ Provide/increase lateral stability,
- ✓ Increase resistance to liquefaction,
- ✓ Transfer the load to a more competent (deeper) foundation soil.

Over the last 25 years many significant technologies and methods have been developed and introduced to assist the geotechnical engineers in providing an effective solution for construction on complex and weak soils. An example of possible foundation design is show in Figure 2.1.

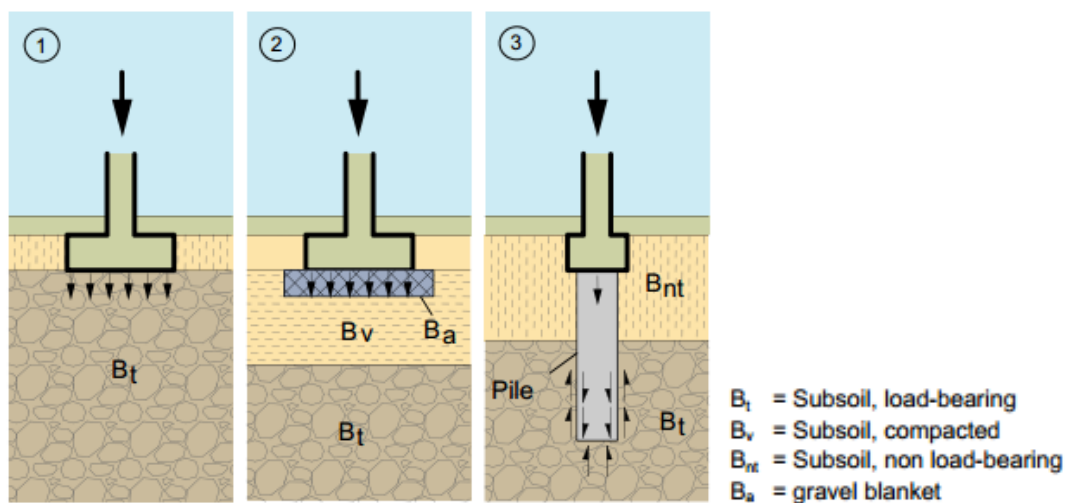


Figure 2.1. Different subsoil conditions and possible foundation methods (Bauer Group – Brochure, 2012)

The simplest stabilization processes are compaction and drainage. The other can be to improve the gradation of particle size further stabilization might be achieved by some binders inclusion into the soil (Rogers et al, 1996). There are many ground improvement techniques, they can be briefly divided into two categories (FM5-410, 2012):

✓ Mechanical stabilization

Ground improvement is accomplished by a physical process, through altering the physical nature of soil particles by induced vibration, compaction or by applying barriers and nailing into the soil.

✓ Chemical stabilization

Soil stabilization is achieved mainly by chemical reactions between a stabilizer (cementation materials) and the soil material (pozzolanic materials). Unbound material can be mixed with cement, lime, fly ash, bitumen, biopolymers or combination of those)

After a chemical stabilization soil should have a higher strength, lower permeability and lower compressibility (Keller brochure 32-01E). The decision about which technology to use, in each case must be based on that which parameter of the soil have to be modified. Available technologies are briefly presented in Table 2.1. Main ground improvement methods will be investigated in detail in the upcoming chapters.

Conducting a laboratory test together with a field examination in order to determine engineering properties might be crucial for a successful soil stabilization. It was observed that the laboratory tests may produce higher strength than the same material tested in the field, however it will help to judge the performance of stabilized materials in the field. What is more, results from the test conducted in the laboratory will enhance the recognition on the choice of binders and their amounts (EuroSoilStab, 2002).

Table 2.1. Ground improvement categories, functions and methods
(Ground Improvement Methods, 2006)

Category	Function	Method
Consolidation	Accelerate consolidation and increase shear strength	1.) Prefabricated Vertical Drains 2.) Surcharge
Load Reduction	Reduce load on foundation and reduce settlement	1.) Geofoam 2.) Foamed Concrete 3.) Lightweight fill
Densification	Increase density, bearing capacity, and frictional strength of granular soils. Decrease settlement and increase resistance to liquefaction	1.) Vibro-Compaction 2.) Dynamic Compaction by falling weight impact
Reinforcement	In soft foundation soils, increases shear strength, resistance to liquefaction, and decreases compressibility	1.) Stone Columns
Deep Soil Mixing	Physio-chemical alteration of foundation soils to increase their tensile, compressive, and shear strength; to decrease settlement; and/or provide lateral stability and/or confinement	1.) Wet mixing methods 2.) Dry mixing methods
Grouting	To form fill voids, increase density, increase tensile, and compressive strength	1.) Permeation Grouting 2.) Compaction Grouting 3.) Jet Grouting
Load Transfer	Transfer load to deeper bearing layer	1.) Column Supported Embankment (CSE)

2.2. Ground improvement techniques

2.2.1. Stabilizing agents

These chemical additives range from waste products to designed and manufactured materials, including lime, fly ash, cement, bitumen and proprietary chemical stabilizers. The additives can be used to improve native engineering properties of many various types of soil. The effectiveness of these stabilizing agents depends mostly on the soil being treated and the applied amount of additive. When a wrong kind or a wrong amount is placed, it may result in devastating the success of the project. For a proper implementation it is necessary to (CAT brochure, 2006):

- ✓ have a clear idea about the required result,
- ✓ understand the types of soil and their characteristics on site
- ✓ understand the usage of additives, how will they interact with the environment,
- ✓ chose a proper mixing technique,
- ✓ realize how the improved soil will behave.

Cement

Portland cement is a mixture of minerals, made up of oxides of calcium, silica, alumina and iron. Cement is considered as the oldest binding agent since the invention of soil stabilization technology. It can be treated as primary stabilizing agent or hydraulic binder, because cement used alone can bring the stabilizing required effect (Sherwood, 1993). Cement reaction does not depend on the soil minerals, and the key role plays here the reaction with water, that may be available in any soil (EuroSoilStab, 2002). This is why cement can be used to stabilize a wide range of sites.

Hydration is a process under which cementation takes place. The process starts when stabilization agent is blended with water and other components to use for a designed application, resulting into hardening phenomena. The hardening of cement will attach soil as glue, but it will not modify the entire structure of soil (EuroSoilStab, 2002). The hydration process and strength increases take place for the most part between 24 hours and 28 days, although the cement will continue to hydrate at decreasing rates as long as free moisture is present.

Roughly all types of soil may benefit from the strength gained by cementation. However, the most significant results are achieved when used with well-graded materials, which possess enough fine particles to produce floating aggregate matrix. Plenty types of cement are available in the market; these are ordinary Portland cement, blast furnace cement, sulfate resistant cement and high alumina cement. Generally, the selection of cement relies on the type of soil to be treated and the required final strength. Soils treated with cement have:

- ✓ decreased cohesiveness,
- ✓ increased strength,
- ✓ decreased volume expansion or compressibility.

Additional information and references about work with cement-stabilized soils can be found in Prusinski (1999).

Fly ash

Fly ash is produced by the combustion of coal in electric power generation facilities. The inorganic matter from coal that fuses during the combustion process, solidifies from the discharged gases and then is collected by special precipitators.

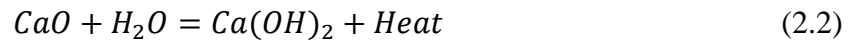
Most of the fly ashes are used as a secondary binders; these binders are not sufficient enough to produce the desired effect on their own. That fore, small amount of activator might be required to form cementitious compound that fulfill the strength improvement conditions.

With the water addition, fly ash hydrates and creates cementitious products similar to those of Portland cement. However, it has less bounding properties compared to lime and cement. Fly ash helps to reduce the plasticity index, swell and gives the soil additional strength. Because of fly ash begins to hydrate immediately after the addition of water, soil strength and density strongly depend on compaction time. Any delays in compaction may decrease those parameters dramatically. The rate of hydration for fly ash is more progressive than for Portland cement. When compaction becomes delayed, hydration products may begin to bond soil particles in a loose state and then disruption of these aggregations will be required to densify the material. More information about the fly ash stabilization can be found in Ferguson's (1993) and Nicholson's (1993) works.

Lime

Lime supplies an economical way of soil stabilization. It is produced by heating limestone at an elevated temperature and mostly used with highly plastic clays for subgrade improvement. In lime modification a strength increase is achieved by cation exchange capacity rather than cementation (Sherwood, 1993). In the clay stabilization process, agent performs two main functions: flocculation and cementation. The first one reduces the plasticity index of a soil, what improves the workability and lower the swell potential of the soil. Cementation is a slow process that occurs after the soil compaction, increasing its strength and durability. What is more, cementation also produces a working platform for construction process and significantly increases the durability of the groundwork (Little, 1995).

Nowadays there are two main types of lime use in construction, *quicklime* (calcium oxide) and *hydrated lime* (calcium hydroxide). Heated at high temperatures limestone produces a quicklime and the water addition to the quicklime produces a hydrated lime. It is explained in the following chemical equations (Milburn and Parsons, 2004):



The first one presents the reaction when limestone is heated to produce quicklime and a carbon dioxide as a by - product. The second equation shows that the water addition to quicklime results in hydrated lime together with heat as a by - product.

Lime stabilizations technology is being widely used in geotechnical and environmental engineering. Some of the applications include encapsulation of contaminants, rendering of backfill, slope stabilization, highway capping and foundation improvement (e.g. lime pile or lime stabilized soil columns) (Ingles and Metcalf, 1972).

However, presence of some organic materials or minerals like sulphur may hinder the lime stabilization process. For instance gypsum will react with lime and can swell, what will affect the strength of soil.

Bitumen

Bitumen is a viscoelastic material that occurs naturally or is produced in a petroleum distillation process. It is the pitch, mainly used to make asphalt. During the soil stabilization it does not lead to an increase in rigidity. Moreover, no cracking occurs through the operational time of the produced layer of the improved soil (Murgala, 2011). Such a stabilized soil layer can be used as a supporting base of bitumen type.

Soil stabilization methods apply liquid bitumen produced with solvents. Important fact is also the price, this kind of bitumen is very expensive. Furthermore, it may have a negative impact on soil and ground water as a result of solvent penetration. Asphalt cement, tar, cut back asphalt, and asphalt emulsions are used for bituminous soil stabilization. Construction method, soil type and weather are all the factors in choosing which bitumen to apply. This stabilization agent makes soil stronger and resistant to water and frost (CAT brochure, 2006).

2.2.2. Deep soil mixing

Deep soil mixing (DSM) is a ground improvement method that mixes binders into the soil at a specific depth to strengthen the in-situ soil properties without its excavation or removal. In this technique wet or dry agent is injected into the soil and then blended by mechanical or rotary mixing tool (Porbaha et al, 2005; EuroSoilStab, 2002). The system is most applicable in soft soils. Cohesionless soils are easier to mix than cohesive soils. Depending on design, various patterns may be produced: single patterns, block patterns, panel pattern or stabilized grid pattern (Figure 2.2.). The aim DSM is to produce a stabilized soil mass which may continuously interact with natural soil and not to produce a too stiff stabilized soil mass like a rigid pile (column) which could independently carry out the design load.

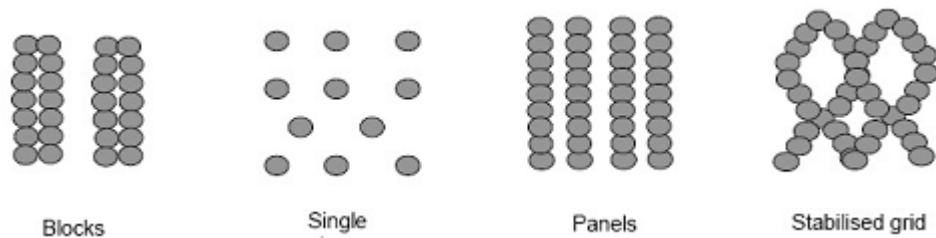


Figure 2.2. Typical patterns of Deep Soil Mixing (Keller brochure, 2008).

Deep soil mixing can be used for many various applications including excavation support, settlement reduction, soil stabilization, foundation support, and reduction of liquefaction potential. A deep soil mixing process is shown in Figure 2.3.

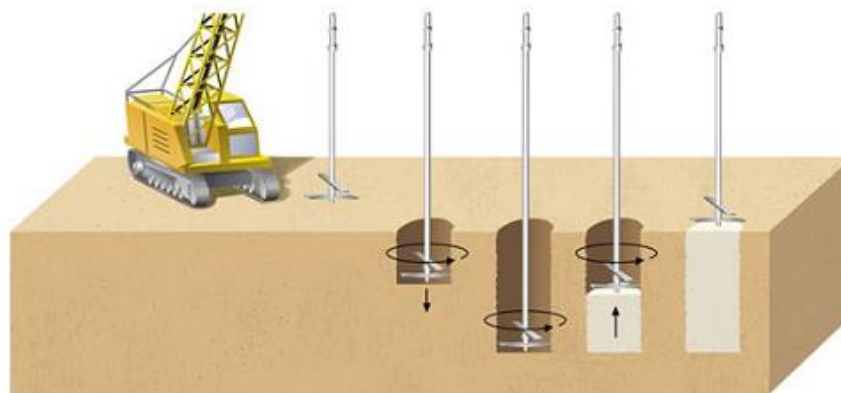


Figure 2.3. Schematic showing overall process of DSM technique (Keller brochure, 2008).

For wet soil mixing, the stabilizing agent is delivered in a slurry form. Slurry volumes usually range from 20 to 40% of the soil volume that is mixed. The most commonly applied are Portland cement, fly ash, ground blast furnace slag, and other additives. For dry soil mixing, exactly the same materials are pumped dry by using of compressed air. Before any application can take place a laboratory testing is used to determine proper mix energy and grout proportions (Ground Improvement Methods, 2006).

Wet DSM techniques are typically used for large-scale structural support improvement. As this method is quite expensive, requires mobilization of many devices at the same time and accompanying auxiliary batch plants, it is uneconomical for small projects. Wet deep soil mixing has been used to stabilize soil to provide an improved foundation bearing capacity and for seismic stabilization. Main usage is for settlement control and shear strength improvement under embankments.

Dry DSM techniques are used primarily for soil reinforcement and settlement reduction. Lime, cement, or lime-cement columns have been applied to improve soft, cohesive soils. They were used to reduce total and differential settlements (similar to stone columns). Lime-cement columns are stiffer and less compressible than untreated surrounding soil; therefore, can carry a greater portion of the applied stress and reduce total and differential settlement.

2.2.3. Jet grouting

Grouting contains many techniques that employ injection of a stabilizing agent into soil or rock formations via boreholes, in order to improve its physical characteristics, enhance strength, density, permeability, and homogeneity. Jet grouting uses high pressure, high velocity erosive jets of water and grout to remove some of the soil and replace it with cement based grout. The mixture of soil and grout is called soilcrete. Jet grouting is applied in various soils, from clays to gravels, however with different degrees of effectiveness. Jet grouting might be used in following cases (Ground Improvement Methods, 2006):

- ✓ Water Control,
- ✓ Underpinning,

- ✓ Scour Protection,
- ✓ Settlement Control,
- ✓ Excavation Support,
- ✓ Liquefaction Mitigation,
- ✓ Treatment of Karst.

Jet grouting gives a chance to control the shape (usually a circular column), size and properties of treated soil. Figure 2.4. provides a schematic view on a jet grouting process.

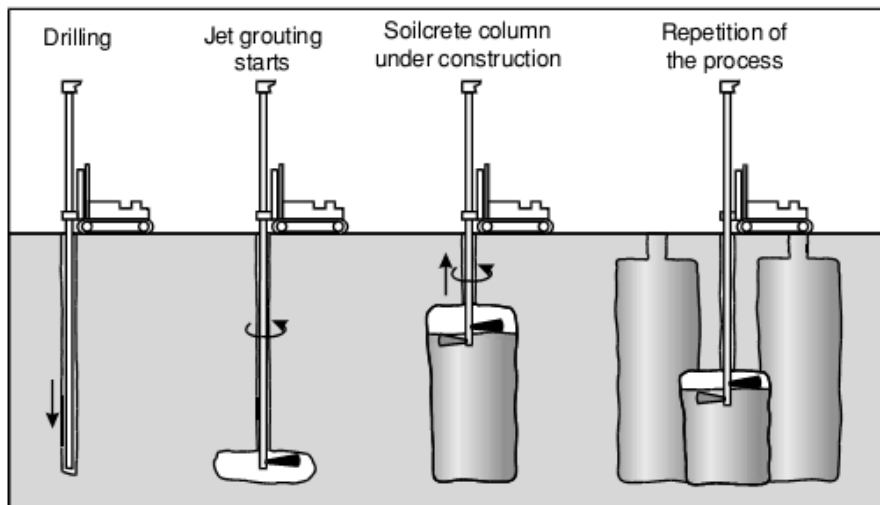


Figure 2.4. Jet grouting procedure (Hayward Baker Inc.).

There are three traditional jet grout systems (Figure 2.5.). Selection of the most decent system is determined by the in situ soil, chosen application, and the needed strength of the soilcrete. There are (The foundation engineering handbook, 2006):

- ✓ The single-fluid system that uses only a high-velocity slurry grout to erode and mix the soil;
- ✓ The double-fluid system that surrounds the high-velocity slurry jet with an air jet. Due to the air jet erosion in the soil is significantly increased;
- ✓ The triple-fluid system that uses a high-velocity water jet with attached an air jet to erode the soil. A lower jet injects the slurry at a reduced pressure.

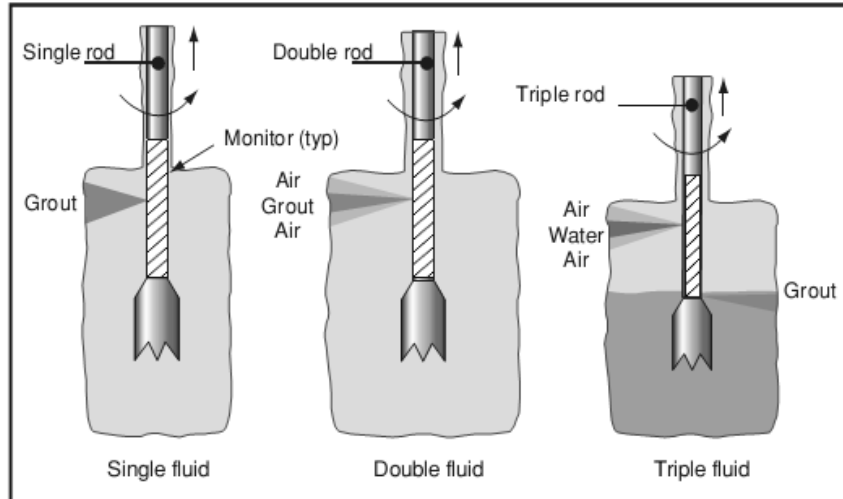


Figure 2.5. Jet grout systems (Hayward Baker Inc.).

2.2.4. Dynamic compaction

Dynamic compaction (DC) known as dynamic deep compaction, was introduced in the 1960s by Luis Menard, however there are some reports of performing dynamic compaction over 1000 years ago. The process relies on dropping a heavy weight on the ground surface to compact soils to depths as great as 12.5 m. The weight can be dropped even from 30 m. Compaction scheme is presented in Figure 2.6.

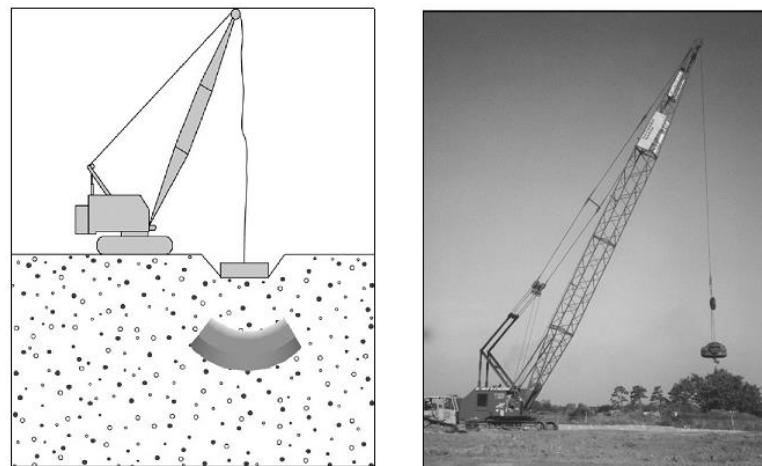


Figure 2.6. Dynamic compaction: schematic and field implementation (Hayward Baker Inc.).

The technique is applied to reduce foundation settlements, seismic subsidence, and liquefaction potential, densify garbage dumps, permit construction on fills, and improve mine spoils. Dynamic compaction method is most effective in highly permeable, granular soils. Cohesive soils absorb the energy and may limit the

technique's effectiveness. In soft organic soils, dynamic compactions have been used for constructing sand or stone columns by repeatedly filling the required material. The craters are typically filled with a clean and free draining granular soil (The foundation engineering handbook, 2006).

2.2.5. Vibro-compaction

Vibro compaction (VC), known also as Vibroflotation was introduced in the 1930s in Europe. The process requires the usage of a down-hole vibrator, which is lowered into the ground in order to compact the soil at appropriate depth (Figure 2.7.). This technique is applied to increase bearing capacity, reduce foundation settlements, and liquefaction potential, and densify loose granular fills. The VC method is more effective in freely draining granular soils (The foundation engineering handbook, 2006).

The vibrator is firstly lowered into the ground, and then the compaction starts at the bottom of the soil. The vibrator might be either raised at a certain rate or can be repeatedly raised and lowered while extraction. Soils surrounding the hole are rearranged into a denser state; relative densities may reach of 70 to 85%. The deepest treatment was performed at 37 m. (The foundation engineering handbook, 2006).

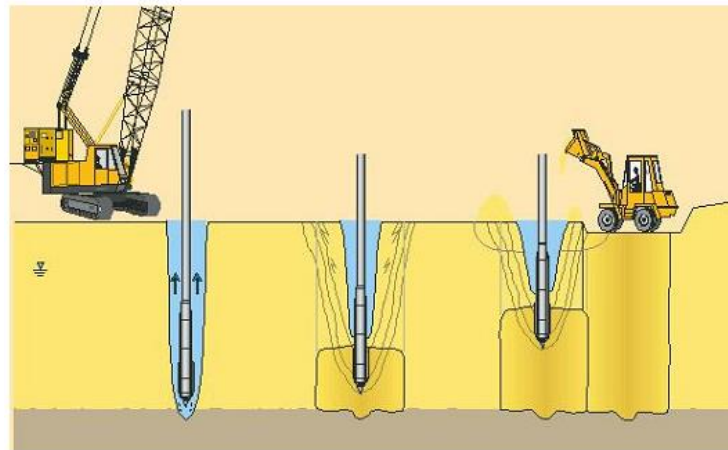


Figure 2.7. Vibro Compaction process (Keller brochure, 2008).

The improvement effectiveness highly depends on the energy of vibrator used, the spacing of vibrator penetrations, the amount of time spent for compaction, and the quantity of backfill added (The foundation engineering handbook, 2006).

2.2.6. Stone columns

Stone columns are constructed by vibration methods, similar to techniques used in the vibro compaction. The difference is that instead of using coarse materials to just fill the hole created by the vibro compaction, stone (or other materials) is placed to form a column (Ground Improvement Methods, 2006). Stone columns might be constructed using either vibro-replacement or vibro-displacement (Table 2.2.).

Table 2.2. Vibro-replacement and vibro-displacement.

Vibro-replacement	Refers to the wet, top feed process in which jetting water is used to aid the penetration of the ground by the vibrator. Due to the jetting action, part of the in-situ soil is washed to the
Vibro-displacement	Refers to the dry, top or bottom feed process; almost no in-situ soil appears at the surface, but is displaced by the backfill material.

Since stone columns take their strength and settlement characteristics mainly from the surrounding soil, they are not sufficient in very soft clays with a thickness greater than the diameter of column. Specially to treat such a soils, vibro-concrete columns were developed. Instead of putting stone to the tip of the vibrator, concrete is pumped to the bottom of the hole. This technique can offer ground improvement that will have similar characteristics with the deep foundations. A schematic column construction is shown in Figure 2.8.

Stone columns care mostly used to improve the stability of slopes, reduce total and differential settlements, increase bearing capacity, and decrease the time for settlements to occur.

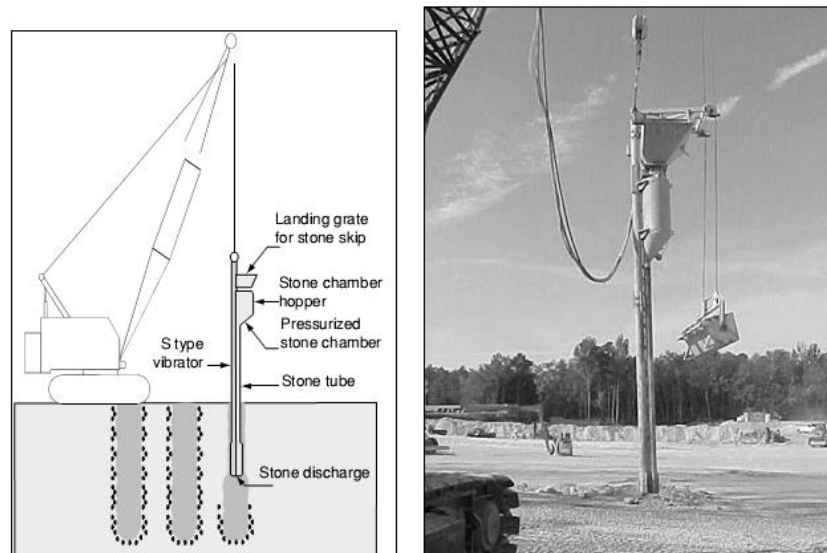


Figure 2.8. Stone column construction: schematic and field implementation (Hayward Baker Inc.).

2.2.7. Soil nailing

Soil nailing is another in situ method for reinforcing, stabilizing, and retaining deep excavations. Relatively small and closely spaced steel bars are placed into the soil mass, to make it locally stabilized. Process is shown in Figure 2.9. This technique has been applied for almost 40 years in Europe and it is more new in the United States. Soil nailing is mainly done for temporary or permanent excavation support, retaining walls, stabilization of slopes, stabilization of tunnel portals and repairing retaining walls.

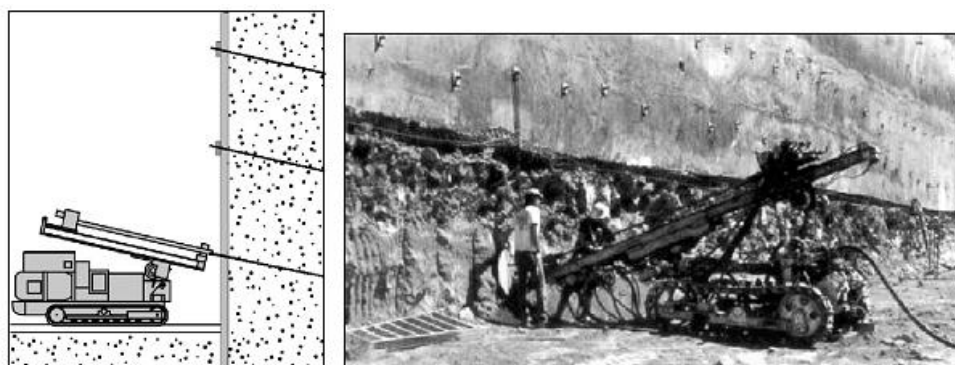


Figure 2.9. Soil nailing: schematic and field implementation. (Hayward Baker Inc.).

It is required for the soil to temporarily stand in a vertical face until a row of nails is installed. That is why, cohesive soils or rock are best suited for this method. Soil nails cannot be easily applied in cohesionless granular soils.

Soil nails technique is designed to give a soil mass an evident cohesion by transferring of resisting tensile forces generated by the steel bars into the ground. Friction between the soil and the steel inclusions holds back the ground movement. The biggest concern for engineers is to be sure that the ground-steel bars interaction is effectively mobilized to control ground displacements and secure the stability with an appropriate safety factor.

2.2.8. Geosynthetics

Geosynthetics are man-made materials used for soil reinforcement. The word is derived from: Geo = earth or soil and Synthetics = man-made. They are usually made from petrochemical-based polymers, which will not decompose from fungal or bacterial action. Some characteristics of those stabilization materials are presented in Figure 2.10.

Geosynthetic materials are used in ground improvement in order to:

- ✓ separate and distribute the loads:
 - improve level-grade soil situations (roads, lane ways, alleys)
 - improve sloped-grade situations (banks, hillsides, stream access points),
- ✓ reinforce soil (soil walls, bridge abutments and soil arches),
- ✓ prevent soil movement – piping (in drainage systems and back fill around water intakes),
- ✓ control water pressure (on foundation walls to allow water to move down to perimeter drains).

There are many different geosynthetic materials; however, they can be grouped in the following categories:

- ✓ geotextiles – used for drainage, reinforcement and separation,
- ✓ geogrids – open mesh-like materials used for stabilization and reinforcement,
- ✓ geocells – cavity-like materials in a web used for stabilization,
- ✓ geomembranes – very low permeability liner or fluid containment materials,
- ✓ erosion control – materials that are biodegradable or non-biodegradable.

The main parameters used in design, are the tensile strength and stiffness of the geosynthetic material, and the soil–geosynthetic interface shear and bond resistance (Brown, 2006).

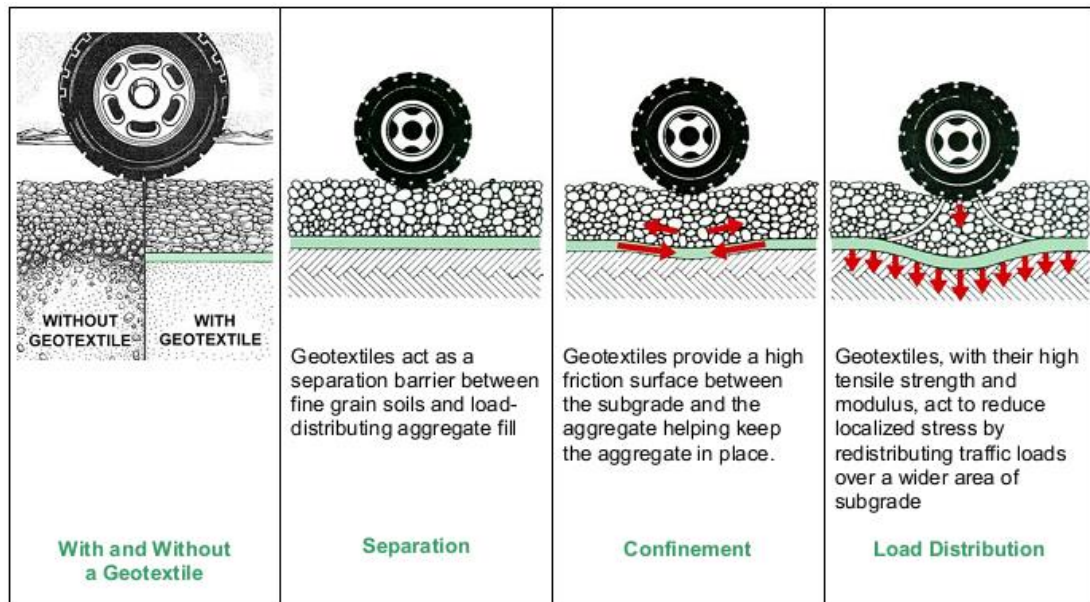


Figure 2.10. The Main Geotextile Characteristics (Brown, 2006).

2.3. Biopolymer applications

2.3.1. General background

One of the recent developments in geotechnical engineering is the application of biotechnology to soil improvement. Natural microorganisms and bacteria can be used to produce a biocement for improving the mechanical properties of soil by biocementation, bioclogging or even biogas. Biocementation is a process where biocement is applied to improve the shear strength of soil, while bioclogging means reduction of its permeability. Biogas technique is a way of using gas bubbles created by microbial activities to modify and reduce the degree of soil saturation. Application of natural bacteria products into ground improvement methods has brought to life a concept of biogeocivil engineering (Jonkers and Loosdrecht 2010).

Use of biopolymers gives a chance to reduce the generation of hazardous substances currently used for ground improvement design, what could increase in public acceptance of the soil treatment. This delivers a product for multiple specific uses, and a long-lived, but ultimately biodegradable, material without the environmental

concerns (Cabaniss et al., 2005, Goto et al., 2001, Decho 2010). Biopolymers have some important characteristics, such as supreme viscosifying power, high resistance to shear degradation, kind of pseudoplasticity, stability at various ranges of temperature and pH. Thanks to their characteristics chemical structure, biopolymers may have many very useful applications. (Khachatoorian et al., 2003).

When single soil particles are linked together by the biopolymer matrix, their mobility is limited, therefore following processes occur (Griggs, 2010):

- ✓ erosion is control,
- ✓ reduction of suspended solids,
- ✓ reduction in heavy metal transport,
- ✓ increase in strength of the soil surface layer.

Biopolymers might be also act as soil fixation agents for excavation control. Moreover, independently from plugging effect, they can bind soil particles, metals and other biopolymers, due to the ability of creating cross-linking networks. Therefore, the biopolymer application into the soils may result in the creation of impervious barriers. The concept of bacteria usage in creating a biobarriers in highly permeable soils like sands in order to stop or at least reduce contaminant plumes migration has recently gained more attention (Mitchell, 1997). Such a barriers are formed by reducing the permeability characteristics of the soil due to the pore clogging process. Bacterial cells over the time develop forms of biofilms, decreasing the void space or filling the voids, thanks to its viscous properties (Bouazza et al, 2001). Heavy metal transport in the ground is a concern for both water and air erosion of soil and needs special treatment to be controlled. Presence of these contaminants in soil creates a number of environmental worries about the water (Larson et al., 2012). There are many polymer additives commercially available, that can easily stabilize heavy metals in soils include (Weston et al. 2009).

Cross-linking is a chemical process of connection two or more molecules by a covalent or ionic bind using a linking agent, those cross-linkers should contain minimum two reactive groups. Functional groups of that can be targeted for coupling include primary amines, carbohydrates, sulfhydryls, carbonyls, and carboxylic acids (most of these are present in biopolymers). Selected cross-linked biopolymers are resistant to biodegradation and have the potential for healing (repair) applications.

Due to the cross-linking process molecular mass of the biopolymer increases, many bonds between biopolymer chains are created, therefore any break in chain is less or even non-effective on its stability. For enhancing the strength of polymers and decreasing their biodegradability cross-linking agents are applied, they are chosen up to the functional groups of biopolymers. The network, developed by this process can slow or even stop the migration of contaminants, mostly because of increased viscosity, reduced hydraulic conductivity and greater stability (Knox et al., 2010).

Studies showed that the strength parameters of soil can be improved by either inclusion of slime-forming bacteria to the ground to produce a biopolymer inside it, or by direct application of biopolymer from slime-forming bacteria or other commercial products such as guar gum, agar, and sodium alginates to the soil structure. In the case of this research, a direct application of biopolymer, namely xanthan gum will be discussed.

There are two common ways to prepare biopolymer – soil mixtures. The first technique relies on mixing a dry biopolymer with water, producing a viscous gel, which is going to be applied to the soil surface. In the second method dry biopolymer is mixed into the soil, and then water is being added. By using any of those application methods, when soil gets wetted, biopolymer will create a linking within the soil matrix. (www.silverson.com)

2.3.2. Material – Xanthan gum

Biopolymers are produced by complex biosynthesis process in bacteria and algae. A scheme showing biopolymer production process is presented in Figure 2.12. In the work of Mitchell and Santamarina (2005) a description of common microbial constituents and processes taking place in soils, with a focus on those biopolymers that could be considered for ground improvement.

Plenty of different polymer-based products for soil strengthening are already available; such as polyhydroxybutyrate (PHB), guar gum, xanthan gum, agar, polyglutamic acid (PGA), and chitosan (Tingle et al. 2007). Those gain popularity thanks to the ease of handling, competitive price and greater eco-friendliness compared to other traditional soil stabilization agents such as asphalt, lime, and cement.

Many soil-stabilizing emulsions are copolymers of ethylene or vinyl acetate or are acrylic copolymers. When soil is mixed with a synthetic polymers its strengths may get similar to that mixed with cement. However, the polymer treated soil will have more flexibility than that mixed with cement and also its toughness will increase. Therefore such a soil will have an increased resistance to cracking due to a higher failure strain before yield (Newman et al., 2005).

Xanthan gum is a polysaccharide that is widely used for thickening and stabilizing emulsions and suspensions. Very often is combined with other rheology modifiers, particularly guar gum to give greatly increased effects. Xanthan gum is not easily degraded by microorganisms (Cadmus et al., 1982). Biopolimer – sand mixture is presented below in Figure 2.11.



Figure 2.11. Biopolymer – sand viscous mixture (Knox et al., 2010).

Xanthan gum might be dispersed into hot or cold systems; many grades are available, including some specially designed for easy dispersion. Biopolymer powders have a strong weakness to form lumps when added to the water, therefore numbers of dispersion and hydration methods were developed to overcome this inconvenience. Some of the techniques are (www.silverson.com):

- ✓ Slow addition of the powder into the vortex in an agitated vessel. Once dispersed mixing continues to allow the product to hydrate;
- ✓ Xanthan gum maybe premixed with other powdered ingredients such as sugars which reduces the formation of agglomerates by separating the particles;

- ✓ Similarly the gum may be dispersed into non-aqueous phase liquids such as oils, alcohols or glycols. This is then added to the aqueous phase allowing the gum to hydrate.

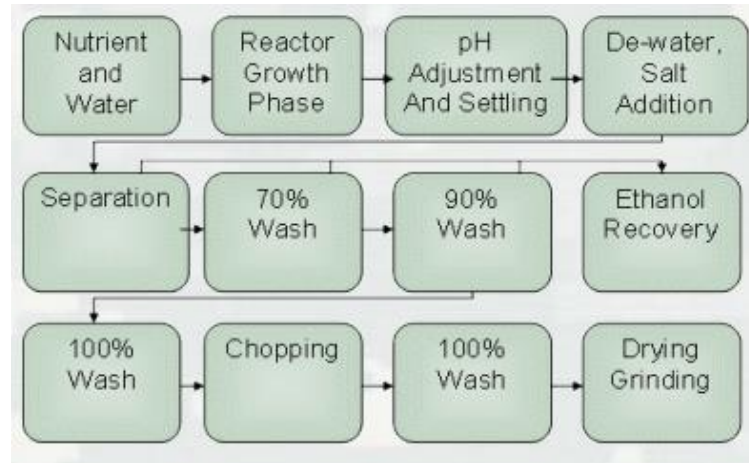


Figure 2.12. Schematic of biopolymer production process (Griggs, 2010).

2.3.3. Research overview

The use of biopolymers for ground improvement is widely referred in the literature. Several works (Li et al., 1993; Yen et al., 1966; Martin et al., 1996; Stewart and Fogler, 2001) took into account biopolymer applications and producing some microorganisms in the soil as a plugging agents used for construction of impervious barriers.

Furthermore, Cabalar et al. (2009) claimed that biopolymers inclusion improved the shear strength of sand and DeJong et al. (2010) wrote that soil stiffness, compressibility, hydraulic conductivity and volumetric response could be arbitrated by means of biological processes. Also, Bouazza et al. (2009) used guar gum, sodium alginate and xanthan gum for inclusion up to 2% (by dry weight) into a silty sand and found that biopolymers may significantly lower the permeability values.

A new method to use biopolymers for reducing soil permeability has received more attention in recent years. Ivanov and Chu (2008) went through many studies where a relatively cheap and ecofriendly biopolymers were adapted to fill pores in granular media and therefore reduce hydraulic conductivity and strengthen the material through cementation. Possible biopolymer applications for reducing permeability were also

studied by Blauw et al. (2009), while Thullner and Baveye (2008) investigated the problem by some numerical modeling approaches.

Other researchers (Yang et al., 1993, 1994; Momeni et al., 1999; Karimi, 1997) also presented valuable information on biopolymer inclusion into the soil matrix; they proved that microorganisms improved the soil structure by increasing the shear strength and decreasing the permeability, thus reducing the contaminants leaching.

Khachatoorian et al. (2003) studied biopolymers inclusion and their results have shown mild to good plugging effects, what gave a permeability reduction in sandy soil, during the 11-day experimental period. This was because of development of a large flow resistance, gelation that occurred over a short length of the sand-pack column. The best plugging effects were obtained for PHB (polyhydroxybutyrate), when permeability ratio reduction factor reached 10^{-14} over 11 days of duration, followed by chitosan and PGA (polyglutamic acid) with a reduction factor of 10^{-7} . Results are shown in Figure 2.13. As predicted, hydraulic conductivity for all studied biopolymers was much lower than for clean distilled water, indicating that any of these biopolymers could be efficiently used in field applications of soil remediation.

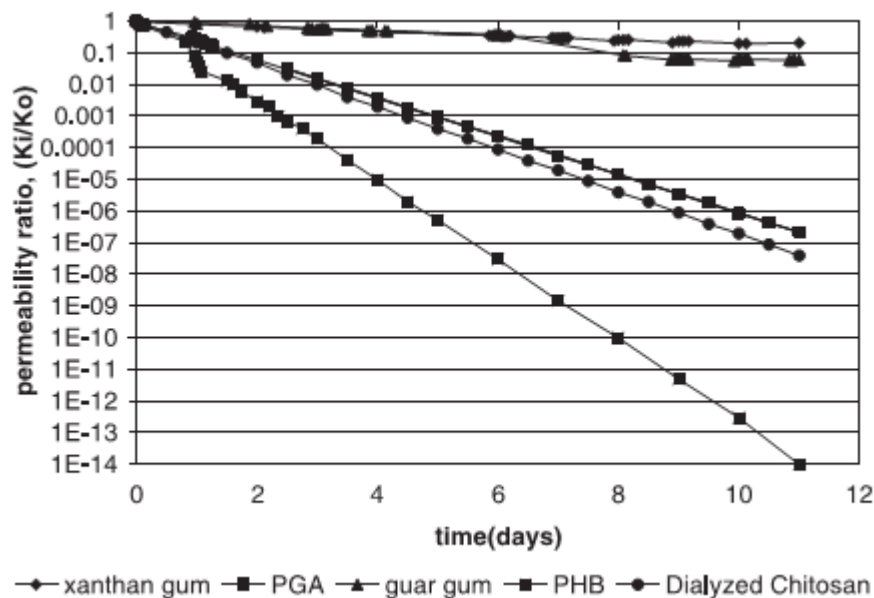


Figure 2.13. Permeability ratio versus time for xanthan, PHB, PGA, guar gum and dialyzed chitosan solutions (Khachatoorian et al. 2003).

Bouazza et al. (2001) has summarized the results of several studies and showed a clear decrease in the hydraulic conductivity for several different porous media, while several different types of treatment were applied. The results in Table 2.3. show that low hydraulic conductivities ($<10^{-9}$ m/s) could be achieved with clayey silts and sands.

The results in Figure 2.14. show the relative effectiveness of three different types of biopolymers - xanthan gum, sodium alginate, and guar gum. From the studies it was conducted that xanthan gum inclusion was more effective than sodium alginate, and guar gum addition was found to be the least efficient. In the study 2 % of biopolymer was required to reach the target hydraulic conductivity of 10^{-9} m/s.

Table 2.3. Summary of hydraulic conductivity results (Bouazza et al., 2001).

Matrix	Hydraulic Conductivity (m/s)		Material added
	Initial	Final	
Berea sandstone	1.3×10^{-6}	5×10^{-8}	<i>Bacillus subtilis</i> (dead)
Sandstone cores	3×10^{-6}	9×10^{-7}	<i>Klebsiella pneumoniae</i> UMB
Sandstone cores	3×10^{-6}	4×10^{-8}	<i>Klebsiella pneumoniae</i> UMB (resuscitated)
Clayey silt	5×10^{-8}	8×10^{-10}	Xanthan Gum (1%) at 30% moisture content
Clayey silt	10^{-6}	1×10^{-8}	Xanthan Gum (1%) at 13.7% moisture content
Clayey silt	10^{-8}	5×10^{-10}	Xanthan Gum (1%) at 20% moisture content
Clayey sand	10^{-6}	3×10^{-10}	Guar Gum (2%)
Clayey sand	10^{-6}	2×10^{-10}	Sodium alginate (1%)
Clayey sand	10^{-6}	1×10^{-10}	IDPAC (1%)
Clayey sand	10^{-6}	3×10^{-11}	Xanthan Gum (1%)
Silty sand	10^{-7} to 10^{-8}	1×10^{-10}	<i>Beijerinckia indica</i>

The hydraulic conductivity reduction was shown to be mainly dependent on the initial porosity of the material, the clogging mechanism, the concentration and kind of a biosubstance that was applied, the compaction moisture content, and the curing time of the samples (Bouazza et al., 2001). Results can be seen in Figure 2.14.

Generally, a significant reduction in permeability values has been observed for a lower initial hydraulic conductivity of the sand, an increase in compaction water content, and a longer curing time.

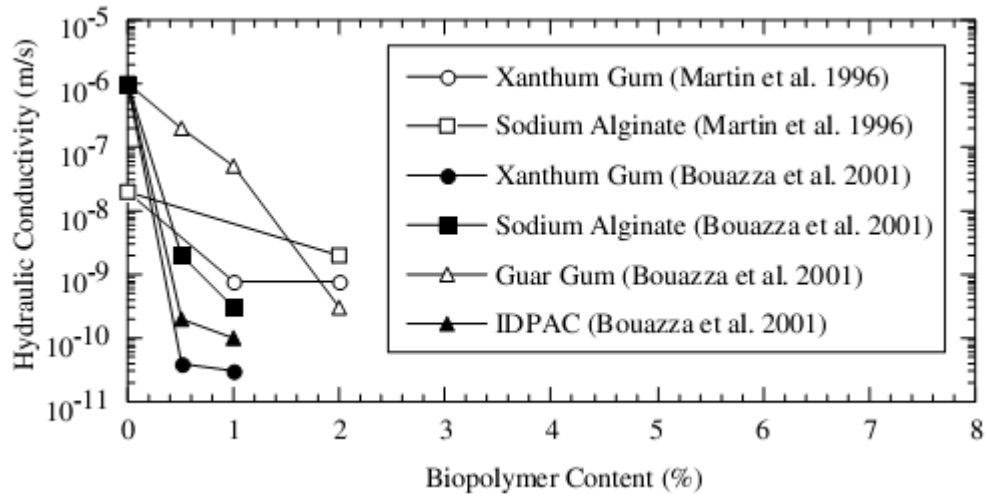


Figure 2.14. Hydraulic conductivity of soil – biopolymer mixtures (Bouazza et al., 2001).

Khatami and O'Kelly (2013) investigated some mechanical properties of sand treated with agar and starch biopolymers. According to the biopolymer concentration, the unconfined compressive strength of sand treated biopolymers ranged from 158 to 487 kPa. Triaxial compression tests with various confining pressures also determined that the biopolymers effectively increased cohesion and stiffness of the treated sand.

A series of triaxial compression tests were conducted on the dry sand and sand treated with solutions containing 1%, 2% and 4% of agar (by weight). The results data showed that higher agar inclusions gave higher compressive strength and stiffness for the biopolymer – sand mixtures (Figure 2.15.). For the maximum deviatoric stress values the same correlation was observed. Internal friction angle and cohesion values for different ratios are presented in Table 2.4.

The improvement in characteristics of sand treated with agar and modified starch (biopolymers) have been found to be directly dependent on the amount of agar as the main component and starch as the additive.

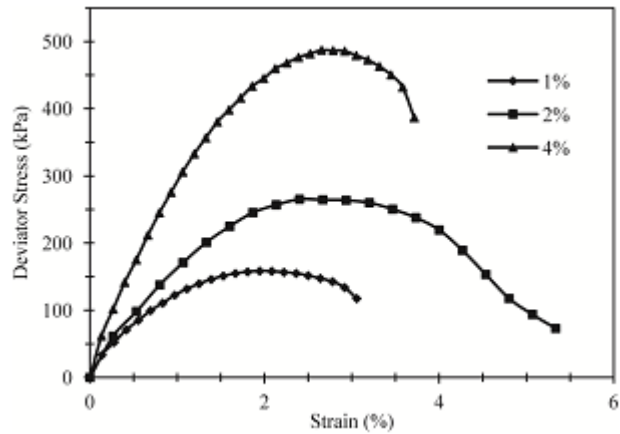


Figure 2.15. Unconfined compression tests of sand treated with agar
(Khatami and O'Kelly (2013))

Table 2.4. Mohr-Coulomb parameters for sand treated with biopolymers
(Khatami and O'Kelly (2013))

Method \ Conditions	Non-linear optimization		Linear least squares	
	c (kPa)		c (kPa)	
Untreated sand	0	32.3	0	33.1
1% agar	62	24.7	49	25.4
2% agar	111	25.6	105	26.4
4% agar	190	26.3	222	23.6
1% agar and 0.5% Starnol 600	187	17.5	187	17.5
1% agar and 1% Starnol 136	240	17.6	245	17.4

In another work, Griggs et al., (2010) studied a possible application of biopolymers in dispersive soils; he examined soils in the matter of piping erosion and slope stability. The appearance of surface runoff water from soil treated at various rates of biopolymer is presented in Figure 2.16. Higher loading rates demonstrate decreased suspended solids and turbidity.

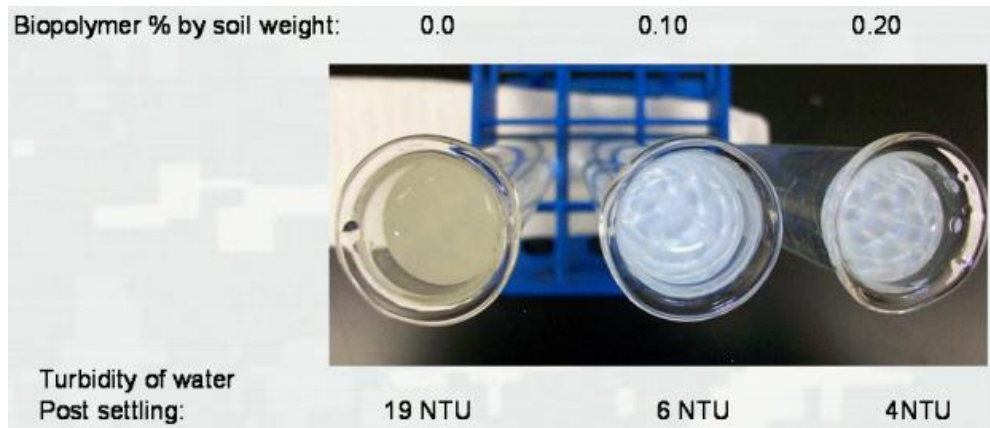


Figure 2.16. Suspended solids and turbidity (Griggs et al., 2010).

The treatability study on the biopolymer application in order to improve slope stability and reduce loss of sediment in surface water runoff indicated that the biopolymer inclusion:

- ✓ effectively maintained the slope stability of a simulated berm,
- ✓ reduced significantly transport of soil particulates in surface runoff,
- ✓ performed effectively in soils (clays) with a high concentration of fines.

This study of sediment transport showed that the biopolymer application into the soil was able to significantly reduce surface water erosion and particulate transport in leachate, what is presented in Figure 2.17.

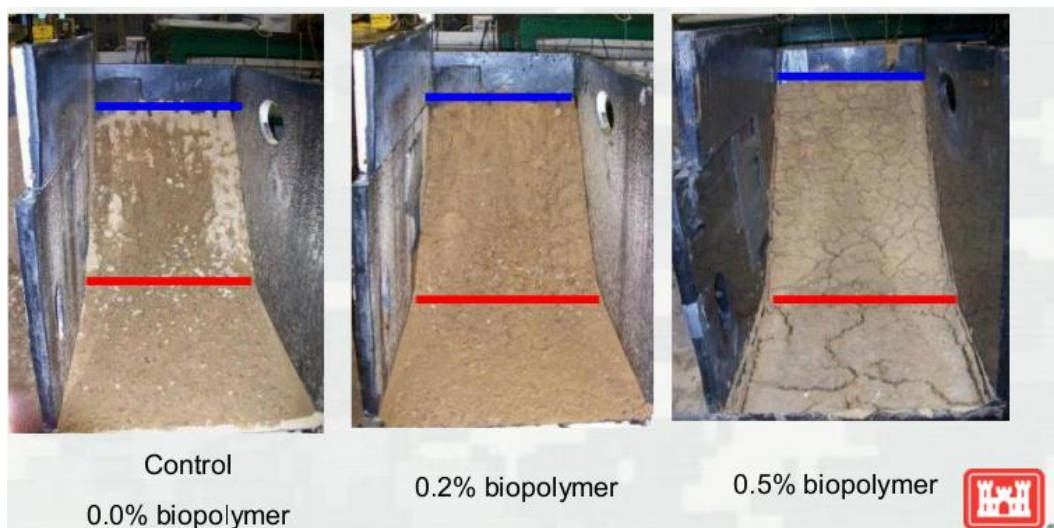


Figure 2.17. Erosion control and slope stability (Griggs et al., 2010).

In a research done by Larson et al., (2012) heavy metal transport was studied by using the mesoscale rainfall lysimeters for a static system, and for dynamic, larger,

live-fire lysimeters. The rain simulations were conducted at weekly intervals. The static lysimeters were running for 12 weeks, what simulates 1 year of weathering. However the LFL controls were running for 20 rain events, 1.7 years of weathering. Different biopolymer concentration were prepared and tested, each at least for a minimum 4 rain events.

The total mass of Pb (particulate and dissolved forms) that was detected in the leachate and runoff water from control (clean, untreated) and biopolymer-soil mix after 16 weeks (what gives 1.25 years of simulated weathering) is shown in Figure 2.18. for the static lysimeter system. The data showed a 97% reduction in total Pb in the runoff water from biopolymer-soil mixture, and a 94% reduction in leachate from the same soil. Biopolymer added to the soil of the lysimeters was successful at decreasing the concentration of lead, particularly in surface water runoff.

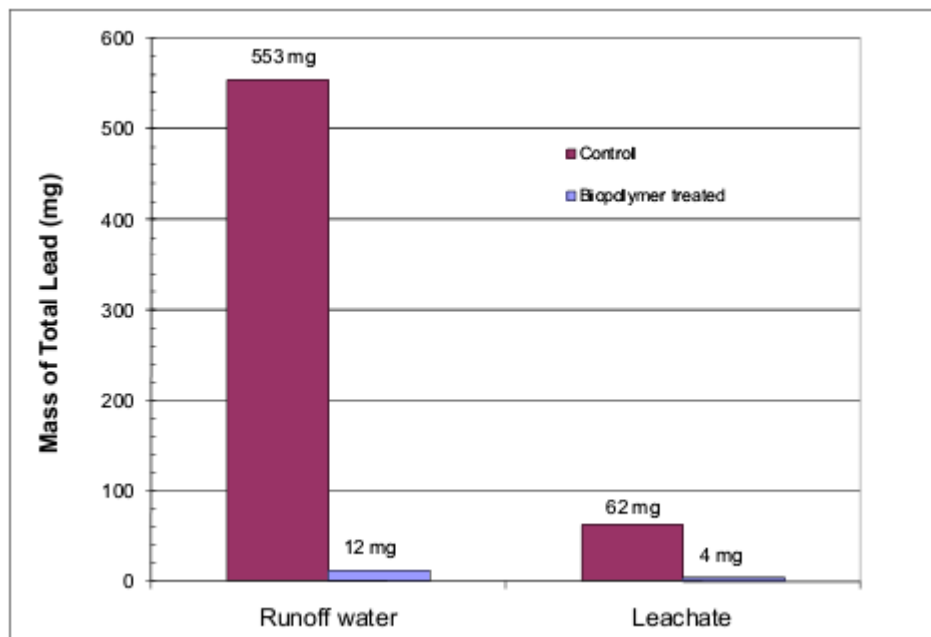


Figure 2.18. Heavy metals in biopolymer treated soils (Larson et al., 2012).

CHAPTER 3

EXPERIMENTAL STUDY

3.1. Materials and their properties

3.1.1. Xanthan Gum

The commercially available biopolymer material was obtained from a local food store in a powder form. Worldwide production of xanthan gum comes out from the bacteria *Xanthomonas campestris*. Xanthan gum is as a hydrophilic colloid to thicken and stabilize water-based suspensions (Figure 3.1.). It is widely used in the drilling industry to thicken drilling fluids, and is very stable under various temperatures and pH (Bouazza *et al.*, 2009).

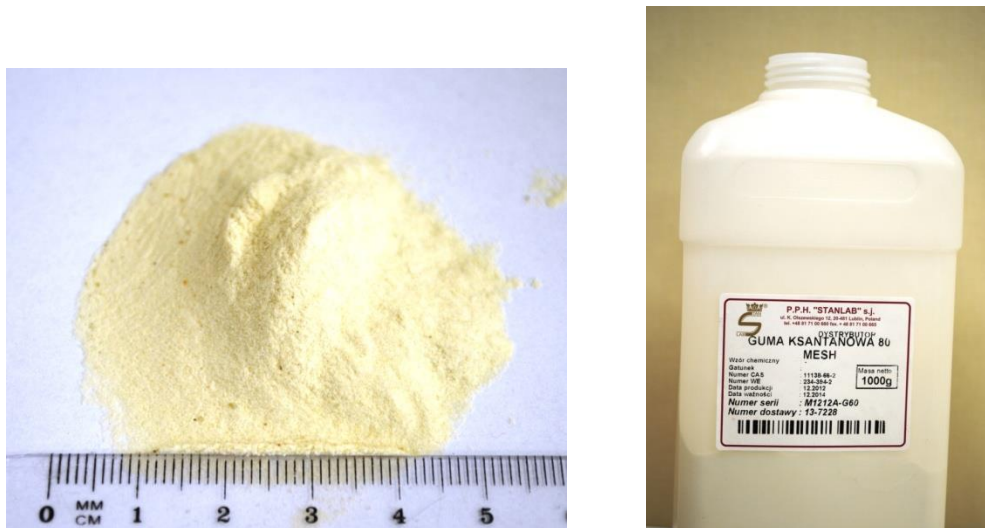


Figure 3.1. Xanthan gum.

3.1.2. Coarse Narli Sand

The Narli River Sand was obtained from the southern-central of Turkey, near Gaziantep. The specific gravity of the grains was found to be 2.68. A gradation of

the sand falling between 1.18 mm and 2.00 mm was artificially selected to provide uniform specimens for visual classification purposes.

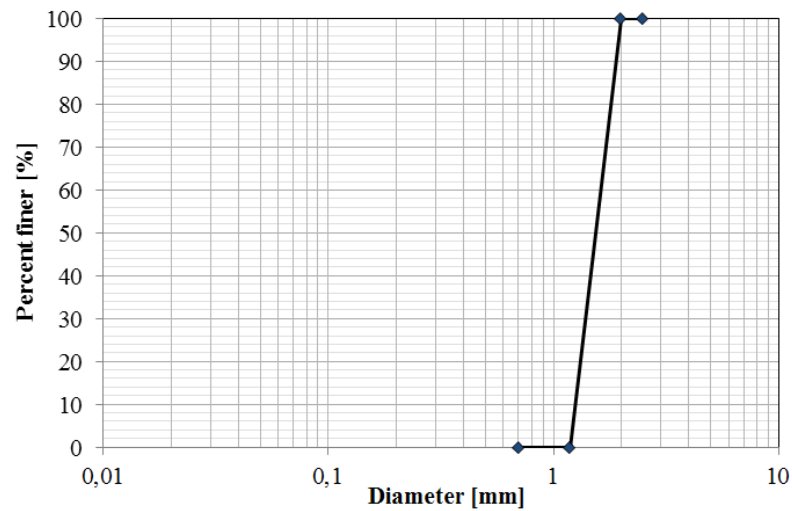


Figure 3.2. Particle size distribution curve for Narli sand.

Table 3.1. Index properties of Narli sand.

Property	Value
Minimum void ratio, e_{min}	0,60
Maximum void ratio, e_{max}	0,79
Relative density, R_D	35%
Uniformity coefficient, C_u	1,53
Curvature coefficient, C_C	0,88



Figure 3.3. Narli coarse sand.

3.1.3. Medium Quartz Sand

The commercially available sand was obtained from regional sources near Warsaw. The specific gravity of the grains was found to be 2.68. A gradation of the sand falling between 1.00 mm and 0.071 mm was artificially selected. The grain size distribution curve of medium sand taken for the investigations is presented in Figure 3.4., while index properties of this soil in Table 3.2.

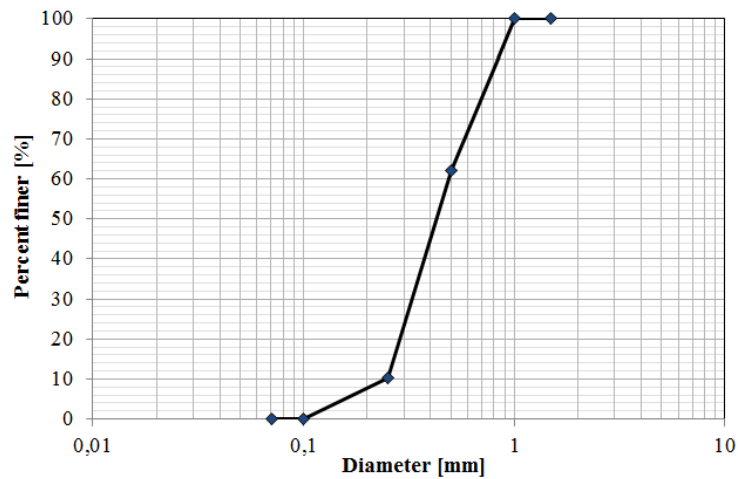


Figure 3.4. Particle size distribution curve for quartz sand.

Table 3.2. Index properties of quartz sand.

Property	Value
Minimum void ratio, e_{min}	0,45
Maximum void ratio, e_{max}	0,70
Relative density, R_D	45%
Uniformity coefficient, C_u	1,96
Curvature coefficient, C_c	0,84



Figure 3.5. Quartz sand from Warsaw.

Soil contains rectangular, quartz grains with coefficient of uniformity $C_u = 1,96$ and coefficient of curvature $C_c = 0,84$.

3.2. Methods and sample preparation

3.2.1. Hydraulic Conductivity

Part I – Coarse Narli Sand (Turkey)

For all the tests samples were prepared at relative density of 35%, which was found to be 1.56 Mg/m^3 . All works done in this stage were performed in Geotechnical Engineering Laboratory at University of Gaziantep in Turkey.

Permeability of soil is a measure of its capacity to allow a fluid to flow through it. Most applications as fluid consider only liquid, that is usually understood to be water (Head, 1994). All soils consist of solid particles and empty voids between them,

generally, the void are connected with each other, what enables water to pass through them, it means soils are permeable to water. The degree of permeability is estimated by applying a hydraulic gradient across a sample of soil, that is fully saturated, and measuring the consequent flow of water. The coefficient of permeability is evinced in terms of a velocity.

The flow of water through soils, from gravel and sand to impervious clay is directed by the same physical laws, even though a clay can be million times less permeable than sand. The flow properties were first studied by Henry Darcy in France (Darcy, 1856). He determined that under steady conditions of flow through sand samples of various thicknesses and under different pressures, the flow rate was always proportional to the fall in hydraulic head per unit thickness of sand, *the hydraulic gradient*. This assumption, known as Darcy's law, has been found to be commonly valid for the flow of water in soils, except a case of high velocities when turbulence occurs.

Darcy's law:

$$Q = A * k * i * t \quad (3.1)$$

Where:

Q – volume of water flowing through the sample [cm³],

A – cross section area of the sample [cm²],

k – coefficient of permeability [cm/s],

i – hydraulic gradient [-],

t – duration of the water flow [s].

Hydraulic gradient:

$$i = \frac{h_1 - h_2}{L} = \frac{\Delta h}{L} \quad (3.2)$$

Where:

Δh – fall in hydraulic head [cm],

L – length of the sample [cm].

Factors affecting permeability (Head, 1994):

- ✓ Particle size distribution
- ✓ Particle shape and texture

- ✓ Mineralogical composition
- ✓ Voids ratio
- ✓ Degree of saturation
- ✓ Soil fabric
- ✓ Nature of fluid
- ✓ Type of flow
- ✓ Temperature

A method that is used for measuring permeability of a soil highly depends on the characteristic of the material. In this stage a constant head permeability tests were carried out. The test maintenance is presented diagrammatically in Figure 3.6. The hydraulic gradient in the specimen was determined by using two piezometer tubes inserted in the permeability cell, denoted by a , b or c (in the case of this research only two piezometers were used). The water levels in those tubes are the heights h_a and h_b , respectively above bench level, which was taken as the common datum level. Considering piezometer tubes and distance between the points of insertion into the sample, denoted by x , the hydraulic gradient i can be easily calculated. If a quantity of water Q passes through the specimen in a time t , the coefficient of permeability of the sample is given by equation:

$$k = \frac{Q}{t \cdot A \cdot i} \text{ [cm/s]} \quad (3.3)$$

Samples for testing were prepared by mixing sandy soil with xanthan gum by dry weight. Five different mixtures were prepared, containing respectively 0.0; 0.1; 0.25; 0.5 and 1.0% of biopolymer. Firstly sand was washed and dried in an oven at approximately 105 °C. Required amounts of sandy soils and xanthan gum were blended together, under dry conditions and then spooned, without vibration, into the mould (with a diameter of 8 cm, and height of 23.6 cm) with thin layers of sand (Figure 3.9.). When the mould was completely filled, the top cap was placed and then the specimen was saturated with water. The specimens were left for a curing, for 1, 3, 7, 14, and 28 days of time. The moulds (permeability cells) with the specimens can be seen in Figure 3.10. and the details of permeability cell construction are shown in Figure 3.8.

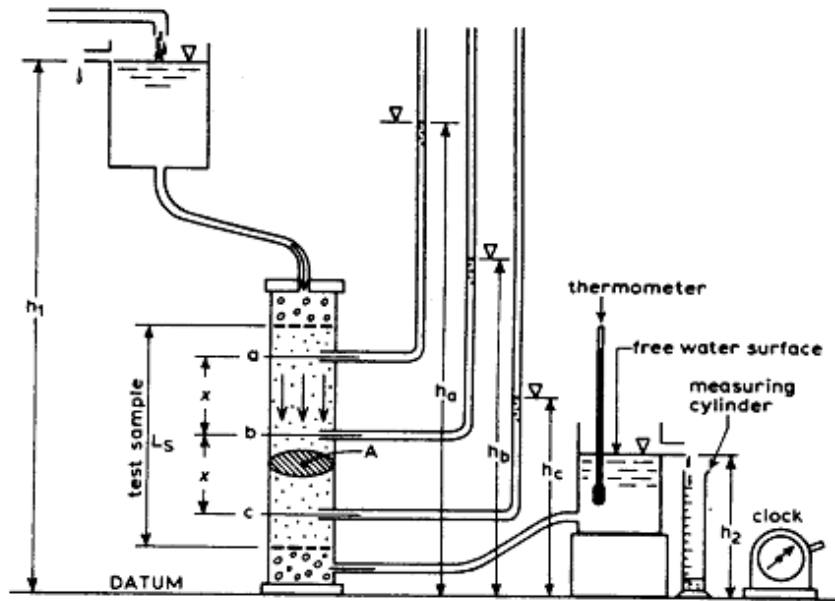


Figure 3.6. Principle of constant head permeability test (Head, 1994).

Calculated rate of water flow q (ml/min) versus time was plotted on a graph for each of the samples. The rate of flow varied at first, but then decreased slightly to reach a constant steady state value. Plotting the graphs was necessary to make sure that the final hydraulic conductivity values were correct. An example of such a graph is given in Figure 3.7.

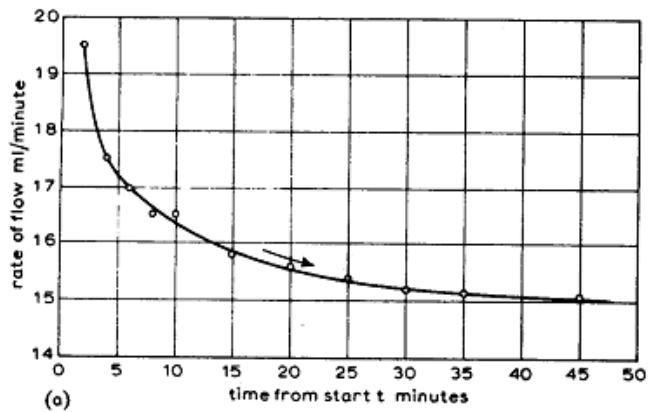


Figure 3.7. An example of water flow versus time curve.

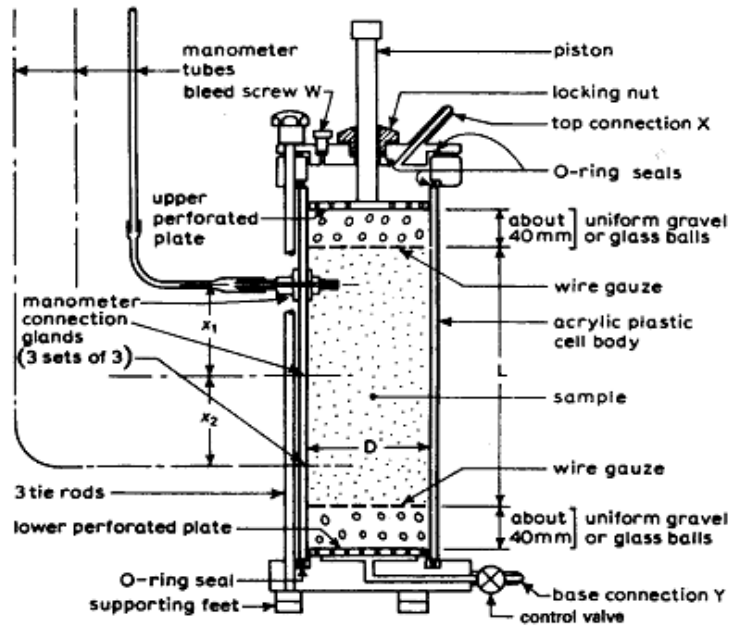


Figure 3.8. Details of constant head permeability cell (Head, 1994).



Figure. 3.9. Samples in the molds while curing [8.0 x 23.6 cm].

All samples were made saturated from the bottom to top. A series of constant head permeability tests were performed according to ASTM D2434. The hydraulic conductivity of each sample was reported by the average of the last three measurements. Two tests were done for all sand - xanthan gum mixtures and the average was taken as a final value. In this stage 42 tests were carried out, all under same room temperature of 24°C.



Figure. 3.10. Constant head permeability apparatus.

Part II – medium quartz sand (Poland)

All tests in this phase were conducted in Water Center Laboratory in Geotechnical Engineering Department at Warsaw University of Life Sciences in Poland.

In this stage, when a better graded and containing smaller particles sand was investigated, more accurate testing apparatus was necessary to use. Tests were conducted in a triaxial cell, where hydraulic gradient and confining pressure were controlled. Permeability apparatus with the specimen can be seen in Figures 3.12. and 3.13. This device provides facilities for maintaining a water flow through a specimen under a known difference of pressure on its top and bottom, and for measuring the rate of flow, when the sample is subjected to a known effective stress. Using those measurements the soil permeability can be easily calculated. Several tests, under a various effective stresses can be carried out on the same specimen without removing it (Head, 1994).

Advantages for using triaxial cell in permeability measurement:

- ✓ Mixture can be first saturated by applying a back pressure, reducing air bubbles;
- ✓ The test can be carried out under effective stresses and pore water pressure that represents the field conditions;
- ✓ It is easy to measure small rates of water flow;

- ✓ Both, constant and falling head procedures can be applied;
- ✓ A wide range of hydraulic gradients might be applied and measured accurately;
- ✓ There is no cell wall effect, that gives non-uniform flow conditions;
- ✓ All the test is automatized, reading can be made very often.

An example of triaxial permeability test is shown in Figure 3.11. The cell confining pressure must be greater than the inlet and outlet water pressures, otherwise the sample would be destroyed. The coefficient of permeability k was determined by the same means as described in paragraph 3.2.1. Part I, measuring the volume of water that passes through the sample in a known time:

$$k = \frac{Q}{t \cdot A \cdot i} \text{ [cm/s]} \quad (3.3)$$

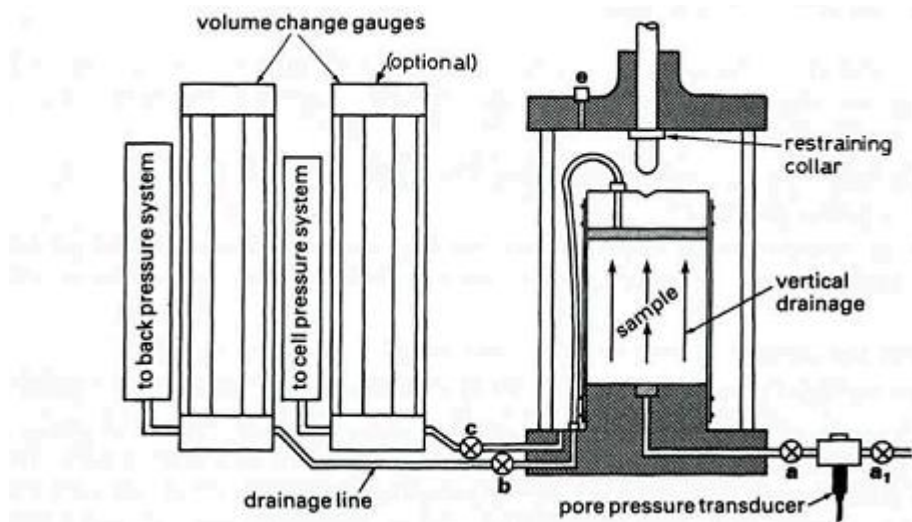


Figure 3.11. Arrangement of the triaxial apparatus (Head, 1994).

When only possible the applied hydraulic gradient values should be compatible with those like to occur in the field. However, for some soils with very low permeability, it might be necessary to use substantially greater hydraulic gradients, so that the laboratory tests would not last for a too long time.

For all the tests samples quartz sand was used, prepared at relative density of 45%, which was found to be 1.66 Mg/m^3 .

Samples were arranged by mixing sandy soil with xanthan gum by dry weight. For medium sand four different mixtures were prepared, containing respectively 0.0; 0.5; 1.0 and 1.5% of biopolymer. The required amounts of sandy soils and xanthan gum were blended together under dry conditions; later water was added, reaching 10% of each dry sample weight. Then mixture was spooned into the mold (with a diameter of 69 mm, and height of 70 mm) with thin layers of sand. When the mold was completely filled, using plastic bags and foil, it was tightly sealed to prevent moisture loose. Then samples were left for curing, respectively for 1, 3, 7, 14, and 28 days of time. In this stage samples were not fully saturated while curing time.



Figure 3.12. Permeability apparatus: 1- specimen; 2- triaxial cell; 3- cell pressure controller; 4- pore water controller; 5- out flow cylinder; 6- sample mould.

The permeability tests were conducted on clean sands and sand with xanthan gum at three different contents. A series of constant head permeability tests were performed according to ASTM D2434. Tests were conducted in a triaxial cell, where hydraulic gradient and confining pressure were controlled.



Figure 3.13. Permeability cell.

All samples were made saturated from the bottom to top. Pressure at the top of each sample was set to be 0 kPa, while at the bottom applied pressure took values of 5, 10, 20, 30, 50 and 100 kPa (Head, 1995). Confining pressure in the cell was set to be 50 kPa (for bottom pressure of 5, 10, 20 and 30 kPa), 100 kPa (for 50 kPa) and 200 kPa (for 100 kPa). The hydraulic conductivity of each sample was reported by the average of the last three measurements. All tests were conducted under room temperature (22 °C).

3.2.2. Oedometer tests

All consolidation tests were conducted in Water Center Laboratory in Geotechnical Engineering Department at Warsaw University of Life Sciences in Poland.

The oedometer consolidation test is used to determine the consolidation characteristics of soils, namely the compressibility of the soil. Whenever a load is placed on the ground (i.e. due to a structural foundation, some settlement will occur even if applied pressure is within the bearing capacity of that soil. The limitation of settlement may sometimes be more significant in foundation design than the bearing capacity requirements (Head, 1994).

Consolidation in sand and gravels take place in a short time, normally during the construction proceeds, and those exceptionally cause major problems. However, in clay soils, due to their low permeability, settlement may take place in much longer periods – months, years or decades. Therefore the estimation of settlement and the time of its completion are very important factors in foundation design.

The problem of long term consolidation of clays was introduced by Terzaghi in his publication in Vienna in 1925. Terzaghi formed a theoretical approach to the consolidation process and designed the first consolidation apparatus, named oedometer (from Grek *oidema*, swelling). Later in the USA test on various sized specimens were carried out by Casagrande (1932), Gilboy (1936), and Rutledge (1935). The mathematical theory of consolidation process was published by Terzaghi and Frohlich in 1936.

The test is normally carried out by application of a sequence of vertical loads to a laterally confined specimen. The vertical compression under each load is being observed for a period of time, usually up to 24 h. Since no lateral deformation is permitted, it is a one-dimensional test, where the one-dimensional consolidation parameters are derived. Details of a typical oedometer consolidation cell are shown in Figure 3.14.

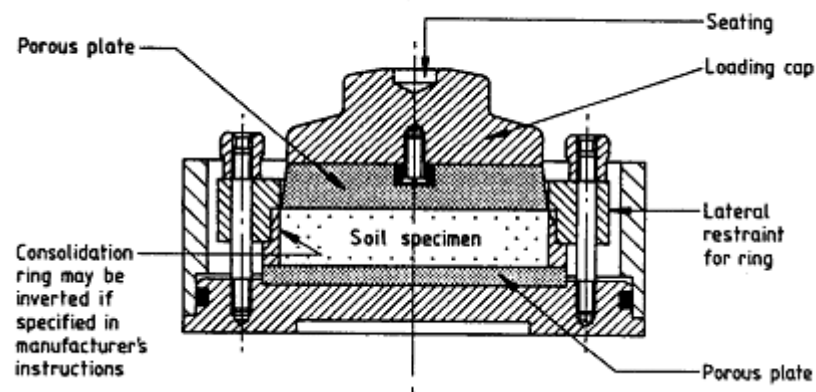


Figure 3.14. Details of the oedometer cell (Head, 1994).

Soils consist of solid particles between which are spaces (voids) that might be filled by air or a liquid, usually water. When a specimen is subjected to a compressive pressure its volume tends to decrease, what takes place due to the compression of the solid grains and compression of the water within the voids between grains.

The volume change during consolidation test occurs only in the voids. The change in height ΔH , from initial height H_0 , corresponds to a change in voids ratio Δe from an initial voids ratio e_0 (Figure 3.15.).

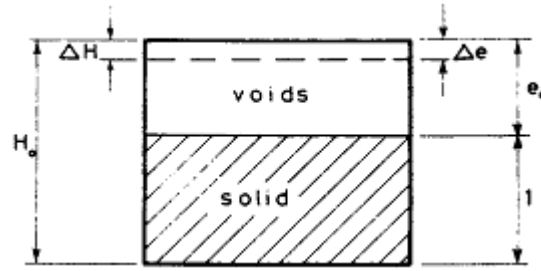


Figure 3.15. Representation of voids ratio change (Head,1994).

Change in voids ratio is determined as follow:

$$\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0} \quad (3.4)$$

$$\Delta e = \frac{1+e_0}{H_0} * \Delta H \quad (3.5)$$

$$\Delta e = \frac{\Delta H}{H_s} \quad (3.6)$$

Where H_s is the equivalent height of solid particles, which depends only on the initial conditions of the test sample and remains constant. It defined as:

$$H_s = \frac{H_0}{1+e_0} \quad (3.7)$$

Two compressibility coefficients were derived from the consolidation test to indicate the compressibility of the specimen:

- ✓ Coefficient of volume compressibility m_v ,
- ✓ Compression index C_c .

Coefficient of volume compressibility indicates the compressibility per unit thickness of the soil, it can be known also as a modulus of volume change. Defined by the equation:

$$m_v = \frac{e_0 - e_1}{(1+e_0) * \Delta \sigma'} \quad (3.8)$$

Where:

- e_1 – void ratio at the start of the load increment $\Delta \sigma'$;
- $\Delta \sigma'$ – load increment in a single loading stage.

The compression index C_c is equal to the slope of the filed consolidation curve plotted to a logarithmic scale of pressure σ , in the linear range. This straight line is represented by the equation:

$$C_c = \frac{e_o - e}{\log_{10} \frac{\sigma_o + \Delta\sigma}{\sigma_o}} \quad (3.9)$$

Numerically C_c is equal to the change in voids ratio for one log cycle of pressure change. It is a dimensionless number.

Samples were prepared using medium quartz sand, at relative density of 45%, which was estimated to be 1.66 Mg/m^3 . Sandy soil was mixed with xanthan gum by dry weight, containing respectively 0, 0.5, 1.0 and 1.5 % of the biopolymer. Before testing sand was washed and dried in the oven. After bending together soil and biopolymer 10 % of water (by weight) was added and then placed in a ring with diameter of 7.5 cm, and height of 2 cm. Equipment that was employed for testing is presented in Figure 3.16.

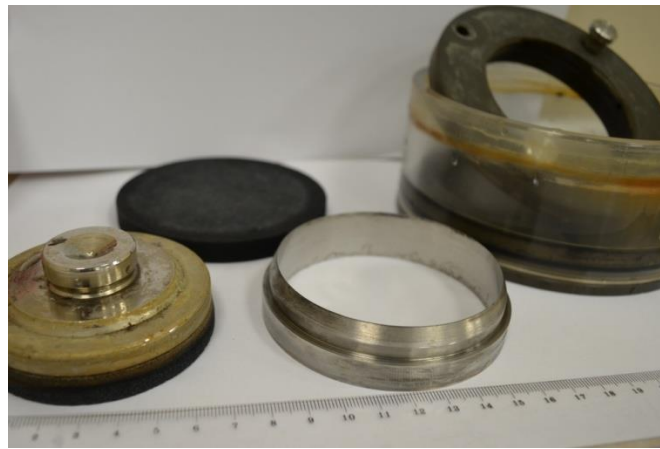


Figure 3.16. Oedometer equipment.

In this stage of laboratory investigation four samples were tested. After ring with the specimen was placed in the oedometer apparatus, mould was filled with a distilled water for saturation. The consolidation machine is shown in Figure 3.17. For proceedings a modern, fully atomized apparatus was used, namely The Automatic CONSolidation System (ACONS). Together with digital height readers was connected to a computer, where necessary software was available.

All tests were conducted according to ASTM D2435. Investigation program consisted of 10 steps. Firstly, a pressure of 1 kPa was applied to make sure that loading frame has touched the sample's surface and then pressure increased to values of 25, 50, 100, 200 400 kPa, later for unloading stage, respectively 200, 100, 50, 25, 0 kPa. Each of the pressures was applied to the sample for 24 hours consolidation time. Due to digital equipment accessibility the readings were done every 60 seconds. Tests were conducted under room temperature (22 °C).



Figure 3.17. Oedometer apparatus that was used during experimental works.

3.2.3. Triaxial tests

All the tests in this stage were conducted in Water Center Laboratory in Geotechnical Engineering Department at Warsaw University of Life Sciences in Poland.

The triaxial test is widely performed and one of the most versatile geotechnical test, that allows the shear strength and stiffness of soil to be determined and used in geotechnical design. The test gives ability to control specimen drainage and take measurements of pore water pressures. Standard parameters obtained from the test

are the angle of shearing resistance ϕ' and cohesion c' , although the shear stiffness G , compression index C_c and permeability k might be estimated. The general set-up of a triaxial cell is shown in Figure 3.18.

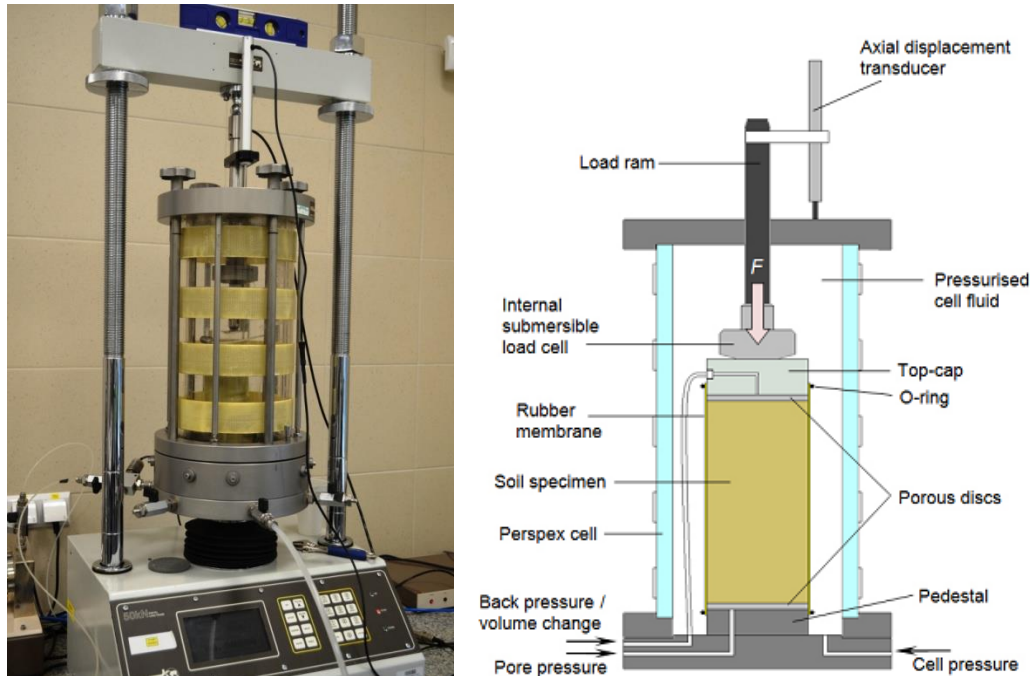


Figure 3.18. General set-up of a soil specimen inside a triaxial cell
(www.gdsinstruments.com)

Three primary triaxial tests can be conducted in the laboratory:

- ✓ Unconsolidated Undrained test (UU)
- ✓ Consolidated Undrained test (CU)
- ✓ Consolidated Drained test (CD)

For investigation the consolidated drained (CD) test was chosen, it applicable to describe a long term loading response, providing geotechnical parameters designated under the effective stress control. The set shear rate was slow enough to allow small water pressure changes.

Stresses applied to the biopolymer – sand mixture while running a compression test are presented in Figure 3.19. The confining stress σ_c is set by pressurizing the cell fluid surrounding the specimen, it is equal to minor principal stress σ_3 . The deviator stress q is created by applying an axial strain ϵ_a to the mixture. The deviator stress acts in addition to the minor principal stress σ_3 in the axial direction, together equal to the major principal stress σ_1 .

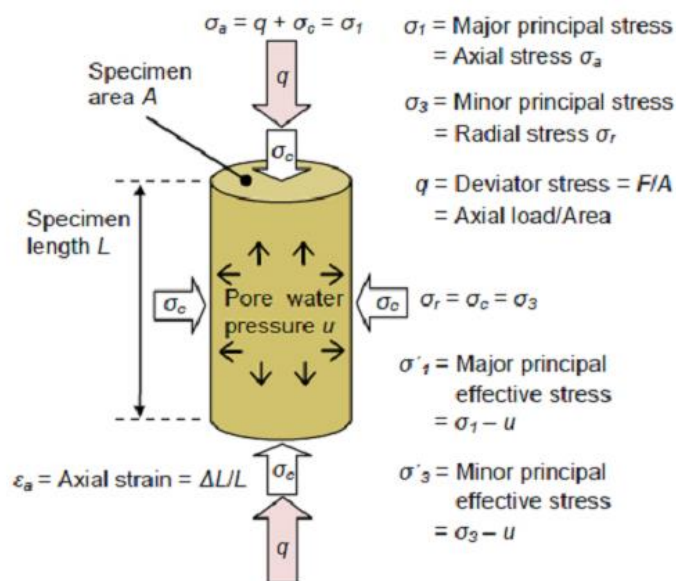


Figure 3.19. Specimen stress state during the triaxial compression (www.gdsinstruments.com).

For testing procedures 6 samples, each two containing respectively 0.0, 0.5 and 1.5 % of xanthan gum were prepared. Specimen had a diameter of 70 mm and height equal to 140 mm, what gives 2:1 height-to-diameter ratio. Samples were prepared using a medium quartz sand, at relative density of 45%, which was estimated to be 1.66 Mg/m³. Before testing sand was washed and dried in the oven. After bending together soil and biopolymer, 10 % of water (by weight) was added and before placing in the triaxial machine samples were sealed within a rubber membrane. Sample ready for testing in presented in Figure 3.21. Following the initial preparation mixtures were saturated than consolidated and sheared, creating conditions that approximate to those in-situ. During the shear stage mixtures were loaded axially. The saturation process is required to ensure all voids within the specimen are filled with water and that the pore pressure transducer and drainage paths are properly de-aired. The consolidation stage was use to put the mixtures into the effective stress required for shearing. It was conducted by increasing the cell pressure while maintaining a constant pore water pressure. The specimen was sheared by applying an axial strain ϵ_a at a constant rate through upward movement of the load frame platen. Biopolymer – sand mixture preparation is shown in Figure 3.20.



Figure 3.20. Biopolymer – sand mixture under preparation.

Each mixture containing respectively 0.0, 0.5 and 1.5 % of xanthan gum was tested in two stages, 1 sample for each stage. In the first part of investigation, confining pressure σ_c (cell pressure) reached the value of 100 kPa, while the pore water pressure u (back pressure) was equal to 50 kPa that gave the minor effective stress σ'_3 equal to 50 kPa. In the second stage, confining pressure σ_c was set to be 250 kPa, the pore water pressure $u = 50$ kPa, in result the minor effective stress σ'_3 reached the value of 200 kPa.

The respond of soil during the shear stage was monitored by plotting the deviator stress q against the axial strain ϵ_a . The stage was continued until identification of the peak deviator was possible.



Figure 3.21. Prepared sample ready for triaxial testing.

Values of effective stresses were calculated as follow:

$$\sigma'_1 = \sigma_1 - u \quad (3.10)$$

$$\sigma'_3 = \sigma_3 - u \quad (3.11)$$

Where:

σ'_1 – major effective stress [kPa],

σ_1 – major principal stress [kPa],

u – pore water pressure [kPa],

σ'_3 – minor effective stress [kPa],

σ_3 – minor principal stress [kPa].

During the test confining pressure was set a constant value, and deviator stress was being increased until the failure criteria represented by Mohr-Coulomb theory occurred. The failure mechanism is shown in Figure 3.22. After the tests were completed, maximum stresses and vertical strain determined, shear strength parameters (c' - effective cohesion, ϕ' – effective internal friction angle) could be estimated.

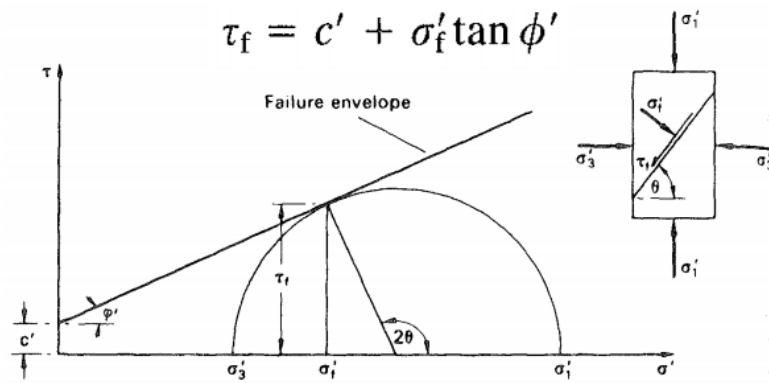


Figure 3.22. Mohr-Coulomb failure criterion (Bishop and Henkel, 1962).

For each sample Mohr-Coulomb circles, stress paths and failure envelopes were drawn, due to that the failure envelope characteristic were assessed and shear strength parameter could be designated. The point on each stress path corresponding to the failure criterion of maximum deviator stress was marked, and the line of best fit was drawn through the set of points. The angle created by the line and the horizontal axis is denoted by θ , and its intercept with vertical axis is denoted by t_o .

The shear strength parameters (c' , φ') are derived from the following realtions:

$$\sin \varphi' = \tan \theta \quad (3.12)$$

$$c' = \frac{t_0}{\cos \varphi'} \quad (3.13)$$

Test procedure was based on ASTM D7181 – 11 Method for Consolidated Drained Triaxial Compression Test for Soils.

3.2.4. Unconfined compressive strength tests

All the tests were conducted in Water Center Laboratory in Geotechnical Engineering Department at Warsaw University of Life Sciences in Poland.

The unconfined compression test is usually used to measure the shearing resistance of cohesive soils, which might be undisturbed or remolded specimens. An axial load is applied by use of either strain or stress control condition. For cohesive soils the unconfined compressive strength is described as the maximum unit stress obtained within the first 20 % strain.

For investigation, specimens remaining after hydraulic conductivity tests were employed. Hard and solid cylindrical samples are shown in Figures 3.23. and 3.24.



Figure 3.23. Solid sand-xanthan gum samples (a).

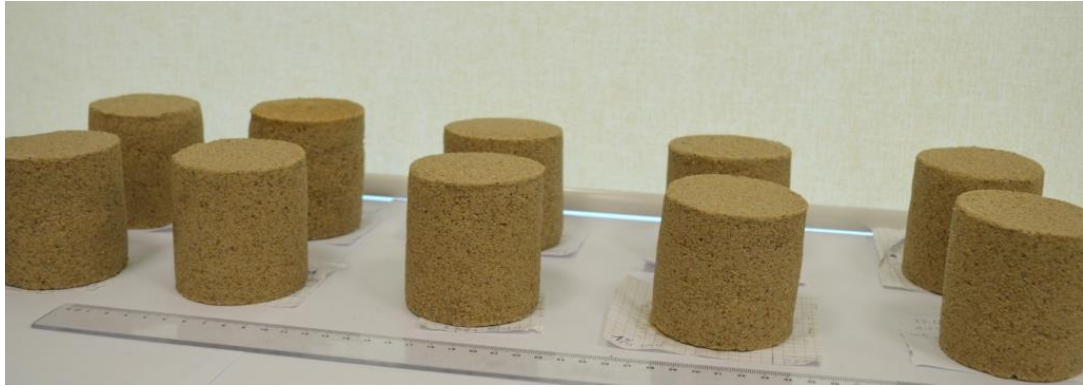


Figure 3.24. Solid sand-xanthan gum samples (b).

Biopolymer – sand mixtures due to the hardening process got solid. After being taken out from triaxial permeability cell, specimens were left for drying, in the room temperature, for about 2 months. Cylindrical tubes had a diameter of about 70 mm and were approximately 70 mm high. Fifteen samples have been tested, for each of them curing time (before permeability investigation) was estimated. Process and sample preparation described in details can be found in section 3.2.1. Hydraulic conductivity, Part II.

Unconfined Compressive Strength (UCS) testing is carried out using a digitally-controlled INSTRON testing machine (see Figure 3.25.), with measurement of the applied stress, overall sample height, and strain values.



Figure 3.25. INSTRON – mechanical testing system.

Samples used for testing could be considered as rock material, because of their stiffness. The compressive strength is probably the most widely applied and quoted rock engineering parameter. Under uniaxial loading conditions the maximum stress that rock sample can sustain is referred as uniaxial compressive strength (σ_c or q_u).

The most useful way to present mechanical behavior of intact rock is the complete stress – strain curve of the compressive strength test. The stress – displacement relation is presented in Figure 3.26.

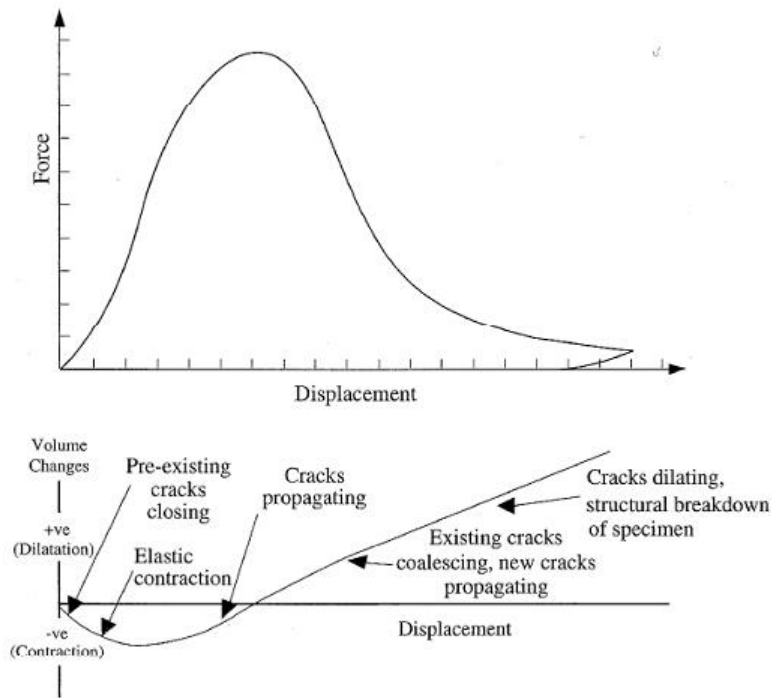


Figure 3.26. Force – displacement curve (Hudson and Harrison, 2007)

The stress values applied to the ends of the sample (σ_1 , or major principle stress) is computed as follows:

$$\sigma_1 = \frac{F}{A} \quad (3.14)$$

Where:

σ_1 – unconfined compressive strength [kPa],

F – maximum force/load applied to the sample [kN],

A – cross-section area of the sample.

Tests procedure was based on:

- ✓ ASTM D2166 Standard Test Method for Unconfined Compressive Strength of Cohesive Soil
- ✓ ASTM D7012 - 13 Standard Test Methods for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures

CHAPTER 4

RESULTS AND DISCUSSION

4.1. Hydraulic conductivity

Part I – coarse Narli sand (Turkey)

The level of saturation was controlled, in saturated material imposed flow is realized in steady flow conditions. It implies that volume of water flowing into a specimen must be equal to volume of water flowing out of it. Should be emphasized that, with verification of steady flow conditions any permeability tests cannot be consider as a reliable.

The results regarding hydraulic conductivity of biopolymer treated sand are shown in Table 4.1. Addition of 0.1% xanthan gum to the coarse sand decreases its permeability to almost half of the initial value (Figure 4.1.) Addition of 1.0% xanthan gum changes the permeability from $7.16 \cdot 10^{-3}$ m/s to about $5.75 \cdot 10^{-5}$ m/s, which is much less than 100 times.

Table 4.1. Hydraulic conductivity values of biopolymer treated coarse sand.

Ratio [%]	Permeability [m/s]				
	Curing time				
	1 day	3 days	7 days	14 days	28 days
0,00	7,16E-03	7,16E-03	7,16E-03	7,16E-03	7,16E-03
0,10	3,07E-03	4,28E-03	5,09E-03	6,41E-03	6,92E-03
0,25	2,20E-03	2,52E-03	2,94E-03	4,02E-03	6,25E-03
0,50	8,90E-04	1,55E-03	1,72E-03	3,38E-03	5,74E-03
1,00	5,75E-05	3,07E-04	7,09E-04	1,62E-03	3,55E-03

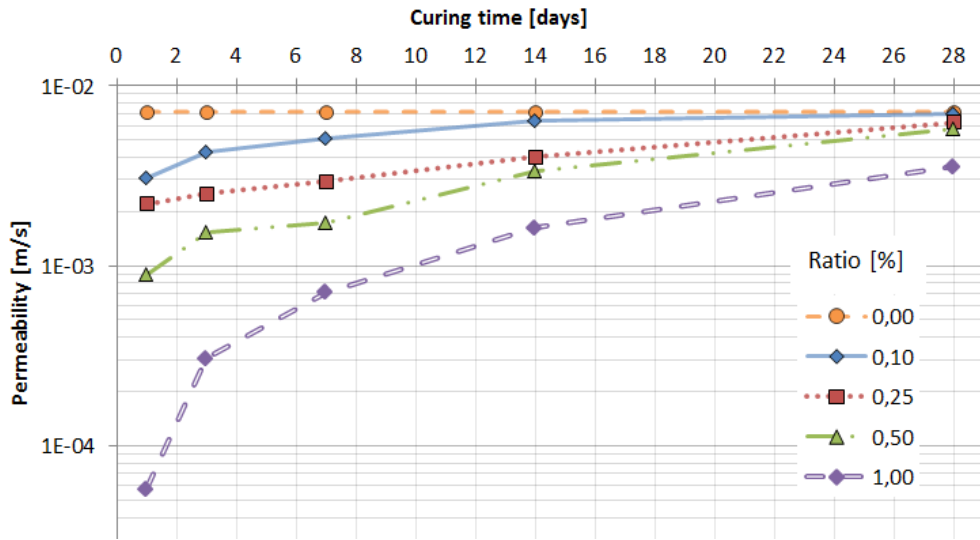


Figure 4.1. Hydraulic conductivity of a biopolymer treated Narli sand.

As mentioned before, the effect of the biopolymer inclusion, stable permeability decrease was observed. Hydraulic conductivity highly depends on the amount of biopolymer added to the sample, as shown in Figure 4.2.

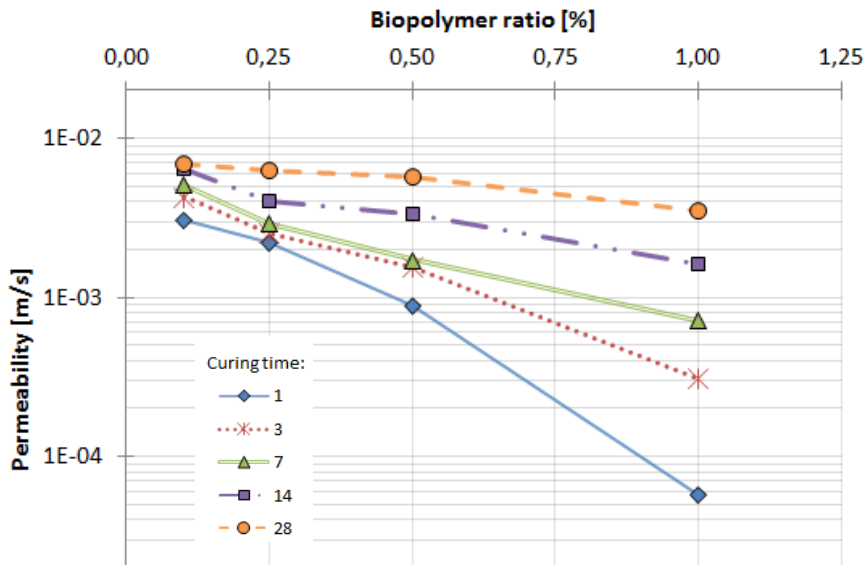


Figure 4.2. Effect of biopolymer content on hydraulic conductivity of coarse sand

The aging effect on the sample is not very well understood, it seems that biopolymers undergo some partial decomposition in the soil, but the process is not fully known. Bio substance may behave in a different manner while placed in field, where more living organisms are present and physico-chemical properties of the soil are other than those in a laboratory. Some tests in the natural environment must be done to possess

more knowledge about bio reactions going on in the sand and a long term observation is required to understand the aging process of sand biopolymer mixture.

Viscous characteristics of xanthan gum have a significant meaning for stability of the soil. As observed in Figure 4.3. samples with greater amount of the biopolymer acts much more stable, the soil particles glued to each other and created a linked structure. That gives another conceivable way to apply those chemicals in the ground, for instance in slope or road embankment stability.

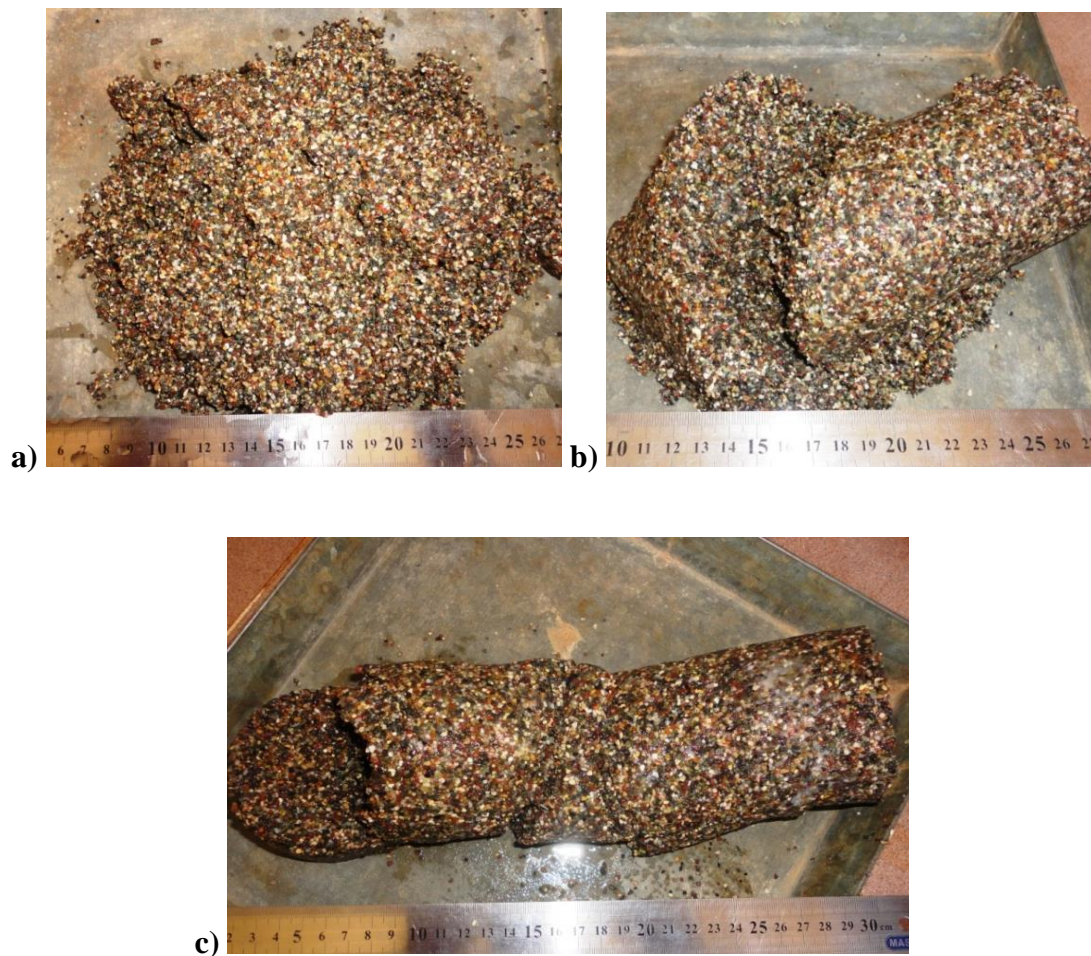


Figure 4.3. Samples after testing: a) ratio 0.1%, b) ratio 0.5%, c) ratio 1.0%.

Part II – Medium Quartz Sand (Poland)

The testing results on hydraulic conductivity of biopolymer treated sand are shown in Table 4.2. Ninety tests have been conducted, for each xanthan gum ratio it was 30 tests, depending on bottom pressure applied and the conditioning time.

Permeability (hydraulic conductivity) decreases when more xanthan gum is added to the sample. For example, addition of just 0.5% xanthan gum to the sand decreases the permeability to almost 0.001% of the initial value. Addition of 1.5% xanthan gum changes the permeability from $8.46 \cdot 10^{-5}$ m/s to about $2.84 \cdot 10^{-11}$ m/s, which is less than 1 000 000 times.

Table 4.2. Hydraulic conductivity of biopolymer treated sand for various curing time.

Biopolymer ratio [%]	Curing time [days]	Permeability [m/sec]					
		Pressure applied [kPa]					
		5	10	20	30	50	100
0,5	1	8,65E-10	9,02E-10	1,13E-08	8,71E-08	5,25E-07	1,55E-06
	3	6,26E-10	2,42E-09	5,30E-08	3,76E-07	1,33E-06	1,83E-06
	7	4,25E-09	4,42E-09	9,89E-08	9,75E-07	1,45E-06	2,14E-06
	14	3,12E-09	4,92E-08	2,80E-07	7,49E-07	1,68E-06	2,35E-06
	28	3,14E-08	2,03E-07	9,02E-07	1,53E-06	2,67E-06	2,59E-06
1,0	1	1,18E-10	1,24E-10	4,67E-10	7,23E-10	7,12E-09	1,98E-07
	3	3,68E-10	3,94E-10	7,07E-10	2,75E-09	1,79E-08	9,09E-07
	7	4,98E-10	5,54E-10	6,74E-10	3,81E-09	3,57E-08	1,29E-06
	14	2,12E-09	2,64E-09	1,47E-08	7,17E-08	4,09E-07	1,42E-06
	28	2,69E-09	2,50E-09	5,56E-09	2,00E-07	3,34E-07	1,03E-06
1,5	1	2,84E-11	7,33E-11	1,28E-10	2,20E-10	5,08E-11	9,25E-08
	3	3,40E-11	3,38E-11	1,54E-10	7,53E-11	2,59E-11	7,90E-08
	7	4,39E-11	3,90E-11	7,73E-11	4,91E-11	3,20E-11	1,41E-07
	14	2,18E-11	2,51E-11	1,20E-10	7,98E-10	2,70E-10	6,39E-07
	28	6,84E-11	5,69E-11	2,80E-10	7,27E-10	8,66E-10	1,99E-07

According the pressure applied and sample dimensions, for each of them different hydraulic gradient value was estimated, all results are presented in Table 4.3.

Table 4. 3. Values of hydraulic gradient applied to each sample.

Biopolymer ratio [%]	Curing time [days]	Hydraulic gradient [-]					
		Pressure applied [kPa]					
		5	10	20	30	50	100
0,5	1	7,41	14,81	29,63	44,44	68,15	88,89
	3	7,29	14,58	29,15	43,73	58,31	75,80
	7	7,35	14,71	29,41	43,76	56,18	68,12
	14	7,41	14,81	29,63	44,44	51,85	59,26
	28	7,26	14,51	29,03	36,28	44,99	53,70
1,0	1	7,49	14,97	29,94	44,91	74,85	149,70
	3	7,41	14,81	29,63	44,44	74,07	125,93
	7	7,59	15,17	30,35	45,52	75,87	135,05
	14	7,11	14,22	28,45	42,67	71,12	105,31
	28	7,27	14,53	29,07	43,60	72,67	116,28
1,5	1	7,46	14,93	29,85	44,78	74,63	149,25
	3	7,41	14,81	29,63	44,44	74,07	148,15
	7	6,98	13,97	27,93	41,90	69,83	139,66
	14	7,27	14,53	29,07	43,60	72,67	145,35
	28	7,24	14,47	28,94	43,42	72,36	144,72

Figures 4.4 - 4.6. Present three graphs with different xanthan gum (biopolymer) content, where the change of permeability, according to the hydraulic gradient applied for various curing time is shown. It is seen that Hydraulic conductivity increases as the applied pressure increases, and as the curing time gets longer. Permeability of the specimens remains low, under greater hydraulic gradient for higher xanthan gum ratio. As presented in the graph for 1.5% biopolymer content, water flow through the sand is stable, for all the samples (different curing times) for a hydraulic gradient up to 70. In that case improved sand might be considered as impermeable, reaching values between $2.18 \cdot 10^{-11}$ and $8.66 \cdot 10^{-10}$ m/s.

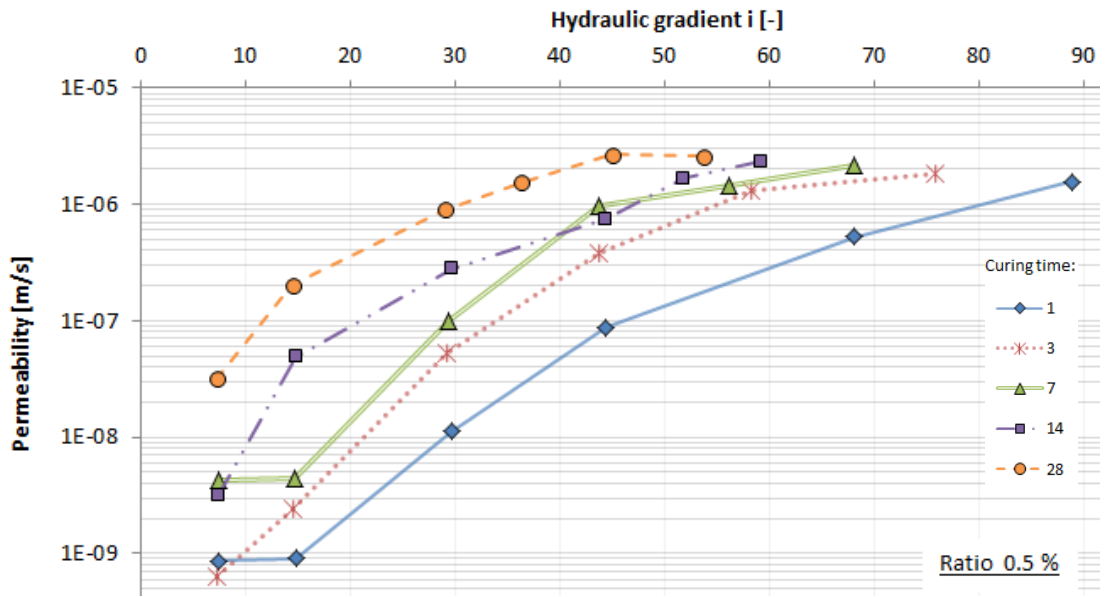


Figure 4. 4. Hydraulic conductivity of a 0.5 % biopolymer treated quartz sand.

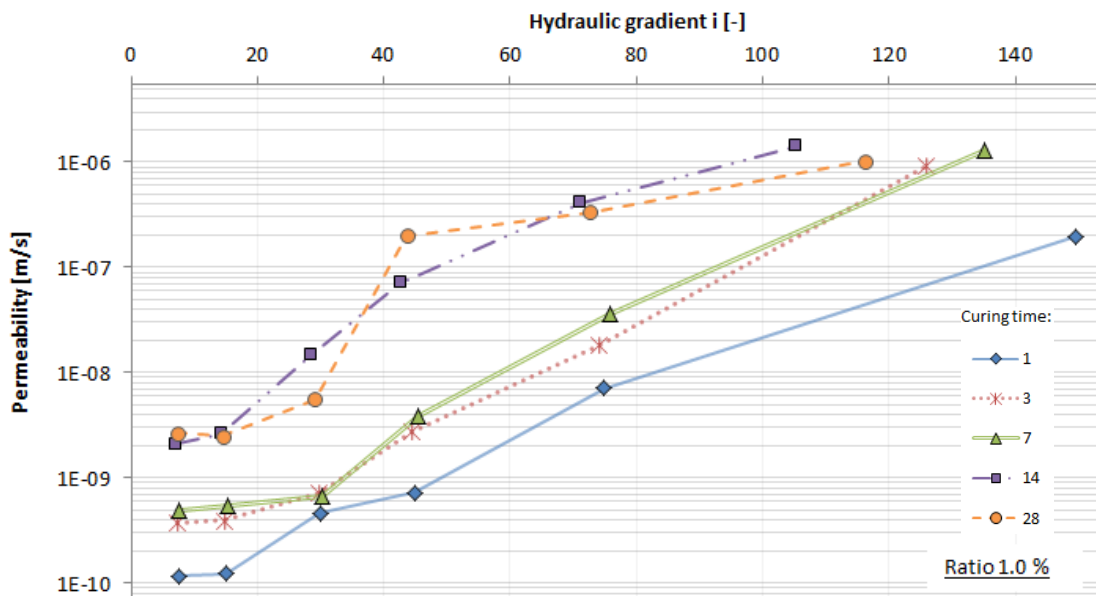


Figure 4. 5. Hydraulic conductivity of a 1.0 % biopolymer treated quartz sand.

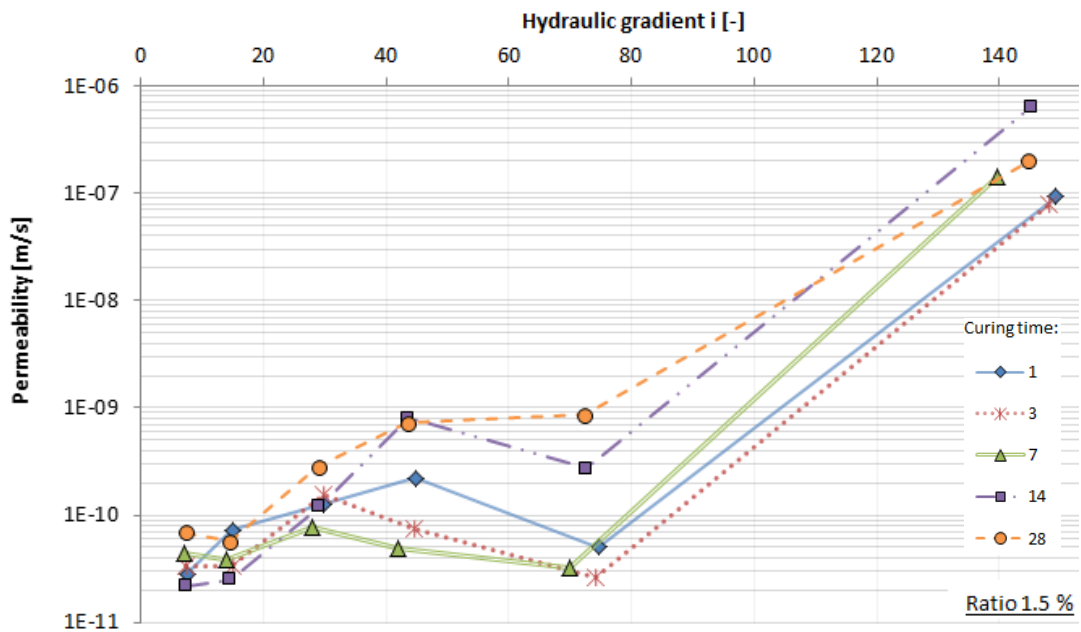


Figure 4.6. Hydraulic conductivity of a 1.5 % biopolymer treated quartz sand.

Effect of time on hydraulic conductivity of the soil biopolymer mix under various pressures is shown in Figures 4.7. - 4.12. When the long-term behavior of seepage barriers is a main objective, observing short term behavior is a good evidence of the possible technology development. A longer ageing time generally achieved a lower conductivity, but work with coarse and medium sand shows different relationship. It can realized that the permeability (hydraulic conductivity) of a biopolymer treated sand increases with time, for the 0.5% mixture under pressure of 30 kPa, it increases from $3.76 \cdot 10^{-7}$ m/s at 3 days to $1.53 \cdot 10^{-6}$ m/s at 28 days. The ageing influence gets lower, when the xanthan gum content increases, for 1.5% mixture under pressure of 30 kPa, it increases from $7.53 \cdot 10^{-11}$ m/s at 3 days to only $7.27 \cdot 10^{-10}$ m/s at 28 days. Permeability remains low and stable for all 28 days, up to the hydraulic gradient value of 70.

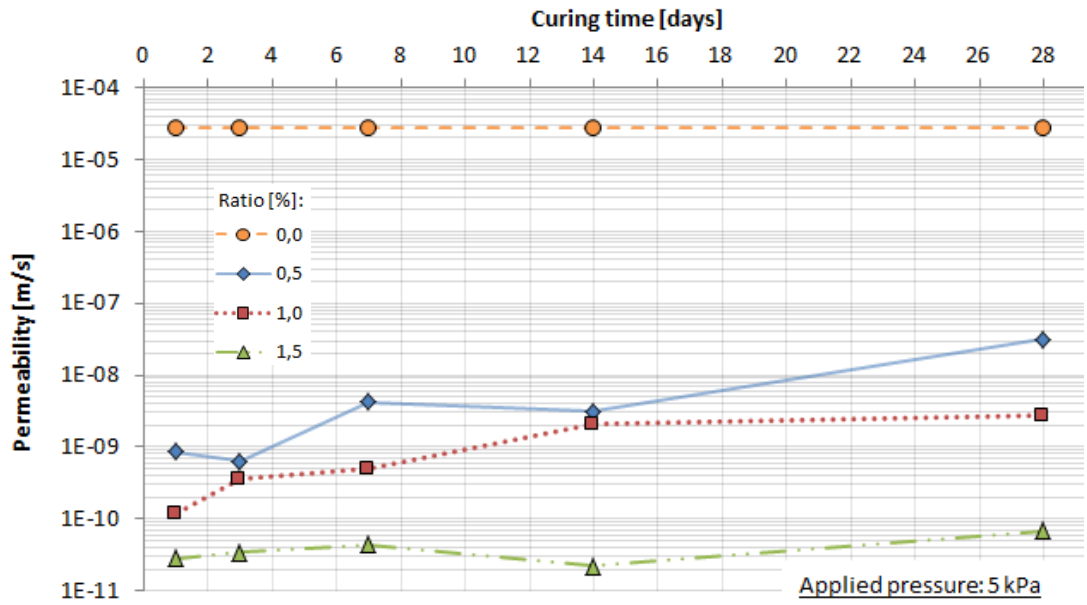


Figure 4.7. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 5 kPa.

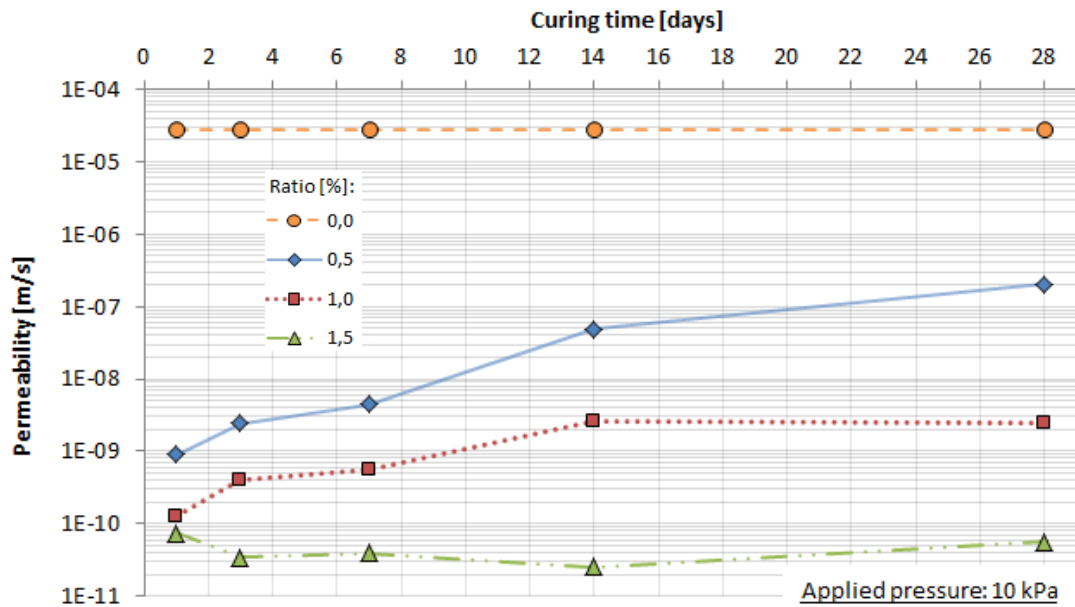


Figure 4.8. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 10 kPa.

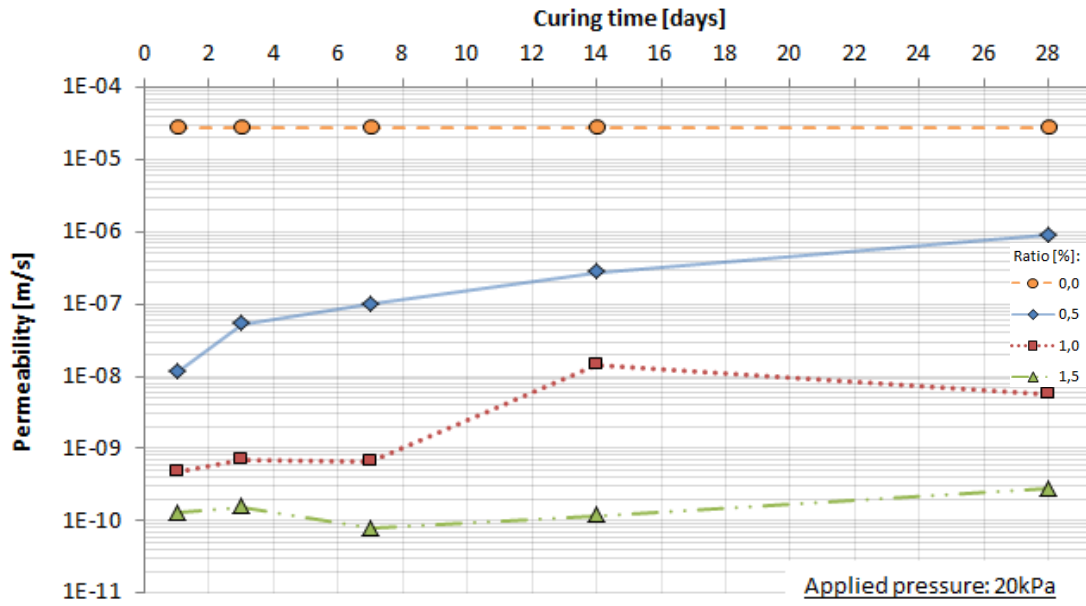


Figure 4.9. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 20 kPa.

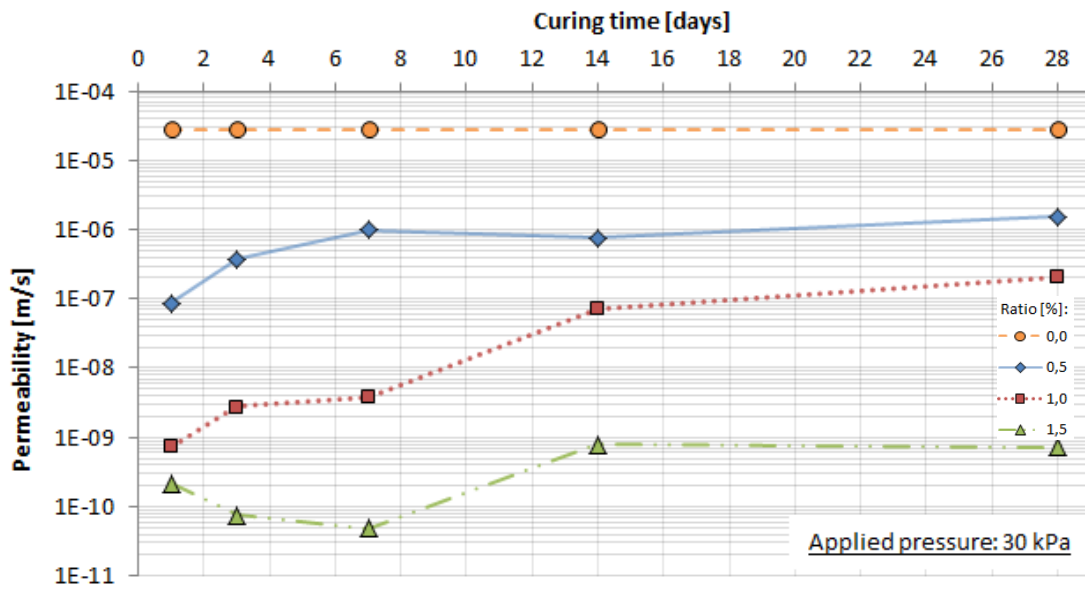


Figure 4.10. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 30 kPa.

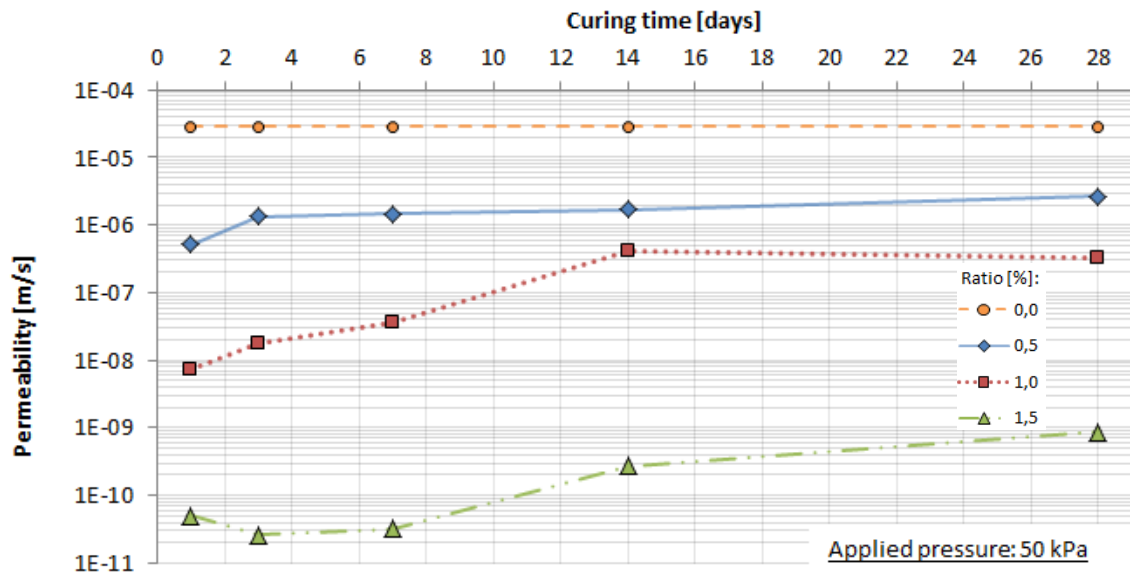


Figure 4.11. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 50 kPa.

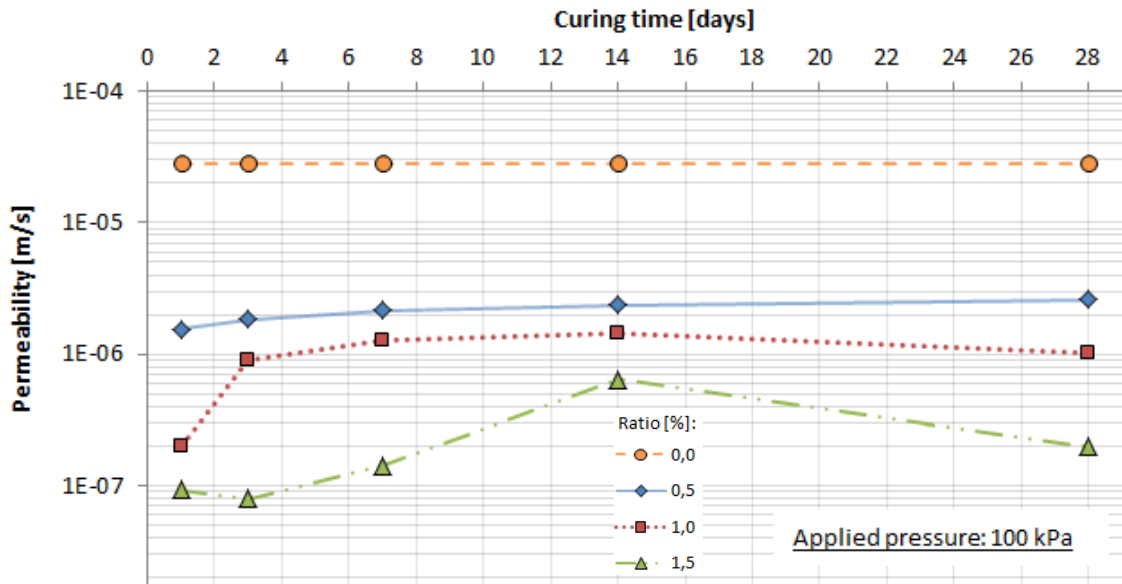


Figure 4.12. Effect of time on hydraulic conductivity of the soil biopolymer mix under pressure of 100 kPa.

A polymeric chain is significant for permeation of the grout. When the biopolymer is placed in the soil matrix, it is desired to undergo some form of cross-linking in order to enhance strength and decrease its mobility in the ground. Cross-linking connects polymeric chains through chemical reactions, which might be initiated by temperature rise, change in pressure and pH. The process can form a comprehensive lattice in the soil matrix, which rigidifies the whole polymeric structure, enhance its mechanical strength and reduce permeability (Khatami, O’Kelly, 2012).

As the effect of the biopolymer inclusion, stable permeability decrease was observed as shown in Figures 4.13. – 4.18. When the biopolymer ratio increases permeability decreases of no account of conditioning time and pressure applied. It determines a possible usage of that chemical material to create impervious barriers in the soil.

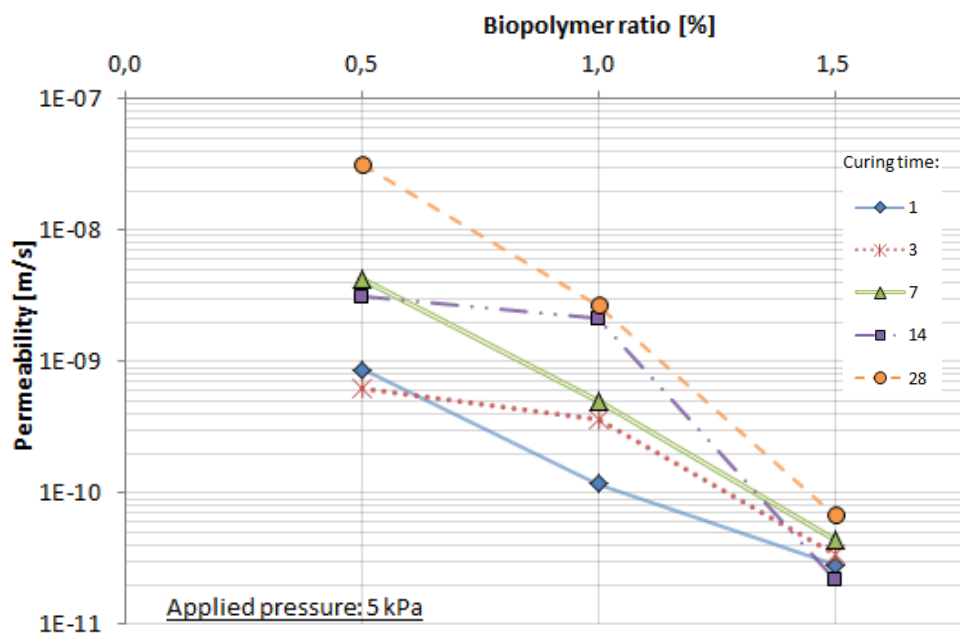


Figure 4.13. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 5 kPa.

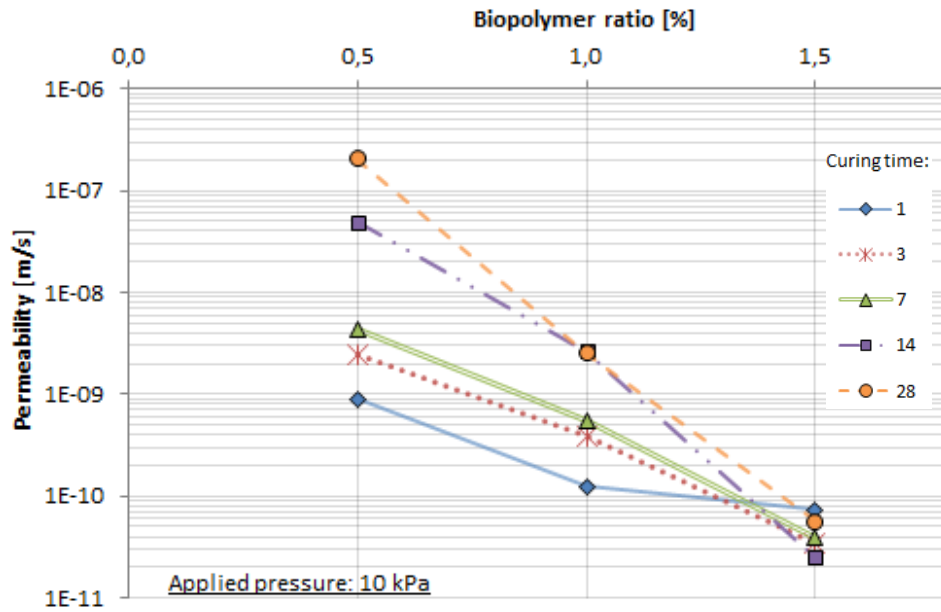


Figure 4.14. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 20 kPa.

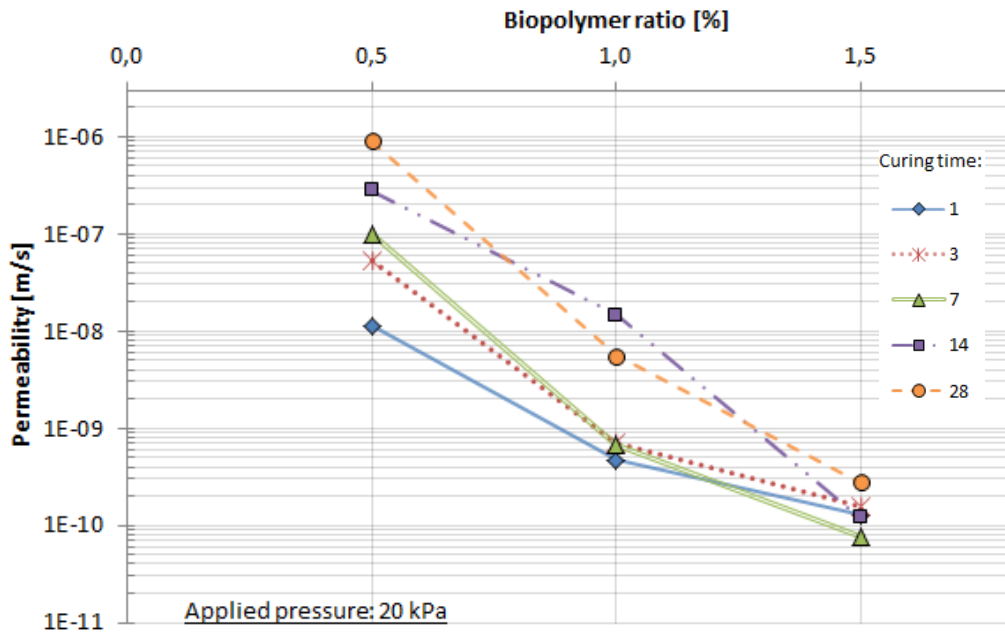


Figure 4.15. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 20 kPa.

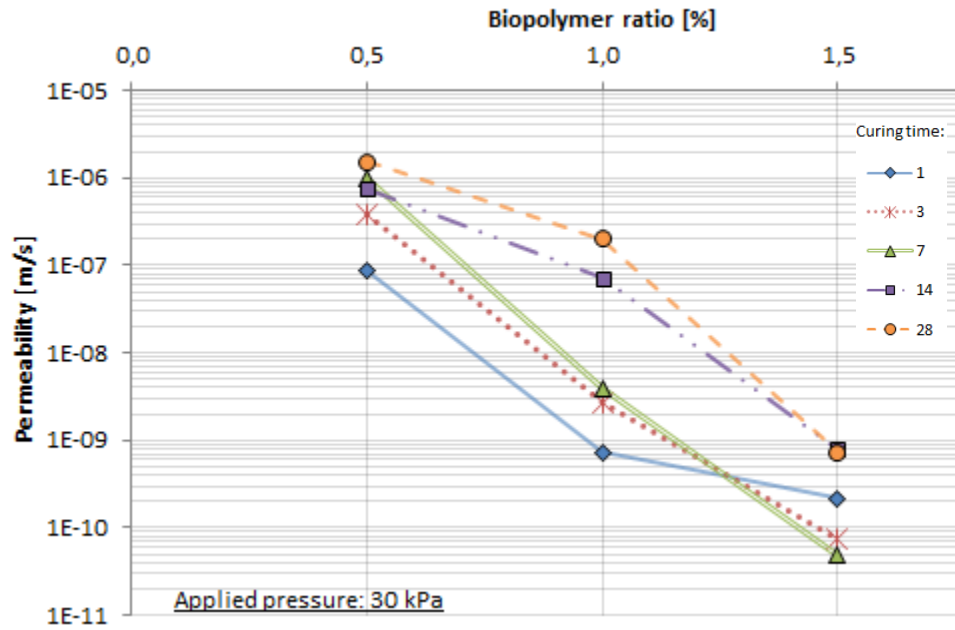


Figure 4.16. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 30 kPa.

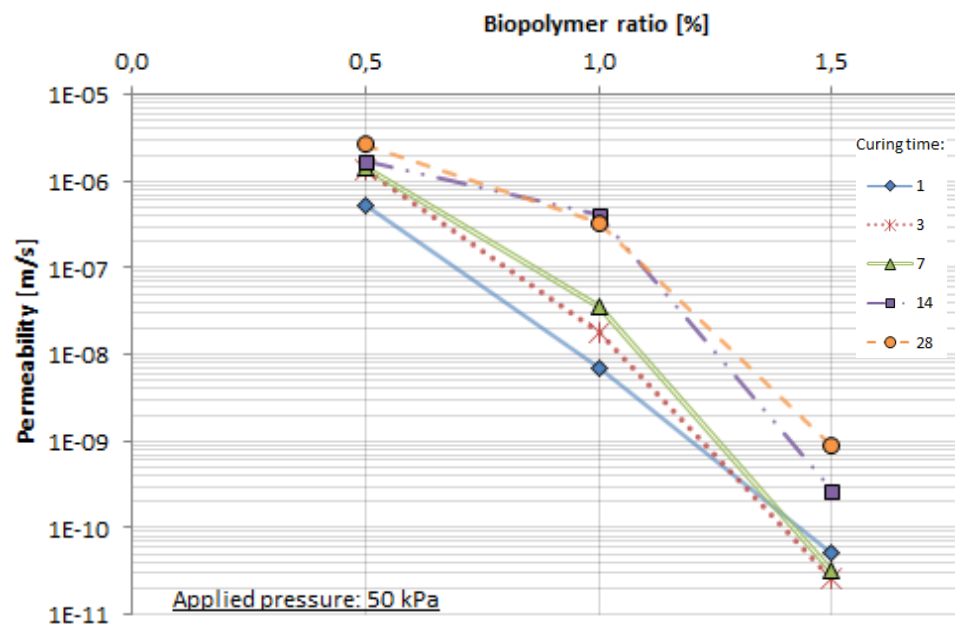


Figure 4.17. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 50 kPa.

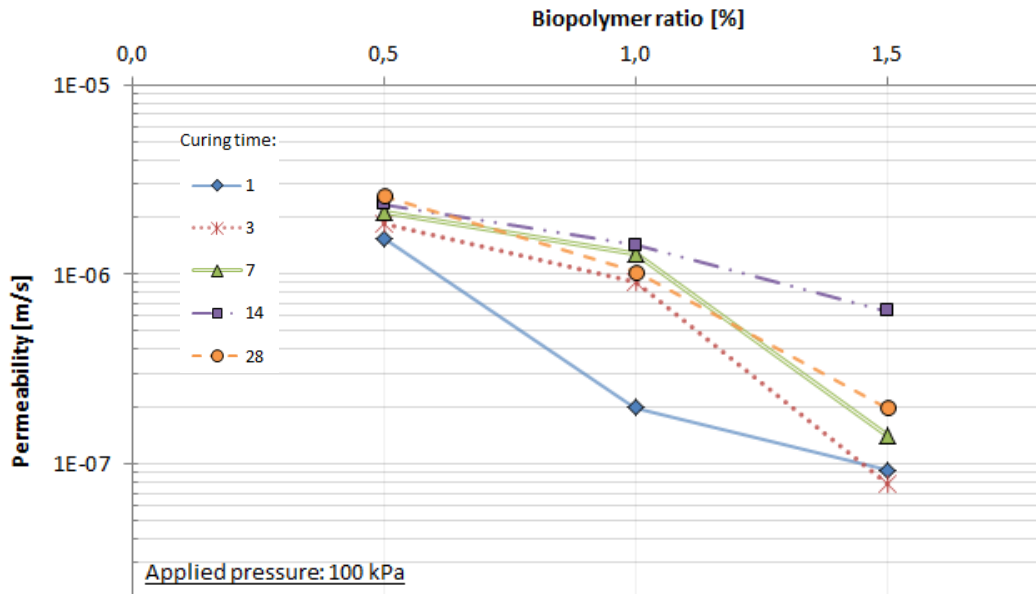


Figure 4.18. Effect of biopolymer content on hydraulic conductivity of the soil under the pressure of 100 kPa.

Viscous characteristics of xanthan gum have a significant meaning for stability of the soil. As observed in Figure 4.19., samples mixed with the xanthan gum appeared to be much more stable. As in the previous tests sand grains sticked to each other, and created a linked structure. That gives another conceivable way to use of such products in the ground, for instance in slope or road embankment stability.



Figure 4.19. Rigid samples after testing.

Great differences in laboratory test results for two sands come mainly from the grain size distribution type. Biopolymers are faster decomposed and being more washed out in coarse sand. As medium grained sand has higher density, particles better adhere to each other and it is easier to create a biopolymer-sand linked network. Another is the applied technology. When a triaxial cell was used and confining pressure applied, it prevented water to flow between the membrane enclosing sample and the sample's edge. However, for testing coarse sand, where a simplified technic was adopted, such a water flow was possible and uncontrolled. It means that the results for medium sand should be treated as more reliable.

4.2. Oedometer tests

In this part of experimental investigation one dimensional compression tests were conducted on sand and its mixtures with different biopolymer content. Samples were tested fully saturated with water and under various effective stresses applied. The effect of xanthan gum inclusion has been examined. Detailed results for each of four samples are presented in Tables 4.4.-4.7.

Table 4.4. Consolidation test results for sample with 0.0 % of biopolymer.

Ratio 0.0 %					
Load [kPa]	Total Settlement [mm]	Stage settlement [mm]	Void ratio e [-]	Coefficient of vol. compress. m_v [m²/MN]	Compress. index C_c [-]
1	-	-	0,611	-	-
25	0,283	0,283	0,588	0,5896	-
50	0,328	0,045	0,584	0,0913	0,0120
100	0,384	0,056	0,580	0,0569	0,0150
200	0,518	0,135	0,569	0,0683	0,0359
400	0,830	0,312	0,544	0,0801	0,0835
200	0,633	-0,197	0,560	0,0514	0,0527
100	0,531	-0,102	0,568	0,0527	0,0273
50	0,503	-0,028	0,570	0,0288	0,0075
25	0,473	-0,030	0,573	0,0615	0,0080

Table 4.5. Consolidation test results for sample with 0.5 % of biopolymer.

Ratio 0.5 %					
Load [kPa]	Total Settlement [mm]	Stage settlement [mm]	Void ratio e [-]	Coefficient of vol. compress. m_v [m²/MN]	Compress. index C_c [-]
1	-	-	0,611	-	-
25	0,350	0,350	0,583	0,7292	-
50	0,412	0,062	0,578	0,1262	0,0166
100	0,497	0,085	0,571	0,0868	0,0227
200	0,638	0,141	0,559	0,0723	0,0377
400	0,954	0,316	0,534	0,0816	0,0845
200	0,898	-0,056	0,539	0,0147	0,0150
100	0,823	-0,070	0,545	0,0393	0,0201
50	0,741	-0,082	0,551	0,0855	0,0219
25	0,693	-0,048	0,555	0,0997	0,0128

Table 4.6. Consolidation test results for sample with 1.0 % of biopolymer

Ratio 1.0 %					
Load [kPa]	Total Settlement [mm]	Stage settlement [mm]	Void ratio e [-]	Coefficient of vol. compress. m_v [m²/MN]	Compress. index C_c [-]
1	0	-	0,611	-	-
25	0,576	0,576	0,564	1,2000	0,0332
50	0,729	0,153	0,552	0,3151	0,0409
100	0,851	0,122	0,542	0,1266	0,0326
200	1,029	0,178	0,528	0,0930	0,0476
400	1,342	0,313	0,503	0,0825	0,0837
200	1,21	-0,132	0,513	0,0354	0,0353
100	1,131	-0,079	0,520	0,0420	0,0211
50	1,072	-0,059	0,525	0,0625	0,0158
25	1,013	-0,059	0,529	0,1247	0,0158

Table 4.7. Consolidation test results for sample with 1.5 % of biopolymer.

Ratio 1.5 %					
Load [kPa]	Total Settlement [mm]	Stage settlement [mm]	Void ratio e [-]	Coefficient of vol. compress. m_v [m²/MN]	Compress. index C_c [-]
1	-	-	0,611	-	-
25	0,679	0,679	0,556	1,4146	-
50	0,875	0,196	0,540	0,4058	0,0524
100	1,161	0,286	0,517	0,2991	0,0765
200	1,369	0,208	0,501	0,1104	0,0557
400	1,722	0,353	0,472	0,0947	0,0944
200	1,546	-0,176	0,486	0,0481	0,0471
100	1,441	-0,105	0,495	0,0569	0,0281
50	1,362	-0,079	0,501	0,0851	0,0211
25	1,320	-0,042	0,505	0,0901	0,0112

Variations of void ratio for different xanthan gum content are presented in Figures 4.20. and 4.21. respectively. First one presents complete history of the loading process, second skips the beginning (1st stage) of consolidation and focuses on more precise data. Tests were proceeded under the vertical stress increments of 25, 50, 100, 200 and 400 kPa and then for unloading respectively 200, 100, 50, 25, 0 kPa. As can be noticed, there are significant effects of the biopolymer inclusion on specimens compressibility. All samples were prepared at the same initial void ratio equal to 0.611, but after applying first pressure of 25 kPa, a meaningful differences in settlement occurred. First stage change in void ratio for pure sand reached 0.028, but for sand containing 1.5 % of xanthan gum it was 0.055, what is two times more. Detailed information about void ratio under various stresses applied for all biopolymer contents are also presented in Figure 4.22. It gives a proof that xanthan gum addition makes sand more compressive, where greater values of settlement may occur, what is shown in Figure 4.23.

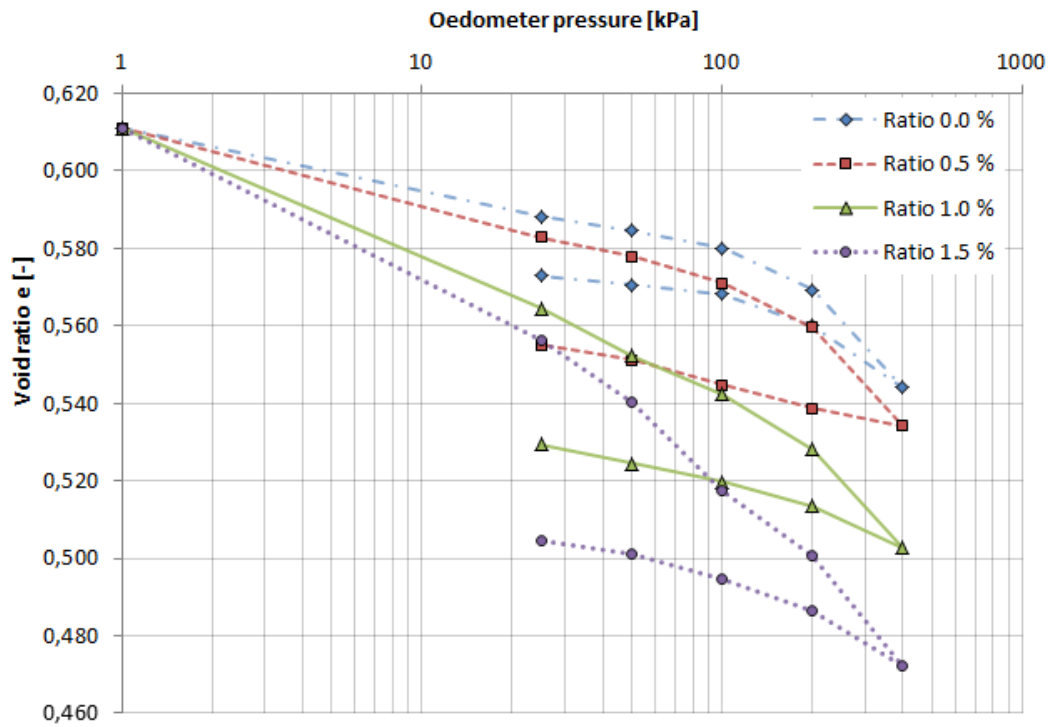


Figure 4.20. Variation of void ratio for different xanthan gum content.

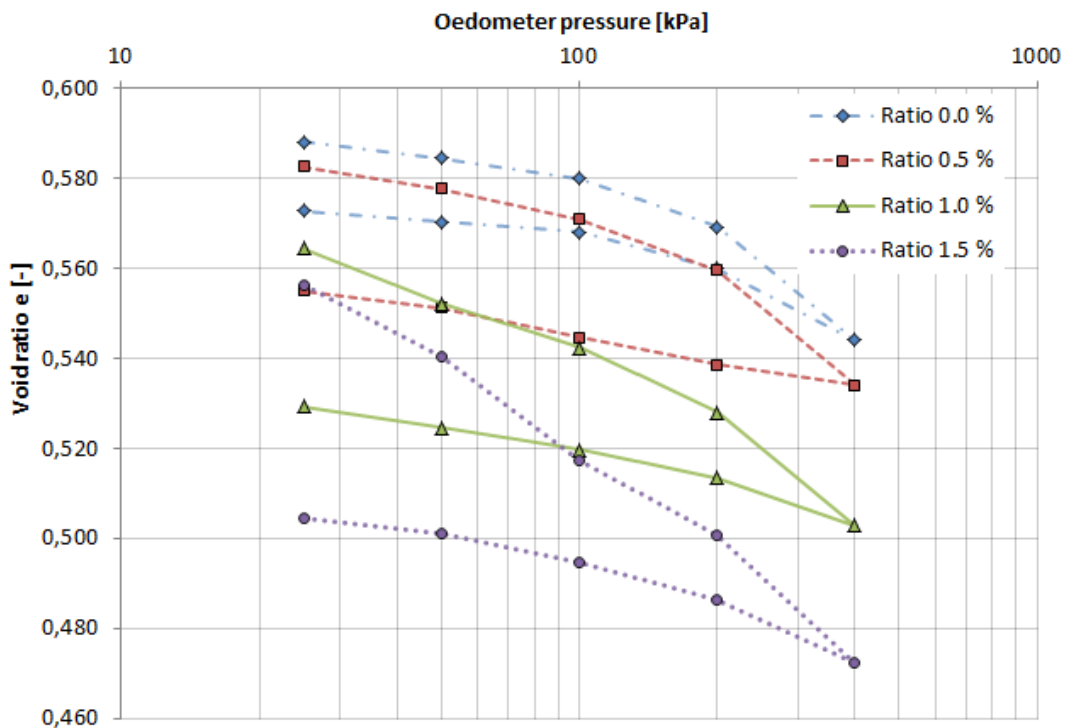


Figure 4.21. Variation of void ratio for different xanthan gum content.

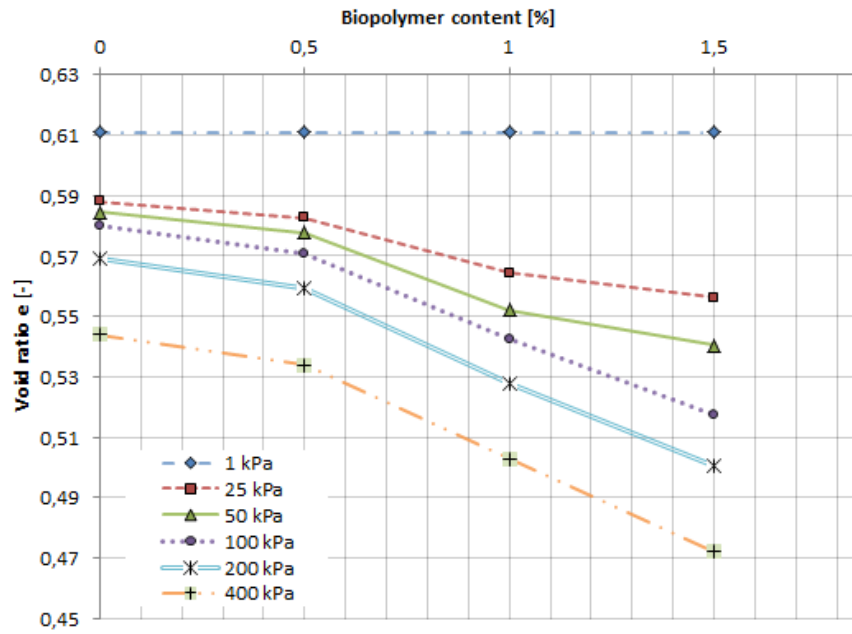


Figure 4.22. Void ratio variation for different biopolymer content.

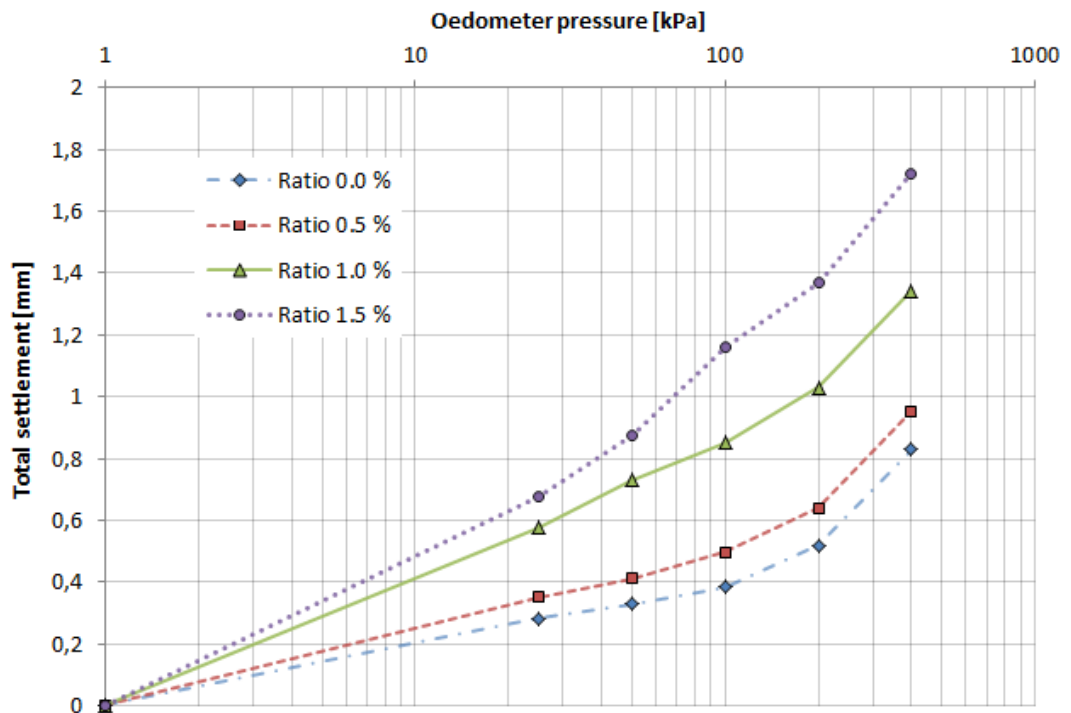


Figure 4.23. Total settlement of the sample for various biopolymer content.

The change in void ratio also gives an indication for differences in compressibility indexes C_c [-] that are presented in Figures 4.24. and 4.25. The greatest variations, according to the biopolymer percentage content are visible for low vertical stresses that are 0 – 100 kPa. For those pressures, value of compressibility index can increase its magnitude even seven times. However, for stresses reaching 200 – 400 kPa no significant difference in compressibility index was observed. Generally it might be assumed that xanthan gum inclusion increases compressibility of tested specimen.

Similar dependences were obtained for coefficient of compressibility m_v [m^2/MN], shown in Figure 4.26. The maximum effect of biopolymer addition is visible for 1.5 % xanthan gum – sand ratio when low stress was applied, 25 – 50 kPa.

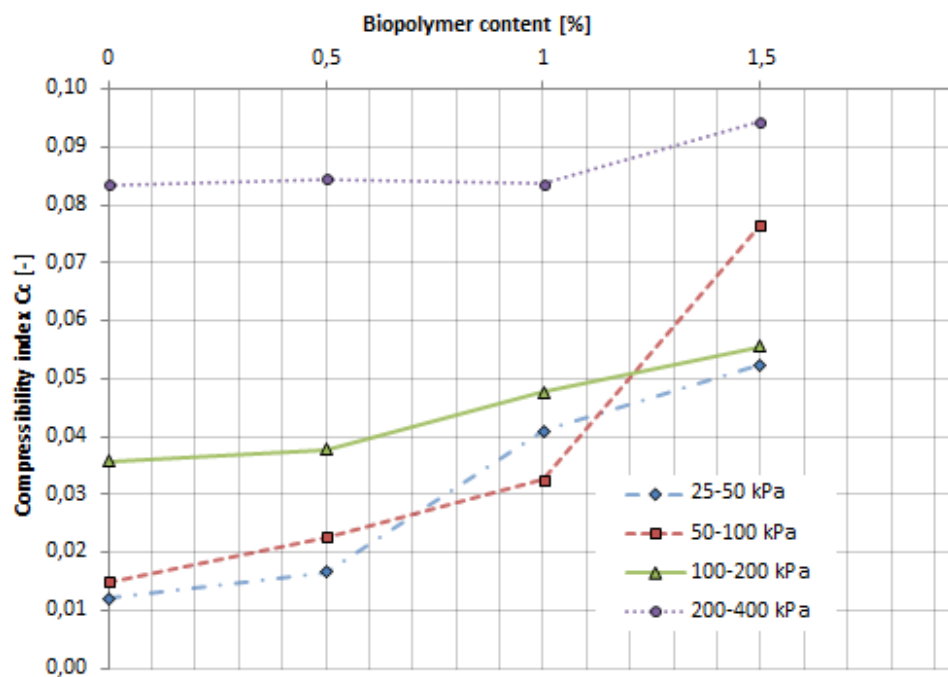


Figure 4.24. Compressibility index of the samples for different biopolymer content.

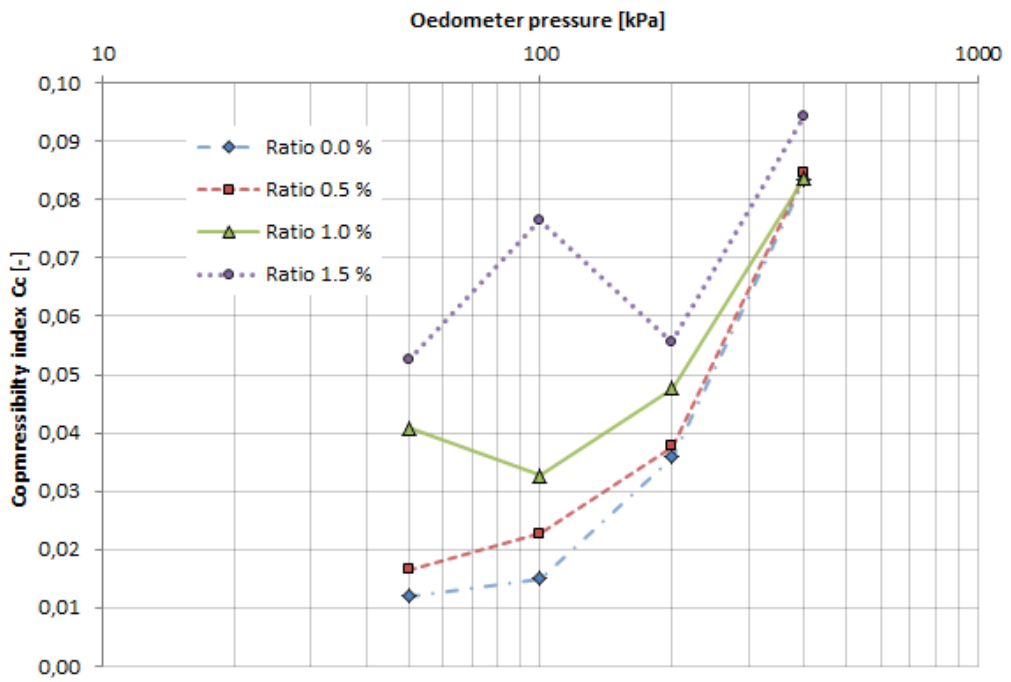


Figure 4.25. Compressibility index under different stresses applied.

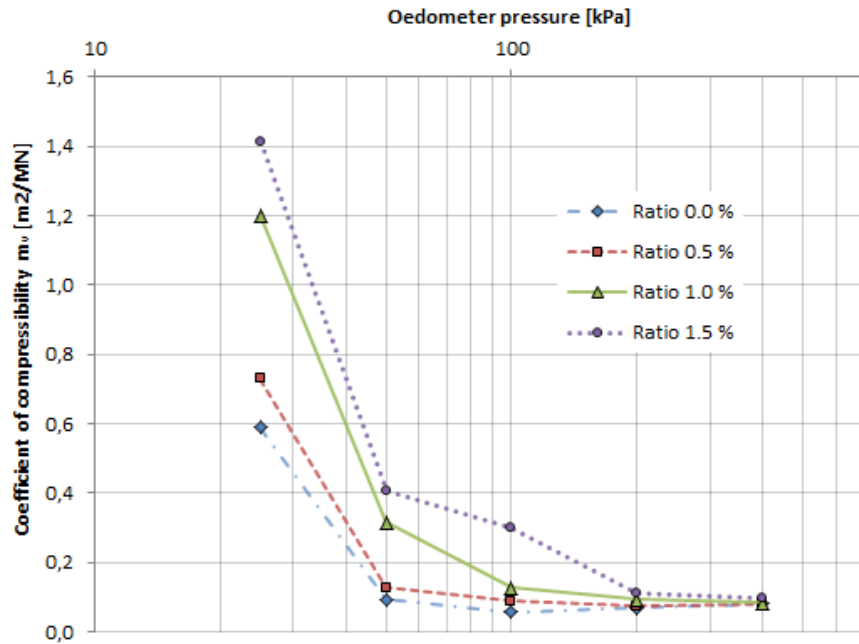


Figure 4.26. Coefficient of compressibility under different pressures applied.

After consolidation process was accomplished, unloading of applied stresses took place, in sequence 400 – 200 – 100 – 50 – 25 – 0 kPa. The values of total rebound are presented in Figure 4.27. It was noticed that addition of small amount of biopolymer decreases the rebound of specimen. For pure sand relaxation reached 0.36 mm, but for 0.5 % xanthan gum content only 0.26 mm. More elastic behavior of sample can be seen for greater amount of the bio substance (1.5 %), when rebound reaches value of 0.40.

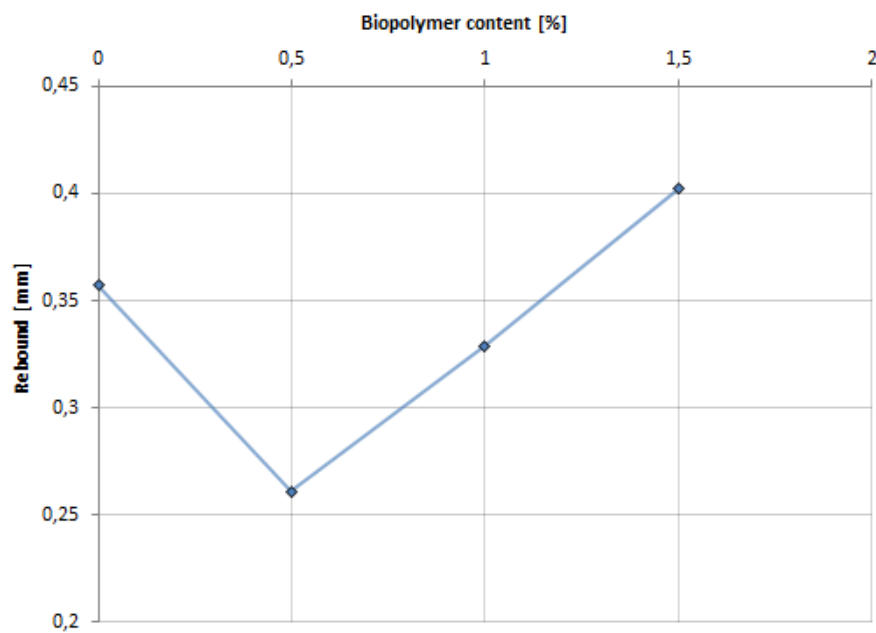


Figure 4.27. Specimen rebounds.

4.3. Triaxial tests

In this part of research the shear strength of biopolymer treated sand was investigated. For that reason a triaxial compression apparatus was used. Stresses applied to the samples are presented in Figure 4.28. In total 6 samples were tested, sandy soil was mix with various concentration of the biopolymer, namely xanthan gum. Two samples were prepared for each ratio, containing respectively 0.0, 0.5 and 1.5 %. Specimens were subjected to a consolidation process before testing, minor principal stress σ_3 (confining pressure) reached the value of 100 kPa for the first test, and 250 kPa for the second one. At the same time pore water pressure was equal to 50 kPa for all the tests. A series of triaxial tests were performed on the fully saturated specimens.

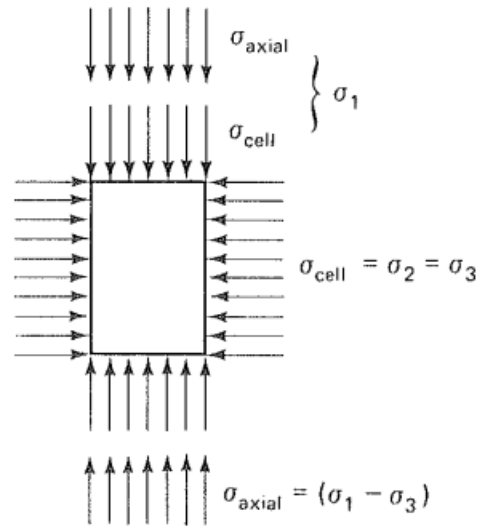


Figure 4.28. Stresses acting on the sample.

The data indicated that higher xanthan gum concentrations produced higher cohesion c for the treated sand. For the internal friction angle, its value increased for 0.5 % biopolymer addition but decreased for 1.5 % ratio, while cohesion for that sample reached nearly 100 kPa. Detailed test results are shown in Table 4.8.

The maximum deviatoric stress, mobilized for a given confining pressure, also increased at higher xanthan gum concentrations. It can be easily observed in Figure 4.29. for minor principal stress σ_3 equal to 100 kPa and Figure 4.30. for σ_3 equal to 250 kPa. The maximum deviatoric stress was calculated as follows:

$$\sigma = \sigma'_1 - \sigma'_3 \quad (4.1)$$

Where:

σ_1 – deviatoric stress [kPa],

σ'_1 – major effective stress [kPa],

σ'_3 – minor effective stress [kPa],

Its value for σ_3 equal to 100 kPa in the pure sand reached 118.5 kPa, while for the mixture containing only 0.5 % of biopolymer it was 198.2 kPa and for 1.5 % - 378.8 kPa. That estimates a great potential of the xanthan gum usage in the soil improvement industry, it shows that even a small amount the biopolymer can significantly change the strength parameter of the soil.

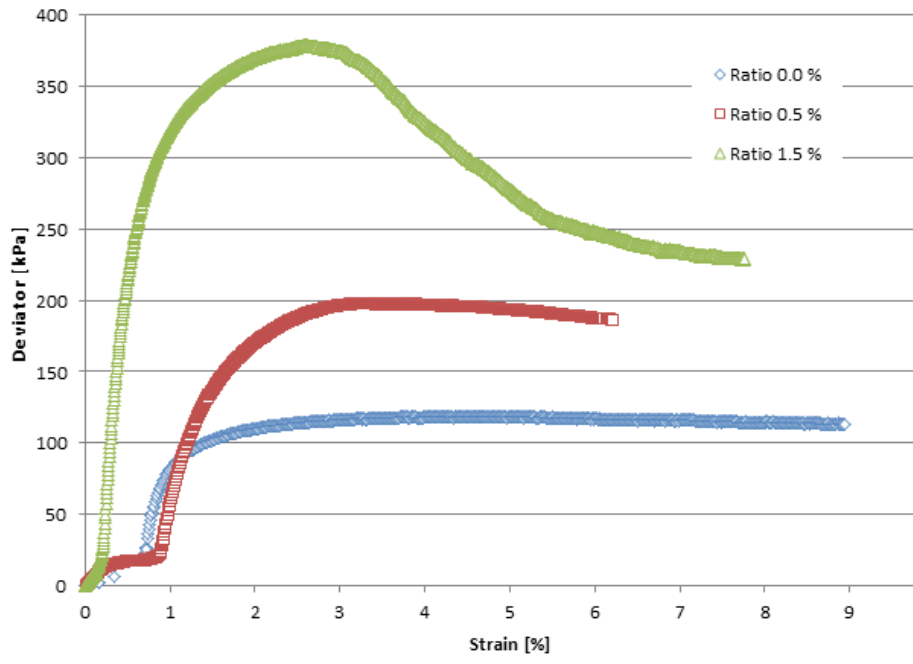


Figure 4.29. Deviatoric stress versus strain for confining pressure of 100 kPa.

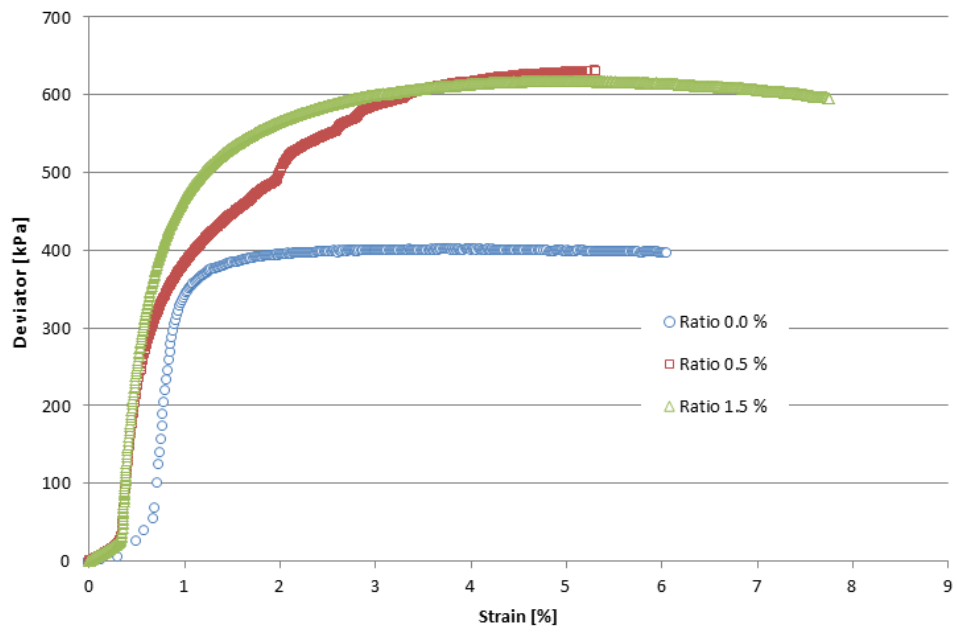


Figure 4.30. Deviatoric stress versus strain for confining pressure of 250 kPa.

Table 4.8. Strength parameters of a biopolymer treated sand.

Strength parameters	Biopolymer ratio		
	0.0	0.5	1.5
φ [°]	29,04	36,27	26,31
c [kPa]	7,14	13,58	92,95

In Figure 4.30. confining pressure was equal to 250 kPa and deviatoric stress reached 401.3 kPa for the pure sand and for specimens with the biopolymer inclusion, respectively for 0.5 % - 632.2 kPa and for 1.5 % - 617.5 kPa. That shows a meaningful influence of xanthan gum addition on the sample's strength parameters.

The calculated data was used for plotting a stress path graphs, they are shown in Figures 4.31. – 4.33. The stress paths were necessary to determine failure envelopes and the shear strength parameters (cohesion c and friction angle φ). Three graphs show the stress growth for each biopolymer – sand mixtures. Two lines represent the stress increment for various confining pressures, namely σ_3 equal to 100 and 250 kPa.

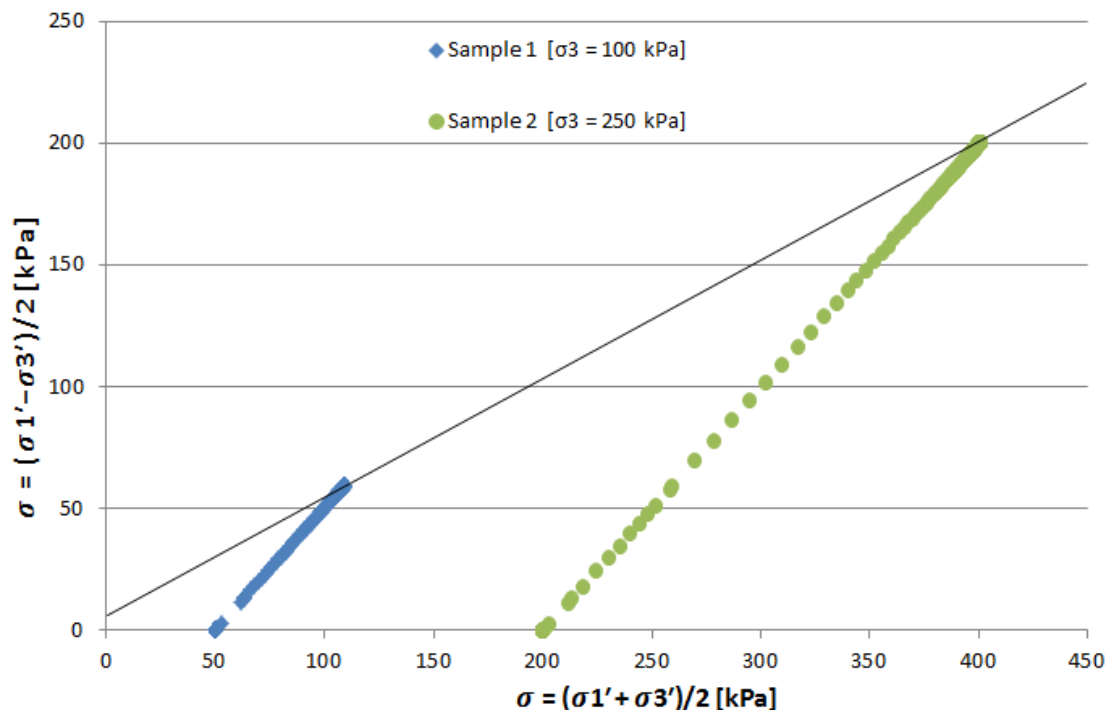


Figure 4.31. Stress paths for specimen with 0.0 % of the biopolymer.

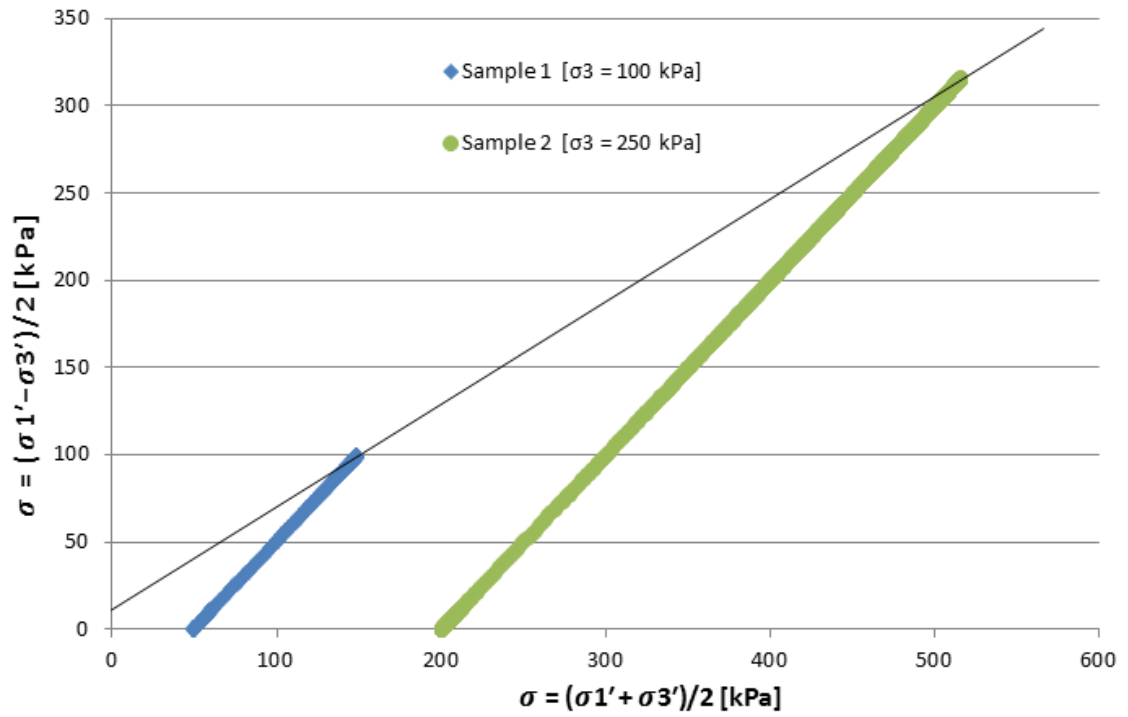


Figure 4.32. Stress paths for specimen with 0.5 % of the biopolymer.

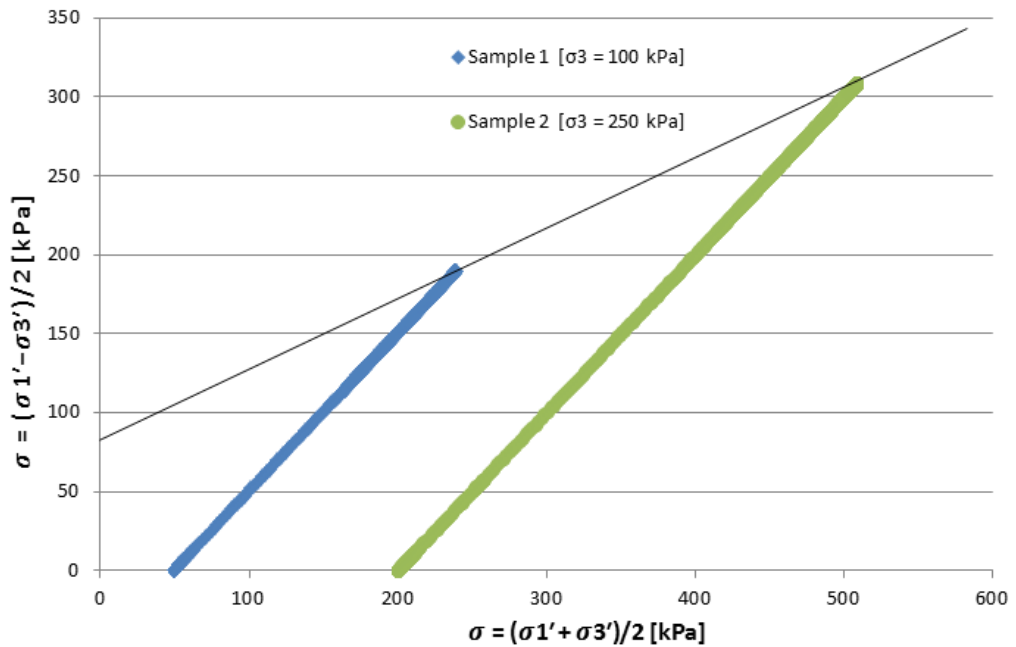


Figure 4.33. Stress paths for specimen with 1.5 % of the biopolymer.

The specimen treated with xanthan gum sheared along a distinct failure plane at a slightly lower axial strain. It is claimed that since biopolymer gel by itself behaves plastically, increasing its concentration introduces some degree of ductility to the treated soil matrix. Addition of 0.5% biopolymer had the effect of increasing brittleness.

The specimens failed in compression either by bursting into sand clusters, by forming a rough shear plane or an intermediate swelling (increasing the cross section area) state for sand treated with less biopolymer solution. Examples are shown in Figure 4.34.

Table 4.8. indicates that the cohesion intercept was directly proportional to the concentration of xanthan gum, for instance, by twofold for 0.5 % ratio and tenfold for 1.5 %. However, the biopolymer inclusion was found to produce a reduction in ϕ from 29° for the untreated sand to 26° for sand treated with 1.5 % xanthan gum solution. It is believed that the coating effect of the biopolymer on the grain surfaces has smoothed the micro-scale roughness, hence reducing the interlocking of the sand grains.



Figure 4.34. Samples after testing.

4.4. Unconfined compressive strength tests

Compression strength of biopolymer – sand mixture has been investigated and presented in this chapter. Fifteen samples were tested under one axial vertical loading. The effect of xanthan gum application was inspected. All specimens were dried for about two months before testing. A detailed research results are shown in Table 4.9.

Table 4.9. Unconfined compressive strength test results.

Curing time [days]	Biopolymer ratio [%]	Height [cm]	Cross-section area [cm ²]	Loading force [kN]	Compressive strength [MPa]
1	0,5	6,86	35,77	5,19	1,45
3	0,5	6,82	35,98	6,62	1,84
7	0,5	6,49	42,18	5,97	1,41
14	0,5	6,84	35,87	4,86	1,35
28	0,5	6,89	35,98	4,07	1,13
1	1	6,72	34,71	7,78	2,24
3	1	6,89	39,02	7,20	1,85
7	1	6,59	37,16	7,17	1,93
14	1	7,09	35,24	7,72	2,19
28	1	6,90	34,92	7,37	2,11
1	1,5	7,10	36,94	7,20	1,95
3	1,5	6,98	36,51	6,75	1,85
7	1,5	7,15	35,13	6,36	1,81
14	1,5	6,94	36,73	9,95	2,71
28	1,5	7,00	36,62	7,47	2,04

The compressive strength is the maximum stress that specimen can sustain, after it is exceeded the specimen may still have some load-carrying capacity which is called residual strength (Hudson and Harrison, 2007), a scheme is presented in Figure 4.35. In case of this study only the peak strength is considered for final result discussion.

Research results regarding mixtures containing respectively 0.5, 1.0 and 1.5 % of xanthan gum showed that biopolymer inclusion can significantly increase the compression strength of soil. Experimental studies demonstrate the compatibility of biopolymers. Depending on the bio substance concentration, the unconfined compressive strength of mixtures ranged from 1.13 to 2.71 MPa. Samples after the testing procedure are shown in Figure 4.38.

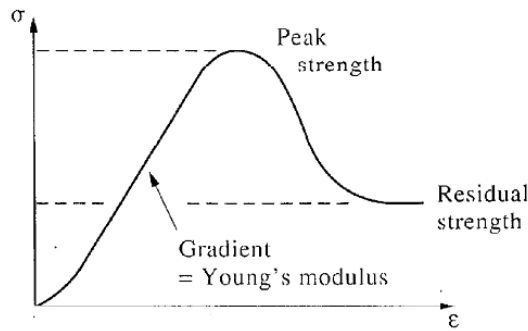


Figure 4.35. Stress – strain scheme (Hudson and Harrison, 2007)

A high compressive strength means that the sand can withstand more stress under allowable strain or deformation. Normally it is impossible to conduct unconfined compression test on sandy soil, due to its cohesionless. In this case biopolymer addition has completely changed the soil parameters, cementation and binding effect occurred. It shows that even 0.5 % xanthan gum inclusion can significantly enhance the soil mechanical properties. Mixtures containing respectively 1.0 and 1.5 % of biopolymer do not show big differences between each sample, except one specimen, which compressive strength reached the value of 2.71 MPa. Results depending on bio substance concentration are presented in Figure 4.36. However it must be noted, that all samples were completely dry while testing, the water addition might cause some decomposition of the particles or more elastic behavior. That case needs to be investigated.

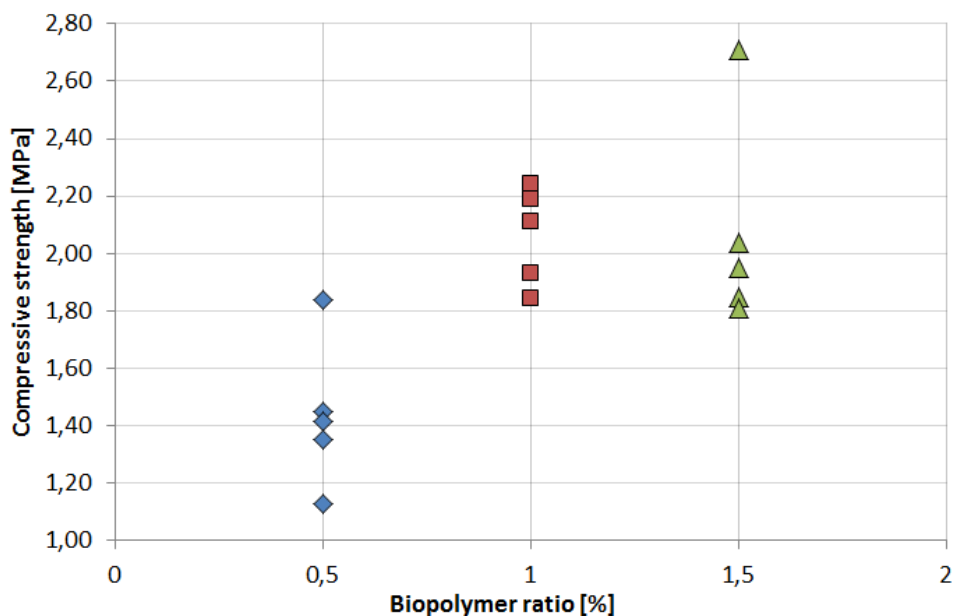


Figure 4.36. Compressive strength of mixtures versus biopolymer content.

The time effect on the strength of each sample was determined. It considers only the curing time, since the mixtures were prepared, sealed and left for conditioning until the hydraulic conductivity tests were conducted. After that all the samples were left for two months drying process. It shows a small time dependence only for 0.5 % ratio samples, others do not seem to be affected. All results are presented in Figure 4.37.

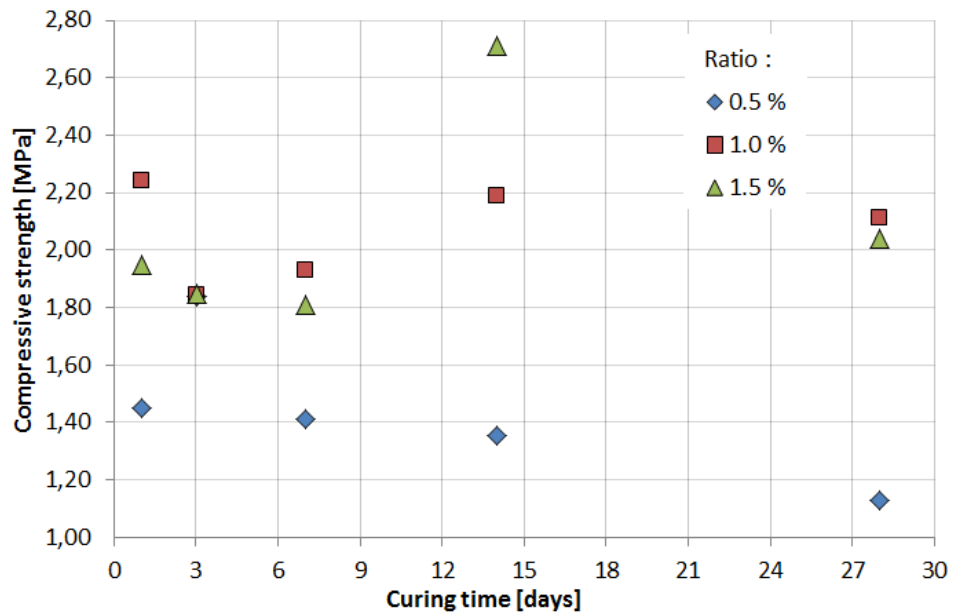


Figure 4.37. Curing time effect on biopolymer treated mixtures.



Figure 4.38. Samples after UCS test.

CHAPTER 5

SUMMARY AND CONCLUSIONS

The objective of the present study was to investigate the behavior of sandy soil and its various mixtures with xanthan gum in the terms of prospective ground improvement application. Some laboratory tests such as hydraulic conductivity, consolidation, triaxial, unconfined compressive strength were conducted. Depending on the findings of deep experimental program discussed in the previous chapters, the study leads to the following conclusions:

- ✓ The biopolymer can link together the individual soil particles within the soil matrix by cross-linking process. The amended soil has greatly reduced mobility, significantly reduced hydraulic conductivity, and compressive strength. It is this change in the physical form of the soil, on a particle level, that results in increased soil strength and decreased soil erosion;
- ✓ As the effect of the biopolymer inclusion, stable permeability decrease was observed;
- ✓ Permeability remains low and stable (10^{-11} m/s) for all 28 days of curing for high xanthan gum content in medium sand. However in coarse sand the effect of time is more visible, all samples after 28 days show permeability values similar to the clean sand ones;
- ✓ Great differences in laboratory test results for two sands come mainly from the grain size distribution type. Biopolymers are faster decomposed and being more washed out in coarse sand;

- ✓ The greatest variations, according to the biopolymer percentage content are visible for low vertical stresses that are 0 – 100 kPa. For those pressures, value of compressibility index can increase its magnitude even seven times;
- ✓ It was noticed that addition of small amount of biopolymer decreases the rebound of specimen. For pure sand relaxation reached 0.36 mm, but for 0.5 % xanthan gum content only 0.26 mm. More elastic behavior of sample can be seen for greater amount of the bio substance (1.5 %), when rebound reaches value of 0.40;
- ✓ The cohesion intercept was directly proportional to the concentration of xanthan gum, for instance, by twofold for 0.5 % ratio and tenfold for 1.5 %.
- ✓ The biopolymer inclusion was found to produce a reduction in φ from 29° for the untreated sand to 26° for sand treated with 1.5 % xanthan gum solution. It is believed that the coating effect of the biopolymer on the grain surfaces has smoothed the micro-scale roughness, hence reducing the interlocking of the sand grains;
- ✓ Normally it is impossible to conduct unconfined compression test on sandy soil, due to its cohesionless. In this case biopolymer addition has completely changed the soil parameters, cementation and binding effect occurred.
- ✓ Mixtures containing respectively 0.5, 1.0 and 1.5 % of xanthan gum showed that biopolymer inclusion can significantly increase the compression strength of soil. Depending on the bio substance concentration, the unconfined compressive strength of mixtures ranged from 1.13 to 2.71 MPa;
- ✓ It shows that even 0.5 % xanthan gum inclusion can significantly enhance the soil mechanical properties;
- ✓ The improvement in performance of sand treated with xanthan gum was found to be directly dependent on the biopolymer concentration;

- ✓ Biopolymers (i.e., xanthan gum) can substantially decrease hydraulic conductivity and improve the strength characteristics of sand without causing environmental toxicity;
- ✓ Biopolymer treatment occurs to be a promising tool to modify and engineer behavior soils. The eco-friendliness and cost of biopolymers also add to their attractiveness for use in engineering applications;
- ✓ However, further studies are needed for better understanding of the use of different biopolymers and percentages with various types of soils.

CHAPTER 6

FUTURE WORKS

The study recommends following points:

- ✓ A detailed observation of physical processes taking place in the soil after biopolymers inclusion; using a scanning electron microscope (SEM) would be very useful;
- ✓ Different ways of biopolymers application must be studied and compared to develop the most effective technique. The ways of mixing stabilizing agent with the soil (dry or wet mixing, higher temperature, extra additives, curing time);
- ✓ Long time in situ observation of a bio treated soil in order to determine its biological behavior (e.g. biopolymers are being consumed by other living organisms in the ground);
- ✓ Investigate possible addition of other stabilizing agents (e.g. cement or lime) to already biopolymer treated soil for further improvement, to achieve better results (prevent polymers from being washed out);
- ✓ More attention is required to understand differences in stabilized soil behavior when it is dry and when wet, also consequences of such a processes;
- ✓ Determination of most sufficient concentrations of biopolymers in the soil.

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