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Ground Improvement Techniques for Tunnel Excavation in Saturated Clay Medium: A Hydro-Mechanical Coupled Analysis of Drains and Capsule Grouting

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Alla mia famiglia.

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Abstract

This thesis investigates ground improvement techniques, focusing on the application of drains and the innovative capsule grouting technology (CGT) for tunnel excavation in saturated clay mediums. The excavation of an underground metro systems frequently encounters geo-materials of poor geotechnical properties, making it essential to improve the ground's strength and stiffness under certain conditions to support the excavation process.

Pre-excavation drainage of clayey layers is typically considered effective. It involves drilling boreholes and activating them for a specific duration. Special attention is directed towards the innovative capsule grouting technology (CGT), which involves injecting grout into capsules (or inflating pockets) made from a special membrane to create a localized densification, ensuring controlled grout distribution within heterogeneous ground conditions.

A comprehensive numerical modelling approach is employed using Plaxis 2D to simulate different scenarios and evaluate the performance of drains and capsule grouting in improving ground conditions. The advanced elasto-plastic constitutive model, specifically the Hardening Soil Model, was implemented to simulate realistic and accurate ground behaviour. The study introduces a transient hydro-mechanical coupled analysis that describes the interaction between the hydraulic and mechanical processes. Additionally, the proposed numerical modelling approach for capsule grouting involves a prescribed volumetric expansion of elements representing the grouted body. The results show significant improvements in isotropic effective stress, active pore water pressure, cohesion, and stiffness. Moreover, parametric analyses on soil permeability and grouting volumetric expansion provide further insight into the conditions that maximize the effectiveness of these ground improvement techniques.

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1 Introduction

Underground structures are increasingly being used in today's world for a variety of needs. Especially in developing cities, the demand for underground infrastructure is ever-growing. Underground structures offer several advantages over surface structures, especially in densely populated urban areas. Underground structures are an ideal solution in mega cities and nearby cities due to the lack of surface spaces. Additionally, underground structures benefit from natural isolation, which provide a protection from extreme temperatures since the temperatures within soil and rock are more moderate and uniform than those on the surface. Underground structures also protect them from severe weather conditions such as hurricanes, tornadoes, and thunderstorms. Moreover, transportation tunnels help to reduce surface noise pollution and enhance air quality by reducing the number of vehicles on streets. These advantages enhance the quality of urban life and environmental conditions.

Tunnelling in problematic ground conditions presents a significant challenge in civil engineering, where solutions for dealing with poor-quality ground are needed. Poor geotechnical conditions such as in saturated clays can lead to a stability problems and excessive deformations during and after excavation. Therefore, these conditions need the implementation of effective ground improvement techniques to improve the strength, stiffness, and permeability of the soil, ensuring the safety and stability of the tunnel structure. Ground improvement is essential for managing and mitigating the risks associated with excavations in poor soils.

The purpose of this thesis is to offer an organized framework for addressing ground challenges and proposing effective solutions.

C**hapter 1** provides the fundamentals of ground improvement, particularly for urban development. It addresses the difficulties posed by poor geotechnical conditions presents various methods for improving soil properties.

Chapter 2 talks about the characteristics of clay soils, focusing on their mechanical and hydraulic properties, as well as the different types of problematic clays and their potential problems.

Chapter 3 presents the process of soil consolidation and its importance for improving soil strength and stiffness, particularly in tunnelling projects.

Chapter 4 describes the grouting technique and its various methods. This chapter highlights the application of grouting techniques in tunnelling. It also explores Capsule Grouting Technology (CGT), including its benefits, equipment, and recent material developments aimed at improving grouting efficiency and control.

Chapter 5 presents the FE numerical modelling and its importance in solving complex geotechnical engineering problems. The hydro-mechanical coupled analysis, the hardening soil model, and the numerical approaches for simulating grouting processes are discussed.

Chapter 6 shows a practical application of drains and capsule grouting in a case study. It shows the results of numerical simulations that compare the effectiveness of using drains alone against a combination of drains and capsule grouting. It also provides insights from parametric analyses on factors such as soil permeability and grouting volumetric expansion.

2 Ground Improvement

2.1 Overview of Ground Improvement

As civilization and urbanization developed, there was a notable increase in demand for land for improved living and transportation. Consequently, several structures, including residential buildings, commercial buildings, high-rise office towers, transportation networks such as highways, railways, tunnels, as well as hydraulic structures like levees and earth dams, have been constructed and will continue to be built continuously. This trend is expected to remain steady or increase in the future. As ideal construction sites with favourable geotechnical characteristics become limited, the necessity to use unsuitable or less suitable sites for construction increases. Accordingly, as engineers seek solutions for dealing with a variety of problematic soil conditions, the need for treating poor-quality ground in urban areas grows.

The literature has employed a variety of terms to describe ground improvement, such as soil improvement, soil stabilization, ground treatment, and ground modification. The term "ground improvement" has been widely used in both academic literature and practical applications, thus making it the selected terminology for this thesis.

The strategies to deal with problematic geomaterials and geotechnical conditions include several options: (1) Avoiding the site (2) Designing alternative superstructures (3) Replacing problematic geomaterials with a proper geomaterials (4) Improving geomaterial properties and geotechnical conditions (Hausmann, 1990). For many projects, improving the geomaterials and geotechnical conditions is becoming more and more relevant. Ground improvement is mostly a practical discipline rather than a theoretical one. This is because it involves working with naturally occurring materials, such as rock and soil, over which we have no control during their formation and whose characteristics make them difficult to model using basic mathematics. As a result, it is crucial to recognize that before implementing a particular soil improvement technique, it is essential to carefully test it and assess its effectiveness. Although certain ground improvements can be executed based on the theoretical understanding of conventional soil mechanics.

Some improvements belong to the first order. For example, consolidation reduces the soil's void ratio. However, reducing the void ratio can result in second-order effects like increased strength and decreased compressibility. Finally, these second-order improvements can have third-order effects like improved stability, reduced settlement, and/or improved liquefaction resistance. Keep in mind that second- and third-order effects typically correspond to the desired results of implementing ground improvement methods. The methods and technologies are primarily used to improve the engineering properties of the ground, including shear strength, stiffness, and permeability. However, their main functions include increasing bearing capacity, controlling settlement, providing lateral stability, forming seepage cut-offs and environmental control, and increasing resistance to liquefaction.

2.2 Classification of Ground Improvement Methods

Several authors and organizations have proposed classifications for ground improvement methods according to different criteria, including (Mitchell, 1981) in his state-of-the-art report for soil improvement, Hausmann (1990), Ye et al. (1994), the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) TC17 committee (Chu et al., 2009), and the SHRP II R02 team led by Schaefer et al. (2012). These contributions are summarized in Table 1. Each classification method has its own benefits, as well as its limitations. The situation is due to the fact that some ground improvement methods can be assigned to one or more categories; this means assigning a specific ground improvement method to a specific category is imperfect because some methods are able to provide improvement by means of multiple principles. For example, stone columns can be applied to provide densification, replacement, drainage, and reinforcement; yet, in the majority of the applications, the main function of stone columns is replacement.

Moreover, this subdivision of interventions is mostly theoretical. In fact, in engineering practice, interventions are frequently combined, involving multiple actions and their resulting effects. For example, combining ground anchors with drains, which are typically used to provide stability to slopes or retaining structures.

Table 1: Classification of ground improvement methods (Han, 2015)

Reference	Criterion	Categories
(Mitchell, 1981)	Construction/function	1. In situ deep compaction of cohesionless soils 2. Precompression 3. Injection and grouting 4. Admixtures 5. Thermal 6. Reinforcement
(Hausmann, 1990)	Process	1. Mechanical modification 2. Hydraulic modification 3. Physical and chemical modification 4. Modification by inclusions and confinement
Ye et al. (1994)	Function	1. Replacement 2. Deep densification 3. Drainage and consolidation 4. Reinforcement 5. Thermal treatment 6. Chemical stabilization
ISSMGE TC17 (Chu et al., 2009)	Soil type and inclusions	1. Ground improvement without admixtures in noncohesive soils or fill materials 2. Ground improvement without admixtures in cohesive soils 3. Ground improvement with admixtures or inclusions 4. Ground improvement with grouting type admixtures 5. Earth reinforcement
(Schaefer et al., 2012)	Application	1. Earthwork construction 2. Densification cohesionless soils 3. Embankments over soft soils 4. Cutoff walls 5. Increased pavement performance 6. Sustainability

	7. Soft ground drainage and consolidation 8. Construction of vertical support elements 9. Lateral earth support 10. Liquefaction mitigation 11. Void filling

2.3 Methods of Ground Improvement

Understanding the diverse array of ground improvement methods is crucial for selecting the most suitable approach for a given project. Table 2 offers a detailed description of these methods, providing insight into their mechanisms, benefits, and typical applications. Consequently, this provides knowledge about the effectiveness of ground improvement techniques, empowering engineers to make informed decisions in geo-engineering field.

Table 2: General descriptions, benefits, and applications of most ground improvement methods (Han, 2015)

Category	Subcategory	Method and Level of Establishment	General Description	Benefit	Application
Densification	Shallow Compaction	Traditional compaction Level = 5	Apply static or vibratory load on ground surface in a certain number of passes to densify problematic geomaterial	Increase density, strength, and stiffness; reduce deformation, permeability, collapsible potential, and ground heave	Suitable for a wide range of fills to a lift thickness of 0.3m; used to compact fill
		High-energy impact roller compaction Level = 2	Apply a lifting and falling motion by a roller with high-energy impact on ground surface to densify or crush problematic geomaterial	Increase density, strength, and stiffness; reduce deformation, permeability, collapsible potential, and ground heave; crush rock and concrete into rubble	Suitable for a wide range of geomaterials to a depth of 2 m; used to improve subgrade and foundation soil and compact fill

		Rapid impact compaction Level = 2	Use an excavator to drop a weight repeatedly on ground surface to densify problematic geomaterial	Increase density, strength, and stiffness; reduce deformation, permeability, collapsible potential, and ground heave	Suitable for granular geomaterials up to 6 m deep; used to improve subgrade and foundation soil and compact fill
		Intelligent compaction Level = 2	Apply and adjust compaction energy based on on-board display from measurements in real time to densify problematic geomaterial	Increase density, strength and stiffness; reduce deformation, permeability, collapsible potential, and ground heave, identify areas of poor compaction, and maximize productivity	Suitable for granular geomaterials; used to improve subgrade and foundation soil and compact fill
	Deep Compaction	Dynamic compaction Level = 5	Drop a heavy weight from a high distance to apply high energy on ground surface, causing liquefaction of saturated problematic geomaterial and densification of unsaturated problematic geomaterial	Increase density, strength and stiffness; reduce deformation, liquefaction, collapsible potential to a greater depth	Suitable for granular geomaterials. collapsible soil. and waste material with less than 15% fines to a depth of 10 m; used to improve foundations
		Vibro-compaction Level = 5	Apply a vibratory force and/or water by a probe on surrounding problematic geomaterial, causing liquefaction and densification	Increase density, strength, and stiffness; reduce deformation, liquefaction, and collapsible potential to a greater depth	Suitable for clean sands with less than 15% silt or less than 2% clay to a typical depth of 5—15 m; used to improve foundations

Replacement	**Shallow Replacement**	Over-excavation and replacement Level = 5	Remove problematic geomaterial and replace with good quality geomaterial	Increase strength and stiffness; reduce deformation, liquefaction, collapsible, and ground heave potential	Suitable and economic for a wide range of geomaterials with limited area and limited depth (typically to 3 m deep and above groundwater table)
	Deep Replacement	Sand compaction columns Level = 5*	Displace problematic geomaterial by driving a casing into the ground and backfill the hole with sand (densified by vibration during casing withdrawal)	Increase bearing capacity and stability; reduce settlement and liquefaction potential; accelerate consolidation	Suitable for a Wide range of geomaterials to a typical depth of 5—15 m; used to improve foundations
		Stone columns Level = 5*	Jet water or air to remove or displace problematic geomaterial by a probe and backfill the hole with stone to form a densified column by vibration	Increase bearing capacity and stability; reduce settlement and liquefaction potential; accelerate consolidation	Suitable for a wide range of geomaterials (undrained shear strength > 15 KPa) to a typical depth of 5—10 m (up to 30 m); used to improve foundations
		Rammed aggregate columns Level = 4	Predrill a backfilled with aggregate. densified by ramming	Increase bearing capacity and stability: reduce settlement and liquefaction potential; accelerate consolidation	Suitable for a wide range of geomaterials to a typical depth of 5—10 m with a deep groundwater level; used to improve foundations

		Vibro-concrete columns Level = 3	Drive a vibrating probe to the ground to displace problematic geomaterial, replaced with concrete	Increase bearing capacity and stability; reduce settlement	Suitable and economic for very soft soil to a typical depth of 5— 10m; used to improve foundations
		Geosynthetic-encased columns Level = 2*	Drive a steel casing to the ground to displace problematic geomaterial. replaced with a geosynthetic casing and fill	Increase bearing capacity and stability; reduce settlement; accelerate consolidation	Suitable and economic for very soft soil (undrained shear strength < 15 KPa) to a typical depth of 5—10 m; used to improve foundations
Drainage, dewatering, and consolidation	Drainage	Fill drains Level = 5*	Place a layer of permeable fill inside a roadway or earth structure	Reduce water pressure and collapsible and ground heave potential: accelerate consolidation; increase strength, stiffness. stability	Suitable for low permeability geomaterial; used for roads, retaining walls, slopes, and landfills
		Drainage geosynthetics Level = 4	Place a layer of nonwoven geotextile or geo-composite in ground or inside a roadway or earth structure	Reduce water pressure and collapsible and ground heave potential; accelerate consolidation; increase strength, stiffness. stability	Suitable for low permeability geomaterial; used for roads, retaining walls, slopes, and landfills
	Dewatering	Open pumping Level = 5	Use sumps, trenches, and pumps to remove a small amount of water inflow in open excavation	Remove water to ease construction	Suitable for a small area, relatively impermeable soil, and lowering of the groundwater table by a limited depth in open excavation

		Well system Level = 4	Use well points and/or deep wells to remove a large amount of water inflow in open excavation	Remove water to ease construction and increase stability of excavation	Suitable for a large area, relatively permeable soil, and lowering of the groundwater table by a large depth for excavation
		Electro osmosis Level = 2	Create electric gradients in soil by installing anode and cathode to induce water flow and collect and discharge the water by a cathode well point	Remove water to ease construction	Suitable for relatively impermeable silt or clayey soil
	Consolidation	Fill preloading Level = 5	Apply temporary surcharge on ground surface for a duration and then remove the surcharge for construction	Increase soil strength; reduce settlement	Suitable for saturated inorganic clay and silt; used to reduce settlement for foundation soil
		Vacuum preloading Level = 3	Apply vacuum pressure on ground surface and/or through drains into the ground for a desired duration and then remove the pressure for construction	Increase soil strength; reduce settlement	Suitable for saturated inorganic clay and silt; used to reduce settlement for foundation soil
Chemical stabilization	**Shallow stabilization**	Chemical stabilization of subgrade and base Level = 5	Mix lime, cement, and/or fly ash with subgrade and base course in field and then compact the mixture; have chemical reaction with soil particles to form a cementitious matrix	Increase strength and stiffness; reduce ground heave potential	Suitable for unsaturated clay and silt; mainly used for roadway construction with a typical lift thickness of 0.3 m or less
	Deep stabilization	Grouting Level = 3	Inject grout into ground to fill voids, densify soil, and have chemical reaction with soil particles to form a hardened mass	Increase strength and stiffness; reduce permeability, liquefaction, and ground heave potential	Different grout suitable for different geomaterial; mainly used for remedying measures or protective projects

		Jet grouting Level = 4	Inject high-pressure cement-based fluid into ground to cut and then mix with geomaterial to form a hardened column by chemical reaction with soil particles	Increase strength, stiffness, and stability; reduce permeability, liquefaction, and ground heave potential	Suitable for a wide range of geomaterials; mainly used for remedying measures and protective projects to a typical depth of 30 m or less
		Deep mixing Level = 4*	Mix cement or lime from surface to depth with geomaterial by mechanical blade to have chemical reaction with soil particles after mixed to form a Cementitious matrix	Increase strength, stiffness, and stability; reduce permeability, liquefaction, and ground heave potential	Suitable for a wide range of geomaterials; mainly used for foundation support, earth retaining during excavation, containment, and liquefaction mitigation
Reinforcement	**Fill reinforcement**	Geosynthetic-reinforced slopes Level = 5	Place geosynthetics in slope at different elevations during fill placement to provide tensile resistance	Increase stability	Suitable for low plasticity fill; mainly used for slope stability
		Geosynthetic-reinforced embankments Level = 5	Place high strength geosynthetic at base of embankment to provide tensile resistance	Increase bearing capacity and stability	Suitable for embankments over soft foundation; mainly used for enhancing embankment stability
		Geosynthetic-reinforced column-supported embankments Level = 3	Place geosynthetic reinforcement over columns at base of embankment to support embankment load between columns	Reduce total and differential settlements; accelerate construction; Increase stability	Suitable for embankments over soft foundation with strict settlement requirement and time constraint

		Mechanically stabilized earth walls Level = 5	Place geosynthetic or metallic reinforcements in wall at different elevations during fill placement to provide tensile resistance	Increase stability	Suitable for low plasticity free-draining fill
		Geosynthetic-reinforced foundations Level = 3*	Place geosynthetic reinforcements within fill under a footing to provide load support	Increase bearing capacity and reduce settlement	Suitable and economic for granular fill over soft soil with limited area and depth
		Geosynthetic-reinforced roads Level = 4	Place geosynthetic reinforcement on top of subgrade or within base course to provide lateral constraint	Increase bearing capacity and roadway life; reduce deformation and base thickness requirement	Suitable for granular bases over soft subgrade
	In-situ ground reinforcement	Ground anchors Level = 4*	Insert steel tendons with grout at end in existing ground to provide tensile resistance and prevent ground movement	Increase stability and resistance to uplift force	Suitable for granular soil or rock; used for temporary and permanent slopes and walls during excavation and substructures subjected to uplift force
		Soil nails Level = 4	Insert a steel bar with grout throughout the whole nail in existing ground to provide tensile resistance and prevent ground movement	Increase stability	Suitable for low plasticity stiff to hard clay, dense granular soil, and rock; used for temporary and permanent slopes and walls during excavation
		Micropiles Level = 4	Insert a steel reinforcing bar in a bored hole, grout in place to form a small diameter pile	Increase stability; protect existing, structures during ground movement	Suitable for a variety of geomaterials; used for slopes, walls. and unpinning of

			(< 0.3 m) and provide vertical and lateral load capacities		existing foundations
Thermal and biological treatment		Ground freezing Level = 2	Remove heat from ground to reduce soil temperature below freezing point and tum geomaterial into solid	Increase strength; reduce water flow and ground movement	Suitable for saturated clay and sand; used for temporary protection during excavation
		Biological treating Level = 1	Utilize vegetation and roots to Increase shear strength of soil or change soil properties by bio-mediated geochemical process, including mineral precipitation, gas generation, biofilm formation. and biopolymer generation	Increase strength and stiffness; reduce erodibility and liquefaction potential	Suitable for cohesive and cohesionless geomaterials; requires more research and field trial before it is adopted in practice

"Level of technology establishment: rating scale I = not established, 3 = averagely established, and = well established (most of the ratings are based on the recommendations by the SHRP Il R02 02-am; however. some ratings with an asterisk * are adjusted or added from the international perspective and the author's judgment).

2.4 Criteria for Selecting Ground Improvement Methods

When selecting a ground improvement method, many factors must be studied. Technical criteria such as stability and settlement are typically of primary concern for an engineer. However, in addition to these, other criteria may also affect the selection of a ground improvement method. Some of the criteria that are expected to be enforced by the client or project requirements can be summarized as follows (Ameratunga et al., 2021):

- Cost, including both capital and maintenance expenses
- Timing for ground improvement.
- Availability of material.
- Impact on adjacent infrastructure.

- Environmental conditions.
- Access and utilization of particular machinery.

Cost, including both capital and maintenance expenses: In the event of a road embankment is designed to allow for settlement corrections every five years rather than for the entire 50-year design life, the capital cost may be low (for example, lower surcharge), but the maintenance cost will probably to be high because corrections (for example, an asphalt correction) may need to be performed on a regular basis.

Timing for ground improvement: Preloading is an economical method for improving soft soil; however, it does require time for the soil to consolidate. Utilizing prefabricated vertical drains can significantly speed up the consolidation process.

Availability of material: In the event that a site is characterized by highly unstable soil conditions, a significant surcharge is needed to improve the ground. Large berms would be required for stability during the work. Additionally, if the site is remote and borrow materials aren't frequent, high-strength geotextiles can be employed to reduce the earth volume required.

Impact on adjacent infrastructure: If a service line nearby to a road widening project may be affected by lateral movement. If surcharging is used, the evaluated lateral movements are considered to be significant. On the other hand, if a piled embankment is used, the differential settlement between the old and new embankments is considered a serious problem. To deal with such concerns, an ultra-lightweight alternative (such as a polystyrene block) is likely to be recommended.

Environmental conditions: If the proposed development is a warehouse facility located on a site with deep soft clays. Surcharging using wick drains has been suggested to be the preferable approach due to the cost difference compared to deep piles. However, additional soil investigations at the final design stage revealed the presence of contaminants inside a portion of the footprint. The wick drain option is ruled out due to environmental considerations. A piled construction or ground improvement with semi-rigid inclusions looks to be the most appropriate solution for the site.

Access and utilization of particular machinery: If a road alignment is adjacent to and closely parallel to a river, ground improvement is necessary due to the site's deep soft clays. Surcharging with wick drains is the most cost-effective approach; however, stability concerns arise regarding the wick drain machine. While a piled embankment is feasible, the piling machine generates heavy loads. Nevertheless, stability challenges could be mitigated through a progressive construction process, utilizing previously finished piles as a working platform.

2.5 Trends and Developments in Ground Improvement

There have been several recent advances in ground improvement techniques. Manufacturers have made important contributions to these advances through continuous innovation and improvements in equipment. Meanwhile, researchers have contributed to improving design methodologies.

A few general trends for future development in ground improvement methods are listed below (Han, 2015):

- Use of combined technologies to develop solutions that are both technically and cost-effective
- Use of intelligent construction technologies integrated with sensors and computer monitoring to enhance both the efficiency and quality of ground improvement processes
- Use of recycled materials and other alternative materials to maximize sustainability in ground improvement techniques.
- Use of specifications that prioritize the end result or performance.
- Use of biological treatment

3 Clay Geomaterial

3.1 Introduction to Clay Soil

Clays, abundantly found on Earth and recently discovered on Mars, capture attention for their combination of widespread availability and extraordinary properties. The JNC has defined 'clay' as 'a naturally occurring material composed primarily of fine-grained minerals, which is generally plastic at appropriate water contents and will harden when dried or fired' (Stephen Guggenheim & R. T. Martin, 1995). Clays are highly heterogeneous in composition and almost invariably exhibit varying mechanical properties. Dry clay presents itself as a solid material; however, upon the addition of water, it softens until reaching a liquid state. It's evident that dry (solid) clay and very wet (liquid) clay will demonstrate different mechanical behaviours. In broad terms, greater clay content within soil contributes to greater plasticity, greater potential to swelling and shrinkage, reduced hydraulic conductivity, greater compressibility, greater cohesion, and a reduced internal angle of friction. Table 3 lists problematic geomaterials including soft clays and their potential problems. These natural and fill geomaterials are the focus of ground improvement efforts.

Apart from problematic geomaterials like soft clay, geotechnical issues might arise from problematic conditions caused by either natural phenomena or human interventions. Natural phenomena refer to geological, hydraulic, and climatic factors, including earthquakes, cavities, floods, wind, and freeze-thaw cycles. Human interventions, particularly the erection of superstructures, substructures, and earth structures, can modify geotechnical conditions, potentially leading to problems in projects such as excavation, tunnelling, swift removal of surface water, and construction of dams.

In engineering applications involving sediments, there is a differentiation between fine-grained or cohesive soils and coarse-grained or cohesionless soils. Within the field of soil mechanics, cohesive and cohesionless soils show significant differences in terms of their consistency and strength. Cohesive soils containing fine-grained materials like clay and silt exhibit significant deformation under load or stress, resulting in a higher settlement. Due to the slow settlement process in cohesive soils, damages may manifest long after construction completion. The soil mechanical behaviour of cohesive soils is particularly linked to changes in water content, affecting not only significant deformations but also significant changes in volume. Conversely, cohesionless soil, also known as non-cohesive soil, consists of coarse-grained granular material comprising sand and gravel particles. Typically, it serves as a stable building surface, exhibiting minimal dependence on water content. With low compressibility, settlement in cohesionless soil is often quite low and occurs immediately upon loading. Moreover, granular soils may exhibit some cohesive properties when containing small amounts of clay mineral particles (5–40%).

Table 3: Problematic Geomaterials and Potential Problems (Han, 2015)

Type of Geomaterial	**Name**	**Potential Problems**
Natural	Soft clay	Low strength, high compressibility, large creep deformation, low permeability
	Silt	Low strength, high compressibility, high liquefaction potential, low permeability, high erodibility
	Organic soil	High compressibility, large creep deformation,
	Loose sand	Low strength, high compressibility, high liquefaction potential, high permeability, high erodibility
	Expansive soil	Large volume change
	Loess	Large volume change, high collapsible potential
Fill	Uncontrolled fill	Low strength, high compressibility, nonuniformity, high collapsible potential
	Dredged material	High water content, low strength, high compressibility
	Reclaimed fill	High water content, low strength, high compressibility
	Recycled material	Nonuniformity, high variability of properties
	Solid waste	Low strength, high compressibility, nonuniformity, high degradation potential
	Bio-based by-product	Low strength, high compressibility, high degradation potential

3.2 Mechanical and Hydraulic Properties of Clays

3.2.1 Consistency and Plasticity

In engineering contexts, consistency stands out as the primary characteristic defining the behaviour of cohesive soils, directly influenced by their water content. It describes the soil's strength and resistance to penetration under in situ conditions. The Atterberg limits, including shrinkage, plastic, and liquid limits, along with the plasticity index, are used to describe the response of cohesive soils to stress. To determine the Atterberg limits of a soil sample, laboratory tests such as the Atterberg limit test, including the liquid limit and plastic limit tests, where the plasticity index is simply the difference between the liquid and plastic limits. However, a high plasticity index typically corresponds to low shear strength, with higher levels of plasticity correlating to increased shrinkage (Bergaya et al., 2013). Additionally, A high liquid limit usually refers to high compressibility and a greater potential for swelling /shrinkage. Moreover, Atterberg limits and plasticity are often useful for classifying fine cohesive soils, as they help to determine the amount of silt and clay in a fine-grained soil. Typically, plasticity tends to increase with clay content.

3.2.2 Strength and Stiffness

Soil strength is basically determined by cohesion and internal friction. Cohesion arises from the adhesion of clay particles induced by electromagnetic forces (Figure 1). It typically occurs in clays and plastic silts. On the other hand, friction is a physical process developed through the friction between soil particles (Figure 2). It typically occurs in sands and non-plastic silts.

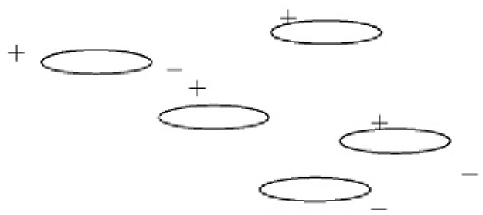


Figure 1: Clay Particles (Bergaya et al., 2013)

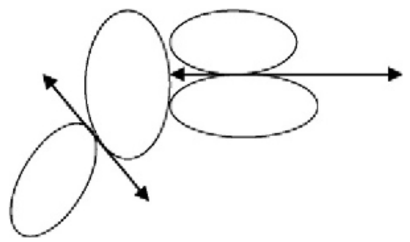


Figure 2: Friction in sand particles (Bergaya et al., 2013)

It's essential to delve into another critical aspect: the shear strength. Shear strength indicates the soil's capability to resist failure when subjected to loads generating shear stresses within it. Furthermore, shear strength can be improved through the application of ground improvement techniques that reduce the void ratio and/or introducing a cohesive (cementing) element. Mohr and Coulomb formulated a failure criterion that establishes a relationship between shear stress (τ) and normal stress (σ) on a plane within a soil mass. It is expressed as: $\tau = c' + \sigma' \cdot \tan(\varphi')$, where τ stands for the effective shear stress on the plane, c' stands for the effective cohesion (drained), σ' stands for the effective normal stress on the plane, φ' stands for the effective internal friction angle. Keep in mind that cohesive soils, like clay or those with a significant clay content, possess high effective cohesion but a low effective friction angle. Conversely, granular soils, like gravel and sand, which contain minimal or no clay, possess no effective cohesion ($c' = 0$) and high effective friction angle. Table 4 presents the average shear strength parameters for cohesive soils.

Water is another key element, as shear strength typically decreases with increased water content. Therefore, shear strength depends greatly on whether soil deformation takes place under fully drained conditions, undrained conditions, or intermediate drainage conditions. In every scenario, different excess pore pressures will develop, leading to different effective stresses and strengths.

The undrained shear strength Cu is frequently employed in soil mechanics to describe the strength of cohesive soils, can be classified according to Table 5.

Table 4: Average Values of Shear Strength Parameters for Cohesive and Organic Soils (Helmut Prinz, 2006)

	φ (°)	**C (KN/m²)**	**C_u (KN/m²)**
Low cohesive soils	25 - 27.5	0 - 5	0 – 40
High cohesive soils	15 – 25	10 – 25	20 – 100
Organic soils	5 - 15	0 – 5	5 – 20

Table 5: Classification of Undrained Shear Strength (Helmut Prinz, 2006)

Very low	< 20
Low	20 – 40
Medium	40 – 75
High	75 - 150
Very high	> 150

Furthermore, soil stiffness is the extent of the deformation of soil resulting from the application of a load. The stiffness of cohesive soil can be improved through compaction and consolidation, while granular soil stiffness is typically increased by densification. Additionally, increasing cohesiveness through methods such as soil mixing or grouting can improve the stiffness of both cohesive and granular soils .

3.2.3 Permeability

Permeability is a fundamental parameter used in Darcy's law, which describes the flow of fluids through porous media. In clays, permeability is related to the non-hydrated free water space, typically resulting in low values. The measured permeability for natural clays generally falls within the range of 1 x 10^-8 to 1 x 10^-10 m/s. In contrast, permeability in sandy soils can vary based on factors such as particle size, fines content, and grain-size distribution. Table 6 shows the range of permeability for various soil types and testing. In the majority of cases, improved ground refers to ground that has been modified to create a zone of reduced permeability, with the aim to control the negative effects of groundwater.

Frequently, construction projects involve work below ground level, and often below the groundwater table.

Table 6: Permeability Values, Classification and Tests (B. J. Barends, 2011)

K [m/s]	1	10^{-2}	10^{-4}	10^{-6}	10^{-8}	10^{-10}
Drainage	Good			Moderate	Poor	
Class	Large	Moderate	Small	Very small	Relatively impervious	
Soil type	Boulders	Gravel	Clean sand	Silty sand	Clay	
Test		Pumping test, constant head test		Falling head test, oedometer		
			Monopole or dipole cone		Piezometer	
Correlation	Grain size distribution			Density	Clay index	

3.2.4 Swelling and Shrinkage

Volume changes in clay can be due either to a swelling or to a process of shrinkage. The extent to which a clay soil swells or shrinks depends on various factors, including its mineral composition, grain-size distribution and plasticity. Smectite and vermiculite show more volume changes during wetting and drying compared to kaolinite and illite (Bergaya et al., 2013). The higher the plasticity of the clay, the greater the likelihood of swelling and shrinkage.

Shrinkage issues with clays may result from water absorption by deep tree roots, drainage, evaporation or other factors leading to a loss in water content. This phenomenon is particularly prevalent in regions where the clay content exceeds 25%, the plasticity index exceeds 30%, and the groundwater level is relatively deep. Additionally, clays can also exhibit a reverse swelling ability. When the volume increase caused by swelling gets obstructed, it leads to the development of swelling pressure. The expected maximum swell heave is typically between 1% and 5% of the clay layer thickness, with a swell pressure of 0.2 to 2.0 MPa. If swell heave is restricted to just 1% to 2%, it can be controlled with small loading. Another method to stabilize swelling clay soils involves treating them with additives like lime, cement, fly ash, and gypsum.

3.3 Clays with Problematic Mechanical Behaviour

3.3.1 Expansive or Swelling Clays

Expansive clays contain minerals such as smectites, which possess the ability to absorb water (Bergaya et al., 2013). This absorption leads to an increase in volume, with greater water absorption resulting in greater increase in volume. Expansions of 10% or greater are frequently observed. Additionally, upon drying, expansive clays may contract, potentially leading to damaging subsidence. Fissures may form in the soil, allowing water to infiltrate deeply. This creates a cycle of shrinkage and swelling, imposing repetitive stress on structures.

3.3.2 Quick Clays

Quick clay is a specific type of sensitive clay whose structure totally collapses when remoulded or disturbed, leading to a sudden decrease in shear strength to nearly zero. It's characterized as clay having a sensitivity of 50 or higher and a fully remoulded shear strength of a value below 0.4 kPa. Additionally, quick clays are primarily composed of non-swelling clay minerals with low activity, characterized by liquid limits lower than their natural water contents (Bergaya et al., 2013). It has been found that the ratio of water content to liquid limit in quick clays typically exceeds 1. When the void ratio in the undisturbed clay is high, the water content could exceed the liquid limit by a significantly, causing the remoulded clay to behave as a liquid.

3.3.3 Black Shales

Clay of this type contains a high organic content or is deposited under anoxic conditions, resulting in swelling regardless of its mineral composition. However, this swelling is caused by the oxidation of pyrite, a chemical reaction commonly observed in black shales (Bergaya et al., 2013). Swelling occurs in black shales when reactions like oxidation, hydration, or carbonation involving particular constituent minerals produce by-products that cause volumes much larger than the original minerals.

Predicting or simulating these reactions during exploration and design is challenging. Hence, it may be preferable to avoid locating structures on pyrite-bearing carbonaceous shales, given that these rocks are the most prevalent hosts of swelling induced by chemical reactions.

4 Consolidation Process

4.1 Introduction to Consolidation

A well-established fact is that reducing the water content in soil can significantly improve its shear strength and stiffness, which is why a prevalent approach for improving soft soil involves reducing its water content through consolidation for example. The significance of consolidation becomes evident when addressing saturated and low permeable soils, particularly fine-grained ones such as clays, as their small pores restrict the flow of water.

Consolidation refers to the gradual compression and deformation of soil when subjected to increased stress or load. Upon application of compressive loading, both the total stress and porewater pressure increase. As time progresses, the excess porewater pressure dissipates, leading to an increase in effective stress. During consolidation, a variety of ground improvements happens, including increased cohesion and stiffness, along with decreased permeability, due to the densification of clay particles and expulsion of water. These processes strengthen the soil's resistance.

Although several ground improvement methods are available for improving the strength and stiffness of clay soils, consolidation is typically the most cost-effective solution for saturated clays.

In the context of tunnelling, low-permeability grounds respond to tunnel excavation in a time-dependent manner. In the short-term, the water content of the ground surrounding the tunnel remains the same, but excess pore water pressures generate and gradually dissipate over time, leading to soil consolidation and additional deformations. In terms of stability, two conditions can be defined: the short-term condition, characterized by undrained shear, and the long-term condition, characterized by drained shear. Advance drainage is an effective technique that can be implemented to improve short-term stability by relieving pore pressure. This increases the effective stresses around the tunnel profile, thus increasing the short-term shear strength of the ground.

4.2 Speeding up the Consolidation with Drains

Theoretically, the time required for 100% consolidation is infinite. However, practical design doesn't necessitate aiming for a complete consolidation, but instead, targets typically range around 95% and 98%. Furthermore, Soft clays commonly have low hydraulic conductivities and given that the rate of consolidation depends on soil permeability, the consolidation time for soft clays might exceed the designated timeframe in the construction schedule. In these situations, consolidation process can be accelerated by installing drains.

Drains aim to speed up the expel of excess pore water pressure by shortening the drainage path length and, particularly, modifying the direction of drainage flow, thus accelerating the consolidation process (Figure 3). The Installation of drains is usually uncomplicated and often economical. Traditionally, sand drains were utilized to speed up the consolidation process. However, prefabricated vertical drains (PVDs) are now the preferred option. Typical installations consisted of drains with diameters ranging from 200 to 450 millimetres.

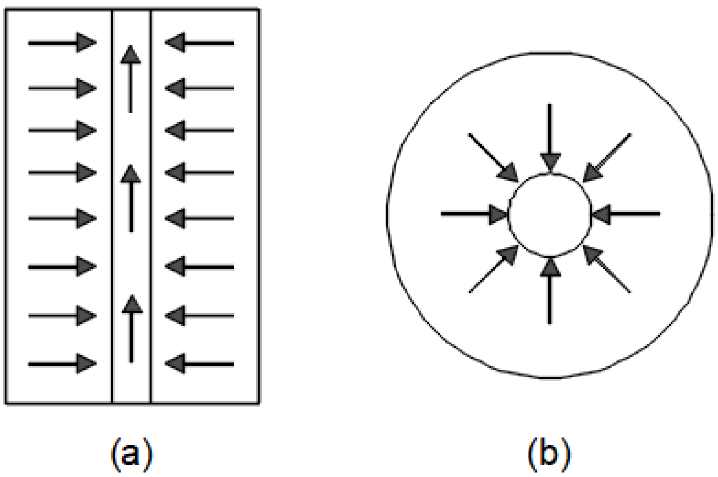


Figure 3: (a) Plane Strain (b) Axisymmetric

4.3 Consolidation Methods

The theory of consolidation holds significant importance in studying deformations within porous materials, particularly within the fields of soil and rock mechanics.

Terzaghi (1925) originally formulated the theory of consolidation while investigating the delayed deformation resulting from the slow expulsion of water through the pores of a low-permeability material subjected to compressive loading. In the scenario of one-dimensional consolidation, he formulated a mathematical representation of the phenomenon by combining Darcy's law, which governs fluid flow through porous media, with his own concept of effective stress. He noticed that deformations in soft soils, like clay, are induced by effective stresses, which are defined as the difference between total stress and pore pressure. However, the application of this theory may encounter limitations in effectiveness due to simplifications made regarding assumed soil behaviour under load, particularly when addressing the use of a linear stress-strain relationship to describe the soil's response to the load. Despite many developments in this field, Terzaghi's one-dimensional consolidation theory remains the most commonly utilized theoretical framework for describing this process.

Biot (1941) provided a logical extension of Terzaghi's one-dimensional consolidation theory to three-dimensional situation and a wider range of materials, such as porous rock. Since then, it has been utilized in addressing various practical contexts. Following that, De Josselin de Jong (1956) and Biot (1956) developed a further generalization to include dynamic problems.

Since analytical solutions for most boundary conditions and more sophisticated soil behaviour are generally not achievable due to the complexity of the differential equations, many researchers rely on numerical methods. Among the preferred numerical techniques for addressing consolidation problem is the finite element method, credited for its ability to handle very complicated situations. Consequently, computer-based FEM models now offer more realistic models of soil behaviour, including plastic deformations and creep, along with the capability to solve complex geometries.

4.4 Improvement of the Effective Stress

According to Anagnostou (Anagnostou, 2018), advance drainage can reduce pore pressure by $\Delta p = \alpha p_0$, where p_0 represents the in situ pore pressure and α represents the effectiveness of the drainage ($\alpha = 1$ for perfect drainage; $\alpha = 0$ for no drainage). The value of α is influenced by the time available for the drainage process, ground permeability, storativity, and the geometric layout of the drains. The relief of pore pressure by Δp leads to an increase in pre-excavation effective stresses, as expressed by the following formula:

$$\Delta\sigma' = \frac{\Delta p}{2(1-\nu)} = \frac{\alpha p_0}{2(1-\nu)}$$

4.5 Improvement of the Mechanical Properties

The expulsion of water during consolidation allows clay particles to rearrange and pack more closely, forming a denser structure. Consequently, this process facilitates the strengthening particle bonds, resulting in cohesion increase. However, undrained cohesion (Cu) can be estimated through various tests such as unconfined compression test, triaxial test, vane shear test (in laboratory or field), or by using a pocket penetrometer. Alternatively, estimation of Cu can be achieved through trigonometric expressions and empirical correlations.

The occurrence of drainage results in an increase in σ0'. Assuming the theoretical case of elastic perfectly plastic ground behaviour without dilatancy, the short-term strength Cu linearly increases with the isotropic (mean) effective stress σ0' according to the following formula (Anagnostou & Zingg, 2013), which is derived from trigonometric considerations.

$$C_u = \sigma'_0(t_D)\sin\varphi' + C'\cos\varphi'$$

Where $\sigma_0'(t_D)$ denotes the effective isotropic (mean) stress at a time t_D after drain activation and ϕ' and c' denote the effective angle of internal friction and the effective ground cohesion, respectively. Meanwhile, evaluating short-term stability involves considering total stresses and is based on the undrained shear strength Cu, assuming a friction angle of zero.

4.6 Improvement of the Groundwater Permeability

During the consolidation process, saturated clays undergo significant changes, notably in their permeability. The expulsion of water reduces the pore pressure within the clay, leading to a decrease in void ratio and an increase in effective stress. Consequently, the permeability of saturated clay soils decreases during consolidation. As a result, the consolidated clay becomes less permeable. (Taylor, 1948) proposed the following empirical linear relationship to describe the variation in permeability of clays during consolidation.

$$k = k_0 \, . \, 10^{\left[\frac{-\,(e_0 - e)}{C_k}\right]}$$

Where k_0 represents the permeability at the initial void ratio e_0, k represents the permeability at any void ratio e, and C_k represents the permeability change index, which is prescribed to be either $0.5e_0$ (Tavenas et al., 1983) or equal to C_c. This form of relationship has become a typical way of representing the variation in permeability with void ratio.

5 Grouting Technique

5.1 Overview of Grouting

Grouting is a term with a wide range of definitions, employed across various industries and performed through different techniques for many applications. In the field of ground improvement, it is broadly defined as injecting a semi-fluid material into soil or rock formations to modify their original geotechnical properties. The semi-liquid material, known as grout, can be formed of various mixtures of substances, which vary according to the objectives of the grouting program and the present conditions at the site and subsurface. The most common type consists of a mixture of cement and water, often supplemented with additives to improve its properties. Also, grouts can be formed from foams, organic resins, solutions (e.g., silicate-based mixtures), and other mixtures. Although grouting is conceptually straightforward, but its implementation, monitoring, and verification in civil and environmental applications can be fairly complex.

5.2 Grouting in Different Mediums

For grouting in soil or loose materials, manchette tubes (TAMs) are essential as they define the grouted body and allow for the injection of the required volume at the specified location, with controlling over pressure and quantity.

Grouting in rock is generally performed in compact to jointed solid rock by injecting a cement suspension through unlined holes or using simple packers with suitable pass lengths.

5.3 Common Grouting Methods and Their Practical Uses

A variety of methods have been developed, each distinguished by its unique approach and applications. Among the five most common types are permeation grouting, compaction grouting, hydro-fracture grouting, compensation grouting, and jet grouting (Figure 4). Each Method is accompanied by its respective definition (British Standards Institute, 2020):

(1) Permeation grouting: a non-displacement grouting method which aims to infiltrate the natural voids, typically filled with water or gas, within a porous medium with grout material. This process is usually conducted at relatively low pressure. It's important to note that permeation grouting is not suitable for fine-grained soils.

(2) Compaction grouting is a displacement grouting method that involves injecting a stiff and viscous grout (mortar) into the soil to create bulbs or lenses. This process displaces and compacts the surrounding soil without fracturing it. It is typically used to densify loose soils.

(3) Hydro-fracture grouting: a displacement grouting method that involves injecting grout under pressure exceeding the local tensile strength and confining pressure of the soil mass, causing it to fracture and forcing the grout to fill these fractures. This method is used to increase the density of plastically deformable materials and to expand the volume of the treated mass when the plastic deformation limit is reached.

(4) Compensation grouting: a particular application of hydro-fracture grouting that, along with precise displacement monitoring, compensates for ground settlements that occur during underground activities, such as tunnelling and excavation, or corrects previous ground settlements.

(5) Jet grouting: a grouting method that involves injecting a high-velocity jet of grout (sometimes with water and/or air) into the soil to create hardened columns or walls. This process erodes and mixes the existing soil with grout, resulting in a denser, more stable soil mass.

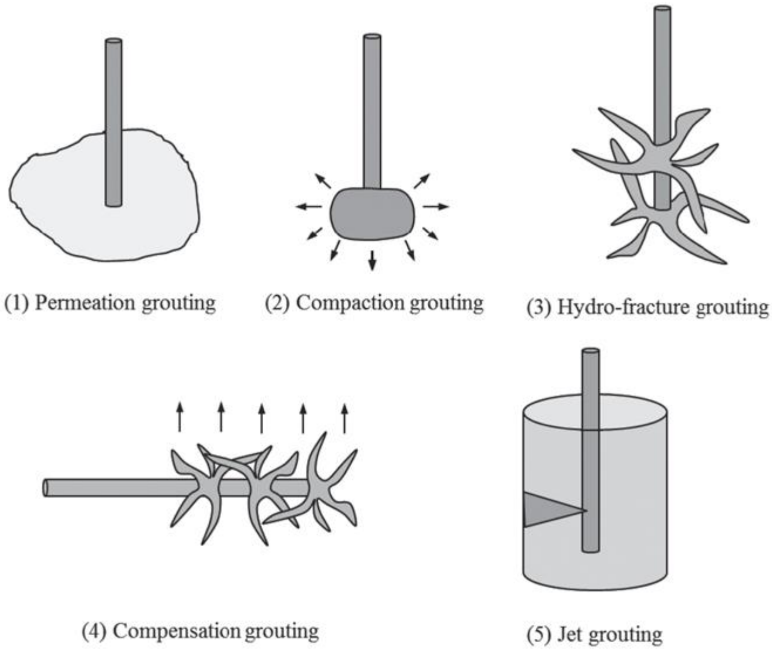


Figure 4: Types of Grouting (Han, 2015)

5.4 Primary Objectives of Grouting

Injecting grout material into soil or rock formations serves several objectives, including (Han, 2015):

- Densification to prevent settlement and reduce liquefaction risks
- Reduction of permeability and control of water
- Increasing granular soil cohesion through soil solidification
- Reduction of clay soil expansion
- Providing additional support to structures already in place

5.5 Grouting in the Context of Tunnelling

Grouting technology represents a specialized aspect of modern tunnelling, offering interventions that are readily available whenever a tunnel faces challenges such as problematic ground conditions, fault zones, or significant water inflow (Kogler, 2013). In recent years, the trend in tunnel construction has featuring significant infrastructure projects facing different conditions and problematic geological conditions. Making the correct selection of a grouting process, involving drilling process, grouting methods, and necessary specialized machinery, can be a critical factor in ensuring both the technical and economic success of a construction project.

5.5.1 Grouting Geometry in Tunnelling

According to the geometric needs, grouting can be provided in four forms (Figure 5):

- Canopy or screen grouting (1)
- Canopy grouting with face support (2)
- Temporary support ring (3)
- Compensation body (4)

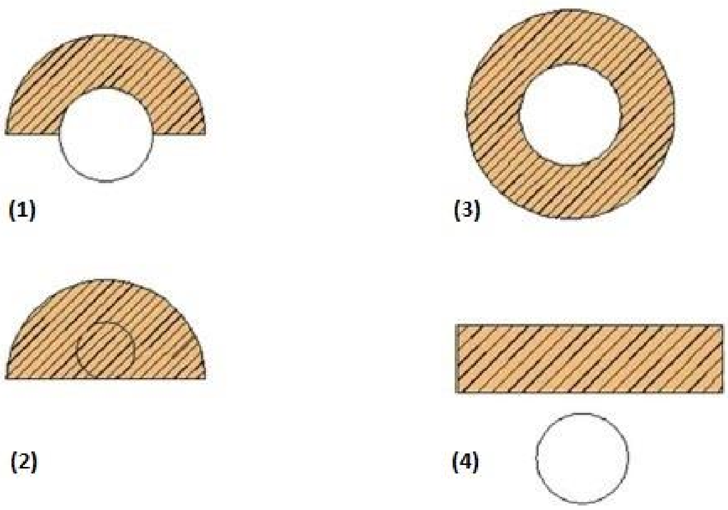


Figure 5: Example of Grouting Profiles (Kogler, 2013)

5.5.2 Grouting Execution in Tunnelling

Grouting execution requires an understanding of site conditions, combined with strategic planning and execution. By considering geometric specifications, structural factors, and accessibility constraints, engineers can optimize the grouting process to achieve the desired results efficiently and effectively. Grouting can be carried out (Figure 6):

- From above ground (1)
- Underground (2 and 3)
- From shafts (4)

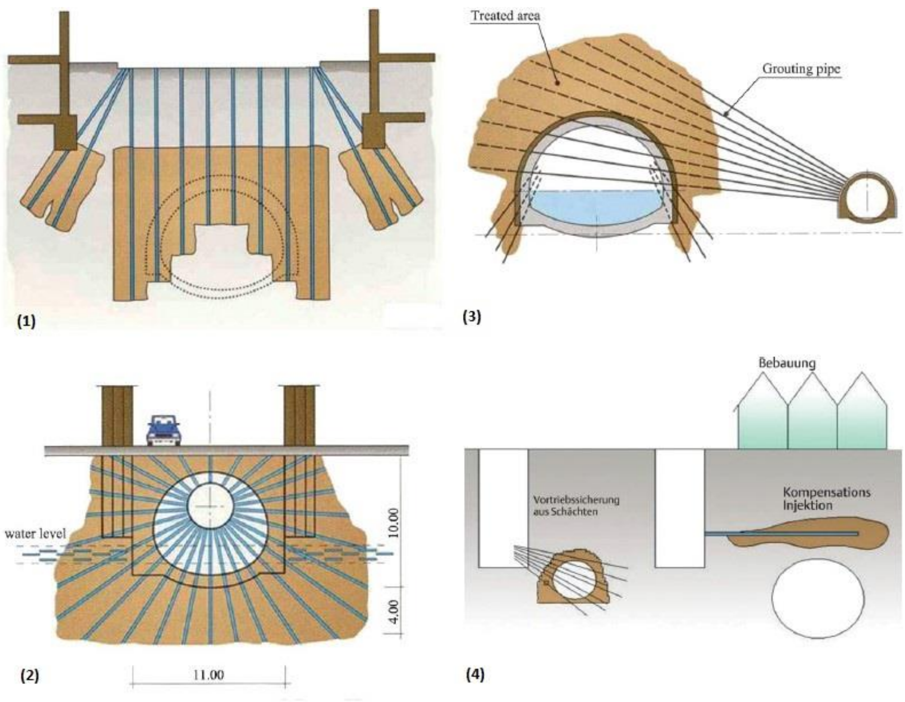


Figure 6: Drilling works: (1) from surface; (2) from adjacent gallery; (3) from pilot gallery; (4) from shafts (Kogler, 2013)

5.5.3 Grouting Objectives in Tunnelling

Grouting serves multiple essential roles for ensuring ground stability and minimizing potential risks during tunnelling operations. Three main purposes include:

- Consolidation grouting for excavation support: This is mainly aims to improve the cohesion and consolidate the ground ahead of a tunnel, specifically addressing the section of ground in front of the tunnel face not yet excavated. Typically, the goal is to improve face stability and create a load-bearing arc over the crown of the excavation profile, consequently enabling longer round lengths and faster advance rates (Kogler, 2013).

- Grouting to reduce permeability: This is typically employed in rock tunnelling below the groundwater level, with the main objective being to seal water-filled fissures before machinery operation starts. In soil contexts, sealing grouting can create an impermeable layer above compressed air tunnelling, thereby mitigating the risk of blowouts reaching the surface (Kogler, 2013).

- Compensation grouting: This is primarily in urban areas, to compensate or counteract settlements caused from tunnel excavation (Kogler, 2013). Typically conducted from shafts situated adjacent to the tunnel, this grouting operation often takes place below the groundwater table.

5.5.4 Grouting Feasibility Across Tunnelling Methods

- In TBM, grouting works conducted from TBM drives pose significant challenges, especially in small diameter tunnels where the restricted space near the cutterhead makes installing a drill nearly impossible. As a solution, modern TBMs are equipped with fixed drills and shield skin openings for drilling ahead of the machine (Kogler, 2013).
- In conventional tunnelling, placing a drilling rig at the face and ensuring space for grouting equipment and materials poses no challenges. This flexibility enables grouting works to be implemented in almost any manner in conventional tunnelling (Kogler, 2013).

5.5.5 Grouting Material: Definitions, Compositions and Applications

In the context of tunnelling and ground improvement, various grouting materials are utilized to address different geotechnical challenges. These materials differ in their composition, properties, and intended uses, which makes them suitable under certain conditions. Table 7 provides a comprehensive overview of different grout mixtures, including their definitions, compositions, and main applications. This summary provides a guide to the use of each grout type for different tunnelling and soil stabilization situations.

Generally speaking, there are two main types of grouting materials: cement-based and chemical. The cement-based grouting materials are commonly used due to their low price, non-toxic nature, good impermeability and high strength. On the other hand, the chemical grouting materials offer excellent fluidity and injectability, but they are expensive and environmentally harmful.

Table 7: Overview of Grouting Mixtures (Kogler, 2013)

Grout	Definition	Composition	Main Application
Mortars and pastes	Suspensions with high solid	Water, cement and sand, possible with additives w/c ratio < 1	Filling of cavities and fissures
Suspensions	Fine suspension of undissolved solids in a liquid	Water, cement or fine cement, if required bentonite, PFA	Waterproofing and consolidation of gravel and sandy soils, joints and fissures in rock
Solutions	Solution of solids in solvents	Water, silicate and hardener synthetic resins and plastics	Waterproofing and consolidation of sands and fine gravel soils
Emulsions	Mixtures of two or more liquids	Water, bitumen, emulsifiers water-insoluble silicate hardener	Waterproofing of very sandy soils
Resin Grouts	Two-component resins with high viscosity	Rubber and chemical hardener, possible with fillers	Waterproofing of very fissures and cracks mechanically effective grouting of rock and building components
PU foams	Polyurethane foam, mostly two components	Polyurethane with chemical hardener	Temporary waterproofing for large water passages in gravel and rock
Polyamide	Melted polyamide granulate	Single-component polyamide granulate with high viscosity	Temporary and permanent waterproofing for large water passages in soil and rock

5.6 Innovative Grouting Technology: Capsule Grouting Technology (CGT)

5.6.1 Overview and Benefits of Capsule Grouting

Capsule Grouting Technology (CGT) is a newly introduced engineering application technique of ideal compaction grouting, developed to control the deformation of soil and adjacent structures, whether they are underground or surface structures (Zheng et al., 2021). It has been effectively implemented in a variety of subway and foundation excavation projects in Tianjin and Zhuhai, China. This technology addresses the challenges posed by heterogeneous ground conditions, where conventional grouting often have difficulty to control grout pathways and maintain uniform volumetric expansion, thus reducing efficiency (Floria et al., 2008).

In CGT, grout is injected under pressure into small, high-strength capsules. These capsules, also described as inflatable pockets, are positioned within a perforated hollow rod called a grouting pipe (Figure 7). As the grout is injected, the capsules expand within the soil layer, applying soil stress and improving ground stability (Figure 8).

This technique provides several benefits, including precise control of grout position, volume, and shape, which ensures more controlled and effective grouting. It also allows for easy calculation and prediction.

The fundamental principle of capsule grouting is represented in Figure 9, which includes the formulas for calculating the final volume and radius of the expanding capsules (Zheng et al., 2022).

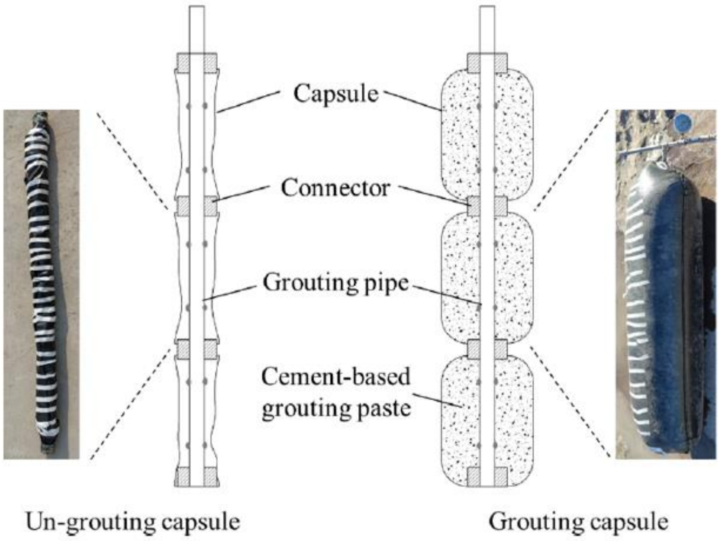


Figure 7: Schematic Diagram of Capsule Grouting Technology (CGT) (Zheng et al., 2021)

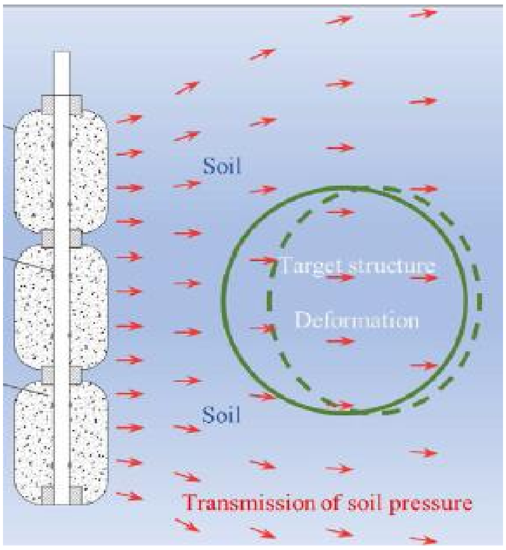
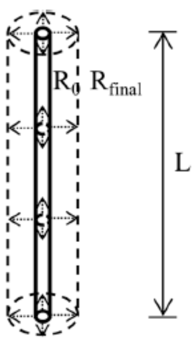


Figure 8: Mechanism of Soil Pressure Transmission and Deformation Control using CGT (Zheng et al., 2022)



$$V_{final}=V_0+\Delta V - V_{pipe}$$

$$R_{final}=R_0+Vel_{normal}\times n_{step}$$

Figure 9: Principle of Capsule Grouting (Zheng et al., 2022)

5.6.2 Equipment & Techniques of Capsule Grouting

Grouting capsules generally consist of an outer layer made from high-strength, anti-puncture geotextile material, and an inner layer of flexible, waterproof plastic film. Given the non-ductile nature and high tensile strength of the grouting capsule material, the capsules are designed with a predetermined capacity and shape to avoid free expansion similar to a balloon, ensuring controlled application (Zheng et al., 2022).

First, the grouting capsules are vacuumized, folded, and fastened with soft tape. Holes are drilled to accommodate the capsules, with diameters matching the size of the pre-buried capsules. After completing the drilling, the grouting capsules and grouting pipes are placed at a specified depth, and fine sand is repeatedly added as backfill until the stabilization occurs (Zheng et al., 2022).

The preparatory mixing tank is responsible for the initial mixing of grout materials. The initially mixed grout is then sent to the secondary mixing tank for a more consistent mix. After achieving a consistent mixture, the grout is moved to the screw grouting pump and injected into pre-buried grouting capsules through a steel-wire-reinforced high-pressure rubber grouting pipe. A flow meter is placed at the pump nozzle, and a pressure gauge is positioned at the connection between the rubber grouting pipe and the pre-buried grouting pipe (Zheng et al., 2022). Figure 10 presents the complete setup of CGT equipment.

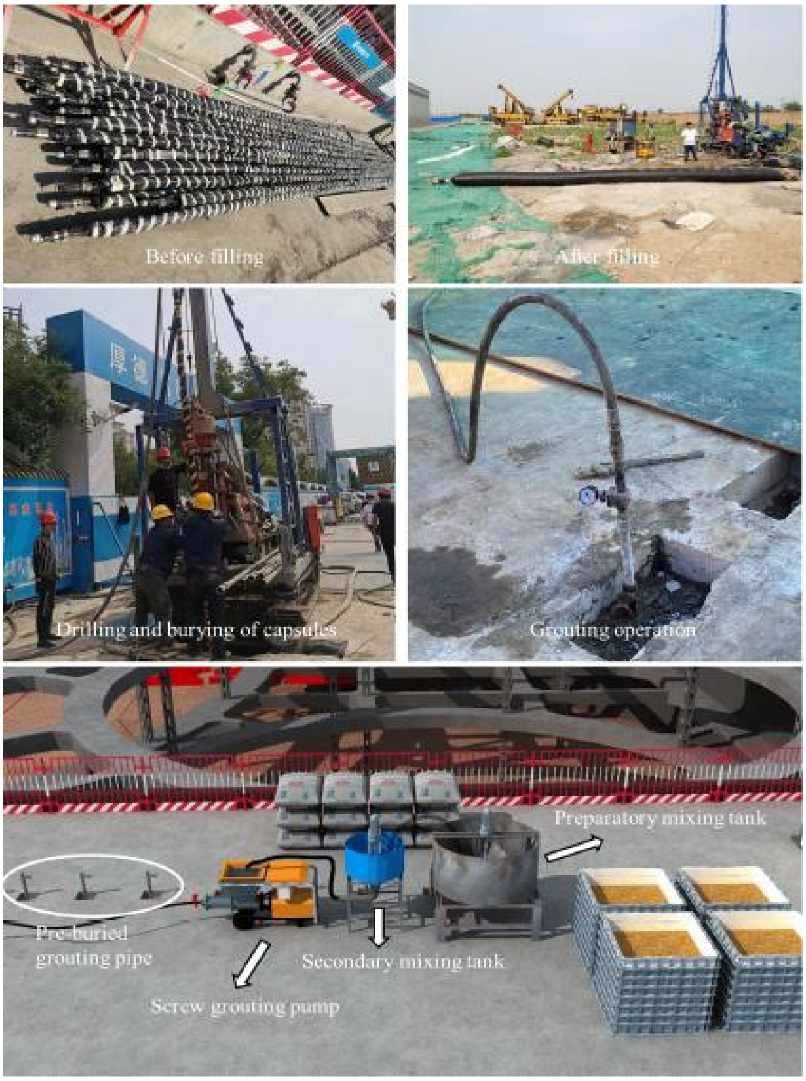


Figure 10: Capsule Grouting Equipment and Procedure (Zheng et al., 2022)

5.6.3 Material Developments of Capsule Grouting

The use of traditional cement-based grout in capsule grouting technology (CGT) reduces the accuracy of structural deformation control, due to its short setting time and poor fluidity. However, many recent studies have been focused on enhancing the performance of cement-based grouting materials, consequently extending their use in civil engineering applications. In addition, multiple grouting technique is used to improve capsule grouting performance, achieving the desired result despite the heterogeneity and complexity of soil.

5.6.3.1 Slow-Setting Cement-Based Grouting Paste (SCGP)

Zheng et al. developed a slow-setting cement-based grouting paste (SCGP) specifically designed for multiple grouting applications in capsule grouting technology (CGT). This retarding grouting material is composed of ordinary Portland cement, bentonite, fly ash, and a retarder (Etidronic acid). The material composition was optimized using an orthogonal test design and multiple linear regression analysis to ensure its suitability for CGT.

For effective multiple capsule grouting, the SCGP must feature excellent properties, including good workability indicated by a good fluidity, great stability indicated by a low bleeding rate, prolonged initial setting time, and sufficient strength to resist deformation. These properties are greatly influenced by the ratio of raw materials and admixtures in the cement-based grouting material.

The recommended composition of SCGP contains a 1.1 water-cement ratio, 30% bentonite, 30% fly ash, and 0.45% retarder. This composition provides excellent properties for CGT, including a longer setting time of 60.1 hours, good flowability of 14.8 cm, a low bleeding rate of 2.41%, and a 28-day compressive strength of 9.8 MPa (Zheng et al., 2021).

5.6.3.2 Retarding and Low-Early-Strength Grouting Material (RLGM)

This study introduced a retarding and low-early-strength grouting material (RLGM) designed for effective multiple grouting in CGT (Zheng et al., 2022). The effects of composition on RLGM properties were determined using response surface methodology. Then material was analysed through X-ray diffraction and scanning electron microscopy, and with analysis of variance (ANOVA) models created to predict its performance.

A key aspect of the RLGM is its retardation property which delays the setting time, allowing for regrouting before the material hardens. Additionally, the low strength property ensures that the hardened grout has low strength, making it easier to perform subsequent re-drilling, pocket placement, and grout expansion. To achieve retardation and low early strength, Etidronic acid retarder (EAR) was selected to delay the hydration reaction of the grout mixture. Also, Polycarboxylate superplasticizer (PCE) was selected to act as a water reducer to increases the fluidity of the mixture.

The recommended composition of RLGM contains a 1 water-cement ratio, 27.5% bentonite, 0.28% PCE, 0.75% EAR. This composition provides excellent properties for CGT, including a longer setting time of 124.1 hours and a 10-day compressive strength of 0.2 MPa (Zheng et al., 2022).

6 FE Numerical Modelling

Geoscientists have faced significant difficulties in dealing with the following aspects, which are necessary for solving modern geoscience problems:

- The complicated geometric configuration and complex interactions within the ground (e.g., twin tunnel interactions, tunnelling in urban areas, groundwater interactions, etc.)
- Heterogeneity of the medium (singular discontinuities, anisotropy)
- Non-linear behaviour of the medium
- Dynamic conditions
- Analysing a large amount of information and data necessary for understanding the controlling operations
- Addressing the three-dimensional nature of the problem
- Complex phases of construction

The rapid progress in modern computer technology has allowed numerical methods to be adopted in nearly all engineering fields: from designing pipelines to large-scale underground tunnels and from simulating the failure of concrete beams to that of extensive double-curvature arch dams.

Mathematically, the finite element (FE) method is used for solving partial differential equations numerically in engineering and applied science, which involves the creation of discretized mathematical models to simulate these problems. The steps of constructing an FE Model include:

- Creating the geometry
- Defining material properties (constitutive law)
- Generating the FE mesh (discretization)
- Applying loads and boundary conditions

- Defining stages (construction sequence) and analysis type
- Computing and displaying results

The results extracted from the model will include the trend of stresses and deformations within the soil/rock medium and the structures at a finite number of points, as well as an evaluation of the stability conditions and existing safety margins. The accuracy of the conceptual model for the problem can be verified by comparing the numerical solution with field observations.

6.1 Hydro-Mechanical Coupled Analysis

The hydro-mechanical (HM) coupling describes the interaction between hydraulic and mechanical processes, which can significantly affect the behaviour of the geotechnical system. In geological environments where soil and rock are present, numerous pores and fractures lead to the coupling of mechanical and hydraulic processes. An important aspect of hydro-mechanical coupled analysis is transient groundwater flow, which describes the dynamic behaviour of groundwater, where groundwater levels and flow rates vary due to factors like pumping and changes in underlying geology or hydraulic properties. This type of analysis is performed when analysing the simultaneous development of deformations and pore pressures in saturated and partially saturated soils due to time-dependent changes in hydraulic conditions (Plaxis, 2023). Typical situations that require a coupled analysis include partially drained excavations and reservoir drawdown behind dams .

Tunnel engineering is considered one of the most important problems involving the coupling between fluid flow and deformation in geotechnical engineering. The excavating of a tunnel in deep and saturated ground induces a time-dependent seepage and consolidation due to the transient nature of the coupled pore pressure-stress interaction. This coupling provides a better understanding of aspects like pore pressure buildup, consolidation, and potential groundwater inflow, which can have an important effect on tunnel excavation stability and overall response. In practical applications, an advancing tunnel is typically simulated using an uncoupled approach, utilizing the steady-state groundwater flow to study the influence of fluid flow on tunnel deformations while ignoring the transient coupled response (Prassetyo & Gutierrez, 2020). However, the coupled approach models the entire excavated length in a single step instead of step-by-step excavation.

There are generally three fundamental algorithms for simulations: full coupling, loose coupling, and one-way coupling. A fully coupling is described by a set of equations, usually consisting of a large system of nonlinear coupled partial differential equations that cover all relevant factors. This approach is often preferred because it theoretically provides the most realistic results, including nonlinear and inelastic mechanical deformation in a multiphase flow. In contrast, one-way coupling involves solving two separate sets of equations, and the information is transferring in only one direction. Loose coupling, similar to one-way coupling, involves solving two sets of equations independently but the information transferred at specific intervals in both directions between fluid flow and geo-mechanical systems. Even though, loose coupling maintains the simplicity of implementation found in one-way coupling.

6.2 Hardening Soil Model

The Hardening Soil (HS) model is an elasto-plastic second-order hyperbolic isotropic hardening model, commonly applied in geotechnical engineering to describe the non-linear behaviour of soils and rocks when subjected to different loading conditions, including the shear/deviatoric loading, compression loading, and unloading/reloading. This advanced constitutive model was developed using the theory of elasto-plasticity with double yield surfaces, including a yield cap, stress-dependent stiffness, and dilatancy. It takes into account the plastic deformation properties of soils, considering the stiffness degradation and strain hardening behaviour found in real soil responses. Consequently, this model provides more realistic displacement patterns under working load conditions, particularly during excavations. Moreover, the predicted ground displacement patterns caused by tunnelling are accurate and don't affect the finite element boundary conditions (Surarak et al., 2012).

The superiority of the Hardening Soil model over the Mohr-Coulomb model is due to its use of a hyperbolic stress-strain relationship rather than a bi-linear one (Figure 11), as well as its ability to control stress level dependency. Using the Mohr-Coulomb model requires selecting a fixed Young's modulus, while real soil stiffness is stress-dependent, making it necessary to estimate stresses within the soil to calculate accurate stiffness values. However, the Hardening Soil model takes away this time-consuming and complex process of selecting input values. Figure 12 compares the variation in Young's modulus (E) variation between the Mohr-Coulomb and Hardening Soil models.

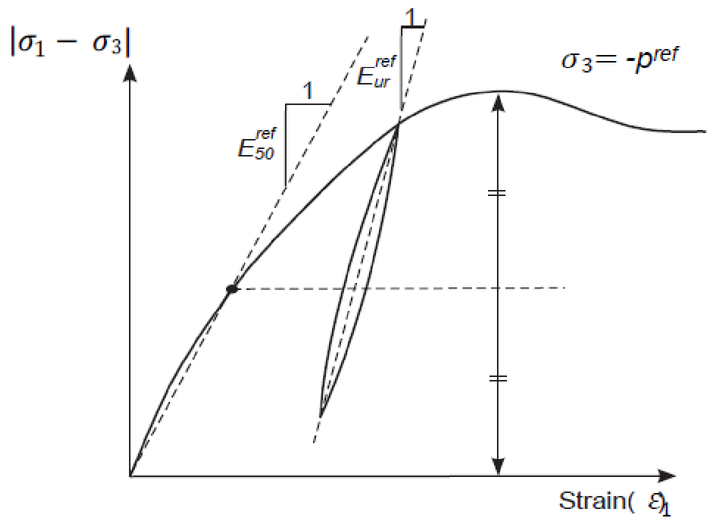


Figure 11: Definitions of E_{50}^{ref} and E_{ur}^{ref} for drained triaxial test results (Plaxis, 2023)

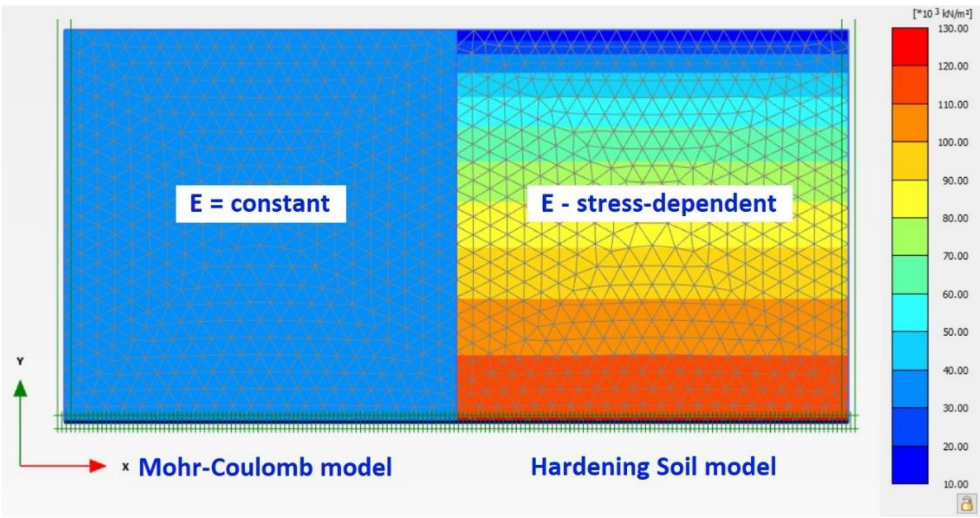


Figure 12: Comparison of Young's Modulus Variation: Mohr-Coulomb Model vs. Hardening Soil Model

Unlike the elastic perfectly-plastic (e.g. Mohr-Coulomb model), the yield surface of a hardening plasticity model expands due to plastic straining in the principal stress space. There are two types of hardening: shear hardening, which represents irreversible plastic strains resulting from primary shear/deviatoric loading, and compression hardening, which represents irreversible plastic strains resulting from primary compression loading.

Table 8 provides a summary of all the parameters of the Hardening Soil model, along with brief descriptions and the laboratory tests required to determine each parameter.

Table 8: Hardening Soil model Parameters and laboratory tests needed for the determination of parameters (Plaxis, 2023)

Parameter	Description and test needed to determine the parameter
E_{50}^{ref}	- Description: Secant stiffness to describe plastic straining due to primary shear/deviatoric loading at a selected reference pressure - Test needed: Drained triaxial compression tests
E_{oed}^{ref}	- Description: Secant stiffness to describe plastic straining due to primary compression at a selected reference pressure - Test needed: Primary loading in oedometer tests
E_{ur}^{ref}	- Description: Stiffness to describe elastic unloading/reloading behaviour at a selected reference pressure - Test needed: Unloading/reloading in triaxial compression test
C	- Description: (Effective) cohesion - Test needed: Drained triaxial compression tests loaded to failure
ϕ	- Description: (Effective) angle of internal friction - Test needed: Drained triaxial compression tests loaded to failure
ψ	- Description: angle of dilation

- Test needed: Drained triaxial compression tests with measurement of volume change

ν_{ur}
- Description: Elastic poisson's ratio
- Test needed: Unloading/reloading in triaxial compression tests

m
- Description: Power of a power law used to describe the level of stress dependency of soil stiffness
- Test needed: Typical range of value $0 \leq m \leq 1$

p^{ref}
- Description: Reference pressure
- Test needed: None (default: $P^{ref} = 100\ KPa$, the atmospheric pressure)

R_f
- Description: Failure ratio
- Test needed: Drained triaxial compression tests

The three stiffness moduli are calculated based on the stress level acting in the soil as simulated by the Hardening Soil model. The following equations are used to calculate the secant stiffness modulus (E_{50}) at 50% of the maximum deviatoric stress, the tangent stiffness modulus (E_{oed}), and the unloading/reloading stiffness modulus (E_{ur}):

$$E_{50} = E_{50}^{ref} \left(\frac{Ccos(\phi) - \sigma_3' \sin(\phi)}{Ccos\ (\phi) + p^{ref} \sin(\phi)} \right)^m$$

$$E_{oed} = E_{oed}^{ref} \left(\frac{Ccos(\phi) - \frac{\sigma_3'}{K_0^{nc}} \sin(\phi)}{Ccos\ (\phi) + p^{ref} \sin(\phi)} \right)^m$$

$$E_{ur} = E_{ur}^{ref} \left(\frac{Ccos(\phi) - \sigma_3' \sin(\phi)}{Ccos\ (\phi) + p^{ref} \sin(\phi)} \right)^m$$

The determination of input parameters in a soil model is generally an essential element when performing numerical analysis of geo-structures. For the Hardening Soil model, these input parameters can be determined by performing a series of conventional triaxial compression tests on the soil. Based on observations, a range of these input parameters can be estimated, with some of them correlated to the Mohr-Coulomb parameters C, Φ, and ψ.

It has been found that for soft clays, the power (m) should be set to 1.0, while for Norwegian sands and silts, the power (m) should be approximately 0.5. Also, the suggested value for the unloading/reloading stiffness E_{ur}^{ref} is equal to $3E_{50}^{ref}$ (Plaxis, 2023). Numerous observational correlations can be found in the literature review.

6.3 Numerical Simulation Methodologies For the Grouting Process

Based on previous research and case studies, the grouting process is numerically simulated using two main numerical approaches: the prescribed strain approach (direct method) and the prescribed stress approach (indirect method). Generally, the overall effect of grouting in the analysis is assessed by the changes in volume in the grouted elements.

In the prescribed strain approach (Figure 13), grout injection is modelled by applying specific strain values to the elements that represent the grouted soil. As described by Nicolini & Nova (2000), this approach simulates the grout injection process using anisotropic inelastic strains, which allows which allows the strain magnitude to vary in different directions and spatially. In practical applications, grouting is carried out by injecting a specific volume of grout into the ground, and in a numerical model using the prescribed strain approach, this volume is directly applied to the soil by specifying a relevant strain distribution. Overall, the extent of the grouting, along with the magnitude and spatial distribution of the applied strains must be specified in advance.

On the other hand, in the prescribed stress approach (Figure 13) described by Soga et al., internal pressure is applied to the elements representing the grouted soil, which induces volumetric strain. This internal pressure is a hypothetical value, chosen to produce the specified strain. By comparing the real volumetric strains with the resultant strain values, the correct value of the internal pressure can be determined for the numerical modelling. Several trials are needed to find the correct pressure value that produces in the real volumetric strain.

Typically, the prescribed strain approach is simpler than the prescribed pressure approach. However, the prescribed pressure approach has the potential to provide more detailed simulations of grout flow into the ground.

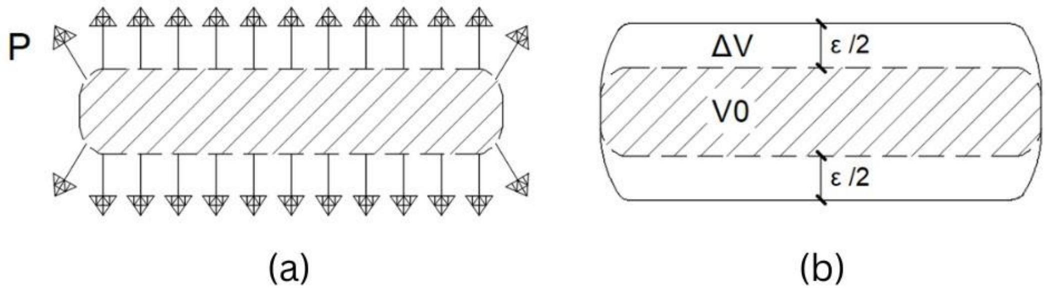


Figure 13: (a) Prescribed Stress (b) Prescribed Strain

7 Case Study: Application of Drains and Grouting

The case study focuses on a railway tunnel, which is designed as a single tube with a truncated conical section, as shown in the Figure 14. The tunnel is located at a depth of 45 meters below ground surface, and has an approximate height of 12 meters and a length of 12 meters. The tunnel will be passing through a mixed heterogeneous formation consisting mainly of fine-grained lithotypes with cohesive behaviour and low permeability properties. The soil in this region is saturated, with the groundwater level typically found 7 meters below the ground surface. The application of consolidation interventions is considered an effective decision to improve the ground properties, providing a stable and safe tunnel excavation.

A small section tunnel created using a Tunnel Boring Machine (TBM), was designed to provide access and facilitate the installation of drains and capsule grouting. Drainage is typically considered an effective and economical method for ground consolidation, as the dissipation of pore pressures provides short-term stability. Additionally, capsule grouting induces a local densification, enhancing the mechanical characteristics of the soil. After the ground improvement phase, the existing tunnel section will be enlarged.

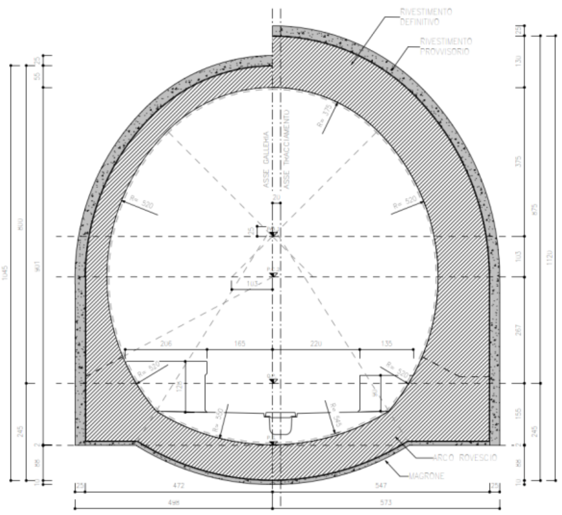


Figure 14: Cross-Sectional Diagram of the Conventional Tunnel

7.1 Geotechnical Conditions

The entire length of the tunnel is characterized by a different formations, where this case study focuses on the section involving a mixed heterogenous formation characterized by cohesive behaviour.

In the area of direct interest, the deposit is characterized by predominantly fine-grained facies. The deposit consists mainly of silty clays and sandy-clayey silts of a greyish colour, with sporadic clasts ranging from millimeter to centimeter in size, from angular to sub-rounded, predominantly calcareous, though with some subordinate greyish marl fragments. When observable, it shows a sub-horizontal parallel stratification. There are interlayers of debris levels with sub-rounded mixed with angular clasts, of varying sizes from 1-2 cm up to a maximum of 10 cm. The coarse levels reach a maximum thickness of 2-3 m. This detailed geological information has been sourced from PINI GROUP.

To sum up, it's a formation consisting mainly of fine-grained lithotypes with cohesive behaviour. Figure 15 and Figure 16 show the cores taken from the mixed formation at different depth intervals, presenting both the fine and coarse components as well as the heterogeneity of this formation.

Figure 15: Cores From 30 to 35 meters in the Borehole Showing the Mixed Formation in its Fine-Grained Component

Figure 16: Cores From 20 to 25 meters in the Borehole Showing the Mixed Formation in its Coarse Component

Geotechnical parameters:

- Volume weight $\gamma = 19.5 \div 20.5\ KN/m^3$
- Effective cohesion $C' = 5 \div 15\ KPa$
- Friction angle $\varphi = 24 \div 28\ °$
- Undrained cohesion $c_u = 100 \div 150\ KPa$
- Modulus of elasticity $E = 80 \div 120\ MPa$

Due to the geological history of the deposit, the material is to be estimated as having a low degree of over-consolidation (unit OCR or slightly higher). For the modulus, a ratio between the compression modulus and the unloading modulus of 2÷3 can be estimated.

7.2 Hardening Soil Model Parameters

In this section, all the input parameters used with the Hardening Soil model in the numerical analysis of this case study are listed out in detail. The following parameters are important for correctly simulating the complex behaviour of soils under different loading conditions. The Hardening Soil model is well-known for describing the non-linear and stress-dependent behaviour and uses a set of parameters to capture both the stiffness and strength of the soil.

- The HS model assumes the use of effective strength characteristics – C' and ϕ', which were determined during the geotechnical investigation campaign.

- The secant stiffness parameter (E50) was determined to be equal to the modulus of elasticity (E), which was obtained during the geotechnical investigation campaign.

$$\boldsymbol{E_{50}^{ref} = E}$$

- The unloading/reloading stiffness parameter (Eur) was estimated as three times the value of E50. This estimation is consistent with the recommendations provided in the geotechnical investigation campaign, which suggests that the unloading/reloading stiffness is typically between two to three times the secant stiffness at 50% of the maximum deviatoric stress.

$$\boldsymbol{E_{ur}^{ref} = 3E_{50}^{ref}}$$

- The secant stiffness for primary oedometer loading (Eoed), was calculated using the following formula, which combines both the bulk modulus (K) and the shear modulus (G):

$$\boldsymbol{E_{oed}^{ref} = \frac{3K + 4G}{3}}$$

Where:

$$\boldsymbol{K = \frac{E}{3(1 - 2\nu)}}$$

$$\boldsymbol{G = \frac{E}{3(1 + \nu)}}$$

- The power parameter (m), which describes the stress dependency of the soil stiffness, was set to 1. This is a common assumption for clay soils related to their stress-strain behaviour. The failure ratio (Rf), which must be less than or equal to 1, is set to 0.9 as a default. Similarly, the reference pressure for stiffness (Pref) is set to 100 KN/m^2 as a default.

- The natural state parameter is a key characteristic of geological models, including the HS model, since it affects the initial phase results of the model. All subsequent computations are based on these results. The lateral pressure coefficient K0 for normally consolidated and over-consolidated soil is calculated using particular formulas. The OCR is set to 1 in this model.

$$\boldsymbol{K_0^{NC} = 1 - sin\varphi'}$$

$$\boldsymbol{K_0^{OC} = (1 - sin\varphi')\sqrt{OCR}}$$

Table 9 summarizes the input parameters used in the Hardening Soil model to ensure a realistic and accurate representation of soil behaviour in numerical simulations. These parameters represent the stiffness and strength properties, as well as other input parameters that characterize the physical and hydraulic properties, such as unit weight and permeability.

Table 9: Geo-mechanical and Hydraulic Input Parameters

Soil Unit	γ [KN/m3]	C' [KPa]	φ [∘]	Ψ [∘]	E_{50} [MPa]	E_{ur} [MPa]	E_{oed} [MPa]	K_0 [-]	m	P_{ref} [KPa]	R_f	ν	k [m/s]
Mixed Formation	20	10	28	0	100	300	135	0.53	1	100	0.9	0.3	10E-7

7.3 Selection of Numerical Software

The following fully coupled analyses were conducted using the Plaxis 2D Ultimate finite element (FE) calculation code, version 2023, which allows a two-dimensional analysis of deformation, stability and groundwater flow in geotechnical engineering and rock mechanics. It's commonly used to design geotechnical structures such as excavations, dams, embankments and tunnels. PLAXIS 2D calculates deformations, soil stresses, water flow and pressures, structural forces and even thermal flow for both 2D plane strain and axisymmetric problems. Plaxis is equipped with various features to accommodate many important aspects of complex geotechnical structures. The features include higher-order elements for improved computational accuracy, automatic generation of finite element meshes with options for global and local mesh refinement, tension-only structural elements for simulation of geosynthetics, joint elements to simulated interface behaviour between dissimilar materials, updated Lagrangian analysis to account for large deformation and displacements, staged-construction algorithm to simulate sequential construction operation. Many different soil models are included in order to take into account specific behaviour of for instance clay, sand and rock as well as the specific behaviour under loading, unloading and reloading of soil.

However, PLAXIS 2D Ultimate further extends the Advanced functionalities for geotechnical problems with transient and fully coupled groundwater analysis, thermal flow analysis, dynamic analysis and the most advanced and state-of-the-art material models. I would like to thank PINI GROUP for their support in providing access to this software, enhancing both the quality and relevance of my research.

7.4 Model Creation

The model has a total height of 92 meters and a total width of 130 meters, which is equivalent to four times the diameter (4D) of the conventional tunnel. This size was chosen in order to consider the influence zone of the tunnel construction, estimated to be approximately 2D.

The model contains a single soil layer composed of the mixed formation, with its properties detailed in Table 9. The geotechnical and hydrogeological properties of this material have been characterized based on a survey conducted by PINI GROUP to assess the zone interested by the tunnel construction. Additionally, the groundwater level is positioned at a depth of 7 meters from the ground surface. The soil is simulated with hardening behaviour to accurately reflect its response under load.

A small section tunnel created using a Tunnel Boring Machine (TBM), was designed to provide access and facilitate the installation of drains and capsule grouting. The diameter of this circular section was 4 meters, and was created using the Tunnel Designer tool of Plaxis 2D. The tunnel lining was modelled with plates, an interface was provided to simulate the interaction between the tunnel lining and the surrounding ground, and contraction was added to simulate the cone shape of the TBM.

The model includes a radial distribution of 10 drains positioned around the conventional tunnel section profile, which is intended to be excavated in later stages after the ground improvement. Each drain has a length of 3 meters, with an equal spacing of 4 meters between them. The activation duration for the hydro-mechanical analysis was set to 5 days.

The capsule grouting was positioned between the drains, following a radial distribution around the conventional tunnel section profile. There are 10 grouting points, each with a length of 3 meters using a single capsule. The diameter of the grouting points is 131 mm, equal to the borehole drilling diameter. For the numerical simulation of the capsule grouting process, a prescribed strain approach was employed. Initially, a cluster was created and assigned the grout material, with its properties presented in Table 10. After that, a specific volumetric strain value equal to 15% were applied to the cluster elements representing the grouted soil. The strain magnitude varied in different directions based on the position of the cluster to accurately simulate the realistic shape of the capsule. Moreover, the activation duration for the hydro-mechanical analysis was set to 5 days and the calculation steps is set to 1000.

Figure 17 represent the model components, highlighting the drains number around the tunnel section.

Table 10: Material Properties of Grout

Parameter	Grout
Soil Model	Linear Elastic
Drainage Type	Non-porous

Unit weight γ $[KN/m^3]$	20
Stiffness E $[MPa]$	20
Poisson's ratio ν	0.2

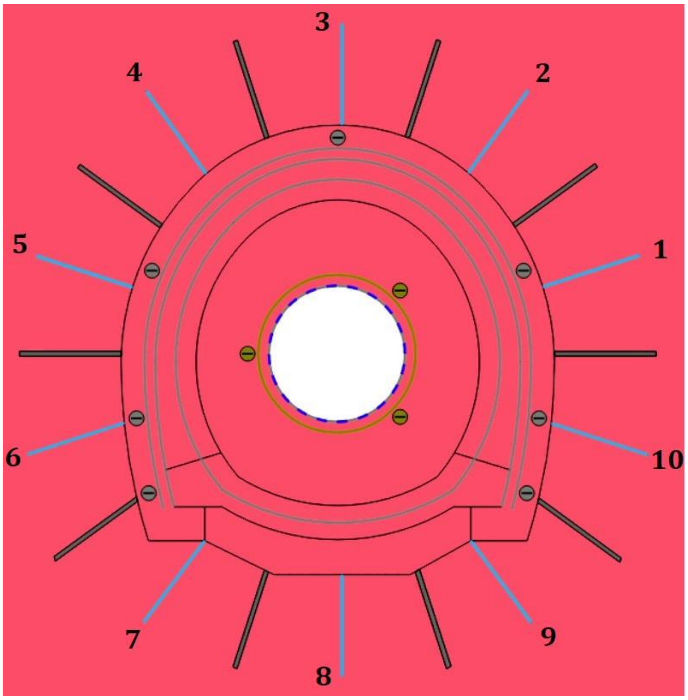


Figure 17: Model Components

7.5 Mesh Generation

Once the geometry model is completely defined, it needs to be divided into finite elements to perform finite element calculations. This collection of finite elements is known as a mesh, which is created in the Mesh mode. Moreover, the input data (properties, boundary conditions, material sets, etc.) is transformed from the geometry model (points, lines, and clusters) to the finite element mesh (elements, nodes, and stress points). Mesh generation relies on a robust triangulation procedure that takes into account soil stratigraphy, structural objects, loads, and boundary conditions (Plaxis, 2023).

To obtain accurate numerical results, the mesh should be fine enough in areas with significant expected stress or strain changes. However, extremely fine meshes should be avoided to prevent long calculation times. For the numerical stability of the calculation, the mesh has to be of good quality, with elements that are regular and not too long or thin. Moreover, the primary soil elements for modelling soil layers and other volumes are 15-node or 6-node triangular elements. It is important to keep in mind that a mesh made of 15-node elements provides a finer distribution of nodes and therefore more accurate results compared to an equivalent mesh of 6-node elements. However, the use of 15-node elements takes more time than using 6-node elements. Therefore, a careful balance between accuracy and calculation time is essential while maintaining the mesh quality.

In PLAXIS 2D, the finite element meshes are generated fully automatically. There are five global levels available, allowing users to select a level to make the mesh finer or coarser globally. Table 11 presents the different levels of element distribution, ranging from 'Very coarse' to 'Very fine', along with the corresponding approximate number of elements generated in the mesh. The precise number of elements is determined by the shape of the geometry and any applied local refinement settings.

Table 11: Levels of Element Distribution and The Corresponding Number of Elements (Plaxis, 2023)

Element distribution	r_e
Very coarse	2.00 (30 – 70 elements)
Coarse	1.33 (50 – 200 elements)
Medium	1.00 (90 – 350 elements)
Fine	0.67 (250 – 700 elements)
Very fine	0.50 (500 – 1250 elements)

In regions where significant stress concentrations or large deformation gradients are predicted, a finer finite element mesh is preferred for greater accuracy, while other regions may not need a such detailed mesh. Therefore, using the local refinement setting is ideal in that situation. Local refinement is determined by a coarseness factor for each geometry entity, indicating the relative size of the elements.

In this case study, a mesh consisting of 15-node triangular elements was adopted to ensure a finer distribution of nodes, which enhances the accuracy of the numerical results. The global mesh coarseness was set to "Very Fine". Additionally, a local refinement zone was created around the tunnel geometry, focusing on the surrounding soil. These layers were refined using a coarseness factor of 0.125. Furthermore, the structural elements and geometric entities, including plates, drains, and clusters, were also refined by a factor of 0.125. The utilization of a very fine global mesh combined with local refinements is crucial for accurately capturing significant stress concentrations, particularly between the drains and the grouting pocket. Figure 18 represents the finite element mesh that have been carried out for the numerical analysis.

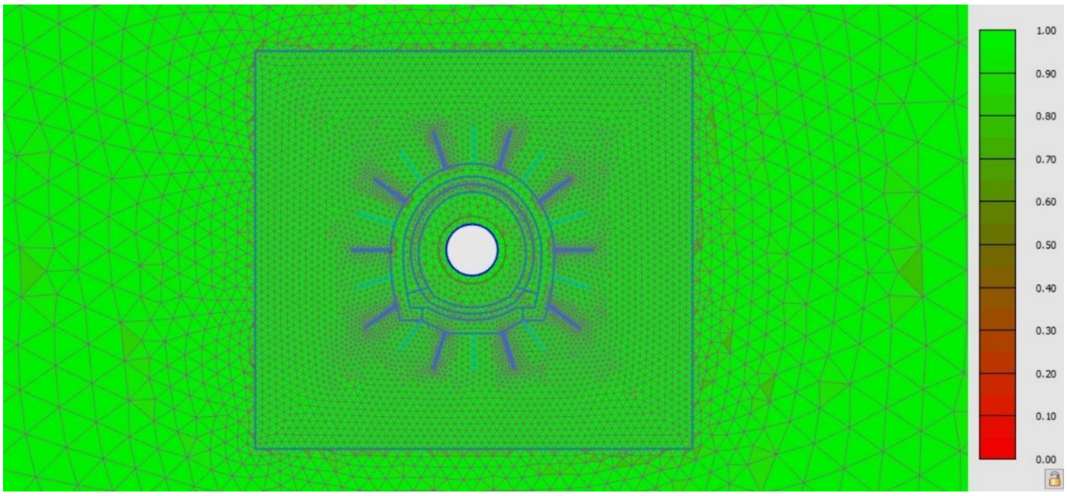


Figure 18: Finite Element Mesh for Numerical Model

To represent the values at a specific location in the numerical model, a measuring point (stress point) has been selected in the middle side of the tunnel geometry (Figure 19). It is positioned midway between the drain and the grouting body, which provides an average value.

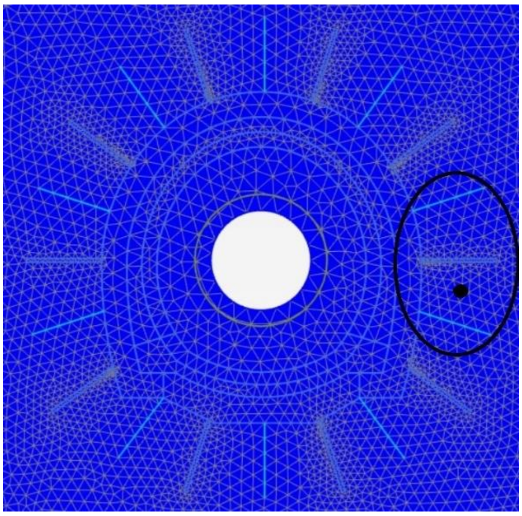


Figure 19: Graphical Representation of the Selected Measuring Point

7.6 Boundary Conditions

Boundary conditions are an important consideration while performing a finite element numerical analysis. A set of deformation boundary conditions for the boundaries of the geometry model are generated according to the following rules:

- The model's bottom boundary is fully fixed, meaning that displacements are constrained in all directions, both vertically and horizontally (Ux = Uy = 0).

- The model's top boundary is free, meaning that displacements are unconstrained in all directions.

- The model's lateral boundaries are normally fixed, meaning that displacements are constrained in the x-direction (Ux = 0) and un constrained in the y-direction.

The hydraulic boundary conditions are essential in any groundwater flow calculation, including fully coupled flow deformation analysis (Figure 20). They define the movement of water within the model, ensuring that it accurately represent real-world hydraulic behaviour. It's important for predicting pore pressure distribution, groundwater flow paths, and the interaction between fluid flow and ground deformation. The boundaries are generated according to the following rules:

- The model's bottom boundary is set to closed, meaning that there is no exchange of flow across it, so it's considered an impermeable boundary.

- Both the model's top and lateral boundaries are set to open, meaning that water can flow in or out freely.

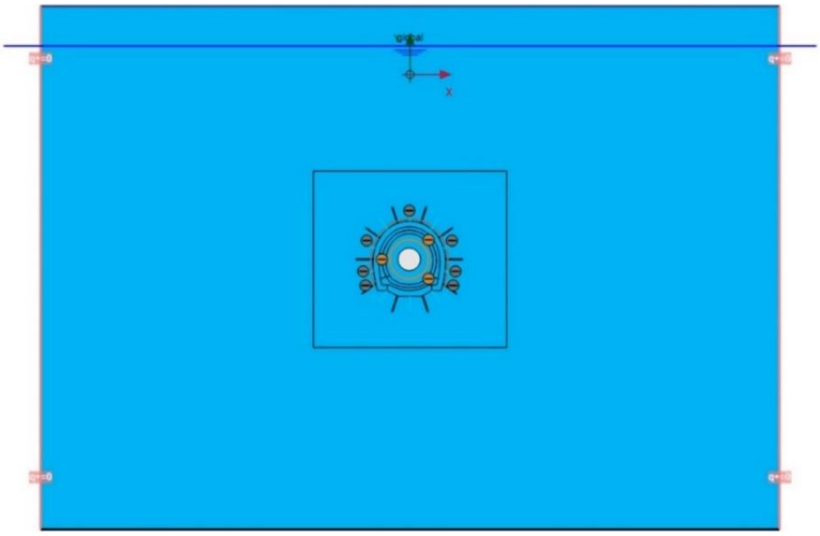


Figure 20: Hydraulic Boundary Conditions for Numerical Model

7.7 Results and Interpretations

7.7.1 Effective Stress Distribution Across Construction Stages

The following output results illustrate the distribution of the isotropic effective stress ($\sigma_0{}'$) around the tunnel geometry at different stages of the excavation and ground improvement process, highlighting the impact of each intervention on the surrounding ground. Note that the isotropic effective stress is an average representation of the effective stress acting equally in all directions within a soil, and it's expressed as the mean of the principal effective stresses.

The initial state shown in Figure 21 is considered the baseline distribution of isotropic effective stress and will be used as a reference for changes in the subsequent stages. Figure 22 shows the stress state after the TBM section construction, including both the excavation and lining installation work. Figure 23 shows the stress state after activating only the drains for 5 days. Figure 24 represents the stress state after activating both drains and grouting with volumetric expansion for 5 days, which highlight the combined effects of both technologies on the isotropic effective stress distribution.

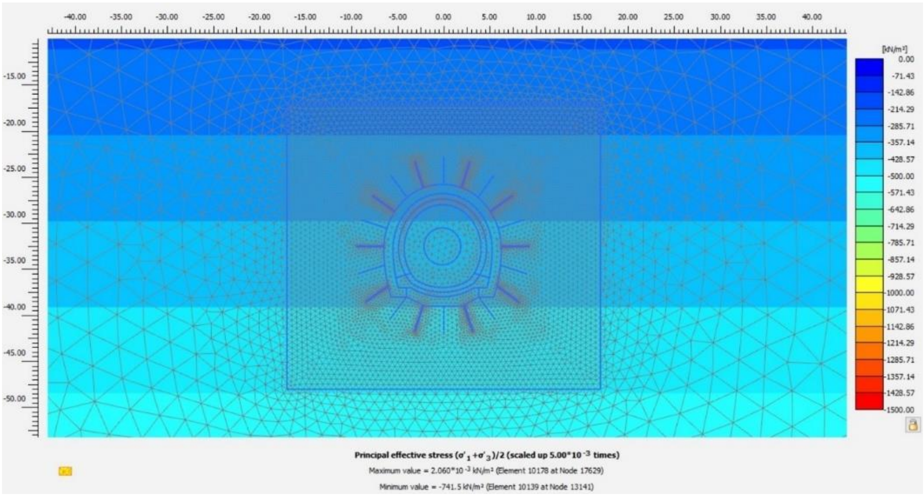


Figure 21: Isotropic Effective Stress Distribution in the Initial State

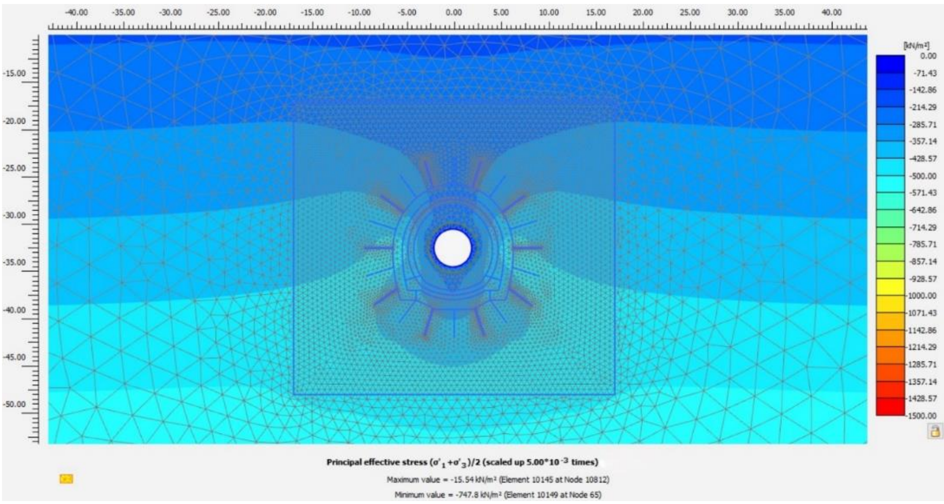


Figure 22: Isotropic Effective Stress Distribution after TBM Section Construction

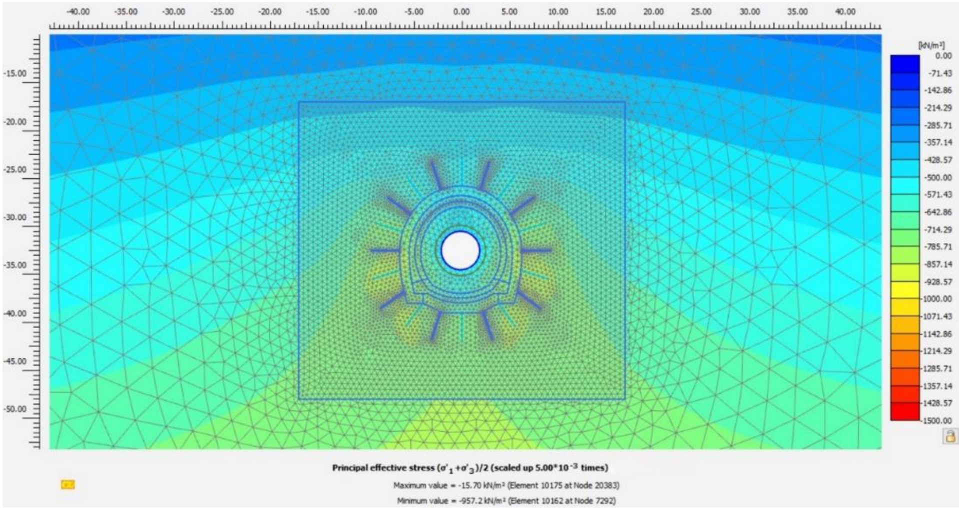


Figure 23: Isotropic Effective Stress Distribution After 5 Days of Drains Activation

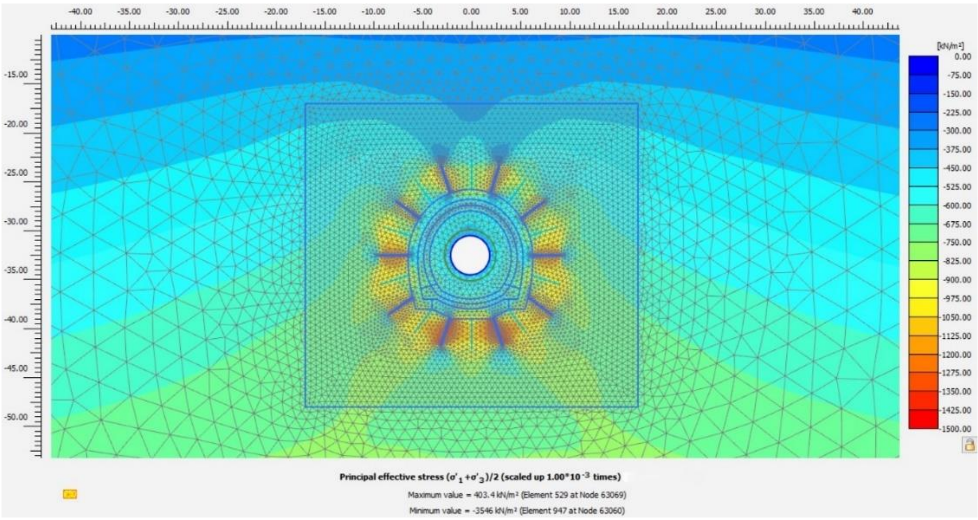


Figure 24: Isotropic Effective Stress Distribution After 5 Days of Drains and Grouting Activation

7.7.2 Comparison of Effective Stress in Drains Only and Drains with Grouting Scenarios

The graph below (Figure 25) shows the variation in isotropic effective stress over time for two case scenarios: "drains only" and " drains with grouting". These stress values are measured at a specific measuring point, as in Figure 19.

The general trend of increasing stress over time is due to the consolidation effect from the removal of water by drainage and the additional compression introduced by the grouting body expansion.

Initially, there is a rapid increase in stress for both case scenarios, reaching a value of 752 KPa for the case of drains only and 810 KPa for the case of drains with grouting after 0.5 day. After that, the effective stress increases gradually over time for both scenarios, whereas the increase in the drains with grouting case is more significant due to the hardening behaviour of the soil. The drains only case reaches a relatively steady state after 5 days of activation, with a value of 793 KPa. In contrast, the drains with grouting case continues to increase strongly over time, reaching 1068 KPa after 5 days.

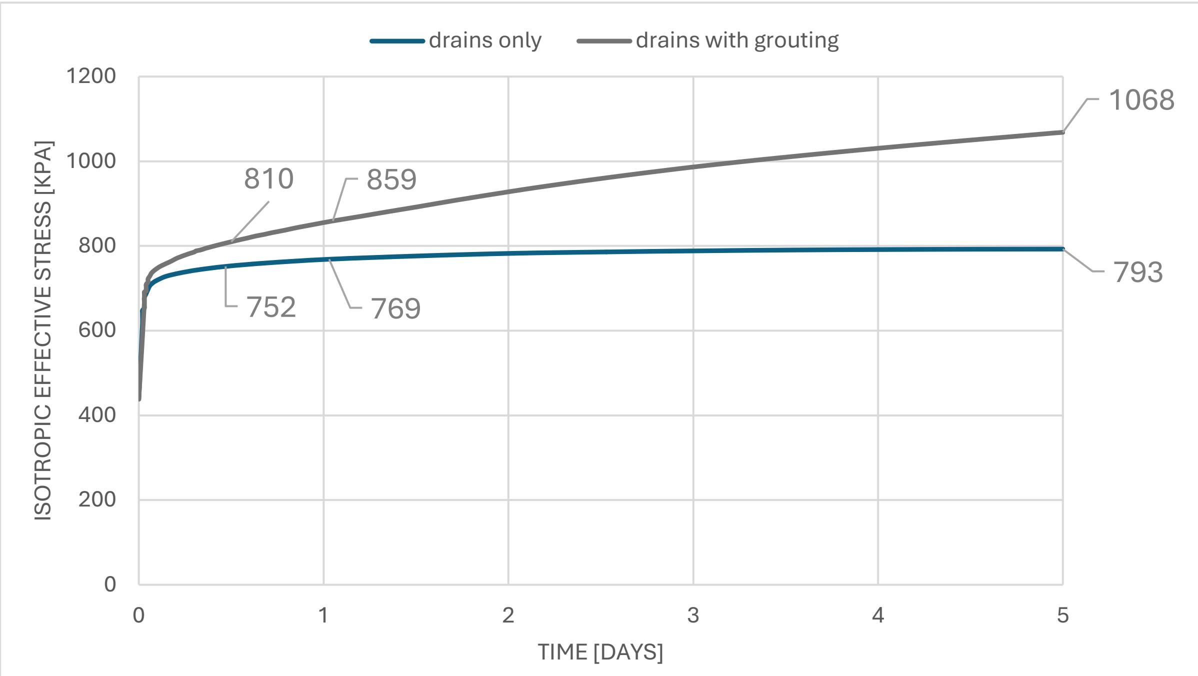


Figure 25: Isotropic Effective Stress Variation Over Time for The Two Case Scenarios ($k = 10^{-7} m/s$)

In terms of percentage change observed after 5 days of activation:

- The isotropic effective stress in the drains only case shows a 106% increase after 5 days compared to the initial state (385 KPa).

- The isotropic effective stress in the drains with grouting case shows a 177.4% increase at 5 days compared to the initial state (385 KPa).

- The isotropic effective stress in the drains with grouting case is 34.7% higher compared to the drains only case after 5 days.

Figure 26 and Figure 27 provide a visual comparison of the isotropic effective stress distribution at 0.5 day, 1 day and 5 days for the two case scenarios: "drains only" and "drains with grouting", with highlighting the measuring point value.

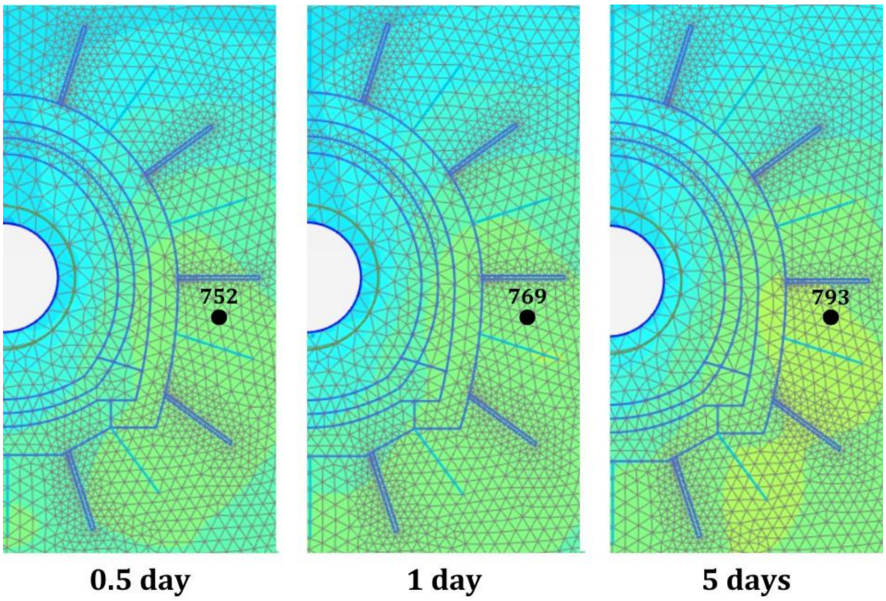


Figure 26: Isotropic Effective Stress Distribution for The Drains Only Case at 0.5 Day, 1 Day, and 5 Days

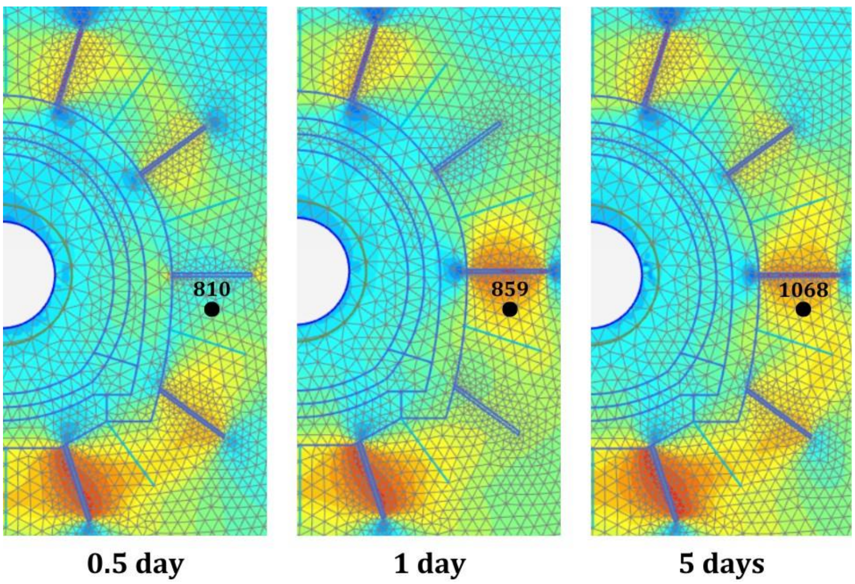


Figure 27: Isotropic Effective Stress Distribution for The Drains with Grouting Only Case at 0.5 Day, 1 Day, and 5 Days

Figure 28 shows the variation of isotropic effective stress along the line joining two drains after 5 days of activating both the drains and the grouting expansion. The graph clearly indicates higher effective stress values near the grouting body and lower effective stress values closer to the drains.

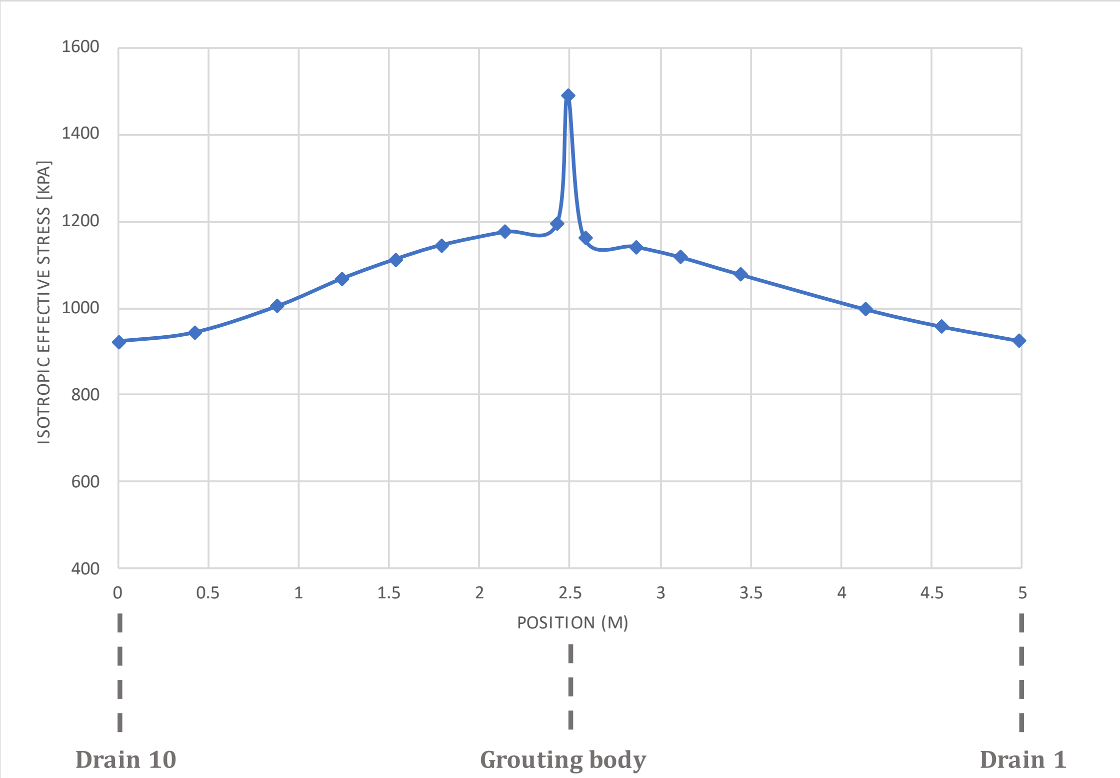


Figure 28: Isotropic Effective Stress Variation Along the Line Joining Two Drains After 5 Days (k = 10^{-7}m/s)

7.7.3 Pore Water Pressure Distribution Across Construction Stages

The following output results represent the distribution of the active pore water pressure (P_{active}) around the tunnel geometry at different stages of the excavation and ground improvement process.

The initial state shown in Figure 29 is considered the baseline distribution of active pore water pressure. Figure 29 shows the stress state after the TBM section construction, including both the excavation and lining installation work. Figure 30 shows the stress state after activating only the drains for 5 days. Figure 31 represents the stress state after activating both drains and grouting with volumetric expansion for 5 days, which highlight the combined effects of both technologies on the isotropic effective stress distribution.

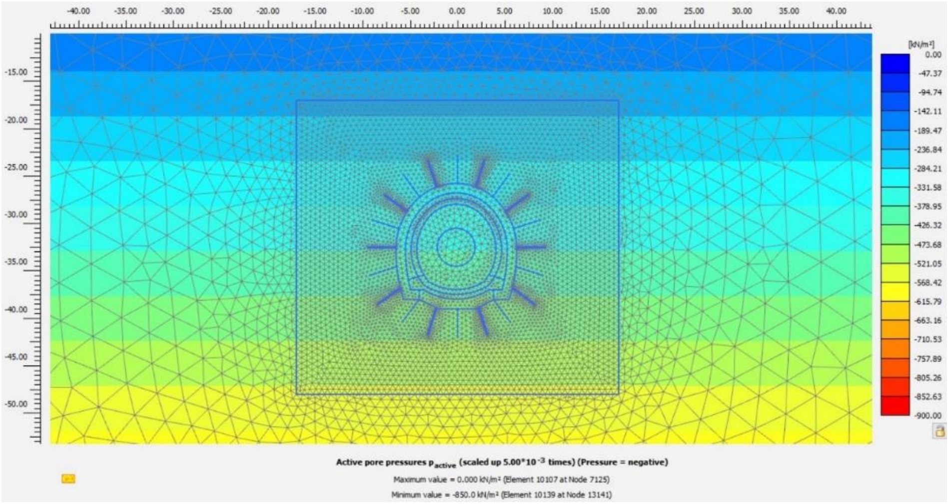


Figure 29: Active Pore Water Pressure Distribution in the Initial State

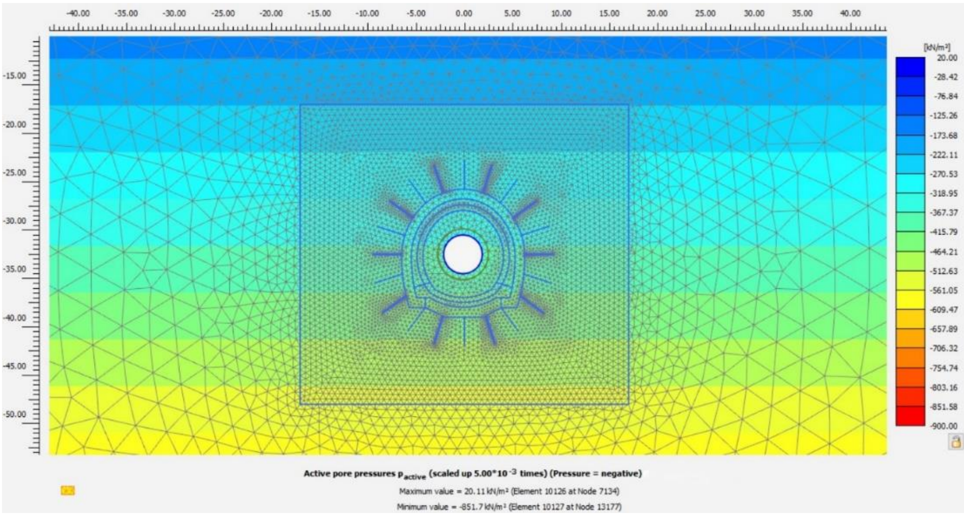


Figure 30: Active Pore Water Pressure Distribution after TBM Section Construction

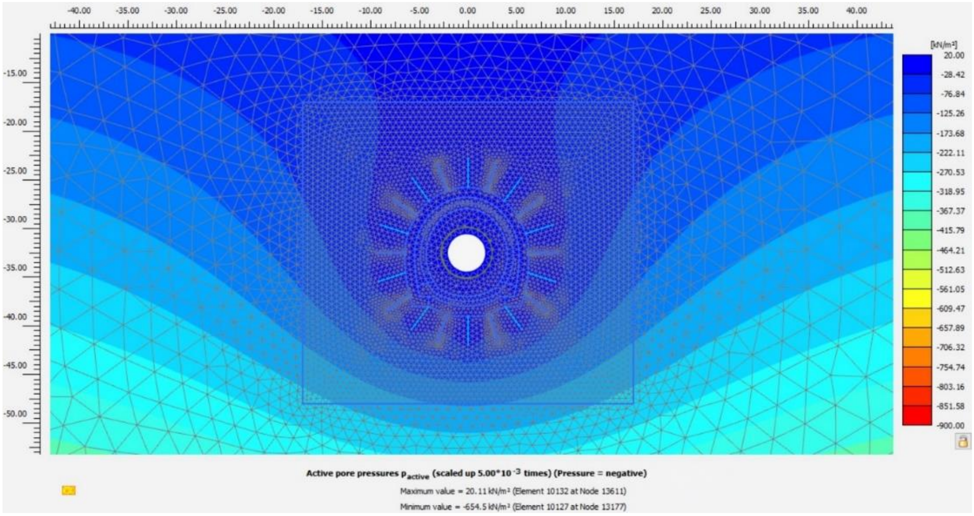


Figure 31: Active Pore Water Pressure Distribution After 5 Days of Drains Activation

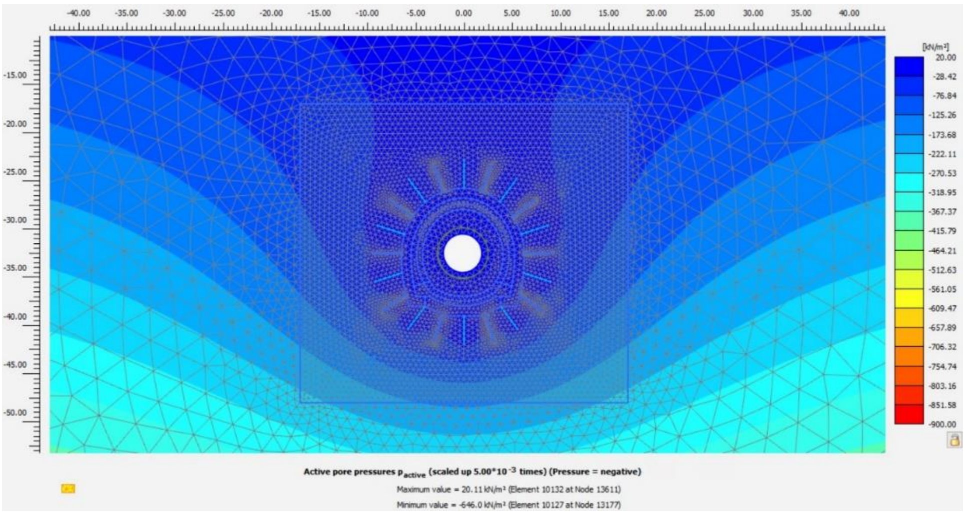


Figure 32: Active Pore Water Pressure Distribution After 5 Days of Drains and Grouting Activation

7.7.4 Comparison of Pore Water Pressure in Drains Only and Drains with Grouting Scenarios

The graph below (Figure 33) shows the variation in active pore water pressure over time for two case scenarios: "drains only" and " drains with grouting". These values are measured at a specific measuring point, as in Figure 19.

The general trend of decreasing pore water pressure over time is due to the consolidation effect from the removal of water by drainage.

Initially, there is a rapid drop in pore water pressure for both case scenarios, reaching 18 KPa for the drains only case and 13 KPa for the drains with grouting case after 0.5 day. Following that, the pore water pressure decreases gradually over time. After 5 days of activation, the drains only case reaches 10 KPa, while the drains with grouting case reaches 5 KPa. This clearly indicates that activating the grouting accelerates the consolidation process, leading to lower values of pore water pressure.

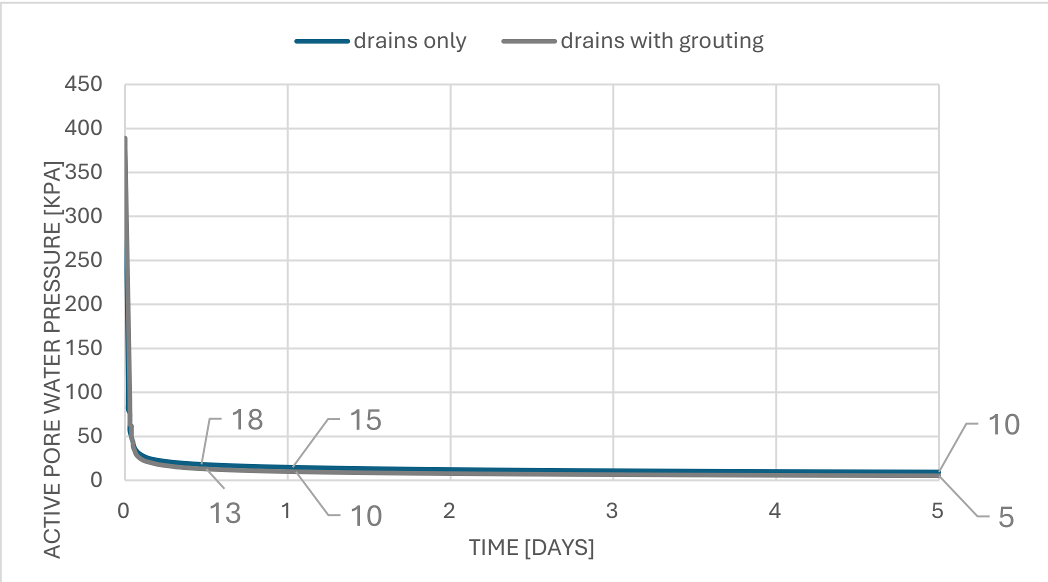


Figure 33: Active Pore Water Pressure Variation Over Time for The Two Case Scenarios ($\mathbf{k = 10^{-7} m/s}$)

In terms of percentage change observed after 5 days of activation:

- The active pore water pressure in the drains only case shows a 97.4% decrease after 5 days compared to the initial state (384 KPa).

- The active pore water pressure in the drains with grouting case shows a 98.7% decrease at 5 days compared to the initial state (384 KPa).

- The active pore water pressure in the drains with grouting case is 50% lower compared to the drains only case after 5 days.

7.7.5 Groundwater Flow

The following output result (Figure 34) represents the distribution of groundwater flow $|q|$ around the tunnel geometry in the case scenario of drains with grouting after 5 days. The maximum groundwater flow values are observed near the drains. Table 12 shows the total discharge in m^3/day for each of the 10 drains, which are identified in Figure 17. This flow rate data is essential for selecting the appropriate diameter for the drains and other design parameters.

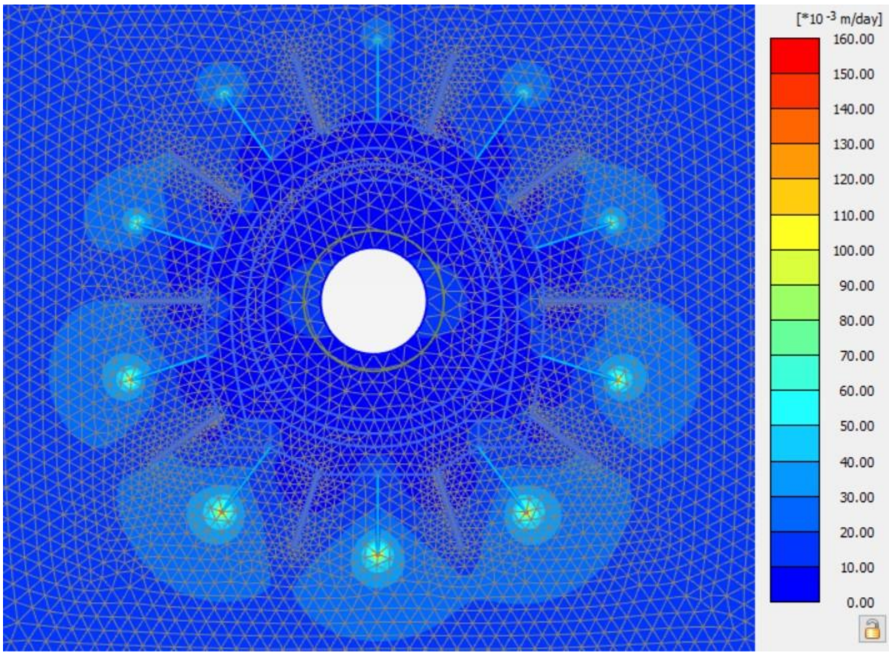


Figure 34: Groundwater flow |q| Distribution after Drains and Grouting Activation

Table 12: Groundwater Total discharge for Each Drain Element

Drain Element	**Total discharge** $[m^3/day/m]$
Drain 1	6.61E-02
Drain 2	4.28E-02
Drain 3	3.32E-02
Drain 4	4.33E-02
Drain 5	6.61E-02
Drain 6	9.92E-02
Drain 7	1.33E-01

Drain 8	1.34E-01
Drain 9	1.30E-01
Drain 10	9.96E-02

7.7.6 Short-Term Strength

Advance drainage improves short-term stability by reducing pore pressure, which increases the effective stresses, consequently increasing the short-term strength (Cu) of the soil. Assuming the theoretical case of elastic perfectly plastic ground behaviour without dilatancy, the short-term strength (Cu) linearly increases with the isotropic effective stress (σ_0') according to the following formula (Anagnostou & Zingg, 2013). Table 13 provides the effective stress (σ_0) and calculated short-term strength (Cu) for two case scenarios over different time intervals (0.5 days, 1 day, and 5 days). These measurements and calculations are done at the selected measurement point, as shown in Figure 19.

$$C_u = \sigma'_0(t_D)\sin\varphi' + C'\cos\varphi'$$

Table 13: Effective Stress and Calculated Undrained Cohesion (Cu) for the Two Case Scenarios Over Time

	Time [days]	$\boldsymbol{\sigma_0}$ $[\boldsymbol{KPa}]$	$\boldsymbol{C_u}$ $[\boldsymbol{KPa}]$	$\Delta\boldsymbol{C_u}$ $[\boldsymbol{KPa}]$
Drains Only	0.5	752	362	262
	1	769	370	270
	5	793	381	281
Drains with grouting	0.5	810	389	289
	1	859	412	312
	5	1068	510	410

In terms of percentage increase observed after 5 days of technologies activation:

- The undrained cohesion in the drains only case shows a 281% increase after 5 days compared to the initial state (100 KPa).

- The undrained cohesion in the drains with grouting case shows a 410% increase at 5 days compared to the initial state (100 KPa).

- The undrained cohesion in the drains with grouting case is 33.9% higher compared to the drains only case after 5 days.

7.7.7 Stiffness After the Improvement

The graph below (Figure 35) shows the variation of stiffness over time for two case scenarios: "drains only" and " drains with grouting". These stress values are measured at a specific measuring point, as in Figure 19.

The trend indicates an increase in secant stiffness over time, with higher values observed in the case of drains with grouting. In terms of percentage increase observed after 5 days of technologies activation:

- The stiffness in the drains only case shows a 121.6% increase after 5 days compared to the initial state ($722\ x\ 10^3$ KPa).

- The stiffness in the drains with grouting case shows a 145.2% increase at 5 days compared to the initial state ($722\ x\ 10^3$ KPa).

- The stiffness in the drains with grouting case is 10.7% higher compared to the drains only case after 5 days.

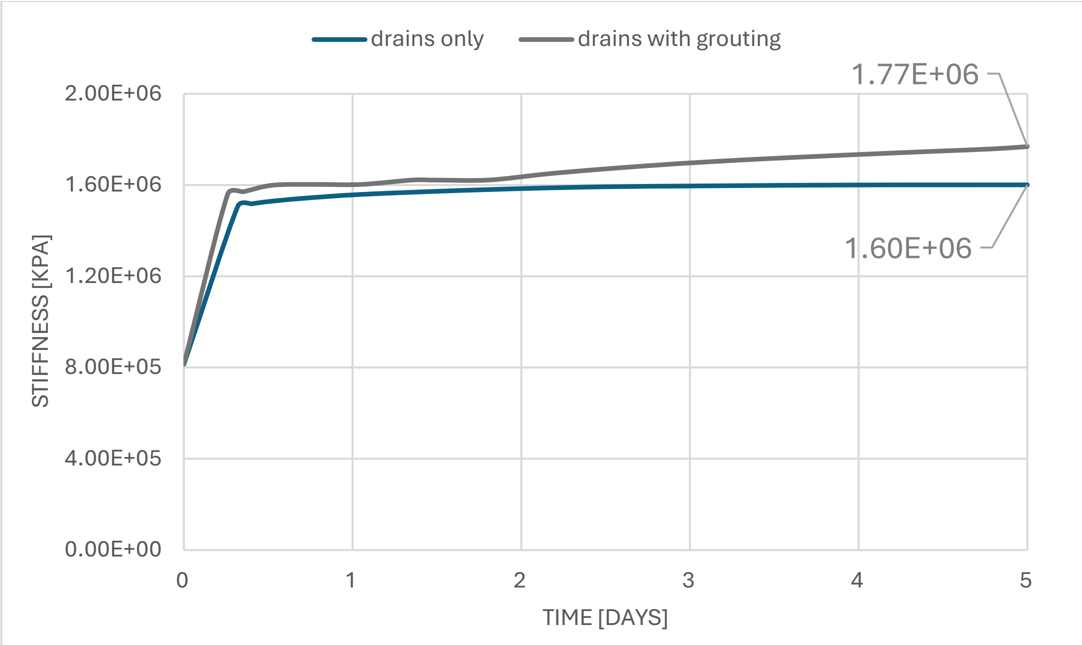


Figure 35: Stiffness Variation Over Time for The Two Case Scenarios ($k = 10^{-7} m/s$)

Figure 36 and Figure 37 provide a visual comparison of the stiffness distribution, with highlighting the measuring point value for the two case scenarios: "drains only" and "drains with grouting".

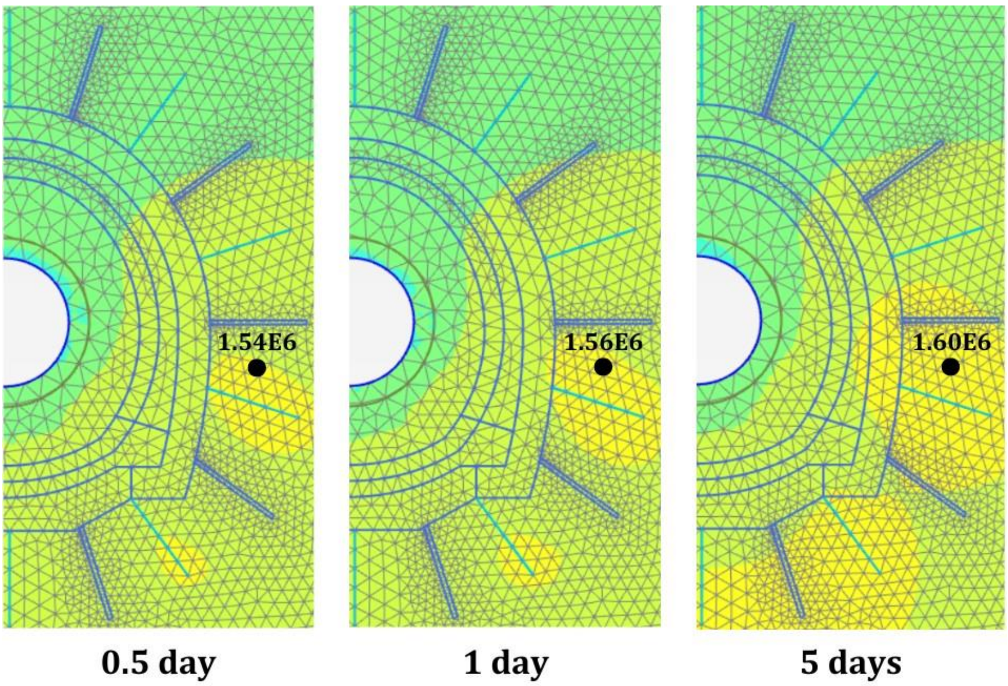


Figure 36: Stiffness Distribution for The Drains Only Case at 0.5 Day, 1 Day, and 5 Days

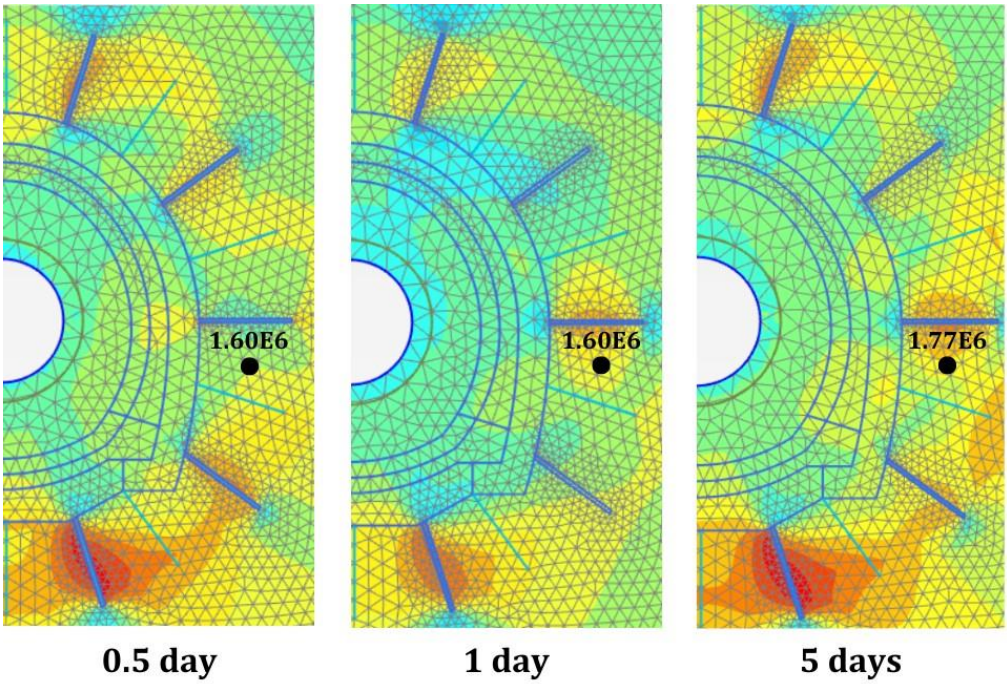


Figure 37: Stiffness Distribution for The Drains with Grouting Only Case at 0.5 Day, 1 Day, and 5 Days

7.7.8 Parametric Analysis

7.7.8.1 Soil Permeability

The consolidation process can be affected by slight variations in the permeability values. Figure 38 and Figure 39 show the variation in isotropic effective stress and active pore water pressure over time for different values of permeability in drains with grouting case. These values were measured at a specific measuring point as shown in Figure 19.

Higher values of soil permeability accelerate the consolidation process. The results prove that increasing permeability lowers active pore water pressure, which leads to higher isotropic effective stress.

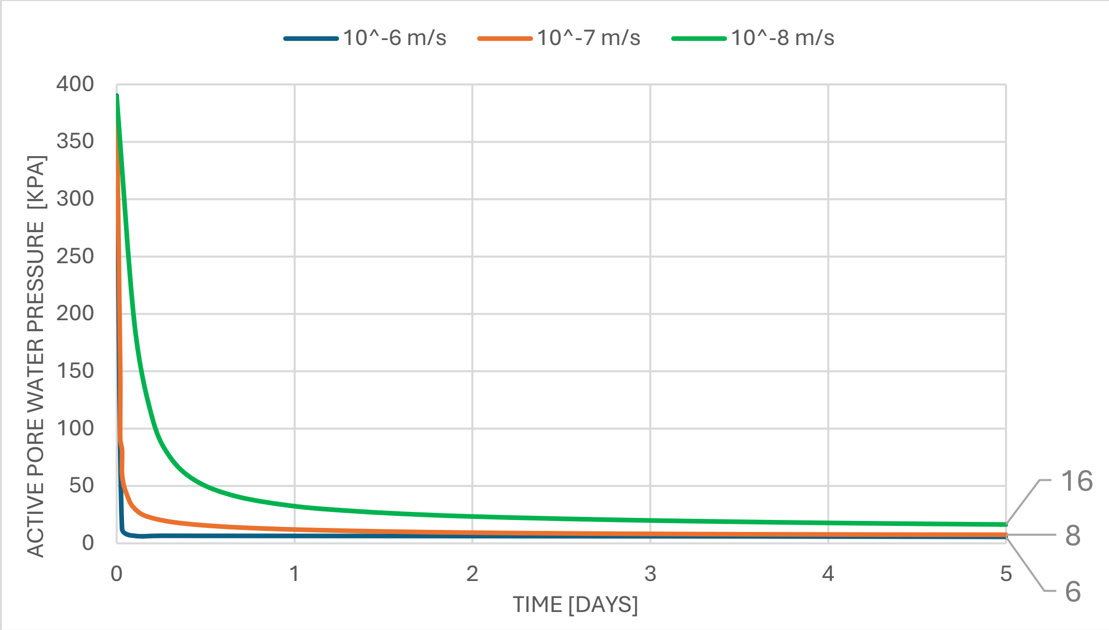


Figure 38: Active Pore Water Pressure Variation Over Time for Different Permeability values

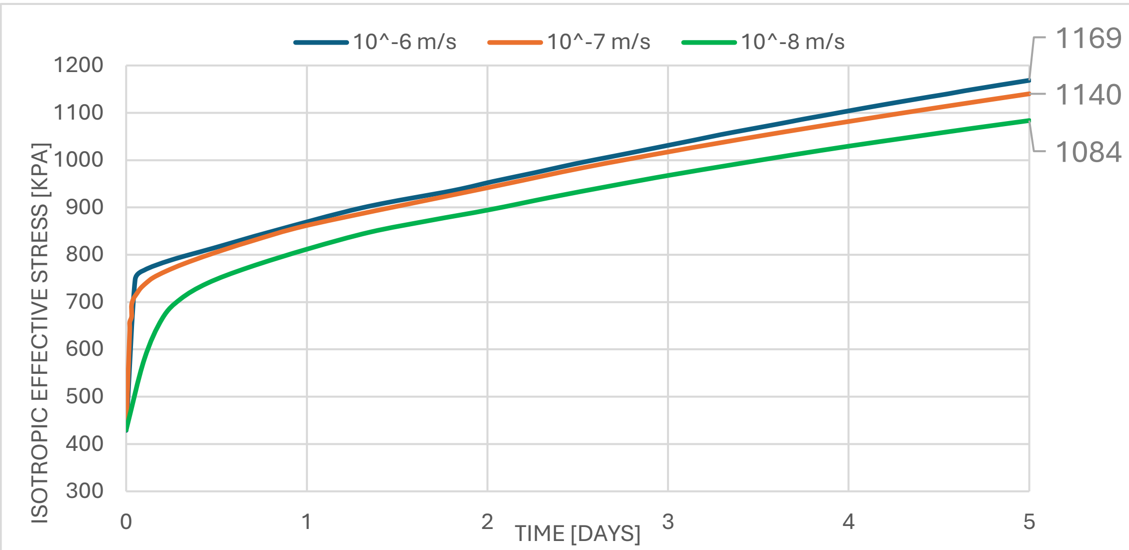


Figure 39: Isotropic Effective Stress Variation Over Time for Different Permeability values

7.7.8.2 Grouting Volumetric Expansion

Figure 40 represents the variation in isotropic effective stress over time for different levels of volumetric expansion in a grouting case, with values measured at a specific measuring point as shown in Figure 19. Higher volumetric expansion gives higher isotropic effective stresses over the analysed period. After 5 days, as the volumetric expansion increases by 10%, 15%, 20%, and 25%, the isotropic effective stresses are 1022 KPa, 1140 KPa, 1233 KPa, and 1315 KPa, respectively. Additionally, in terms of percentage change observed after 5 days:

- The 15% expansion results in an 8% increase in isotropic effective stress compared to the 10% expansion.

- The 20% expansion results in a 15.9% increase compared to the 10% expansion.

- The 25% expansion results in a 23.8% increase compared to the 10% expansion.

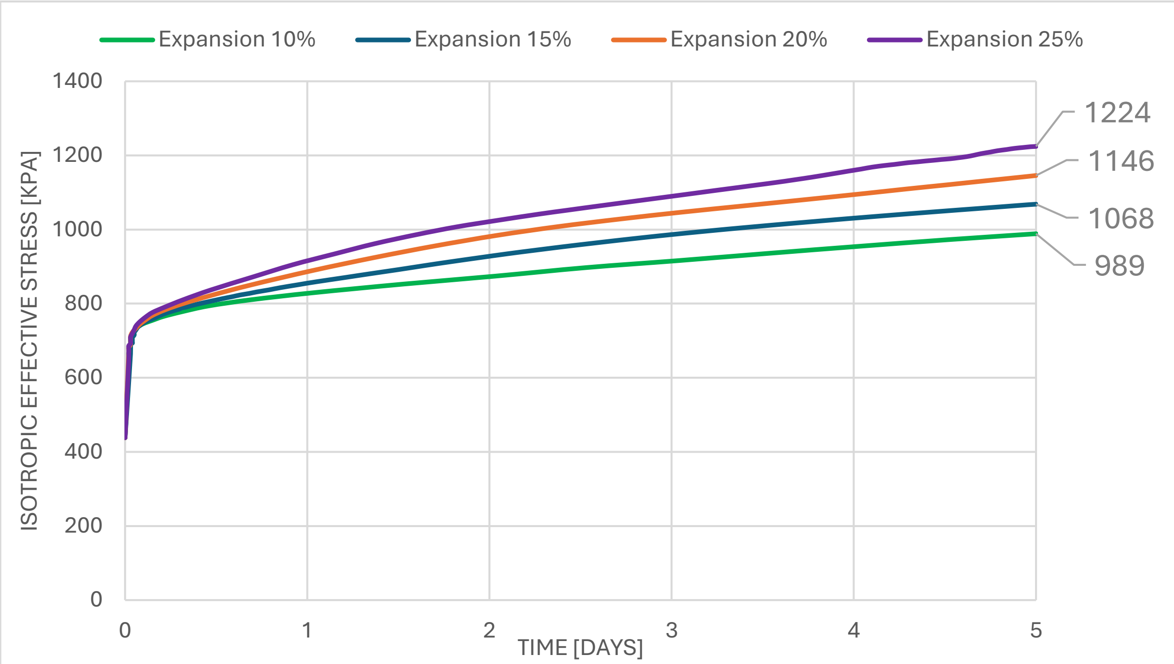


Figure 40: Isotropic Effective Stress Variation Over Time for Different Volumetric Expansions in Grouting ($\mathbf{k = 10^{-7} m/s}$)

8 Conclusion

The poor geotechnical properties of saturated clay can lead to significant difficulties during tunnel excavation, particularly causing ground instability. This thesis presented ground improvement techniques, with a particular focus on the application of drains and the innovative capsule grouting technology (CGT) for tunnel excavation in saturated clay mediums. Pre-excavation drainage of clayey layers is typically considered effective. Additionally, Innovative capsule grouting technology (CGT) was used to create a localized densification, as it addresses the challenges posed by heterogeneous ground conditions.

The numerical analysis was carried out using Plaxis 2D to simulate two different scenarios: ''Drains only'' and ''Drains with grouting'', and to evaluate the effectiveness of these techniques in improving ground conditions. The results show significant improvements in active pore water pressure, isotropic effective stress, cohesion, and stiffness. Based on the input parameters used in this thesis, the following results were observed.

Active pore water pressure decreased significantly after 5 days. In the "drains only" scenario, there was a 97.4% reduction in active pore water pressure compared to the initial state. In the "drains with grouting" scenario, the reduction was even more pronounced, with a 98.7% decrease. These results indicate that the presence of grouting accelerates the consolidation process. Consequently, isotropic effective stress increased greatly after 5 days. In the "drains only" scenario, there was a 106% increase in isotropic effective stress compared to the initial state. In the "drains with grouting" scenario, there was a 177.4% increase.

As a result, both ground strength and stiffness showed notable improvements. Undrained cohesion increased significantly after 5 days. In the "drains only" scenario, there was a 281% increase in undrained cohesion compared to the initial state. In the "drains with grouting" scenario, the increase was even more remarkable, with a 410% rise. Additionally, stiffness increased considerably after 5 days. In the "drains only" scenario, there was a 121.6% increase in stiffness compared to the initial state. In the "drains with grouting" scenario, there was a 145.2% increase.

Finally, a parametric analysis has been carried out on soil permeability and grouting volumetric expansion to understand their impact on the output results. The analysis revealed that higher values of soil permeability accelerate the consolidation process. Furthermore, higher grouting volumetric expansion results in increased isotropic effective stresses over the analysed period.

Future research could investigate the integration of smart monitoring systems with drainage and capsule grouting technologies to provide real-time data on soil and structural conditions. This integration would validate the numerical predictions and manage these techniques by monitoring key parameters such as pore water pressure, stress levels, and soil movement.

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