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Master's Degree Thesis

Evaluating design alternatives for the rehabilitation of the Riserasco tunnel

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2026

إلى عائلتي العزيزة

إلى من كانوا سندي في كل خطوة، ورافقوني بالدعاء والمحبة والصبر

أهديكم هذا العمل عربون امتنانٍ لما قدمتموه لي طوال الطريق

Abstract

The thesis proposes a design alternative for the restoration project of an old existing hydraulic tunnel. The tunnel serves as a hydraulic connection between an impluvium and an artificial lake. The design study is carried out using numerical modelling techniques. A comprehensive review of geological, geotechnical, and environmental data was performed to characterize the ground conditions and define the parameters required for the design.

A numerical model based on the finite element method (FEM) was developed using RS2 software to simulate the excavation, tunnel installation, and backfilling stages. Several excavation and stabilization strategies were evaluated, including open trench slopes with different inclinations, berm configurations, and structural reinforcement using micropiles. Four design alternatives were analyzed and compared. The stability of each alternative was assessed using the strength reduction method to determine the factor of safety, in accordance with Eurocode 7 and NTC 2018 requirements.

The results show that multiple excavation configurations satisfy the minimum safety requirements. Among them, the open trench slope with an inclination of 1.5H:1V represents the optimal solution. This configuration ensures adequate stability while minimizing excavation volume, construction costs, and environmental impact.

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Chapter I

1.1. Introduction

The use of artificial tunnels plays a significant role in construction engineering, particularly in managing rainwater collection and maintaining controlled water pathways. Such infrastructure is especially valuable for small towns, enabling them to store and redirect water efficiently. This concept will be explored through a case study in *Pralormo*, a town in the metropolitan city of Turin, Italy. A hydraulic tunnel that transports water from an impluvium to a nearby lake, has a damaged part that collapsed due to aging. The aim is to make a design for the restoration part of the collapsed tunnel. The tunnel will be constructed beneath a soil layer that requires excavation depths ranging from 5 to 9 meters. The tunnel will be embedded and permanently covered by backfill material. The study includes review of the geotechnical characterization that define the surrounding ground, as well as the properties of the structural tunnel elements and the characteristics of the backfill material used to cover the tunnel after installation.

1.2. Scope of the Thesis and case study

The aim of the thesis is to develop a numerical model that simulates the replacement of an old tunnel, taking into account the excavation stages, the tunnel replacement process, and the backfilling phase. In this thesis, an assessment will be made in order to analyze the excavated volume, to understand how to minimize the size of excavation, to analyze the slope stability. The tunnel was made in order to connect an impluvium and a lake as shown in Figures 1.

Figure 1 Photo showing the location of the tunnel with reference to Spina's Lake. Source: Google Maps

Chapter 2

Artificial tunnels: classification, types and specific aspects for hydraulic purposes

2.1. Introduction

Tunnels are underground structures that are classified according to their purpose, shape, excavation method, and support system (Sood, 1984). Hydraulic tunnels have a specific role within this broad category, since they are designed to convey water for applications such as hydropower, irrigation, drainage, and water supply (Brekke and Ripley, 1993). For this reason, their design does not only involve geotechnical and structural aspects, but also hydraulic performance, watertightness, and long-term durability (Wang et al., 2024). This chapter provides a general overview of the classification and main types of artificial tunnels, with particular attention to those used for hydraulic purposes. It also introduces the main excavation and construction methods commonly adopted in tunnel engineering, as these aspects strongly influence the design, stability, and overall performance of the tunnel (ITA-AITES, 1988).

2.2. Classification of artificial tunnels

Tunnels can be classified according to several criteria, among which cross-sectional shape is one of the most important. The selected shape depends on geological and geotechnical conditions, construction requirements, and the intended function of the tunnel. Among the different tunnel shapes, some sections are mainly chosen for their structural efficiency, while others are selected to satisfy specific hydraulic or construction requirements.

Circular tunnels

Circular tunnels provide good structural efficiency and hydraulic performance, which makes them suitable for the wide range of flow rates and velocities (Figure 2). However, circular tunnels are hard to be excavated and especially if the section area is really small (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984). Besides circular sections, other tunnel shapes are often adopted when greater usable space or different structural behavior is required.

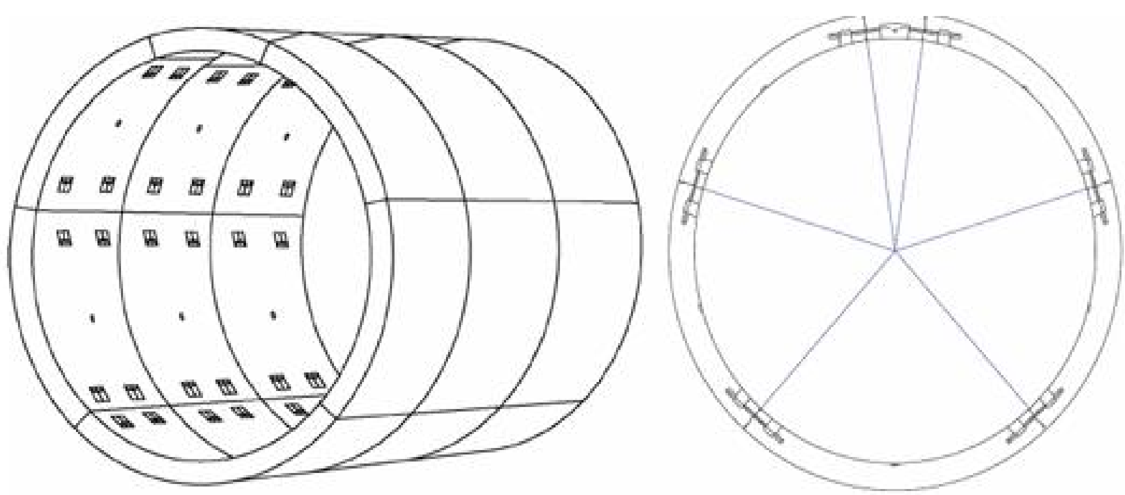


Figure 2 Sketch of a Circular Tunnel (Quing Ai, 2017)

Horseshoe tunnels

These sections provide additional headroom and a larger usable volume. They are structurally strong to withstand external rock and water pressures. Horseshoe-shaped tunnels provide additional headroom and accommodate a larger usable volume. The arched crown offers good structural strength by efficiently transferring rock and groundwater pressures to the side walls (Figure 3).

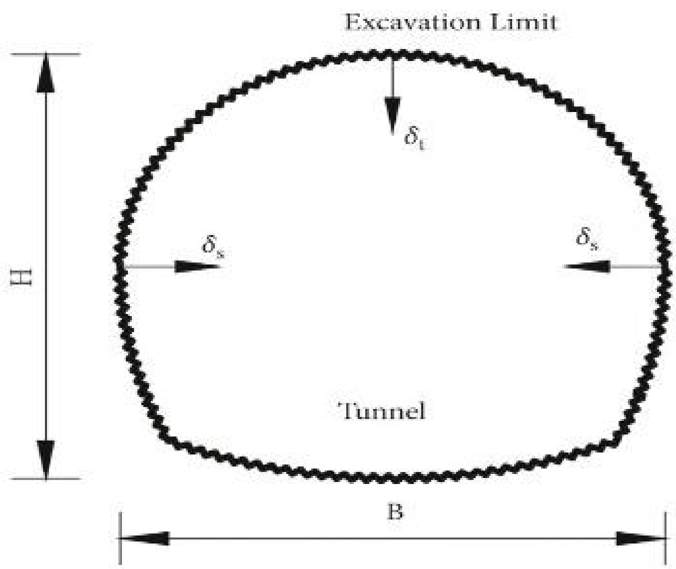


Figure 3 Sketch of a Horseshoe Shape Tunnel

While curved sections are generally preferred for their structural efficiency, other geometries may be adopted when construction simplicity or space utilization becomes more important.

Rectangular tunnels

Rectangular tunnels are used due to their structural efficiency and ease of construction especially for urban environments. In contrast to sections mainly selected for structural or construction reasons, some tunnel shapes are specifically developed to improve hydraulic performance.

Egg-shaped tunnel

An egg-Shaped Tunnel is designed to optimize flow characteristics, reduce sediment deposition, and minimize flow resistance, making them suitable for sanitary sewers and storm water tunnels.

D-shaped tunnel

D-shaped section is used to fulfill storm water management and waste water conveyance (Figure 4). The main advantages of this section are the added width of the invert which gives more working floor space in the tunnel during driving and flatter invert. This helps to eliminate the tendency of wet concrete to slump and draw away from the tunnel sides.

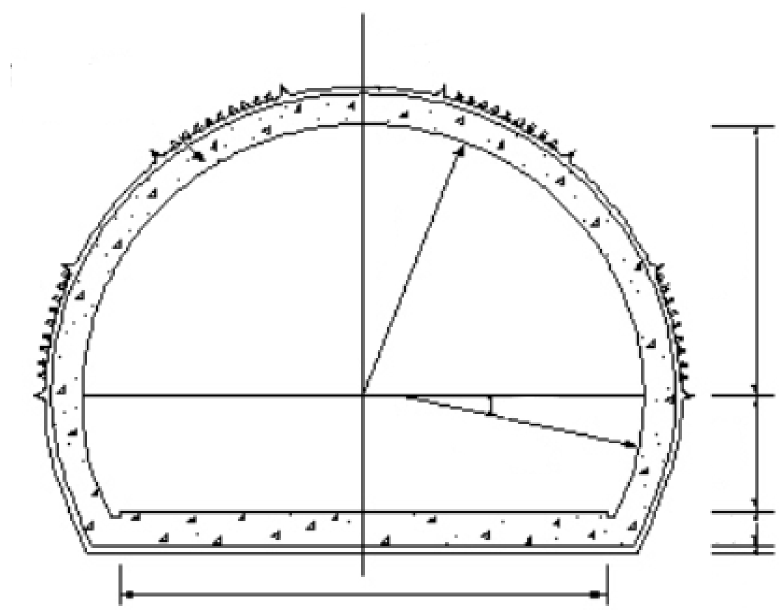


Figure 4 Sketch of a D-shaped Tunnels

Arch-tunnel

An arch tunnel is a curved, semi-circular underground passage designed to efficiently distribute external loads (figure 5). It is used to manage small flow conveyance. The choice of the tunnel is influenced by the clearances specified in view of the vehicles and materials transported in the tunnel, geological conditions, method of driving the tunnel, and material and strength of tunnel lining. Additionally, the choice of the tunnel shape in hydropower projects depends on the hydraulic efficiency needed to transport water to turbines, manage sediment transport and maintain structural integrity. For the Riserasco tunnel considered in this thesis, an arch-shaped section was selected.

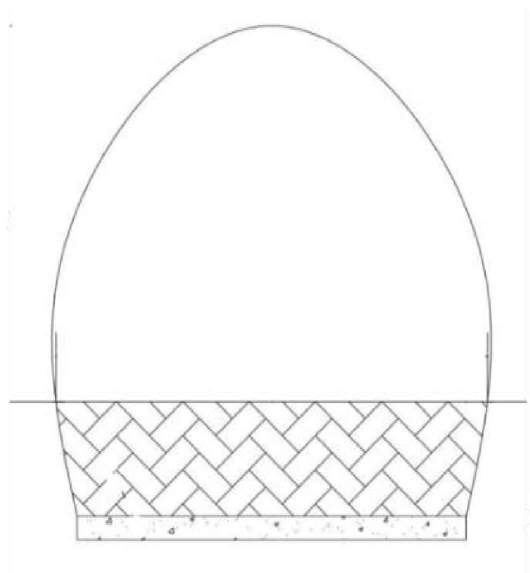


Figure 5 Sketch of arch-shaped tunnel (Consorzio di secondo grado Chierese-Astigiano, 2009)

2.3. Methods of excavation for artificial tunnels

Artificial tunnels can be excavated using different construction methods, ranging from underground excavation techniques to open excavation methods. Among the main methods considered in this study are the sequential excavation method, roadheader excavation, shield tunnelling, the Vertical Tunnelling Method (VTM), and the cut-and-cover method. These methods differ in terms of construction sequence, ground support requirements, applicability, and cost. In order to understand how to choose the best way of excavation, it is important to highlight certain factors, including ground conditions, ground presence, tunnel depth, and sit constraints.

Cut and cover is a method of excavation for artificial tunnels. One of the most important aspects influencing the excavation method is the nature of the surrounding ground. The condition of the rock or soil plays an important role in dictating the method of excavation. Digging in hard rock differs from digging in soft one. Sometimes the rock is so thoroughly crushed or decomposed that it does not require blasting and can be easily excavated by the use of hydraulic shovels. Sometimes, a rock tunnel may intersect buried valley filled with soft-clayey sediment. At times a tunnel may intersect a saturated strata having seepage or flowy conditions. One of the most uprising problems, is when they intersect strata having “Squeezing” or “Swelling” conditions. Squeezing is applied to rock formation which squeezes into excavated opening unless held back by an effective support system (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984). If the clay content has montmorillonite as its main component (which has a great affinity for water) the squeeze is chiefly due to volume expansion and the rock is termed as “Swelling rock”. Several previous case studies are presented to provide additional information for selecting the excavation method. These examples help illustrate how different ground materials influence the choice of excavation technique. Several case studies reported in the literature help explain how different ground conditions influence excavation choices and support measures.

First, the cases of Sierra Madre tunnel near California, U.S.A and Kudremukh Iron Ore Project-slurry pipe line tunnel were carried in a soft ground. In this type of material, engineers use fore poling method in combination with steel supports.

Boards are installed ahead to stabilize the ground until their forward ends are supported by installing the next rib. Spills are set instead of the boards around the sides of the tunnel (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984).

Another case involves soft ground with water-bearing strata. Porous ground produces the danger of loss of ground where the soil is washed into the tunnel (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984). In this case, it is important to make a drainage process (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984). Materials with low permeability such as silty sand are easier to tackle as the quantity water which seeps into the tunnel out of such material is relatively small and therefore does not interface with the tunnelling operations. A big example is the D-shaped head race tunnel of the *Chuckha* project. The presence of the perched water that got suddenly released as soon as the face of the tunnel was removed. There was a very heavy flow of the order of 20 cusecs initially which got reduced to about 3 cusecs after about 48 hours. The tunnel was filled with rock debris up to about RD 570m, where a large fragments of gauge material acting as a barrier against the flow of perched water in the hill mass. After a while, the puncture occurs leading to the release of water that brings with it huge quantities of this material to the tunnel filling it up and blocking the flow. This leads to the collapse of the steel ribs inside the tunnel. The solution was made by removing the debris slowly, first from sides followed by instantaneous erection of steel rib posts. Suitable drainage holes sufficiently deep were driven, one at the crown and one each on the sides to tap any further seepage water (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984).

Furthermore, the main excavation methods commonly adopted in tunnel engineering can be briefly described.

Conventional methods of excavation

Conventional tunneling methods (CTM) are commonly employed when the use of tunnel boring machines (TBMs) is impractical due to challenging ground conditions, budget limitations, or specific project demands. These methods rely on a sequential process of excavation and support installation to maintain tunnel stability (Sood, Manual on the Planning and Design of Hydraulic Tunnels, 1984). Important factors that also affect the choice for the excavation method which is the geological aspects, operational requirements, constraints and hazards (Peila, 2022). In the case of *Lago della Spina*, the tunnel is shallow and the process for construction starts by excavation from the top surface and in the end, an embankment has to be installed. In this case, the most frequently utilized conventional tunneling techniques are classified as follows:

Sequential excavation

Sequential excavation is typically used for large tunnels. (Peila, 2022). It is performed by dividing the face excavation into different phases as described in figure 6. The excavation is carried out in small increments starting from the tunnel crown top. This is followed by a temporary support that has to be installed immediately after each excavation (Peila, 2022). Once the entire tunnel is excavated and temporarily supported, final arch lining is installed (Peila, 2022). This method is suitable to be made for soft soils or ground prone to collapse. It offers flexibility and good support during excavation.

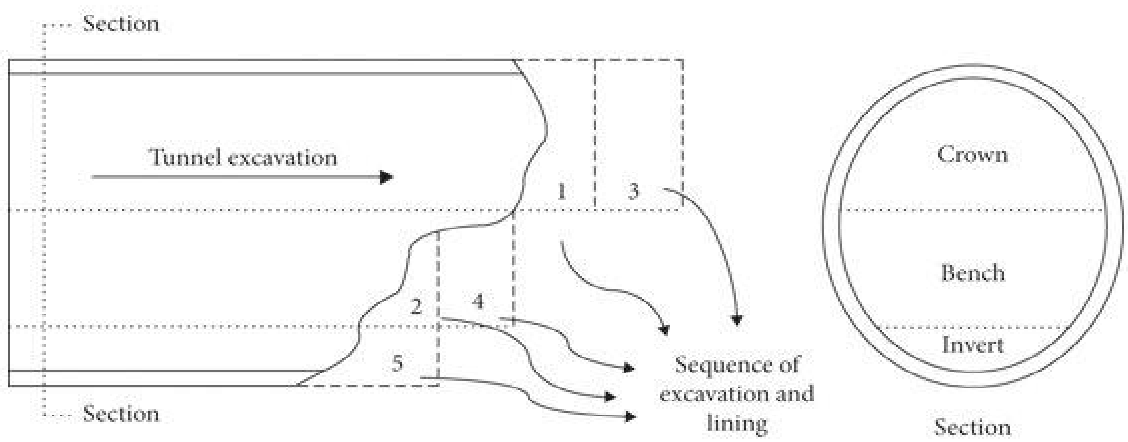


Figure 6 Sequential Excavation Method (Peila, 2022)

Road heading excavation method

The excavation method requires the use of machines (Peila, 2022). A road header machine is suitable for excavation in relatively weak or fractured materials and consists of a cutter head in the front as described in Figure 7.

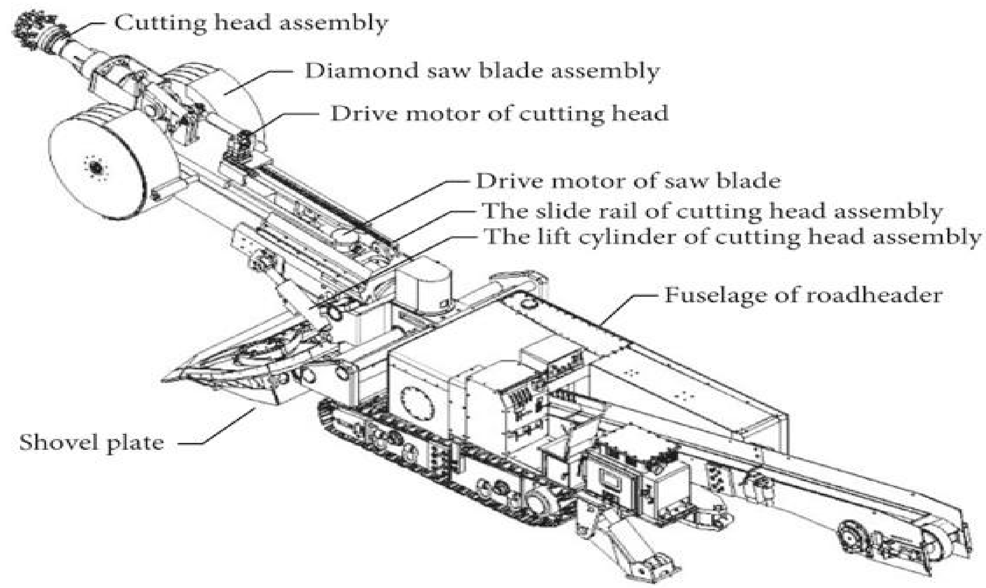


Figure 7 Anatomy of a roadheader machine (Peila, 2022)

Shield method of tunnelling

This method is preferably used in shallow underground excavation where the risk of sudden collapse is high. It is made up of circular shield thick steel plates supported by adequate stiffness. The shield is pushed by hydraulic jacks so the lining can be held in place. Then, the gap that is formed between the lining and the soil is filled with 12 mm size gravel and packed with cement grout (Sood, 1984).

For shallow tunnels constructed from the ground surface, open excavation methods are often more practical than underground excavation techniques.

Cut and cover method

Cut and cover method is a familiar method to excavate tunnels. The name "cut-and-cover" mainly describe the three main stages: cut, construction and cover. Structures constructed using the cut-and-cover method can include underground stations, underpasses, culverts, or hydraulic tunnels., water parks, underpass, culverts, pumping stations, metro stations and running tunnels. The structure held by this method in brief description can be defined as a confined underground as described in the book *Cut and Cover Metro Stations* (Kaul, 2010).

From the general point of view, this method requires three major steps. First stage is done by excavating to the required depth. There are several ways to do this step, that will be discussed later on. The next stage is done by creating the structure needed that will convey the purpose of the construction. Then, the structure is then filled to cover the pre excavated part.

The excavation and construction of the structure is carried out through several ways. Some of them are open cut with "stabilized side slopes" as shown in figure 8 and an open cut with "vertically retained slopes" as described in the figure 9.

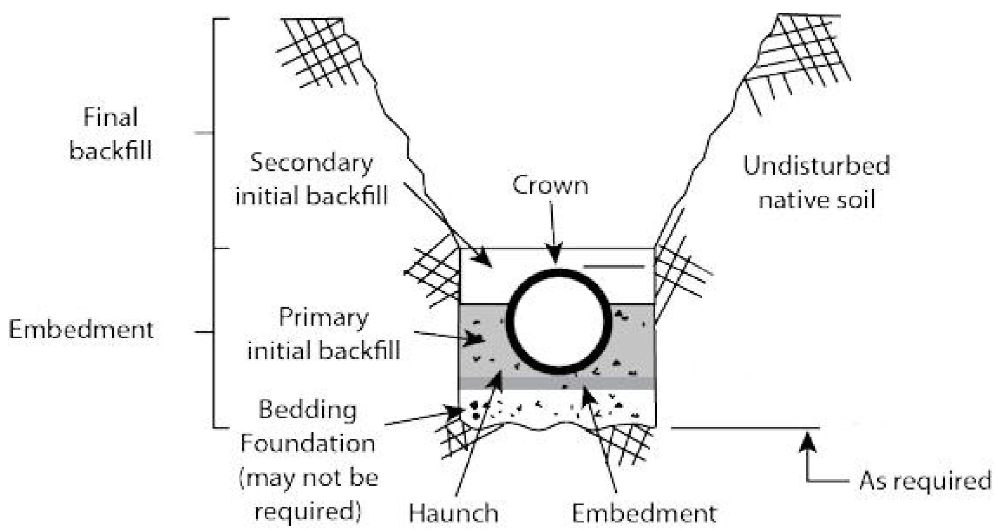


Figure 8 Example of open-cut excavation with stabilized side slopes (www.plasticpipe.org)

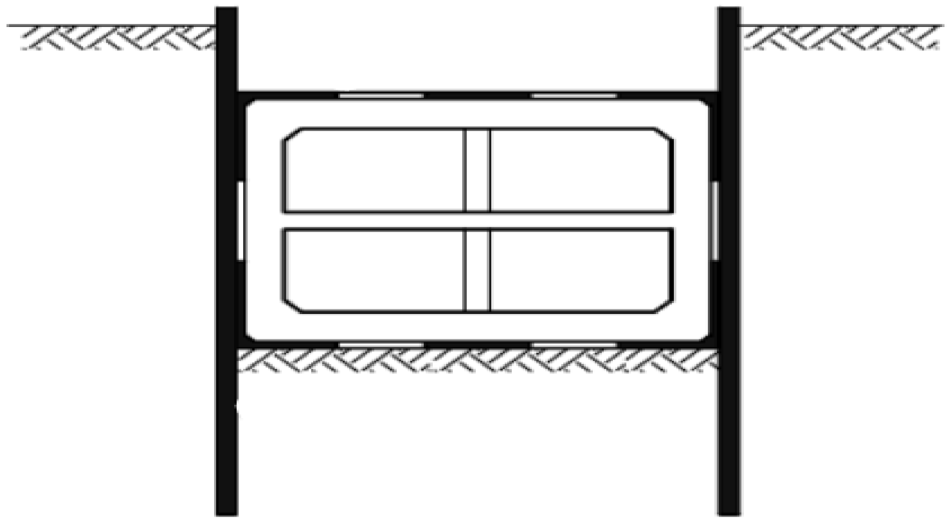


Figure 9 Open cut with vertically retained side slopes (Kaul, 2010)

It is important to mention that the choice for the most adequate way for construction depends on the several factors. The method of excavation in this case study will be the cut and cover method where the way of excavation is a matter of debate and will be discussed later on.

Depending on the available space and the stability of the ground, the cut-and-cover method can be carried out using different excavation configurations.

Open cut stabilized side slopes

One of the typical methods for constructing a cut-and-cover tunnel with stabilized side slopes begins with a general excavation, ensuring proper groundwater control throughout the process. As the excavation progresses, ground support systems must be installed to maintain stability. Once the excavation is complete and adequately supported, the principal structure is constructed within the prepared space. Finally, the process concludes with the replacement and reinstatement of the backfill**,** restoring the surface to its original or intended condition (Kaul, 2010).

According to Kaul (2010, p.32), The extent of the hole to be made in the ground will depend on the size (plan area & depth) of the structure. The design criteria for the sides of the cut must satisfy the stability against slips and failures. It is important to ensure a profiling of the sides to form stabilized slopes that are self- supporting.

Open cut vertically retained side slopes

This method mainly consists of a diaphragm wall that covers the sides of the excavation. The use of this method as structural diaphragm walls in the construction of cut and cover started in the late 1950s. The plan length of the diaphragm walls is dictated by the several aspects like stability, extent of concrete pour, the need to minimize the movements of the surrounding and the structures (Kaul, 2010, p. 101). Due to these aspects, this method is considered expensive.

One of the most familiar ways of construction of the tunnel is the top-down construction. The construction is made by installing a diaphragm walls or sheet pile to form the sides of the excavation. Then, by installing a temporary roof, an excavation can be performed underneath the roof to form the tunnel space and construct the arch lining. Finally, a backfill above the roof after completion is managed.

The stages of construction start by installing a boundary wall to provide the main ground support system. Then a traffic deck could be placed as a temporary retaining structure for soil stabilizing and waterproofing during the construction process. After that, a ground control system is managed. Then, the excavation starts so the wall supports and construction of the structure is maintained. In the end, a necessary backfill is provided (Kaul, 2010).

Vertical tunnel method (VTM)

Vertical tunneling method is usually used for the construction of hydraulic tunnels. It was invented in 1974 (Wang et al., 2021). The process is made first by constructing a horizontal tunnel section by section, where every section is reinforced to maintain the stabilization of the soil around the section. An installation of working platform is followed at the end of the horizontal tunnel. The platform consists of a standpipe section. The standpipe is used to create a jacking force in each excavated section in order to test the soil and prevent a fallback measure (Wang et al., 2021). After the completion of the test. In this case it is possible to reach the point of excavating the horizontal tunnel (Figure 10).

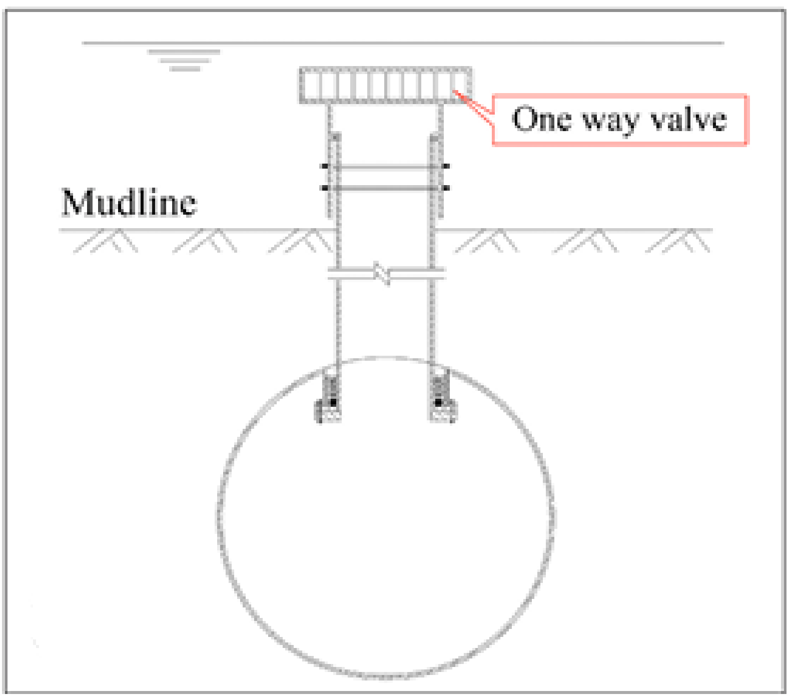


Figure 10 Vertical Tunnelling Method (Wang, 2021)

These excavation alternatives provide the basis for identifying the most suitable construction method for the tunnel examined in this thesis.

2.4. Specific aspects for hydraulic tunnels

Tunnelling for hydraulic purposes requires different calculation methods. Key aspects in order to provide a satisfactory modelling scheme are a relatively small extent of the ground volume affected by the drainage effect of the tunnel, a well-defined and predictable hydro-geological conditions, and a temporary support system (of tunnel face and wall), final lining and water pressure control systems explicitly represented in the model (Peila, 2022). The tunnel must be designed to handle the expected discharge while maintaining non-erosive flow velocities. This is influenced by tunnel slope, cross-section, and lining materials.

Chapter 3

Description of the Riserasco tunnel

3.1. Introduction

The Riserasco tunnel is an existing hydraulic tunnel used to transport water. Due to aging, its structure has deteriorated and experienced collapse, making reconstruction necessary. The tunnel was originally designed to convey water from the Rio Riserasco riverbed to Spina Lake. The tunnel has an arched shape structure with an overall height of 1.4 m and a width of 0.9 m. The drawing of the tunnel is shown in Figure 11.

The project is located in complex ground conditions, and therefore an extensive geological and geotechnical investigation is required on the geological and the geotechnical level. Figure 12 gives an overall view of the tunnel that represents the head, height, and surrounding terrain.

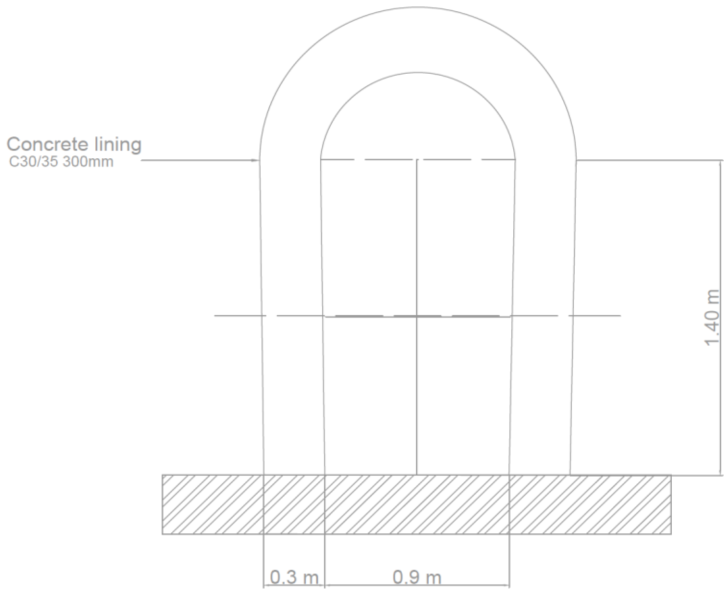


Figure 11 Cross section of the tunnel

3.2. Geological conditions

The report prepared by the Municipality of Pralormo provides the basis for describing the geological conditions that surrounds the tunnel.

The longitudinal geological profile is illustrated in Figures 12 and 13.

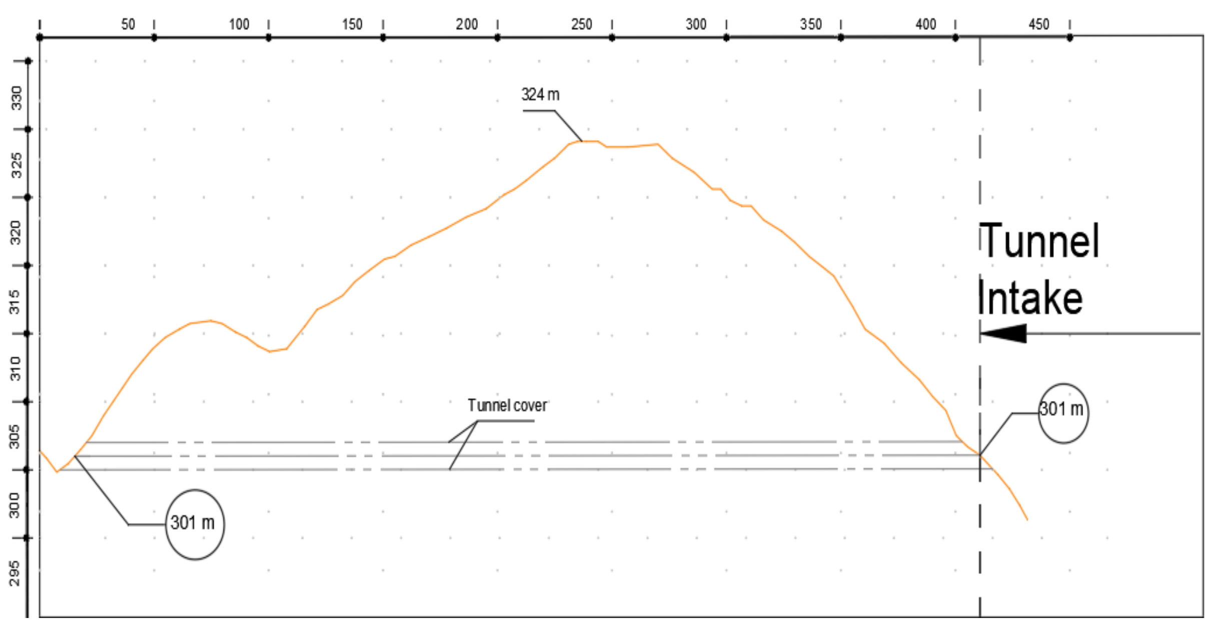


Figure 12 2D elevation map showing the pathway, the slope, and the height of the tunnel

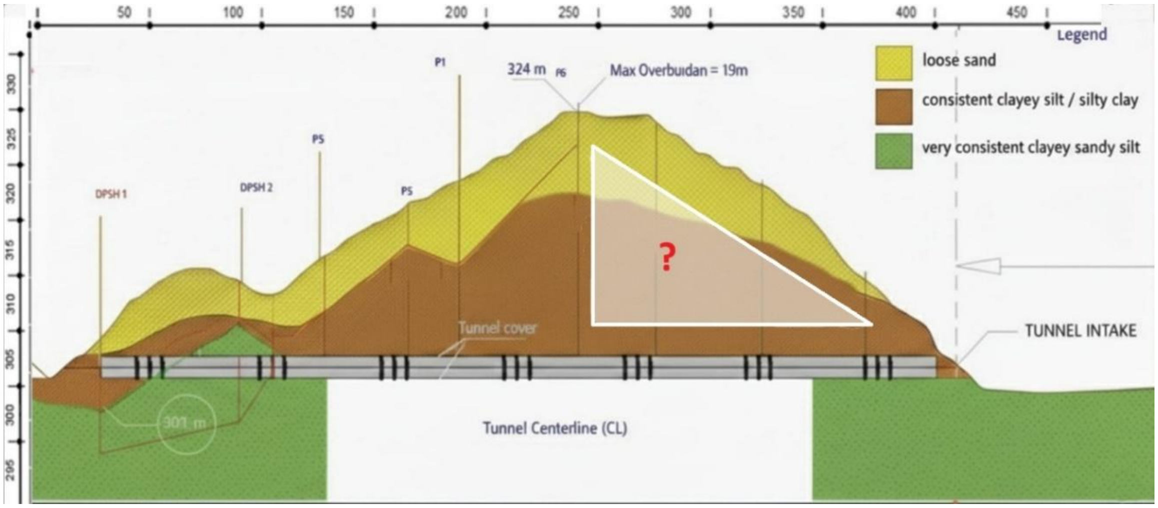


Figure 13 Geological 2D model

Figure 14 Photography of the area surrounding the tunnel.
Source: Actis-Giorgetto (2023, p.9)

Figures 14 and 15 show a photographical presentation of the area surrounding the tunnel and a topographical layout map that shows the geological map.

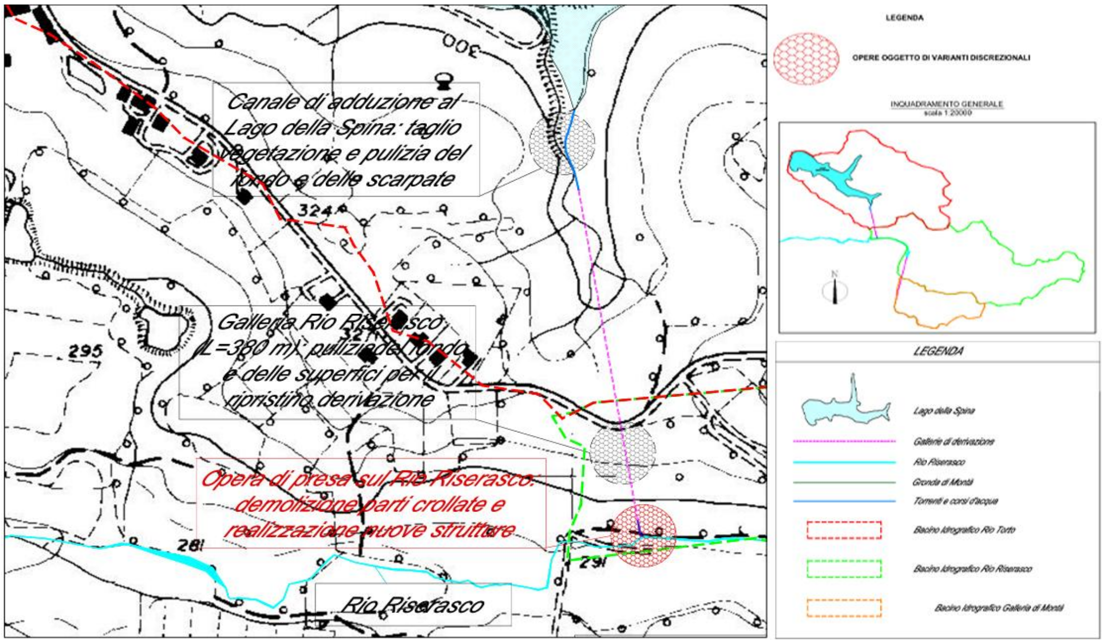


Figure 15 A topographical layout map that shows the geological map representing the elevations and territories surrounding the pathway of the tunnel adaptedfrom Actis-Giorgetto (2023, p.9).

As mentioned in the report "*Progetto di ripristino Galleria del Riserasco - Lago della Spina",* the territory of Pralormo is located in the central part of the Poirino Plateau. Poirino Plateau is composed of Pleistoscene and Holocene deposits overlying Pleo-Pleistocene formations, which include both continental facies (*Villafranchian*) and marine facies (*Asti Sands*). These deposits reflect alternating terrestrial and marine sedimentation influenced by past climatic and tectonic events. This plateau consists of the unconsolidated, surface-level sediments deposited later by the *Po & Tanaro river basins*. Deeper to the ground the *Carmagnola sheet forms* (*Falda di Carmagnola*). This tectonic sheet consists mainly of deformed sedimentary rocks, often covered by subsequent erosion and deposition. Both *Carmagnola sheet* and the *Poirino Plateau*, was formed over each other to form the geological structure of the *Piedmont region*. Analogically, Think of it like a rug on top of a hardwood floor. The hardwood floor (*Carmagnola Sheet*) was put in place first by tectonic movements. Then, over time, dust and dirt (*Poirino Plateau* *sediments*) settled on top, eventually forming a thick layer. Figures 16 and 17 represent the geological maps of the Carmagnola Sheet and the *Poirino Plateau* at a scale of 1:50,000 respectively (*Forno MG, 1982*).

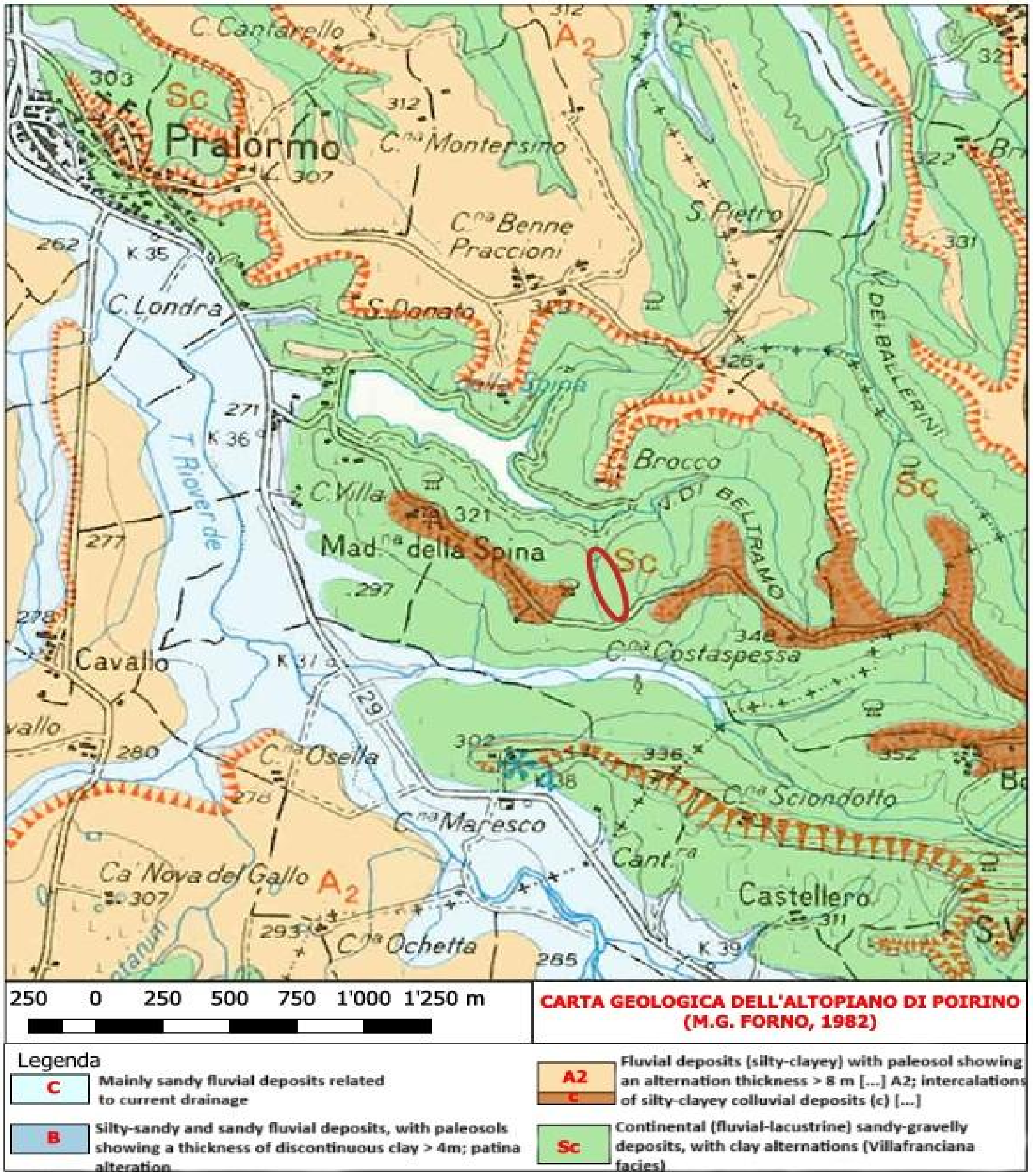


Figure 16 Geological map of the Poirino Plateau. Source: Actis-Giorgetto (2023)

Figure 17 Geological map of the Carmagnola area. Source: Actis-Giorgetto (2023)

From the maps, it can be clearly seen that the soil around the case study area includes fluvial-lacustrine sandy-gravelly deposits with clay alternations (*Villafranchiana facies*) in the *Carmagnola Sheet*, and terrace systems with clayey-sandy-gravelly deposits and yellow-reddish paleosol in the *Poirino Plateau*.

3.3. Geotechnical and stratigraphical context

A geognostic survey was carried out along the tunnel alignment examination along the route of the tunnel to define the structure of the stratigraphic layers and the geotechnical characteristics. This survey consists of 8 tests (Figure 18), 2 dynamic SCPT penetrometric tests carried out with a super heavy dynamic penetrometer (DPSH) and 6 dynamic penetrometric tests carried out with average penetrometer (DPM). DPSH and DPM were converted to SPT. The equivalent SPT blow count was obtained by converting the DPSH results using the ratio between the specific energy per blow (Q) of the DPSH and SPT tests. The conversion coefficient β was calculated from the test parameters by assuming standard equipment characteristics in accordance with ISO 22476-2 and ISO 22476-3. The conversion coefficient is equal to 1.01.

$$Q = \frac{M^2.H}{A.\delta.(M + M')}$$

Where:

- M= hammer mass
- M'= rod mass
- H= drop height
- A= cone area
- δ= penetration step

Conversion Coefficient is:

$$ß = \frac{Q}{Q_{SPT}}$$

The equivalent SPT values were therefore obtained as:

$$N_{SPT} = ß.\,N_{DPSH}$$

The DPM30 test provides the penetration resistance expressed as the number of blows required to penetrate 10 cm. In order to compare these results with the

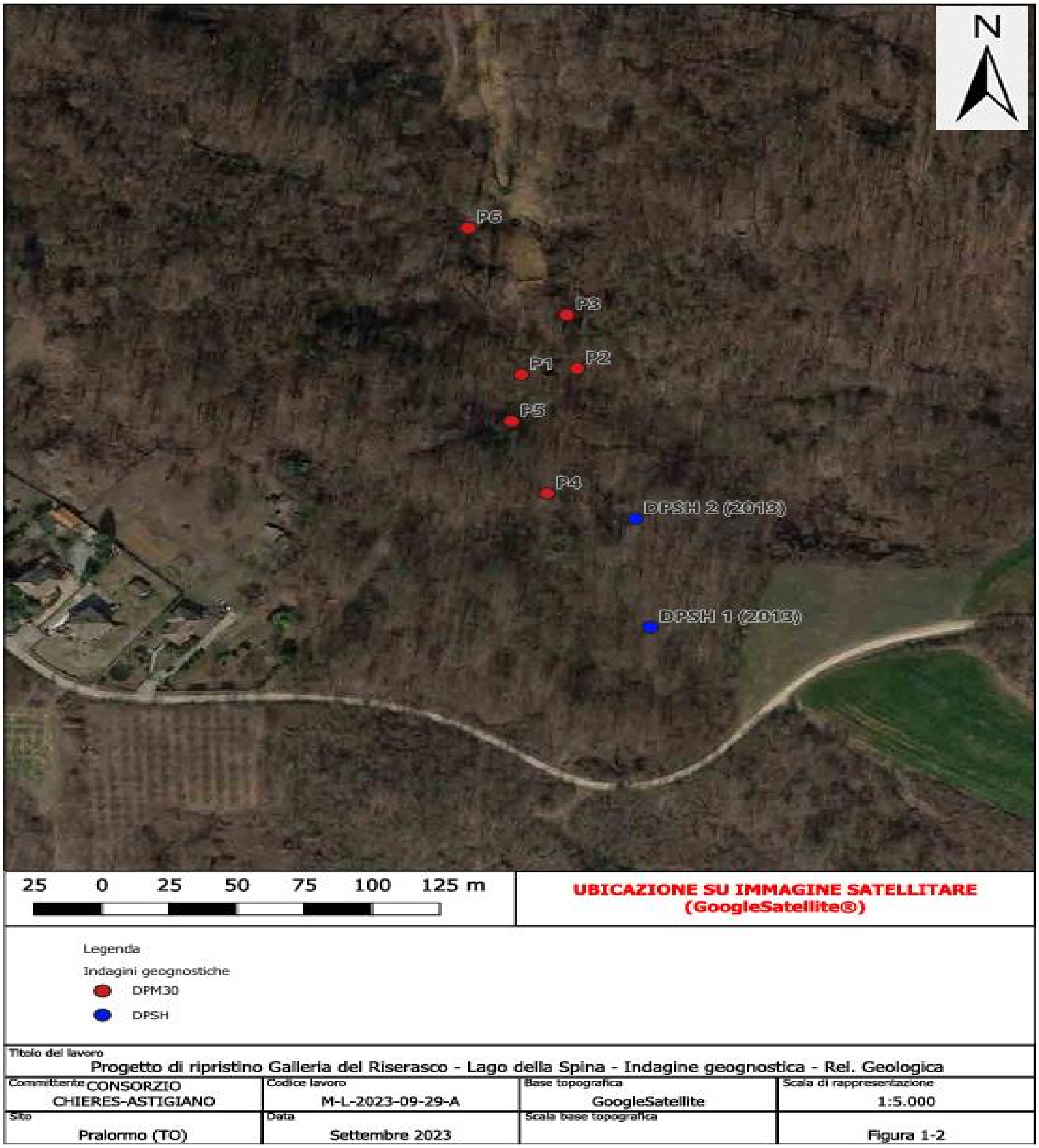


Figure 18 Location of the geotechnical tests. Source: Actis-Giorgetto (2023)

Standard Penetration Test (SPT), the measured values were converted into equivalent SPT blow counts using empirical correlations.

The recorded blow counts were converted into equivalent NSPT values from the DPM and DPSH tests. Then the results are presented in Figures 19 & 20.

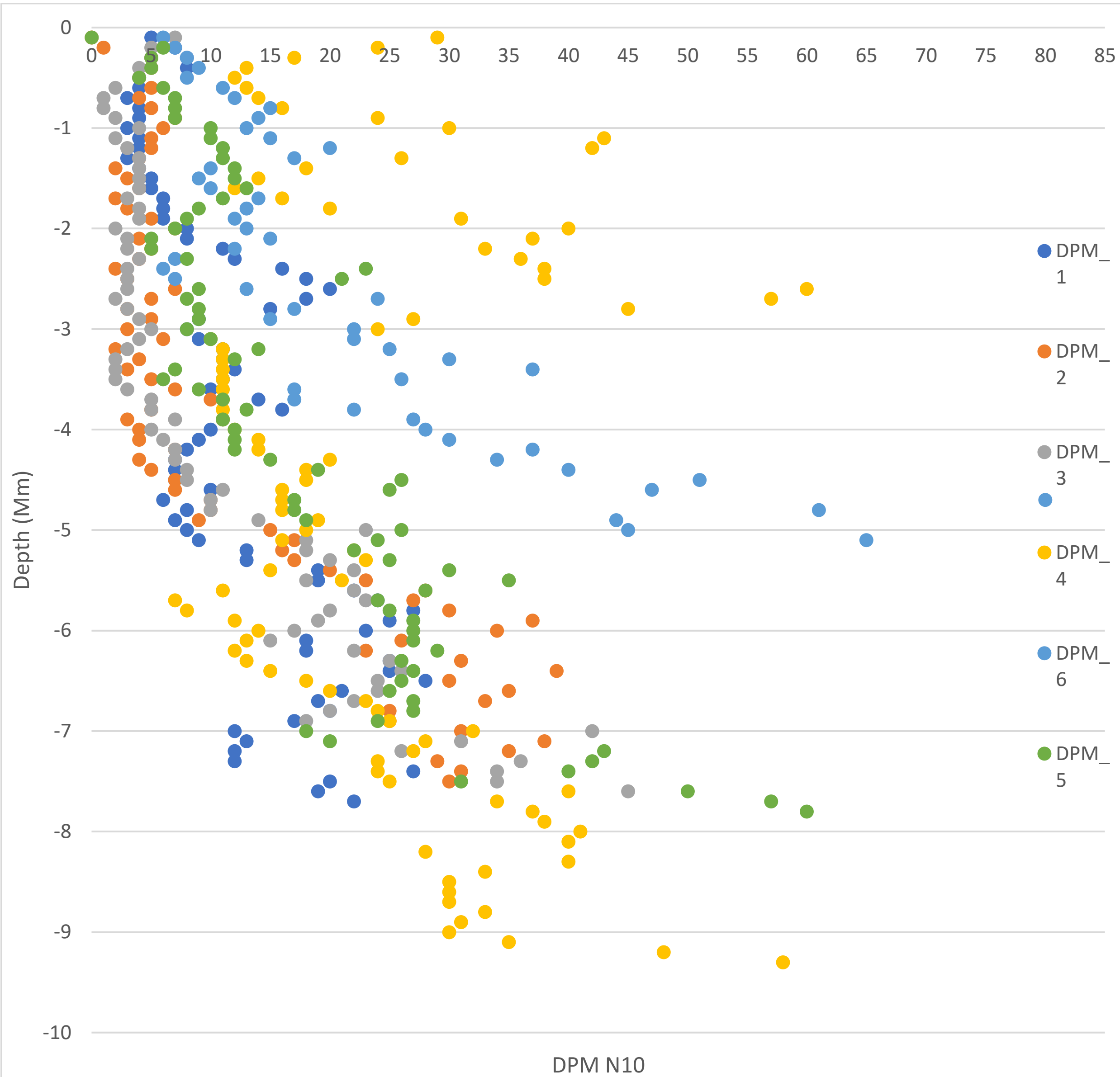


Figure 19 Results of N10 coming from the DPM tests

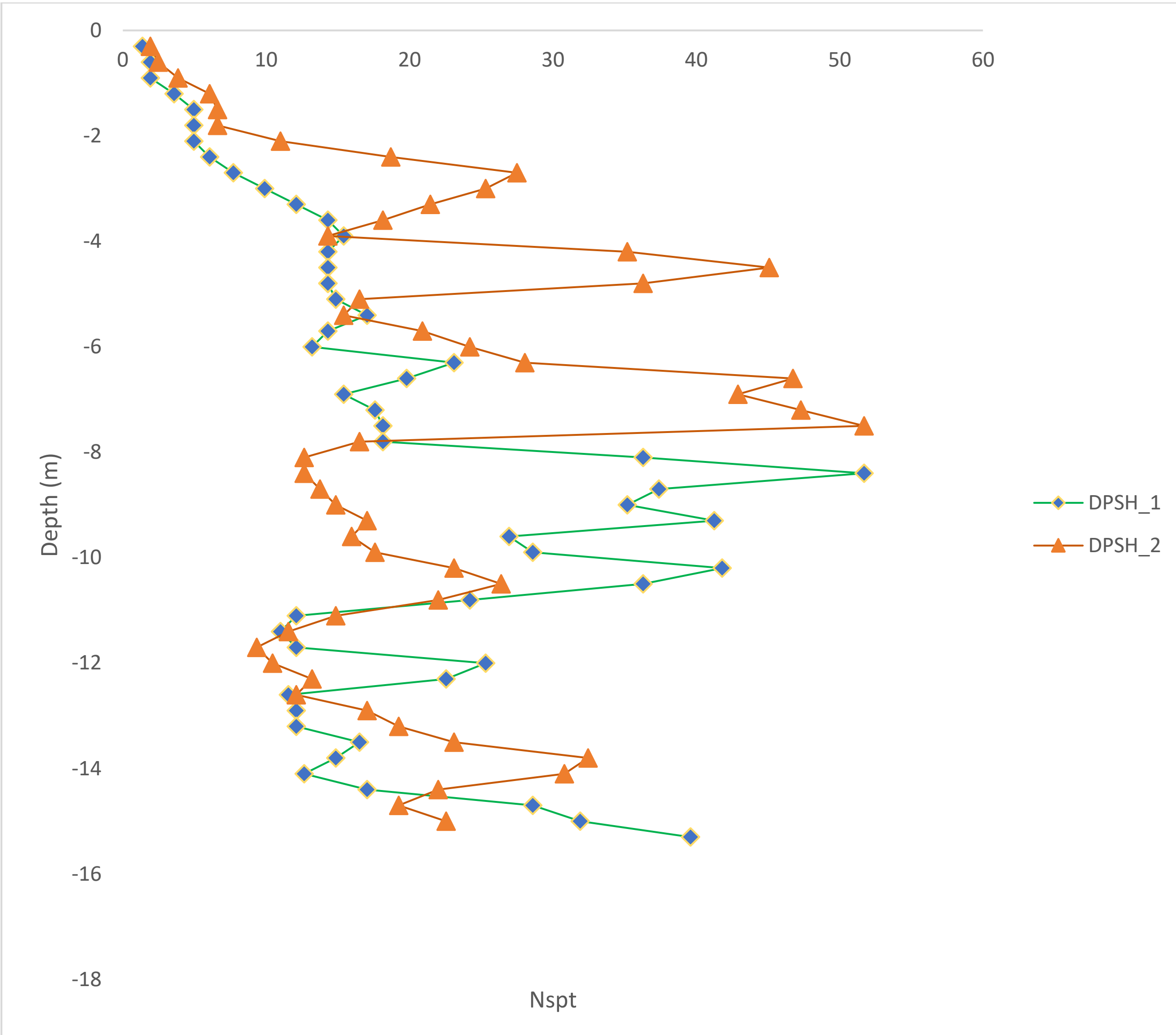


Figure 20 Number of strokes for with reference to the standard penetration test interpreted from the DPSH strokes

As given from the geotechnical report “Progetto di ripristino Galleria del Riserasco - Lago della Spina”, the friction angle of the slope ranges between 30 and 35˚. Then different correlations are done to indicate the physical and mechanical characteristics of the soil.

The soil profile can be divided into three main types which is illustrated in the following table:

Table 1 Geotechnical characteristics of the soil that surrounds the tunnel

Layer	Depth [m]	Lithological unit	Type of material
	0-3	Complex 1	Very loose to loose silty sands and sandy silts
	3-15	Complex 2	Consistent clayey-sandy silts
	Over 15	Complex 3	Very consistent clayey-sandy silts; possible intercalations of medium-densified sands and gravelly sands

NSPT values can be interpreted to obtain a preliminary estimate of the main mechanical properties of the soil at the site. The correlations are then reported as follows:

- Relative density (Dr) for granular materials according to Gibbs & Holtz (1951).
- Peak friction angle (ϕ’) for granular materials.
- Undrained shear strength using the SPT empirical correlation.
- Elastic modulus according to Bowles (1991)

The investigations show that there is no close levels water table to the area under excavation.

The results of the mechanical parameters can be interpreted in the following tables:

Table 2 Geotechnical parameters for complex 1: Silty sands-very loose to loose sandy silts

COMPLEX 1: Silty Sands-very loose to loose sandy silts	
Unit weight γ [kN/m^3]	17.5
Residual shear resistance angle φ [˚]	26.5
Compressibility Modulus Es [MPa]	<5
Young's Modulus E' [Mpa]	<7.5

Table 3 Geotechnical parameters for complex 2: Clayey-sandy silts from firm to very firm

COMPLEX 2: Clayey-Sandy silts from firm to very firm	
Unit weight γ [kN/m^3]	19
Residual shear resistance angle φ [˚]	24
Compressibility Modulus Es [MPa]	5-10
Young's Modulus E' [MPa]	7.5-15
Effective Cohesion C' [KPa]	2-5
Undrained Cohesion Cu [KPa]	75-150

Table 4 Geotechnical parameters for complex 3: Medium-densified sands and gravelly sands

COMPLEX 3: Medium-densified sands and gravelly sands	
Unit weight γ [kN/m^3]	20.5
Residual shear resistance angle φ [˚]	32
Compressibility Modulus Es [MPa]	30-70
Peak shear resistance angle φ_{peak} [˚]	34
Residual shear resistance angle φ_{res} [˚]	32

3.4. Key observations from the site

Following a site visit to the area where the old tunnel is located; a landslide collapse, that is shown in photo1 of the annex, was detected just near the zone of the entrance of the tunnel. There was also detection of several erosion channels. From what can be concluded from the geotechnical report indicated by the municipality, these collapses are directly linked to the collapse that occurred to the tunnel, which indicates that there are no gravitational landslides and there is no possibility to have them in the future. The presence of collapsing slopes suggests the soil does not support vertical excavation well. The exposed soil layers show signs of weak cohesion. The water flow channels indicate a seasonal or continuous surface runoff, which could lead to more erosion. The layers of soil around the slope consists of sand and silt. This means that there are both granular and cohesive materials.

3.5. Environmental assessments

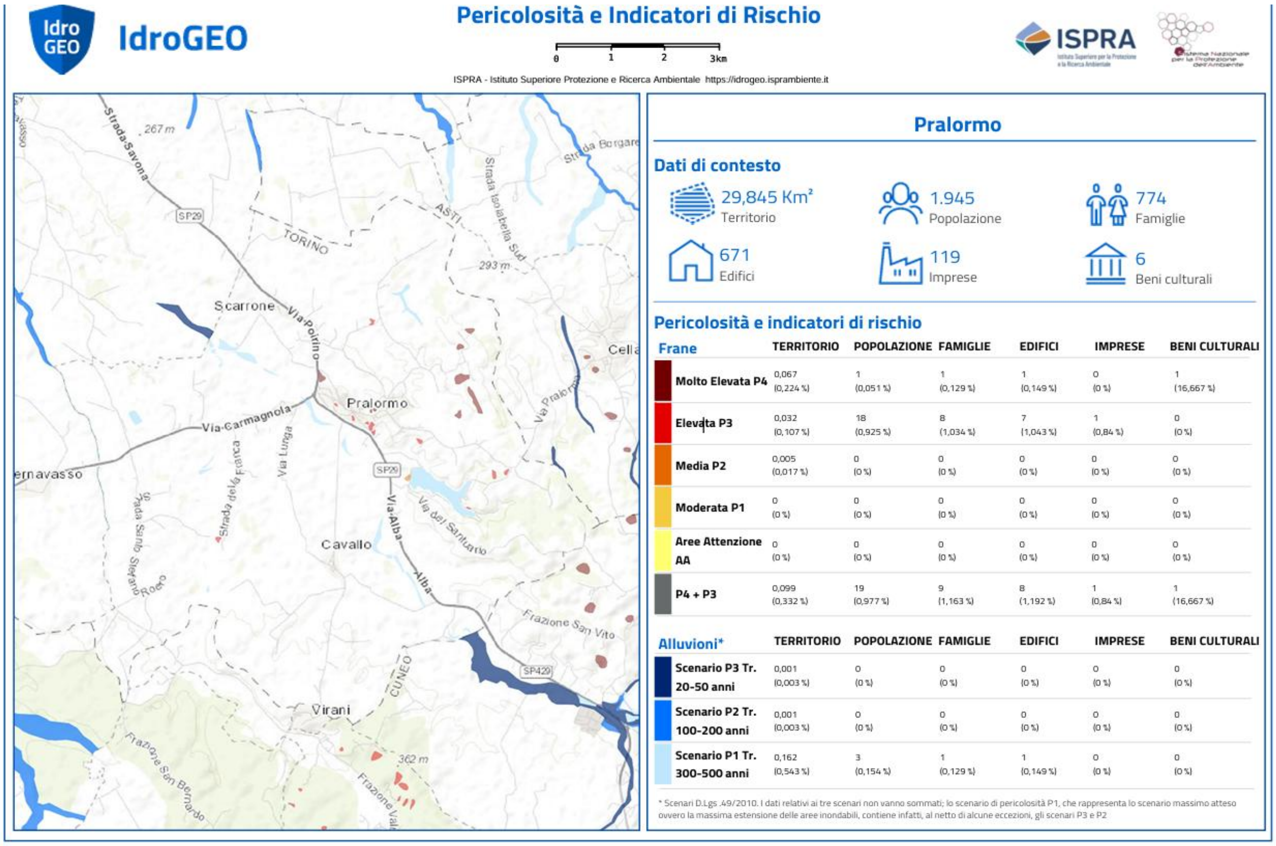
To ensure a reliable design, several checks must be considered. The checks start by knowing the factors that deprives from making a well and long-life shallow tunnel that is made to ensure a good flow of water. This means that the structure has to be installed in a preserved manner and has to be secured from all factors that affects the life of the structure.

One of the key requirements is to maintain stable ground during the different stages of construction and installation of the tunnel. Starting from the excavation, ensuring a stable soil means having a slope that maintains safety and prevents collapse.

3.5.1. Hydrogeological hazard and risk assessment

An important factor before initiating any construction phase is to carry out a hazard assessment around the zone of the construction phase. In order to ensure a safe environment for the structure and to the area around this structure, it is important to highlight the zones that might be at risk. One of the tools to do that is by referring to the historical data and archive research that allows to rebuild landslide events of the past, evaluate it evolution, recurrence time and intensity of the phenomenon. It is fundamental to use, in this process, project databases coming from credible resources. An important data base resource system is the *IFFI (Inventario dei Fenomeni Franosi in Italia)* which is an Italian Landslide Inventory that consists of a computerized cartography and the relative alphanumeric database containing information on landslides surveyed in Italy.

Figure 21 outlines an evaluation for both hazard and the exposure(vulnerability) of the town to landslides and floods for the town “Pralormo”



Frane	TERRITORIO	POPOLAZIONE	FAMIGLIE	EDIFICI	IMPRESE	BENI CULTURALI
Molto Elevata P4	0,067 (0,224 %)	1 (0,051 %)	1 (0,129 %)	1 (0,149 %)	0 (0 %)	1 (16,667 %)
Elevata P3	0,032 (0,107 %)	18 (0,925 %)	8 (1,034 %)	7 (1,043 %)	1 (0,84 %)	0 (0 %)
Media P2	0,005 (0,017 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)
Moderata P1	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)
Aree Attenzione AA	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)
P4 + P3	0,099 (0,332 %)	19 (0,977 %)	9 (1,163 %)	8 (1,192 %)	1 (0,84 %)	1 (16,667 %)

Alluvioni*	TERRITORIO	POPOLAZIONE	FAMIGLIE	EDIFICI	IMPRESE	BENI CULTURALI
Scenario P3 Tr. 20-50 anni	0,001 (0,003 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)
Scenario P2 Tr. 100-200 anni	0,001 (0,003 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)	0 (0 %)
Scenario P1 Tr. 300-500 anni	0,162 (0,543 %)	3 (0,154 %)	1 (0,129 %)	1 (0,149 %)	0 (0 %)	0 (0 %)

* Scenari D.Lgs .49/2010. I dati relativi ai tre scenari non vanno sommati; lo scenario di pericolosità P1, che rappresenta lo scenario massimo atteso ovvero la massima estensione delle aree inondabili, contiene infatti, al netto di alcune eccezioni, gli scenari P3 e P2

Figure 21 Hydrogeological Hazard and Risk Report (www.isprambiente.gov.it)

From Figure 21, it can be observed that the landslide risk is localized but significant where it occurs due to its impact on inhabited and culturally important areas. The hazard levels show a very high (P4) for less than a quarter of 1% of the town's area is at very high risk of landslides and High (P3) landslides hazards exist in small portion of the territory. In the other side the profile shows that flood risk low. This means that the floods have a very limited exposure under realistic scenarios.

3.5.2. Elements at risk

As shown from the geological conditions that were investigated, it is indicated that the stability of the slopes surrounding the tunnel has to be taken into account. Stability of the slopes has to be done on all performed soil cut slopes where the stability is not insured. In other words, all the cases of cuts in weak soils, in transition zones, where high groundwater or seepage forces are present, or/ and with irregular geometry requires a stability analysis.

As a starting point, it is important to identify the critical zones and places that might be affected by the collapse of the tunnel and a future collapse to the area around the tunnel (Figure 22 & 23). Starting with the view of the area surrounding the tunnel and by creating a DEM that can shows the level of steepness where critical section of slopes can be observed in the following figure.

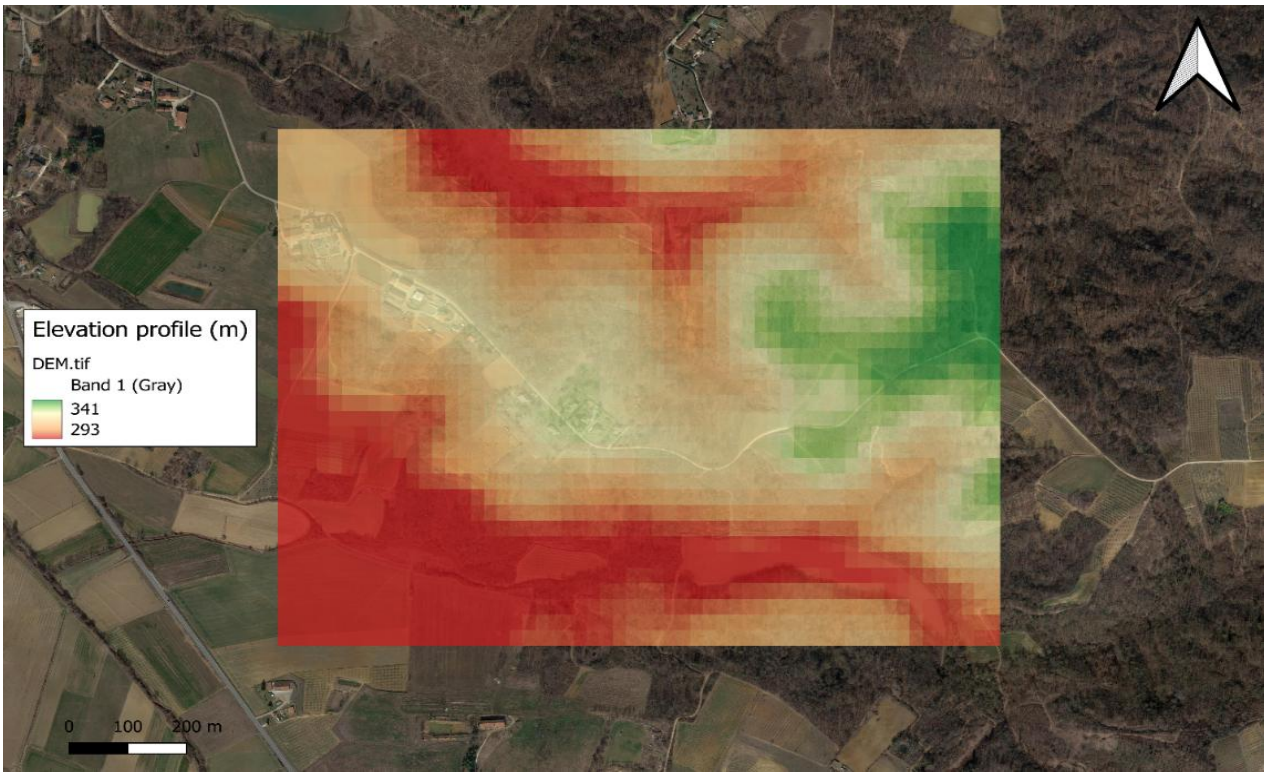


Figure 22 Map with DEM elevation (QGIS)

In the map above, green indicates the steepest parts, while red indicates the gentler areas. In order to have a better vision, it is possible to obtain a 3d map that can emphasize better the area around which the tunnel is going to be built. This map can was done using QGIS.

By referring to the DEM profile as shown in figure 23 and based on the maps, it is possible to start building an opinion choice on what zones are affected by the tunnel.

In the following figure, presentation is made to take into account for the steep part (marked in green), and the building (selected in red) might be existed near the tunnel.

Figure 23 Observation showing a steep part to the right marked by green square and an element at risk marked by a red circle (QGIS)

From the observations presented, the building and the steep zone are located more than 200 m from the area to be excavated, and based on the preliminary assumptions that we assume as engineers is to have a section with a boundary condition up to 10 times the diameter of the tunnel and 200 meters is way far from the boundary conditions. As a matter of fact, by referring again to the DEM profile and going more closely to the section of the tunnel, the zones are safe and is not directly affected by tunnel even if there is a probable occurrence of a collapse.

3.5.3. Section cut

The section selected as the critical section is located about 15 m from the outflow. This section is a good representation for the slope along the tunnel. Figures 24 and 25 show the section of the cut and the 2D layout of the tunnel.

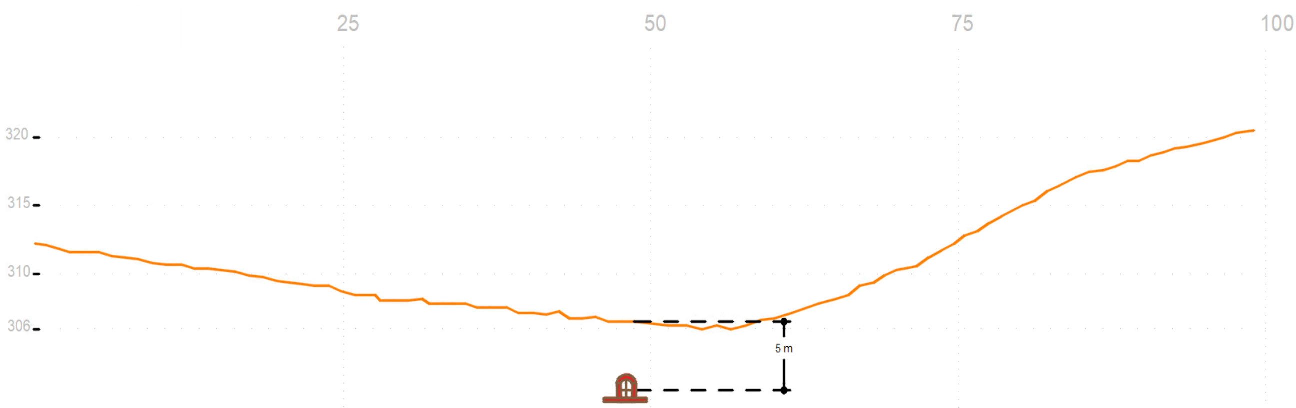


Figure 24 Longitudinal section of the terrain and tunnel alignment

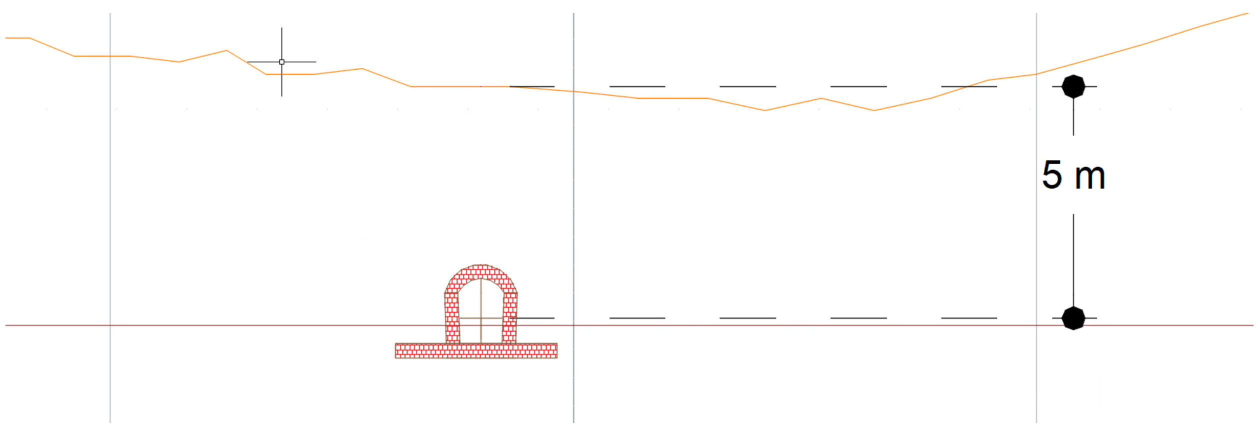


Figure 25 Cross-section showing tunnel position and cover depth

3.6. Preliminary method selection for the construction method

For a shallow artificial tunnel, the first method that generally comes to mind is the cut-and-cover method. The choice of the most suitable method for the construction of the tunnel is influenced by many factors. As mentioned in the book "*Cut and Cover Metro Structures*", the main factors are many starting from the Geological and hydrogeological conditions, the size (depth and width) of the excavation, the location, characteristics of the existence and the disposition of the underground public utilities, the volume and intensity of the traffic of the surface, and the economic situation.

To interpret this choice more clearly, it is vital to discuss some conditions that has to be verified on order to use these methods. Starting from the typical cut and cover method with the stabilized open cut, it is considered as the simplest construction option and a simple design solution. However, for this method to be applied on the field, it is important to verify sets of conditions and discuss the challenges that might be encountered. The following five conditions that has to be satisfied, mentioned in the book "Cut and Cover Metro Structures" (Kaul, 2010), will be provided as follows:

- Adequate width of corridor for construction activity surrounding the perimeter of the structure should be available without involving enormous land-take costs or necessitating demolition of a large number of existing structures.
- The weather is generally favorable during the construction stage.
- The settlement of the surrounding ground and the structures are controlled
- There are no main routes around the construction zone. Otherwise, the routes should be blocked during the period of construction.
- No utilities surrounded within the construction zone or, if present, these can be temporarily diverted.

If these conditions are broadly satisfied, the open cut method can be considered feasible at a preliminary stage.

When these conditions cannot be met, for example in dense urban areas where space is limited or where traffic and nearby structures must remain operational, more advanced construction techniques may be required. One example is the Milano method. In this method, the sides of the excavation are constructed vertically using retaining systems that support the surrounding soil during excavation. This approach reduces the required excavation width but requires specialized construction techniques to maintain soil stability, control ground settlements, and resist lateral earth pressures acting on the support system (Kaul, 2010, p. 39).

To determine the most suitable construction method, these conditions can be compared with the characteristics of the case study under investigation.

The case study: *Galleria del Riserasco,* concerns the rehabilitation of a damaged tunnel that will be replaced located in *Pralormo*. This tunnel is mainly constructed in few meters of depth, and the size of the tunnel is considered to be small with an Arched shaped tunnel. This means, that the shallow depth and low external constraints make open trench construction still feasible, but weak soils require careful temporary stability and drainage measures. Additionally, the area around the tunnel is not urbanized and there are no close buildings that will surround the tunnel. No underground public utilities are present. The tunnel is considered as a light weight structure and therefore the settlement control is more preserved. Finally, no main routes are surrounded by the tunnel's route. All of these conditions can give an overview that the most suitable among the analyzed options method to be used is the cut and cover with an open trench. However, there are two more conditions that has to be taken into considerations, which is the weather control during the construction stage of the tunnel and the size of the land-intake and its following property costs and excavation. From the site visit, there were signs of water runoff described in a water channel. This suggests that the drainage must be carefully considered. In addition, the excavation remains open for the period of construction and this means that further considerations have to be taken into account. The observed collapse risk indicates that unsupported vertical excavation faces would not be stable, so sloped sides or temporary support measures would be required.

From these considerations, the use of an open trench appears applicable in this case with taking into account several aspects. Runoff management has to be taken into account by surface drainage systems, and slope protection. This can be managed by installing diversion ditches, channels, or berms to direct water away from the trench. The slope protection can be made by using a vegetation, or geotextiles. The second aspect to be controlled is the Infiltration control. Infiltration can weaken the soil and lead to trench collapse, especially in cohesive and loose granular soils. Using impermeable barriers, dewatering systems, or compaction of backfill can control this infiltration and reduces water seepage into the trench.

The slope has to be managed in which the excavation slope angle must be selected so that the required safety factor against failure is satisfied, taking into account the soil shear strength parameters. This can be approved by two-step. First step is by following a standard that gives pre-assumption of the slope angle to be assessed. Then, following a numerical analysis or LEM, it is possible to dictate whether this slope angle can maintain the necessary safety conditions.

Chapter 4

Preliminary design of the rehabilitation and stabilization of the Riserasco tunnel

4.1. Introduction

In this chapter, a study was carried out to develop a design for the reconstruction of the hydraulic tunnel, ensuring the safety and stability of the site while restoring its intended function. This process includes defining the tunnel geometry, generating the mesh, applying boundary conditions, and assigning the material properties and their mechanical behavior.

4.2. Design alternatives

Several design alternatives can be considered for the excavation geometry in the cut-and-cover construction method. In this study, open trench slopes of 1.5H:1V, 2H:1V, and 2.5H:1V are considered. The most appropriate configuration will be determined based on the factor of safety obtained from the stability analysis.

Since the tunnel section is evaluated for these three slope configurations, it is necessary to define the limits of the terrain that will be reshaped during excavation, as well as the backfill that will cover the tunnel structure after construction. Figure 26 illustrates the boundaries of the excavation and backfill zones.

This representation allows a clearer visualization of the excavation geometry and helps identify the critical slope configuration during the stability analysis. Figure 27 shows the different open trench slope geometries considered under geostatic conditions.

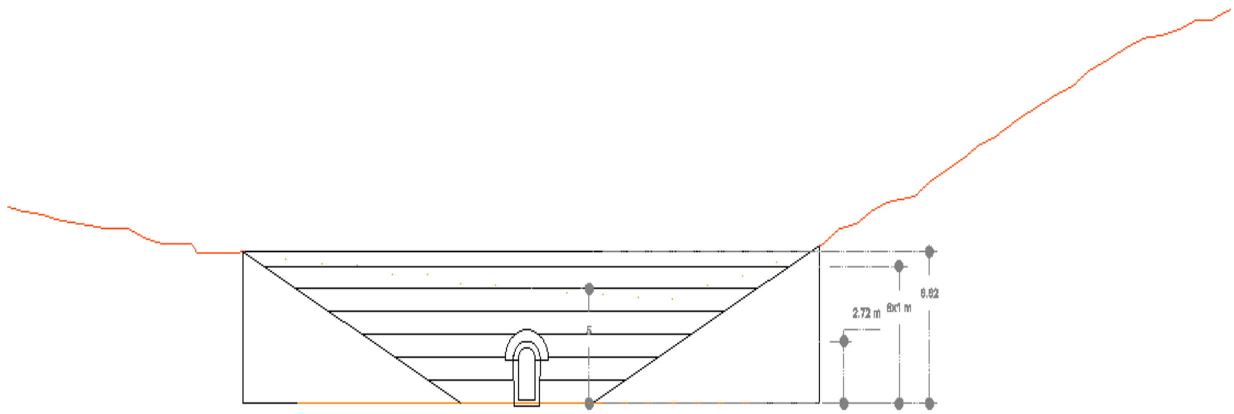


Figure 26 One Layer Slope Layout

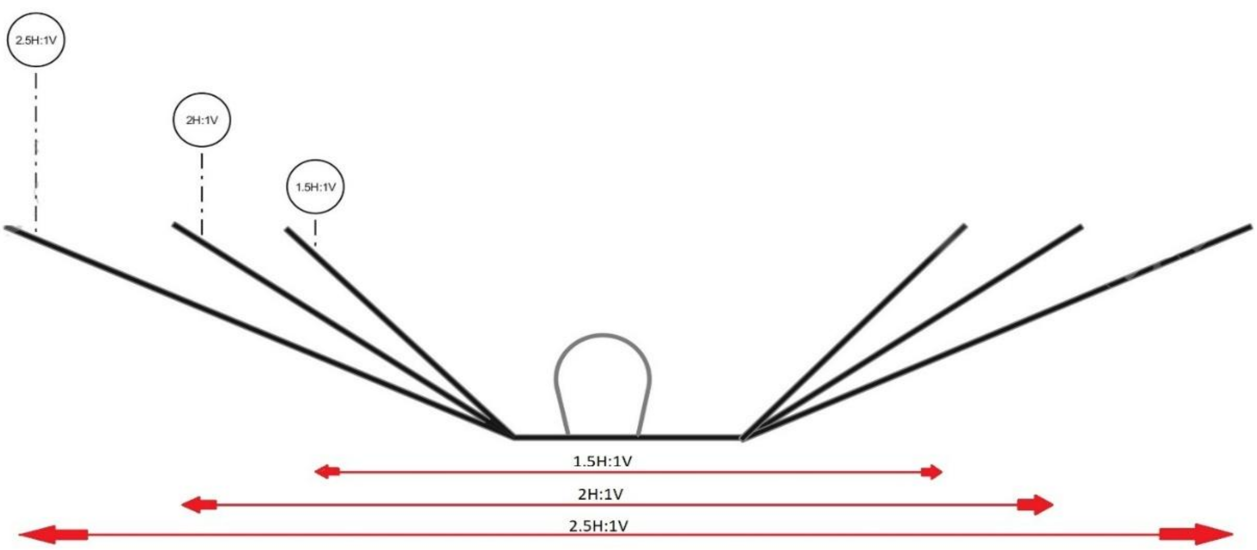


Figure 27 Presentation that shows the effect of the different slopes on the width of the excavation

If one of these three assumptions fails to give an allowable factor of safety, then other design procedures have to be introduced. The other procedures will be the use of berms, vertically retained slope, or a slope coupled with a vertically retained slope.

Other solutions can be made by providing a vertically retained slope that is coupled with the slope excavation, or a slope with berm as shown in figure 28.

Figure 28 Figure that shows the slope, slope with berm, and slope coupled with vertically retained slope

To have an accurate slope geometry, three main rules must be satisfied:

- Operational Safety
- Minimizing geotechnical risks
- Ensuring economic efficiency

For operational safety, it is mandatory to have a slope geometry that ensures that workers, equipment, and surrounding infrastructure are not at risk during both construction and long-term operation. It is also mandatory for operational safety to have adequate factor of safety against sliding and collapse and to have a control on deformation and displacements to avoid progressive failure. This requires an appropriate design for the bench widths and berms if necessary.

For minimizing geotechnical risks, it is important to reduce the instability risks (rotational or translational failures). It is also important for minimizing geotechnical risks control erosion and weathering (rainfall infiltration, frost action, and surface runoff) and seepage pressures (excess pore pressures). Risk minimization involves proper geotechnical investigation, numerical modeling, and mitigation measures (drainage, retaining structures, and reinforcement).

For example, for a cut slope in clayey soil, heavy rain can cause shallow slips. Designing surface drainage channels and installing geotextiles minimizes this risk.

For ensuring Economic efficiency, it is vital to have a geometry that balance safety with cost effectiveness. If it is too steep, this will lead to unsafe and high failure risk. If it is too flat, it might be safe but it will require removing huge volumes of soil/rock, increasing excavation costs and land use. Efficiency means optimizing slope angle, benching, and reinforcements to meet safety standards while reducing unnecessary costs. This will lead to a safe and cost balanced solutions. Figure 29 is an illustration of an optimal slope design.

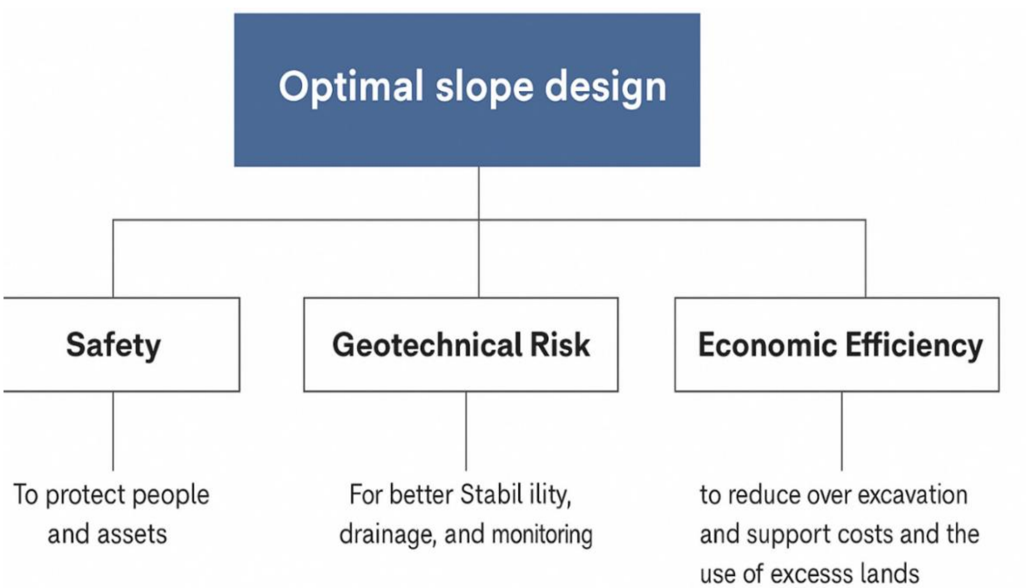


Figure 29 Optimal slope design

Based on these rules mentioned above, it is important to highlight the importance of ensuring a design criterion that minimizes the cost of construction as much as possible. As a matter of fact, it is obvious that choosing a slope with a lower width is important in this case. This will reduce the volume of excavation and the cost related to the its removal.
The choice for the most accepted model will basically depend on the three rules mentioned above. Therefore, based on the economic situation that depends on the excavation volume, the size of the excavation vertically and horizontally, and the cost of the material to be constructed, the shape of the slope will be created.

Retained slopes is an effective solution where it eliminates the excess excavation made by a simple slope. A great example is the micro piling. Micropiles can be installed in areas of particularly difficult, variable, or unpredictable geologic conditions such as ground with cobbles and boulders, fills with buried utilities and miscellaneous debris, and irregular lenses of competent and weak materials. Soft clays, running sands, and high groundwater not conducive to conventional drilled shaft systems cause minimal impacts to micropile installations. Micropiles are commonly used in karstic limestone formations.

Therefore, it is possible to have up to 4 design solutions.

- Design solution 1: Slope 1.5:1 (H: V)
- Design solution 2: Slope 2.5:1 (H: V)
- Design solution 3: Slope 2.5:1 (H: V) with berms
- Design Solution 4: Slope supported by micro piles

Figures 30, 31, 32, & 33 shows the model of design solution 1, 2, 3, & 4 respectively.

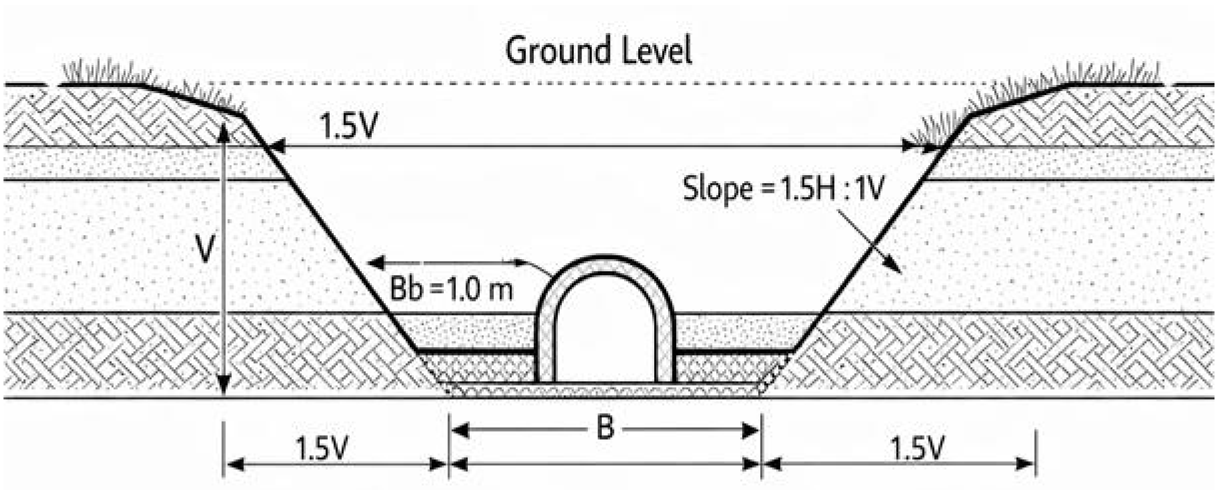


Figure 30 Design solution 1 1.5:1 (H: V)

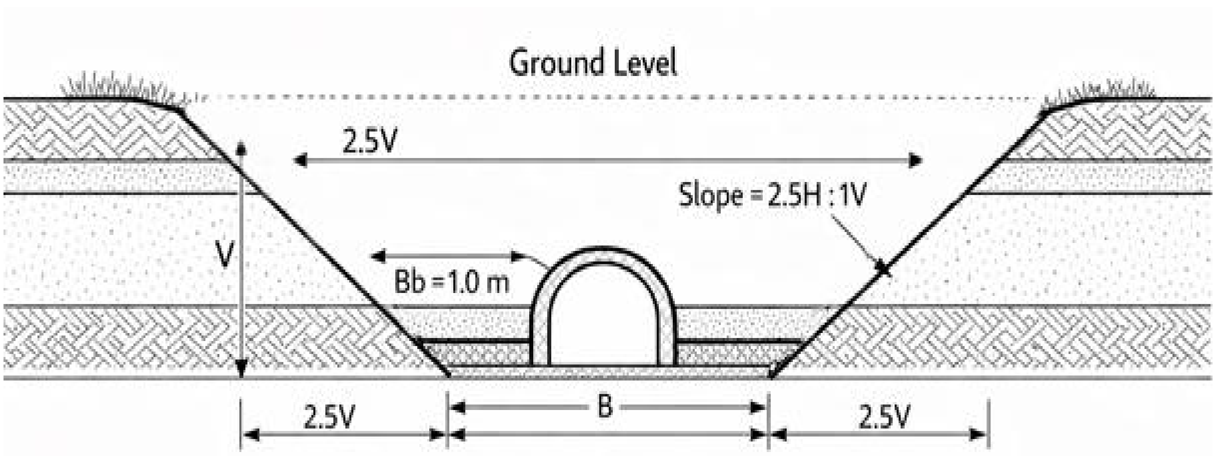


Figure 31 Design solution 2 2.5:1 (H: V)

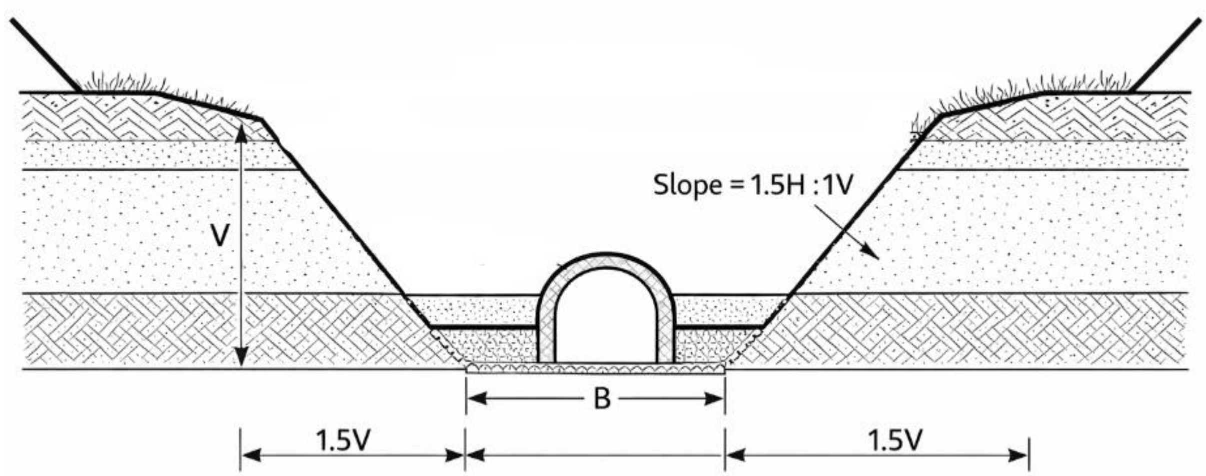


Figure 32 Design solution 3: Slope included by berms

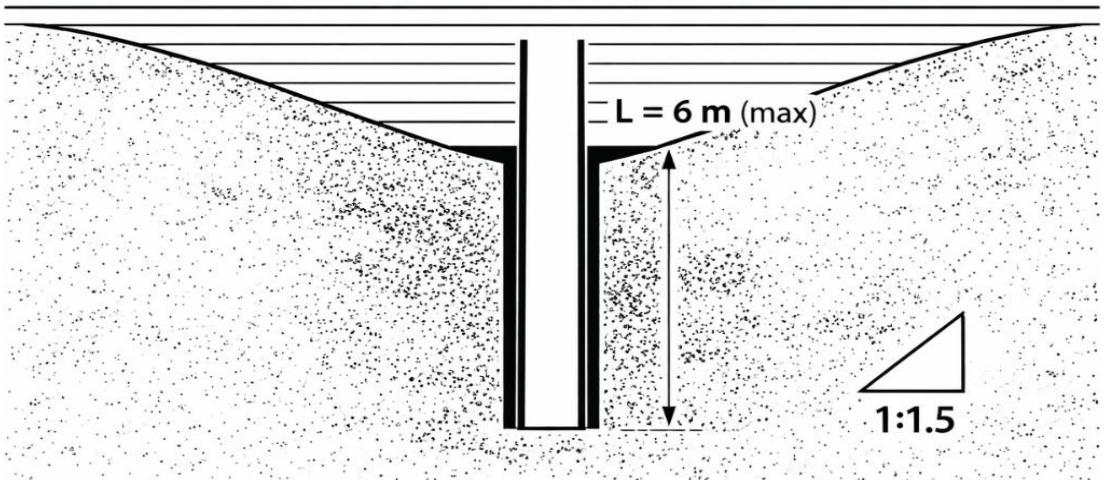


Figure 33 Design solution 4: Slope supported by micropiles

Chapter 5

Numerical Model

The numerical model was developed using the finite element method (FEM) in RS2 software to simulate the excavation, tunnel installation, and backfilling stages. The model represents the geometry of the slope and tunnel, incorporating the stratigraphy, material properties, and boundary conditions obtained from the geotechnical investigation. Its purpose is to evaluate the stress distribution, deformation, and overall stability of the excavation, as well as to determine the factor of safety for the different design alternatives. The following sections describe the mesh setup, domain size, boundary conditions, construction stages, and material parameters used in the analysis.

5.1. Constitutive model and strength parameters

Griffiths and Lane (1999) stated that complex stress–strain models are not always required for slope stability analysis. In many practical applications, soil behaviour can be adequately represented as an elasto-perfectly plastic material following the Mohr–Coulomb failure criterion. Therefore, the soil in the numerical model was represented using the Linear Elastic–Perfectly Plastic Mohr–Coulomb constitutive model.

According to the Mohr–Coulomb failure criterion, the soil behaves elastically until the stress state reaches the failure surface. Once this condition is reached, yielding occurs and stresses redistribute within the soil mass in order to maintain equilibrium.

The shear stress required to induce failure depends on the normal stress acting on the potential sliding plane. The shear strength relationship can be expressed as:

$$\tau = c + \sigma \tan \varphi$$

where c is the cohesion, σ is the normal stress, and φ is the friction angle.

For slope stability analysis, the factor of safety can be introduced by reducing the shear strength parameters:

$$\tau = \frac{c}{F} + \frac{\sigma \tan \varphi}{F}$$

where F represents the factor of safety.

The mechanical properties used to describe soil behavior depend on the soil type. For coarse-grained soils, the shear strength is typically defined by the effective cohesion c′ and friction angle φ′. For fine-grained soils, the behavior under short-term conditions is often described using the undrained shear strength Su, as indicated in Eurocode 7 (Section 11.5).

For long-term slope stability conditions, the soil behavior is generally assumed to be drained, meaning that the shear strength is expressed in terms of the effective parameters c′ and φ′.

Based on these constitutive assumptions, the slope stability analysis was carried out using the Finite Element Method (FEM) combined with the Strength Reduction Method (SRM).

5.2. Finite element method for the calculation of the factor of safety

The Finite Element Method (FEM) can be used as an alternative to the Limit Equilibrium Method (LEM) for slope stability analysis. In FEM-based stability analyses, the factor of safety is determined through a process of repeated calculations in which the shear strength parameters of the soil are progressively reduced until failure occurs.

This approach is known as the Strength Reduction Method (SRM). In this method, the shear strength parameters of the soil are reduced by applying a Strength Reduction Factor (SRF). The reduction process continues until numerical instability is detected, indicating failure of the slope. The value of the SRF at failure corresponds to the factor of safety of the slope.

The reduction of the shear strength parameters can be expressed as:

$$C'crit = \frac{C'}{SRF}$$

$$\tan{(\emptyset')} = \frac{\tan{(\emptyset')}}{SRF}$$

where c' and $\emptyset'$ represent the original cohesion and friction angle, respectively.

Figure 34 represents the effect of the SRF on reducing the strength parameters in a shear stress-normal stress relationship:

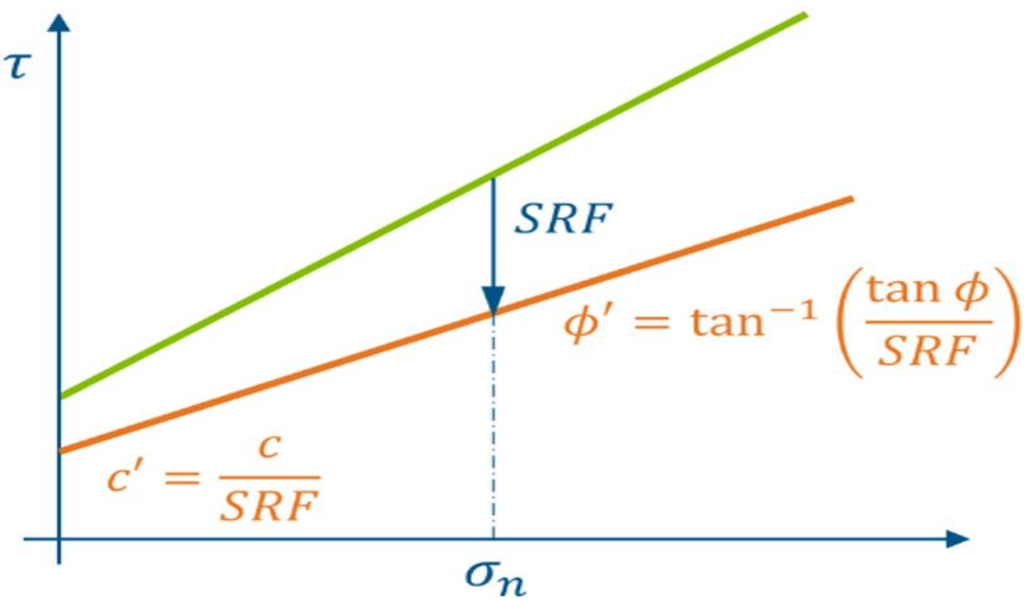


Figure 34 Curve that defines the strength reduction factor (SRF)

5.3. Safety Criteria According to Eurocode 7 and NTC 2018

Numerical modeling plays a crucial role in the design and analysis of hydraulic artificial tunnels, particularly constructed under shallow embankments. One of the basic tools in numerical modelling is the Finite Element Model (FEM). It is widely used for modeling the stress and deformation of tunnel linings in soft soil conditions (Peila, 2022, p. 211). Once the numerical methodology has been defined, the stability results must be evaluated according to the safety criteria specified in the relevant design standards.

EUROCODE 7 (EN 1997-1)

Eurocode 7 (EN 1997-1) suggests that a factor of safety of approximately 1.2 may be acceptable for temporary works when adequate monitoring and control measures are implemented.

Norme tecniche per le costruzioni NTC18

Another suggestion for an allowable factor of safety of the slope is the one that is given by the *Norme tecniche per le costruzioni* NTC18 as described in chapter 6.8.2 (*VERIFICHE DI SICUREZZA (SLU)*) in *table 6.8.1*. Factor of safety (FOS) of 1.1 is often used as the minimum acceptable value of serviceability limit state (SLS) in temporary works, especially in cut and cover tunnels, slope cuts, and temporary excavations. This value is considered an appropriate safety factor. The table is presented in table 6 for ease of reading.

Table 5 Partial coefficients for safety checks of loose material structures and excavation faces

Coefficient	R2
γ_R	1.1

Accordingly, the most appropriate choice for an allowable factor of safety is the one that is provided by *NTC18*.

The safety check for stability must be made based on the global stability check. The global stability check must be performed according to Approach1- Combination 2 (A2+M2+R2), and which will be presented in tables 7 and 8 respectively.

In accordance with §2.6.1, the verification of condition [6.2.1] must be performed under various combinations of partial factor groups: for actions (A1 and A2), for geotechnical parameters (M1 and M2), and for resistances (R1, R2, and R3). The different groups of partial safety factors are selected within two distinct and alternative design approaches. In the first design approach (Approach 1), checks are performed with two different combinations of factor groups, each of which may be critical for different aspects of the same design. In the second design approach (Approach 2), checks are performed with a single combination of factor groups.

Analytical method with characteristic geotechnical parameters can be determined by

- Use the characteristic values of soil parameters (e.g., φ′, c′, cu).
- Divide them by the relevant partial factor γ_M (from Table 6.2.II).

Also apply, when needed, the partial resistance factors γ_R, as specified in the corresponding sections for each type of structure. Partial factors γ_M and γ_R are applied to ensure safety by reducing uncertainties in soil strength and resistance models (Table 7).

Table 6 Partial coefficients for the geotechnical parameters of the terrain

Partial coefficients for the geotechnical parameters of the terrain				
Parameters	**Quantity to which the partial coefficient should be applied**	**Partial coefficients**	**A1**	**A2**
Tangent to the friction angle (degrees)	$\tan \phi'_k$	$\gamma_{\phi'}$	1	1.25
Cohesion (MPa)	γ_{G2}	$\gamma_{c'}$	1	1.25
Undrained resistance	γ_{Qi}	γ_{cu}	1	1.4
Volume of the unit weight	γ_γ	γ_γ	1	1

The partial coefficients for the actions are shown in the following table:

Table 7 Partial coefficients for the actions

Actions					
	Effect	**Partial Coefficients**	**EQU**	**(A1)**	**(A2)**
Permanent Loads G_1	Favorable	γ_{G1}	0.9	1	1
	Unfavorable		1.1	1.3	1
Permanent Loads G_1	Favorable	γ_{G2}	0.8	0.8	0.8
	Unfavorable		1.5	1.5	1.3
Variable Actions Q	Favorable	γ_{Qi}	0	0	0
	Unfavorable		1.5	1.5	1.3

The overall stability check must be carried out according to Combination 2 (A2+M2+R2) of Approach 1, taking into account the partial coefficients shown in tables 6.2. I and 6.2.II for geotechnical actions and parameters (reported for ease of reading respectively in Table 7 and 8).

In table 8, the normalized values and the designed values are presented:

Table 8 Normalized values and the designed values for the geotechnical parameters

Geotechnical Parameters				
	Parameter Name	**Normalized values**	**Designed Values M1**	**Designed Values M2**
Material 1	Unit weight (MN/m3)	0.0175	0.0175	0.0175
	Young's modulus (MPa)	7.5	7.5	7.5
	Tangent to the friction angle (degrees)	0.577	0.577	0.462
	Friction angle (degrees)	30	30	24.791
	Cohesion (MPa)	0	0	0
Material 2	Unit weight (MN/m3)	0.018	0.018	0.018
	Young's modulus (MPa)	15	15	15
	Tangent to the friction angle (degrees)	0.700	0.700	0.560
	Friction angle (degrees)	35	35	29.3
	Cohesion (MPa)	0.005	0.005	0.004

5.4. Failure surface

The critical slip surface is described as a circle. The slope mass to be analyzed is assumed to be a single body or multiple blocks. Circular slip surface is standard way of describing the possible slip surface subjected to failure. The materials that are above and below that slip surface are contained of two main rigid blocks. The lower body stays in a stationary mode and the one above the fault is assumed to be the sliding block. However, the main aim for designing a slope, is to detect the approximate location of the critical slip surface (An-Bib Huang & Hai-Sui Yu, 2018). The stresses applied along the slip surface is made up of the normal stress (N') and the shear stress (τ). Accordingly, it is recommended to introduce a factor of safety relating shear stresses to shear strength by using the Mohr-Coulomb Criterion. Generally, all the areas of the slope should be searched to find the critical slip surface with the minimum factor of safety. By reflecting to the case study, the shape of the slope is considered complex, where more than one slope geometry exists. In such cases, more than one slip surface might be presented. Therefore, precative measures must be taken into account. Figure 35 illustrates these cases.

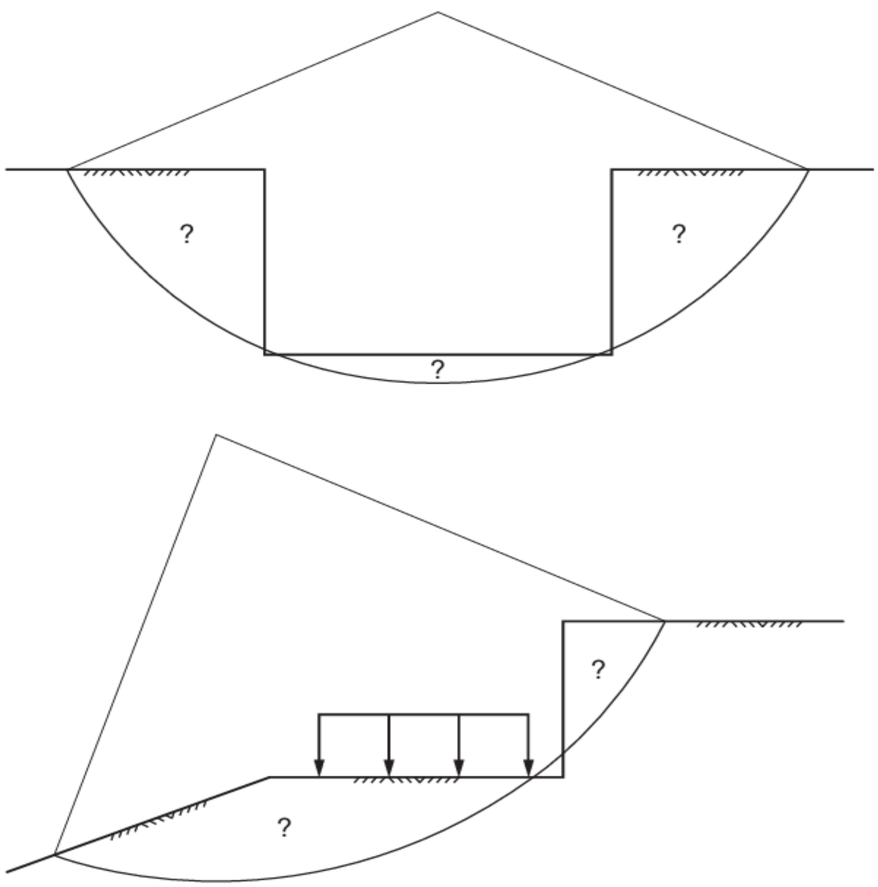


Figure 35 Cases where more than one slip surface might occur (Duncan, &Wright, 2014)

5.5. Geometry, domain size, and boundary conditions

RS2 software works with finite element method (FEM). FEM requires physical or mathematical approximation that is done through a bounded region. The problem domain in FEM is subdivided into discrete elements connected to each other in the form of nodes forming shapes. These shapes can be quadrilateral or triangular, uniform or graded forming a physical approximation to the continuity of displacements and stresses within the continuum (Barla, Rock Mechanics and Rock Engineering).

The size of the boundary condition affects the slope's factor of safety (FoS) in finite element method because the boundaries define the extent of the model and how it interacts with its surroundings, thereby influencing the computation stress distribution and strain field with in the slope. Additionally, the size of the boundary is directly affected by the extension of the slope and its effect on the slop surface. From what can be observed from the slope, the length of the boundary carries the slope that surrounds the tunnel from each side (figure 37).

The results will depend on the change of the strength reduction factor (SRF) which will be validated in chapter 6.

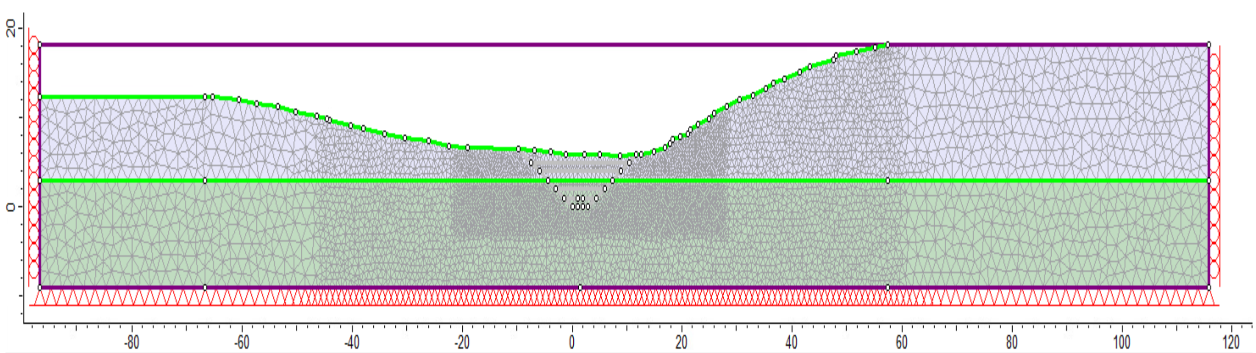


Figure 36 Mesh type on the 2D plane with its advanced mesh regions (RS2)

5.6. Mesh discretization

The mesh generally consists of grade node mesh with grades of fineness that starts fine around the slope and increasing gradually with depth as shown in figure 37.

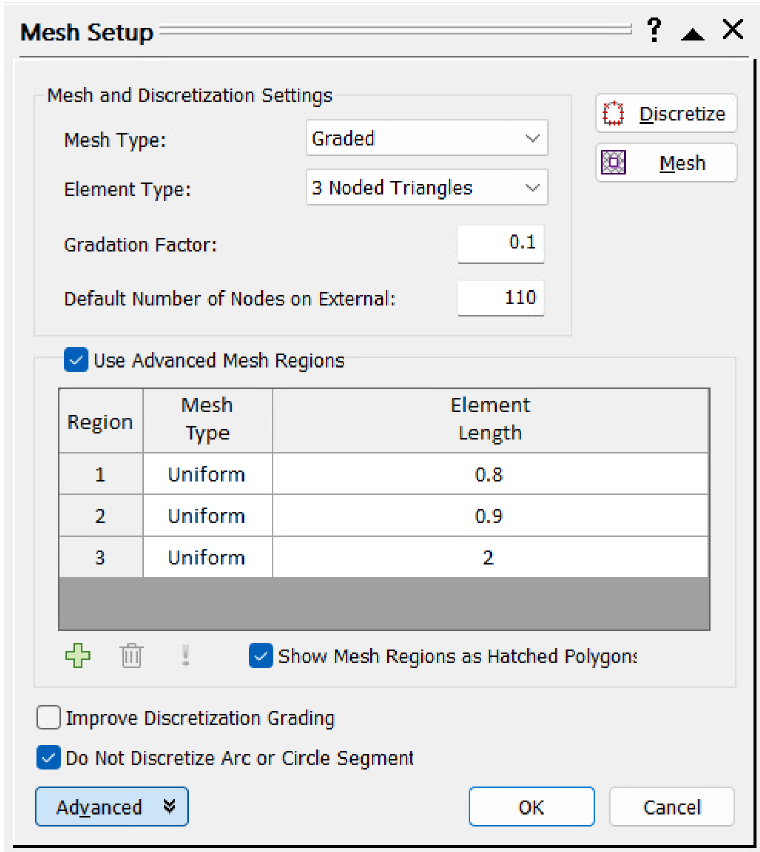


Figure 37 Mesh setup used in RS2

5.7. Material properties

5.7.1. Lining

The tunnel lining is assumed to be a pipeline-type lining. The assumptions of dimensions of this lining mainly depends on the guidelines for the design of segmental tunnel linings (*ITA Working Group 2 – Research*).

The selection of the geometry (thickness, width, and length) is made with respect to the tunnel size and anticipated loading cases. The design philosophy will mainly depend on different codes the is referred to different institutions. One of the codes is the *German Tunnelling Committee (DAUB)*. This committee focuses on the production and installation of the lining segments.

From this code it is possible to obtain preliminary values for the compressive strength of the concrete, the minimum cover of the concrete, and the spacing of the rebar.

The results of these assumptions are provided in the Table 9.

Table 9 Assumptions for the lining provided by the German Tunnelling Committee (DAUB)

DAUB (2013) (German Tunnelling Committee)	
Compressive strength (MPA)	35 to 50
Minimum cover of the concrete (mm)	20 (End faces and bolt sockets) 40 (Surface of Segment)
Rebar spacing (mm)	100 to 150 (Typical range) 90 (Minimum clear spacing)

For the thickness of the lining, several assumptions were made based on several perspectives. As mentioned in section 3.2, thickness of the segmental lining ring, the thickness can be assumed for the segmental lining and then can be optimized during the design stage. The assumption of the thickness for tunnels having a diameter less than 4m can ranges from 150mm to 280mm.

For the reinforcement, specifications for steel bars can be found in local codes and standards. A typical plan view of transverse and longitudinal bars ranges between ϕ10 and ϕ 16, and ϕ 6 and ϕ 16 metrics sized respectively.

Thus, the final results for the dimension can be deduced in the table 10:

Table 10 Parameters of the lining

Compressive strength (MPA)	40
Minimum cover of the concrete (mm)	30
Rebar spacing (mm)	150
Thickness of the tunnel (mm)	200
Transverse reinforcement (mm)	12
Longitudinal reinforcement (mm)	8

5.7.2. Structural support

As already discussed, the use of structural support is one of the options that can be taken into account. It is an effective tool to minimize the width of the slope and hence the width of the excavation that would save space. The use of the structural support is to stabilize the soil mass. Modelling the structure component is used using the FEM. It is possible to use the 2D continuum elements. Since the dimensions of the structural elements are small with respect to the overall geometry, this means that the use of 2D continuum elements would result in either a very large number of elements, or elements with unacceptable aspect ratios.

Beam elements are two-dimensional elements with three degrees of freedom for each node and all nodes are connected to each other. Then, interface elements can be attached on both sides of beam elements to simulate the frictional interaction of a foundation wall with the soil.

The selected support elements are micropiles. As described by AASHTO, chapter 5, table 5-12, a preliminary assumption of 19000 Kpa can be used as an elastic modulus.

5.8. Construction Stages

Numerical modeling for hydraulic tunnels involves simulating the construction process and its effects on the surrounding environment, with particular attention to water-related factors. In order to run a FEM code, it is necessary to define several aspects. These aspects are the spatial domain discretization, the temporal discretization (time step), initial conditions, boundary conditions, integration scheme, constitutive relationships, evolution of geometry and change of material properties, schematization of the structural elements, and dealing with large displacements (Peila, 2022, p. 214).

The construction stages are typically divided into five key phases. The first phase involves establishing the initial stress conditions and the excavation phase, while the second phase applies a removal of the layers made by the excavation process. Excavation phase is divided into different layers. Then after the removal of the layers, the installation of the tunnel is made. Then, the embankment stage is made in a different layer. The Model has to represent the different stages starting from the geostatic conditions until the last stage of backfill (Figure 38). The stages are represented in the table 11.

Table 11 Stages showing the excavation, tunnel construction, and backfill

1	Geostatic Conditions
2	First stage of Excavations
3	Second Stage of Excavations
4	Third Stage of Excavations
5	Fourth Stage of Excavations
6	Fifth Stage of Excavations
7	Tunnel Construction Stage
8	First Stage of Backfilling
9	Second Stage of Backfilling
10	Third Stage of Backfilling
11	Fourth Stage of Backfilling
12	LAST Stage of Backfilling

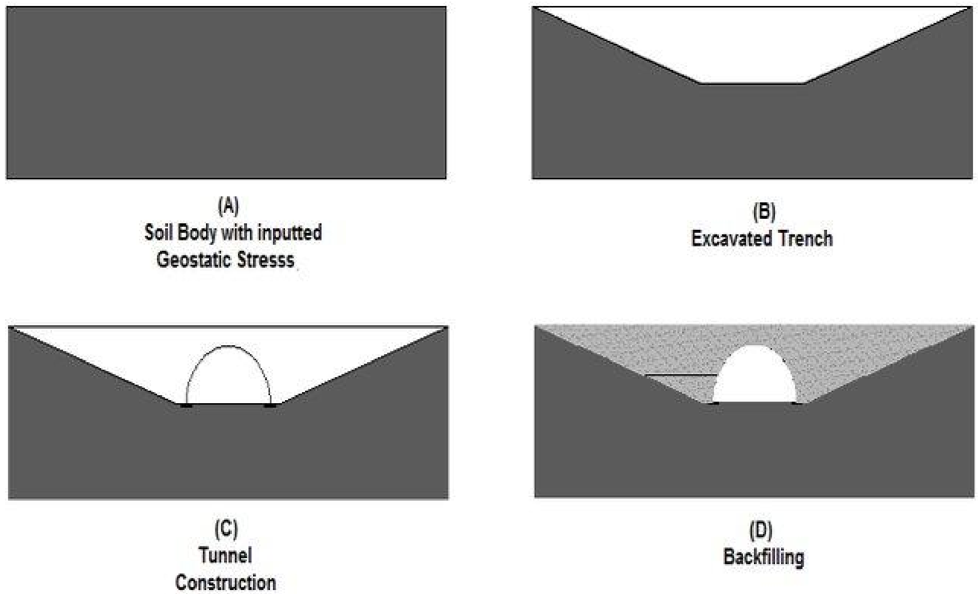


Figure 38 Major Construction stages of the modeled cut and cover (Soumik, Maghoub &Najjar, 2016)

5.9. Results

5.9.1. First design approach: Open trench slope with a grade of 1.5 to 1

5.9.1.1. Factor of safety

The results were obtained for the different excavation cases. It is important that the design offers a safety factor of 1.1 and more for the different excavation procedures. Starting from the design with a slope of a ratio of 1.5 to 1. The factor of safety was shown to be equal to 1.25 in figure 39, which is acceptable.

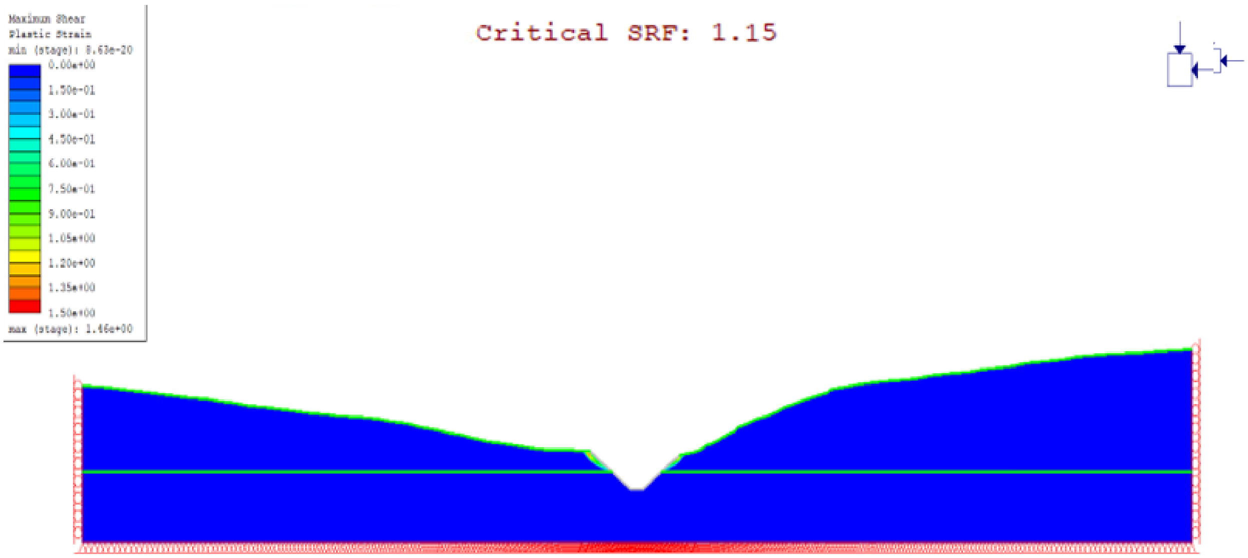


Figure 39 Factor of safety for slope 1.5:1

Another tested model is the slope with a ratio of 1.5:1 reinforced with soil nails. The results show a value of strength reduction factor equal to 1.38 (Figure 40). This value is greater than the allowable factor of safety provided by NTC18.

5.9.1.2. Shear Band & plastic strain

Figure 40 shows the factor of safety for the slope with a ration 1.5:1 with support

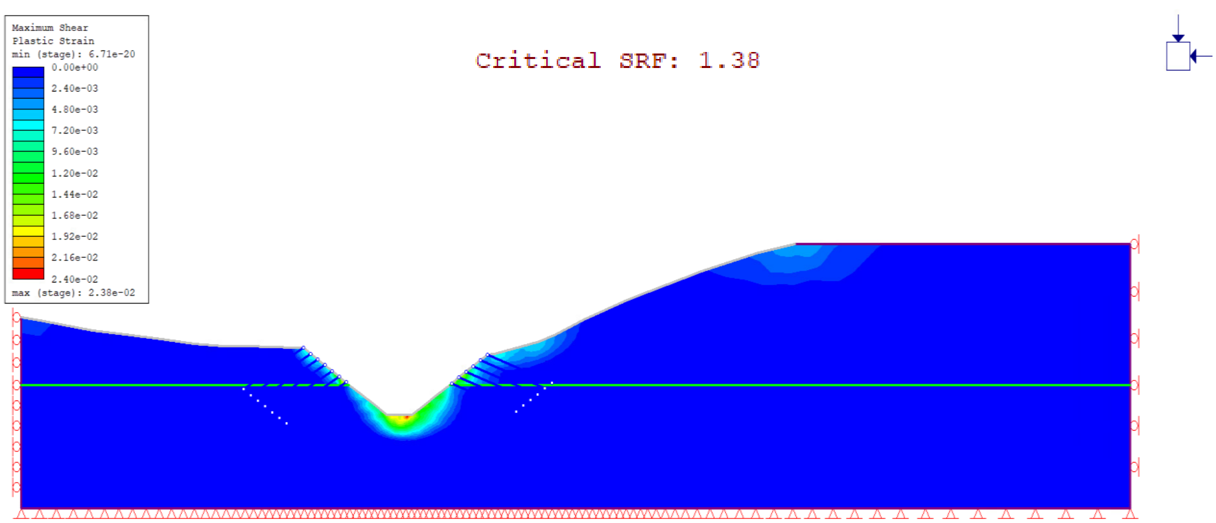


Figure 40 Factor of safer with slope 1.5:1 (RS2)

The shear band is presented for the different stages as described in figure 41

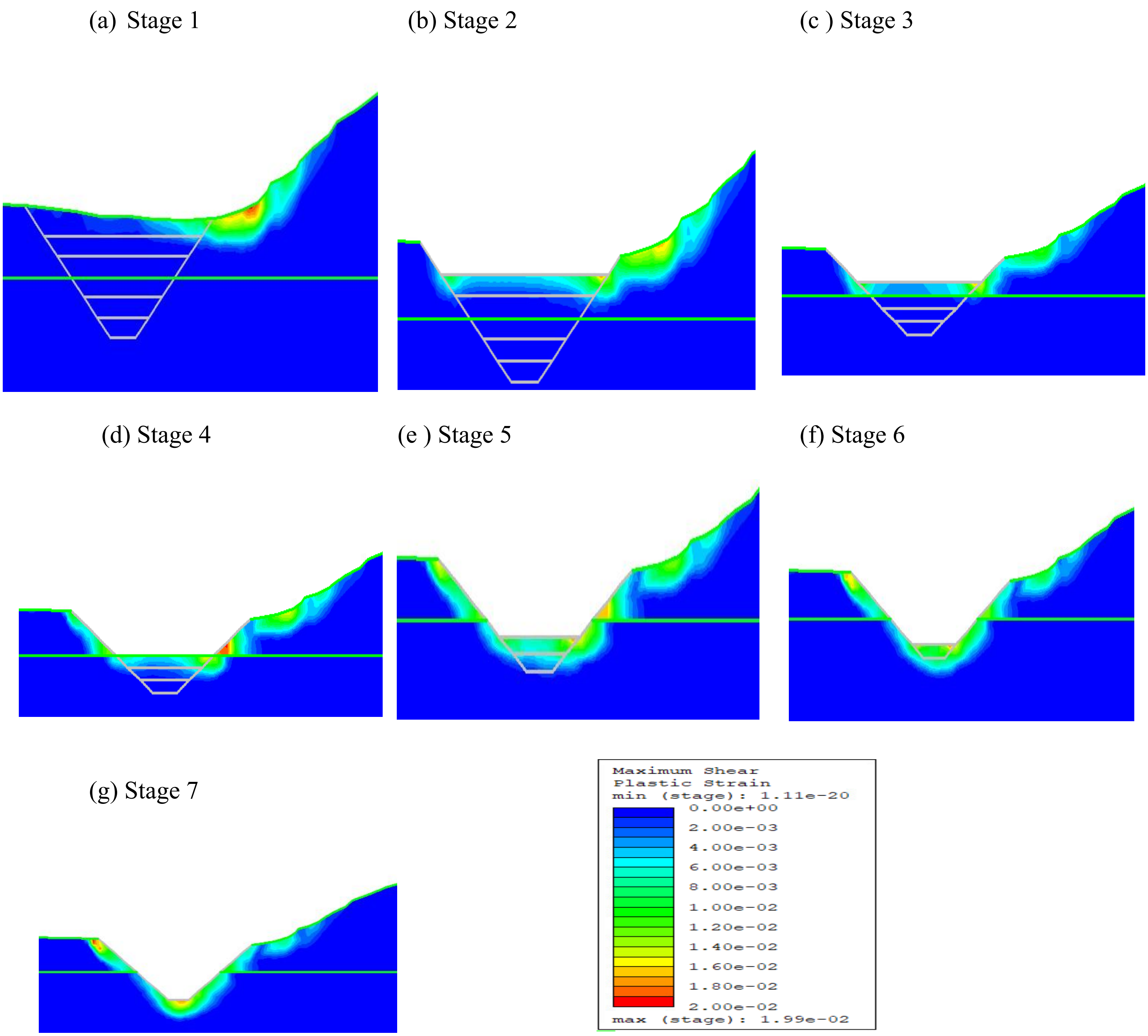


Figure 41 Shear band for the different slope stages and the maximum shear strain contours that illustrate the progressive development of the plastic zone during excavation

5.9.2. Second design approach: Open trench slope with a grade of 2.5 to 1

5.9.2.1. Factor of safety

Another design was simulated and interpreted, which is the slope 2.5:1 (Figure 42). The results show that the factor of safety is equal to 1.26 during the excavation stage. Furthermore, the results for the different stages were described in table 12.

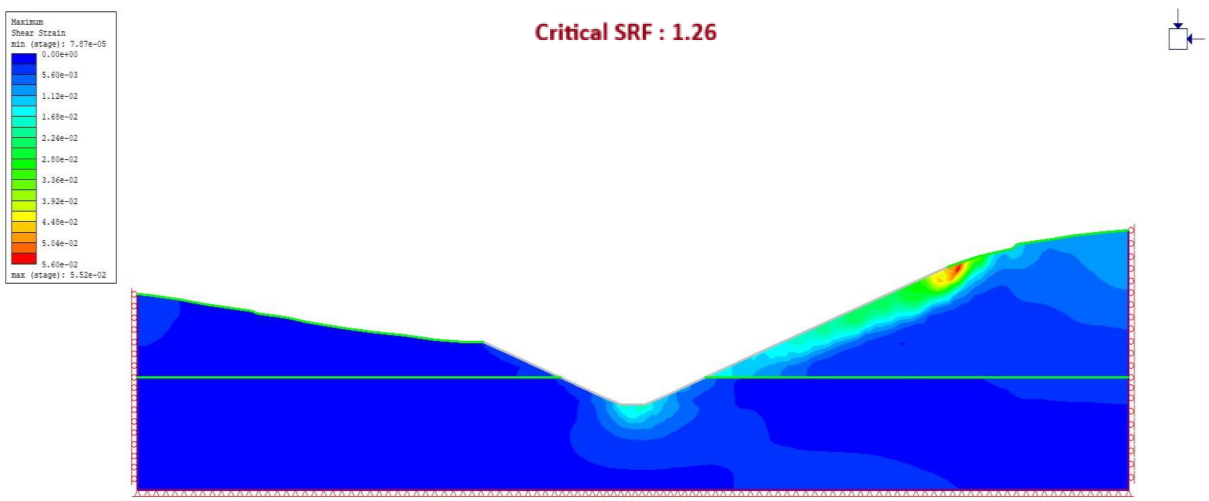


Figure 42 Factor of safety for slope 2.5:1

Table 12 Factor of safety for the different stages for both normal and designed parameters

#	Stages	Normalized values	Designed values	Verification
1	Geostatic Conditions	1.63	1.31	Verified
2	First stage of Excavations	1.63	1.3	Verified
3	Second Stage of Excavations	1.61	1.28	Verified
4	Third Stage of Excavations	1.57	1.25	Verified
5	Last Stage of Excavations	1.57	1.26	Verified
6	Fifth Stage of Excavations	1.57	1.26	Verified
7	Sixth Stage of Excavations	1.56	1.26	Verified
8	Tunnel Construction Stage	1.56	1.24	Verified
9	First Stage of Backfilling	1.52	1.21	Verified
10	Second Stage of Backfilling	1.55	1.22	Verified
11	Third Stage of Backfilling	1.57	1.24	Verified
12	Fourth Stage of Backfilling	1.59	1.26	Verified

The table shows that all the factor of safety for all stages are verified where the values are greater than 1.1. This means that the design can be used and the geotechnical risk is well preserved. Figure 43 shows the presentations of the shear band and the maximum shear plastic strain.

5.9.2.2. Shear band and plastic strain

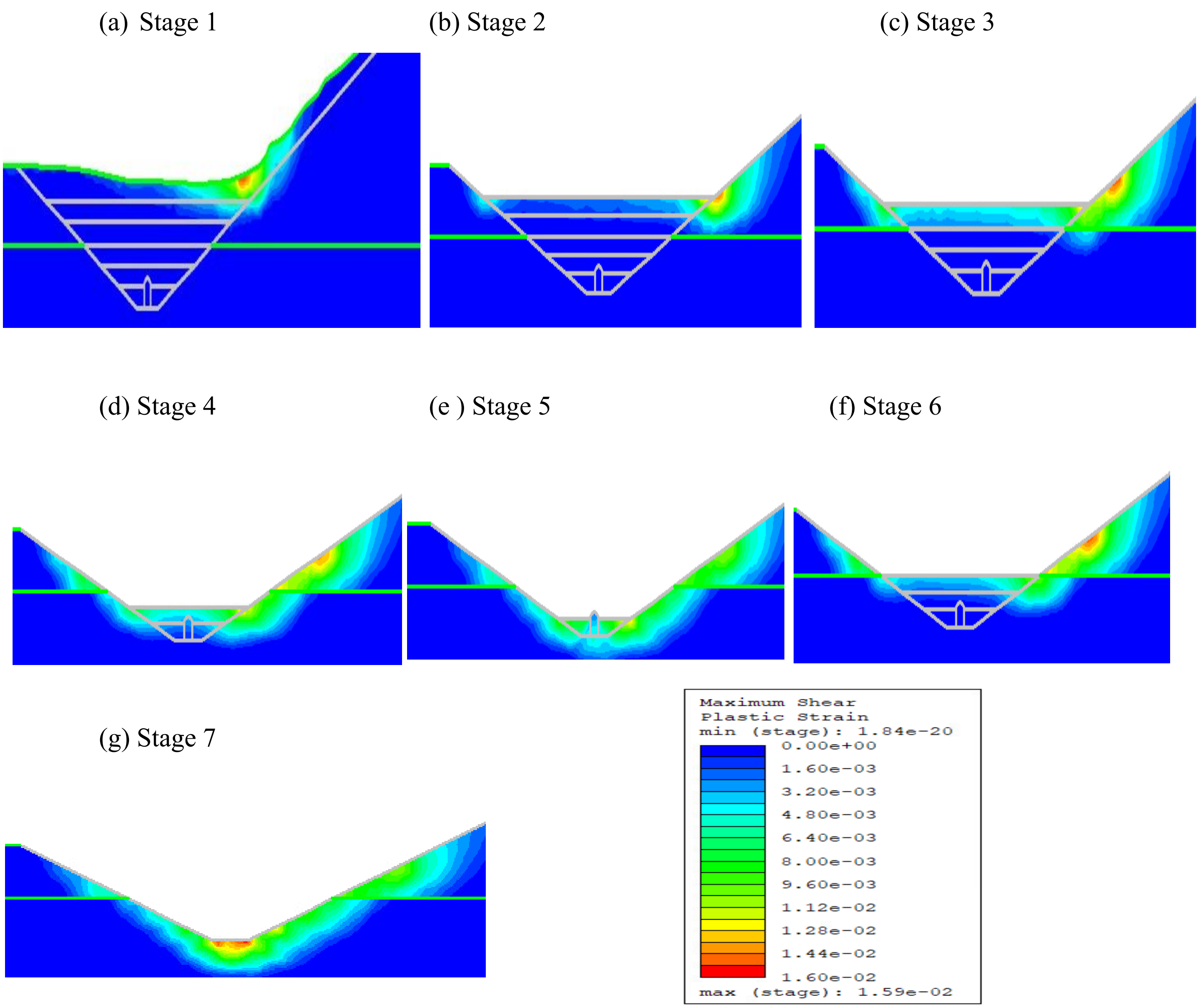


Figure 43 Shear band for the different slope stages and the maximum shear strain contours that illustrate the progressive development of the plastic zone during excavation

5.9.3. Third design approach: Open trench slope with a berm and a grade of 2.5 to 1

The use of berms, does not affect the factor of safety (Figure 44). The slope within the berm is 1.5:1 ratio, that gives a safety factor equals to 0.92. This value is closer to the value given by the same slope without a berm. This indicates that the slope is steep for a longer distance and that the bandit width is still affected by the steep slope. This means that making berms is not an effective solution for a slope that is steep for a longer distance.

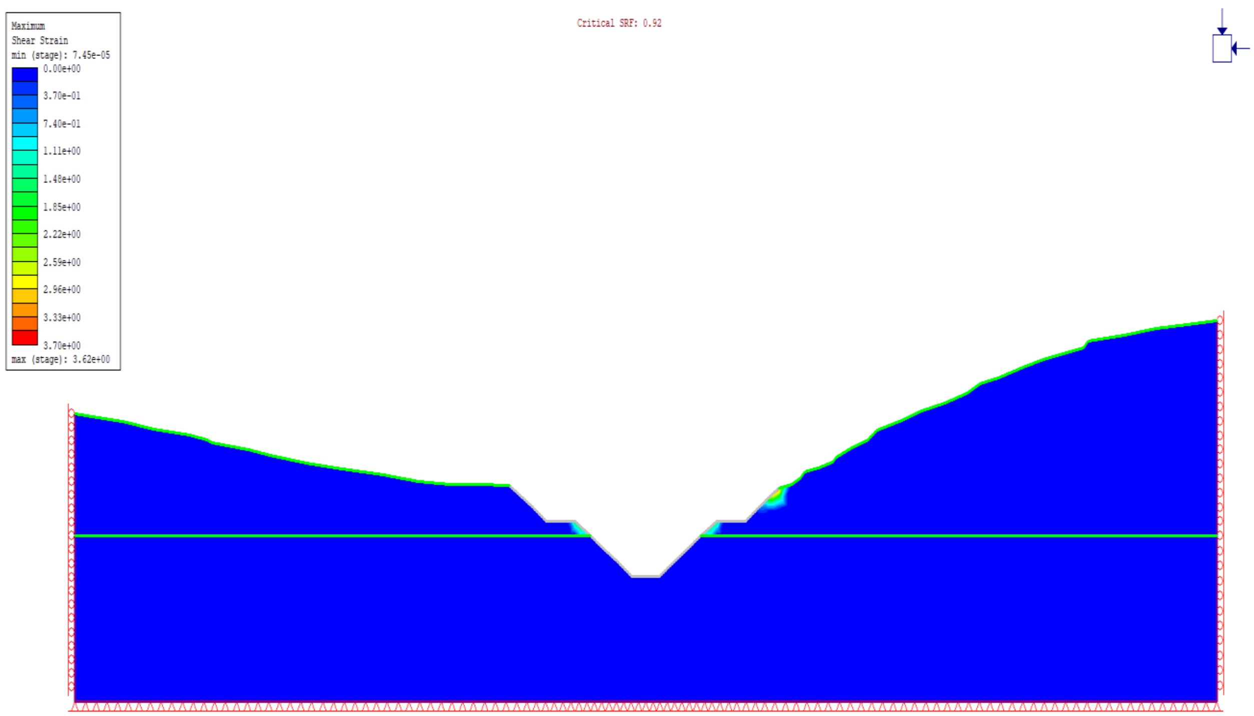


Figure 44 Factor of safety for the slope 1.5:1 with a berm

5.9.4. Fourth design approach: Slope reinforced with piles

5.9.4.1. Factor of safety

Another model was built where piles were installed at the sides to secure stability for the excavation and the tunnel. Having a slope at the top with a ratio of 2.5:1, the results shows that the factor of safety is equal to 1.38. This shows a better result than previous case (Figure 44). Figure 45 shows the presentations of the shear band and the maximum shear plastic strain.

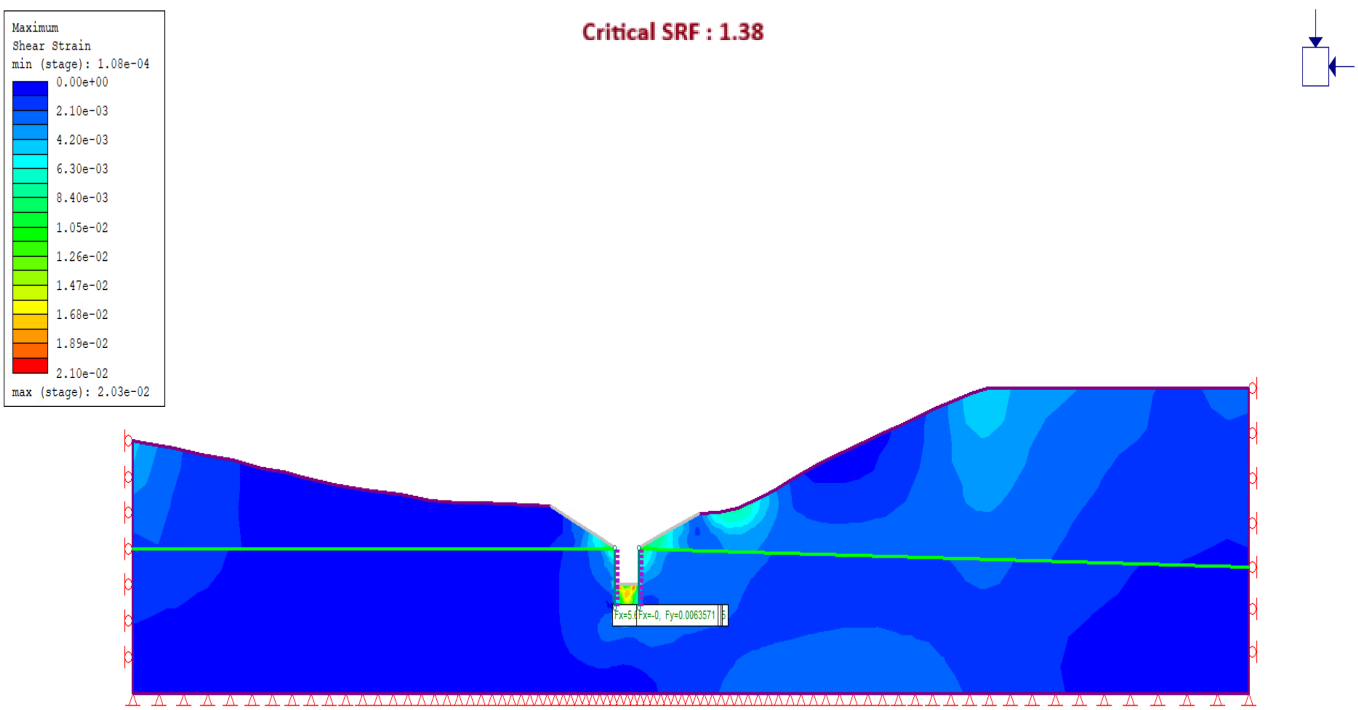


Figure 45 Strength reduction factor SRF for the case that is reinforced with piles

5.9.4.2. Shear band and plastic strain

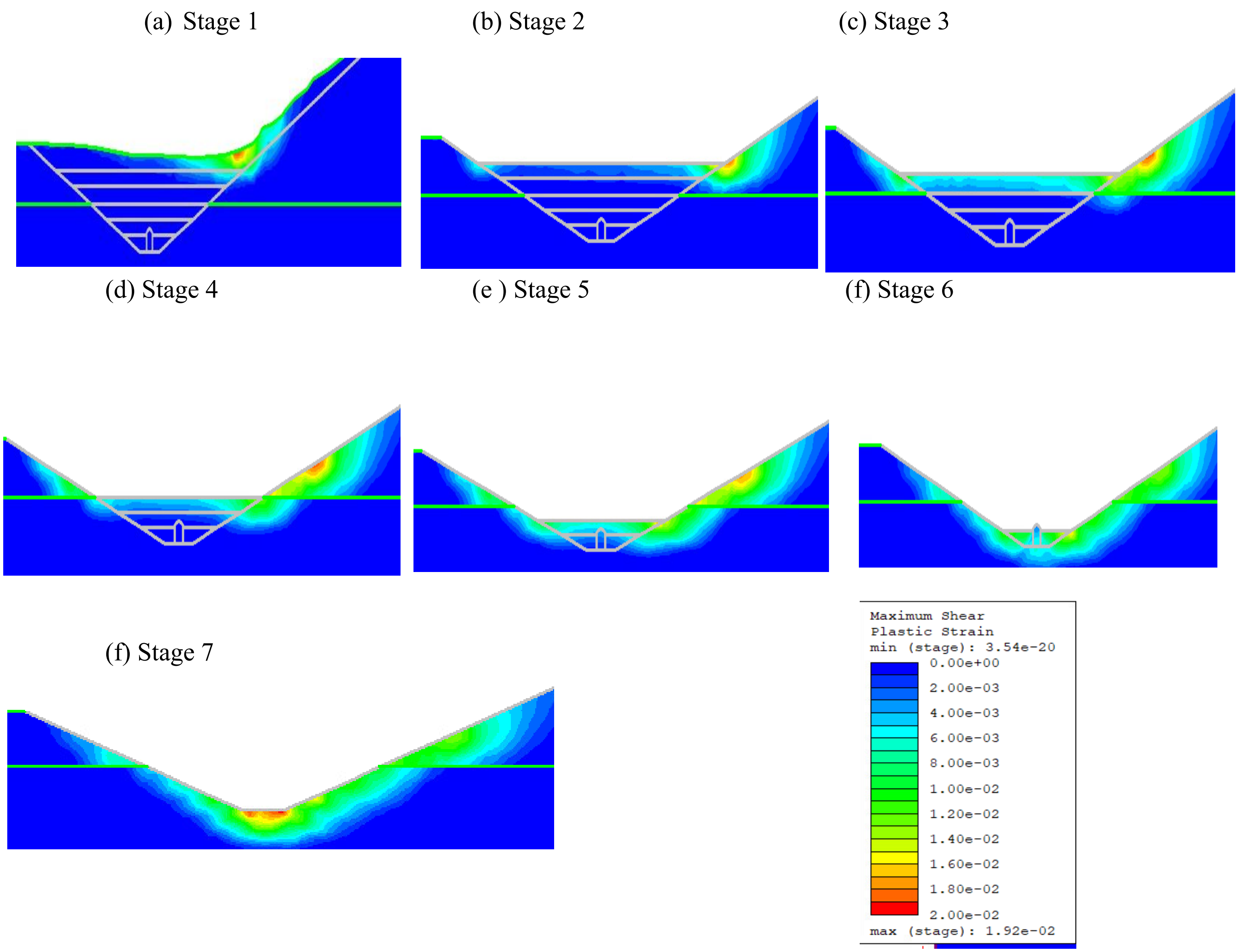


Figure 46 Shear band for the different slope stages and the maximum shear strain contours that illustrate the progressive development of the plastic zone during excavation

Figures 47 and 48 show the orientation in RS2 and the results of the capacity of the liner.

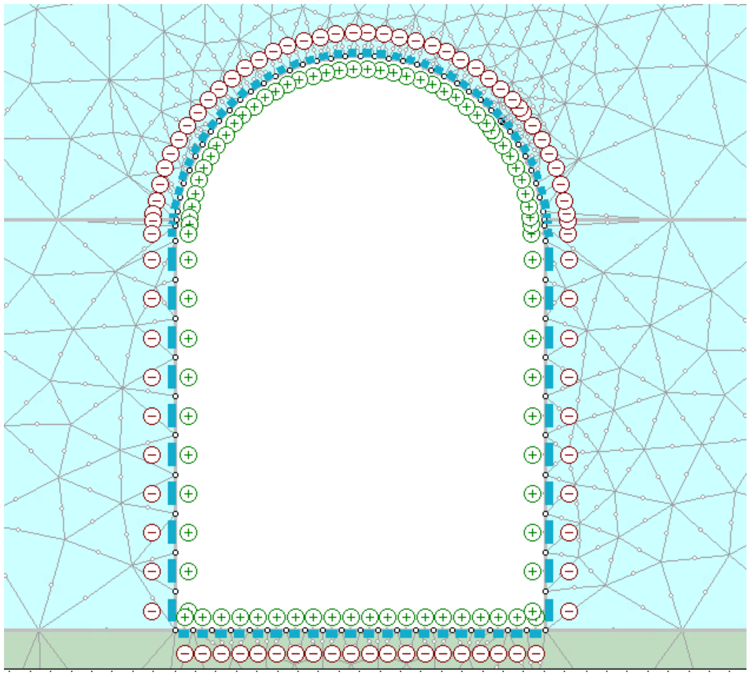


Figure 47 Segment orientation of the liner in RS2

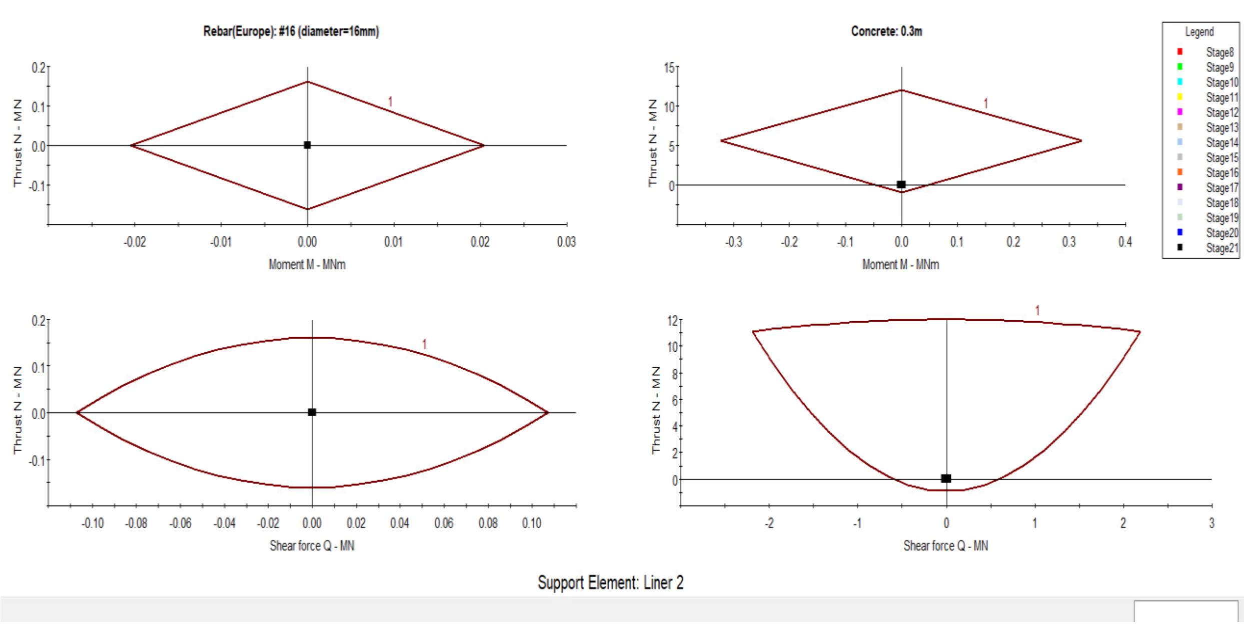


Figure 48 Capacity design of the liner (RS2)

Chapter 6

Cost evaluation for the different design alternatives

6.1. Introduction

This chapter presents the cost estimation for the excavation of the slopes and the demolition of the existing hydraulic tunnel is presented. The unit cost method was used for the cost estimation. This method multiplies the excavated or demolished volume by an appropriate unit price, obtained from reliable references such as Rete Ferroviaria Italiana (RFI) tariffs or recognized construction price lists (Craus, 2015). The analysis considers two slope configurations: 2.5:1 and 1.5:1, as well as the demolition of the masonry tunnel that intersects the excavation.

This chapter provides a general estimate of excavation costs for every design solution. Since there is poor data coming from the supervisors and since there is intentions by the geotechnical engineer to give more data and emphasis about this domain, a general inclusion of data was put. For simplicity, the cost of excavation will be calculated with liner meter.

6.2. Slope 2.5:1

6.2.1. Excavation

The excavation of the slope with a ratio of 2.5:1 requires the removal of a soil area of approximately 132 m². The soil is assumed to consist of sand and clay, corresponding to non-lithoid material as defined in the RFI tariff.

The excavation volume is calculated as:

$$V_{2.5:1} = 132 \times 1 = 132 \text{ m}^3$$

The depth of excavation is on the range of 5 m. Therefore, the excavation volume is subdivided according to the depth ranges defined in the RFI tariff: Excavation with a limited and/or restricted section in soils of any nature and consistency, excluding lithoid materials.

6.2.2. Equipment

Excavation is assumed to be performed using standard mechanical equipment, such as hydraulic excavators with a bucket suitable for sand and clay soil. No specialized equipment is considered, as the slopes are within standard construction parameters and accessible with conventional machinery.

6.2.3. Cost

Using the unit cost method, the excavation cost for the 2.5:1 slope is calculated as:
The excavation cost is calculated using the following RFI BA tariff items:

- BA.MT. A.3003.A of unit cost: 6.14 €/m³

The excavation cost is therefore:

$$C_{exc}, 2.5{:}1 = 132 \times 6.14 \approx 12{,}157 \text{ €}$$

6.3. Slope 1.5:1

6.3.1. Excavation

For the excavation of the slope with a ratio of 1.5:1, the expected soil area is approximately 71.4 m². The excavation volume is computed as:

$$V_{1.5:1} = 71.4 \times 1 = 810.5 \text{ m}$$

The unit cost method is applied similarly, using the same RFI reference for excavation of non-rock soils.

6.3.2. Equipment

The same mechanical excavation equipment described for the 2.5:1 slope is assumed, as the soil type and excavation technique remain unchanged.

6.3.3. Cost

Applying the unit cost:

$$C_{\text{exc}} = 71.4 \times 6.14 \approx 438.4 \text{ €}$$

6.4. Slope reinforced with micro piles

6.4.1. Excavation volume

For the excavation of the slope with a ratio of 1.5:1 supported by micro-piles, the expected soil area is approximately 27 m². The excavation volume is computed as:

$$V_{1.5:1} = 27 \times 1\text{m} = 27 \text{ m}^3$$

The unit cost method is applied similarly, using the same RFI reference for excavation of non-rock soils.

6.4.2. Cost

The cost of micropile installation depends on several factors as described in the FDHA for design and construction of micro piles. The factors that influence the micro pile costs are the micro pile materials like threaded steel casing, and steel reinforcing rods, the equipment cost like grouting and drilling equipment, the labor cost, and the load testing costs. FHWA specified in chapter 10, that a general cost is about 300 **€** per linear meter of micro pile. Therefore, for each micro pile that has a length of 6 meters costs about 1800 **€**. The same mechanical excavation equipment described for the 2.5:1 slope is assumed, as the soil type and excavation technique remain unchanged. Table 13 shows a summary table of cost for excavating and supporting of 1 m^3 of soil. It is important to mention that the actual cost for the slope that is supported with micro piles will have an extra 300 **€** for every linear meter of construction.

Table 13 Cost summary table

Slope	Excavation Volume (m^3)	Unit Cost (€/m^3)	Excavation Cost (€)
Design solution 1: 2.5:1(H: V)	132	6.14	810.5
Design solution 2: 1.5:1 (H: V)	71.4	6.14	438.4
Design solution 4: 1.5:1 (H: V) + Micro piles	27	6.14	166

From an economic point of view, this solution requires a smaller excavation volume compared to flatter slope configurations, resulting in lower excavation, transportation, and construction costs. Since no additional reinforcement measures are required, this solution also simplifies the construction process. Therefore, Design Solution 1(1.5H:1V) represents the most appropriate solution, as it ensures sufficient stability while minimizing excavation volume and overall construction cost.

Chapter 7

Conclusion

This thesis describes a numerical study carried out to evaluate the most suitable design alternatives for the reconstruction of the hydraulic tunnel “Galleria del Riserasco”. The geotechnical and geological investigations were carried out to best identify the soil parameters. Several excavation and stabilization strategies were analysed using the finite element modelling (FEM) and the RS2 software. The strength reduction method was applied to evaluate the factor of safety for each design alternative throughout all construction stages, including excavation, tunnel construction, and backfilling. The results indicate that several design alternatives satisfy the minimum safety requirements specified by the NTC 2018, with factors of safety greater than 1.1 during all critical stages of excavation. However, the selection of the most appropriate solution must consider not only stability but also economic, environmental, and practical construction aspects.

Among the investigated options, Design Solutions 1, 2, and 4 provide acceptable safety conditions. Nevertheless, each alternative presents different practical implications. Design solution 2 (slope 2.5H:1V) requires a significantly larger excavation volume, resulting in increased land consumption and the removal of surrounding vegetation. Although design solution 4 provides an adequate factor of safety, it requires the installation of micropiles, which would considerably increase construction costs.

In contrast, design solution 1 (slope 1.5H:1V) offers a balanced compromise between safety, economic feasibility, and land use. While its factor of safety is slightly lower than some of the other alternatives, it still satisfies the minimum stability requirements while limiting excavation volumes and avoiding expensive structural reinforcement.

The rehabilitation of the Riserasco hydraulic tunnel in Pralormo therefore required a solution capable of addressing the structural failure while respecting the geotechnical constraints of the Poirino Plateau. In this study, RS2 finite element modelling was used to simulate the construction phases of four different excavation strategies and evaluate their stability conditions.

The analysis indicates that the 1.5H:1V open-trench slope (Design Solution 1) represents the most appropriate solution for the project. Although other configurations provide slightly higher factors of safety, they require substantially larger excavation volumes or additional structural elements, leading to increased environmental impact and construction costs. By adopting this solution, the municipality can achieve a design that ensures technical reliability, economic sustainability, and reduced land disturbance, while maintaining the required stability conditions.

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Appendix A

Site Photographs

Pralormo, 10040 Metropolitan City of Turin

Photo 1: Existing Collapse at the side of the tunnel’s entrance from the North

Photo 2: Existing Collapse at the side of the tunnel's entrance from the North

Photo 3: Erosion

Photo 4: Entrance of the tunnel

Photo 5: Water movement

Photo 6: Another photo showing the entrance of the tunnel

Photo 7: Spina's Lake