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SLOPE STABILIZATION TROUGH A GABION WALL: CASE STUDY OF A LANDSLIDE IN SOUTHERN POLAND

A thesis submitted in partial fullfilment of the requirements for the degree of Master of Engineering (MEng) in Civil Engineering

Curriculum : **Geotechnical Engineering**

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DEDICATION

*I dedicate this thesis to my family, especially to my lovely wife **MBETCHOM Monique Amélie** and my son **WANDJI NKEMEGNI Elohim Béni***

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LIST OF ABREVIATIONS AND SYMBOLS

FoS	Factor of Safety
τ_{max}	Maximum shear stress
τ	Shear stress
$E_{resisting}$	Resisting Young Modulus
$E_{driving}$	Driving Young Modulus
$M_{resisting}$	Resisting moment
$M_{driving}$	Driving moment
h	Height of the element to break
U	Interstitial force of water acting at the base
β	slope angle
S	resistance at the plane failure surface
ϕ'	Friction angle
N	Normal force
T	Tangential force
W	Weight of the slice
S_a	Available resistance force along the slip surface
C'_a	Sum of drained cohesion forces along the slip surface
ϕ'_a	Peak drained friction angle for the material
N'	Sum of effective normal contact forces along the slip surface
S_r	Required resistance force
R_d	Weight of the slidable mass
r_d	Distance from the center of the arc to the line of action of R_d
Cu	Undrained shear strength along the slip surface
R	The radius of the arc
La	Length of the arc
θ	Angle of the circular slip surface
$\bar{N}$	Normal force acting on the slice
$\bar{T}$	Tangential force acting on the slice
α	Angle at the base of the slice with the horizontal line

l	Length of the sliding surface of the slice
LE	Limit Equilibrium
FE	Finite Element
SRM	Strength Reduction Method
FD	Finite Difference
LA	Limit Analysis
DLO	Discontinuity Layout Optimization
ASTM	American Society of Testing Materials
PVC	Poly Vinyl Cloride
Zn-5Al-MM	Zinc-5% Aluminum - Mischmetal alloy
P_a	Pressure distribution acting on the wall
γ_s	Density of backfill soil
H	Wall height
K_a	Coefficient of active pressure
q	Uniformly distributed surcharge on the top of the backfill surface
P_h	Horizontal component of P_a
P_v	Vertical component of P_a
FoS_{slig}	Factor of safety against sliding
F_r	Resisting force
F_s	Sliding force
W_v	Sum of vertical force
FoS_{over}	Factor of safety against overturning
M_r	Resisting Moment
M_o	Overturning moment
W_g	Weight of the gabion
e	Eccentricity
σ_{max}	Maximum pressure under the base
q_{all}	Allowable soil bearing pressure
FoS_{bear}	Factor of safety for bearing capacity
C'_n	Cohesion of soil layer n
L_n	Length of the anchor in soil layer n

ϕ'_n	Friction angle of soil layer n
N_c & N_q	Lateral resistance
EC7	Eurocode 7
GEO	Geotechnic limit state
EQU	Equilibrium limit state
A	Actions
M	Material
R	System resistance
E_d	Action effect
R_d	design value of resistance
°C	Celsius
%	Percentage
f_c	Compressive strength
MC	Mohr Coulomb
Engi	Engineering
σ_0	Pre-stress
n	Anchor spacing
D	Anchor diameter
PH	Potential of Hydrogen
fact	factor
resis:	resistance
stren:	strength
comp:	compression

ABSTRACT

The main objective of this work is to analyze the slope stability through a gabion wall, with the case of an unstable slope in the city of Sacz, located in southern Poland. To achieve this objective, a comprehensive review of stability analysis methods and different methods of slope stabilization is presented. In addition, an in-depth study on gabion retaining walls is highlighted; this is done to understand their functioning and dimensioning system, as well as the different degradations they are exposed to. The numerical analysis are done using the software LimitState:GEO. The latter uses the Discontinuity Layout Optimization method. The slope stability analysis is done in three steps. The following results of the safety factor are obtained: 0.9884 for the existing landslide, 1.082 for the landslide with gabion wall, 1.159 for the landslide with gabion wall reinforce by two anchors, 1.242 for the landslide with gabion wall reinforce by three anchors, 1.242 for the landslide with gabion wall reinforce by four anchors. For a better safety, it is necessary to reinforce our gabion wall by four anchors since it takes into account possible additional loads due to a redesign of the road in the future. To highlight the influence of certain parameters on slope stability, a parametric analysis is made. The friction angle at the interface gabion-gabion and the friction coefficient at the interface gabion-soil interfaces have a significant influence on the safety factor. The anchor spacing should be taken into account since when it increases, the slope stability decreases. The level of the water table should be controlled because its influence is very negative on the stability of the slope.

Keywords: slope stability, safety factor, retaining wall, gabion, reinforcement, numerical analysis.

RESUME

L'objectif principale de ce mémoire est l'analyse de la stabilité d'une pente à travers un mur en gabion, cas d'un talus instable dans la cité de Sacz, situé au sud de la Pologne. Pour atteindre cet objectif, une étude progressive et objective est faite sur les méthodes d'analyse de la stabilité et les méthodes de stabilisation des pentes. De plus, une étude approfondie sur les murs en gabion est présentée ; ceci dans le but de mieux comprendre leur système de fonctionnement et dimensionnement, ainsi que les différentes dégradations auxquelles ils sont exposés. Notre analyse est faite à travers le logiciel LimitState : GEO. Ce dernier utilise la méthode « Discontinuity Layout Optimization ». l'analyse de la stabiité du talus est faite en trois étapes. Les résultats suivants du facteur de sécurité ont été obtenues : 0,9884 pour le talus existant, 1,082 pour le talus stabilisé avec le mur en gabions, 1,159 pour le talus stabilisé avec le mur en gabions renforcé par deux ancrages, 1,242 pour le talus stabilisé avec le mur en gabions renforcé par trois ancrages, 1,242 pour le talus stabilisé avec le mur en gabions renforcé par quatre ancrages. Pour une meilleure sécurité, il faut renforcer notre mur en gabion par quatre ancrages car prenant en compte d'éventuelles charges supplémentaires dû à un réaménagement de la route dans le futur. Afin de mettre en exergue l'influence de certains paramètres sur la stabilité du talus, une analyse paramétrique est faite. L'angle de friction à l'interface gabion-gabion et le coefficient de friction à l'interface gabion-sol une influence non négligeable sur le facteur de sécurité. L'espacement entre les ancrages doit être pris en compte car plus il est grand, moins le talus est stable. Le niveau de la nappe phréatique doit être contrôlé car son influence est très négative sur la stabilité d'un talus.

Mots clés : stabilité de pente, facteur de sécurité, mur de soutènement, gabion, renforcement, analyse numérique.

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GENERAL INTRODUCTION

The landslide phenomenon is considered as a permanent natural danger in all countries of the world. The extend of the damage caused by this phenomenon can result in huge losses of human life and materials that can be evaluated to several million dollars. This must be an object of interest.

The design of a road may encounter technical and/or natural difficulties that must be taken into account because a well-constructed and well-maintained road network is essential for economic growth. Road construction can be done in a mountainous area. It is the case of this study which is located in a sub-mountain area of southern Poland, and therefore necessary to find a solution allowing effective retention of the soil.

The retaining structures appear as an obvious solution to overcome the problems related to earth movements. The construction of a gabion wall was chosen as a solution for the stabilization of the slope in this case study. It is therefore important to perform numerical analysis using a software to study all the possible cases to obtain the critical failure surface and define the conditions in limit state.

The main objective of this study is to analyze the stability of the slope through an anchored gabion wall. In addition, a parametric analysis is done in order to point out the role of some parameters in the stability of the slope. The study will be conducted through a numerical modeling of the slope and numerical analysis of some representative cases, using the software LimitState:GEO which is based on the Discontinuity Layout Optimization method.

To achieve the above objective, this work is subdivided into two parts:

- The first is the Literature review, consisting of two chapters. Chapter 1 represents the general concepts of slope stability analysis and stabilization methods, in which we present the different methods of analysis and stabilization of slopes. Chapter 2 is a presentation of Gabions wall illustrating a profound understanding of gabion wall, its construction standards, its design method and the different failures that may appear later
- The second starts by chapter 3, presenting the methodology we used. A general presentation of the method we used to analyze the gabion wall is done. Then, it ends by chapter 4, Numerical model of a Gabion wall and interpretation of the results, in which a preliminary part is dedicated to the presentation of the project. Then follows a presentation and

interpretation of the different results of the numerical analysis and finally a parametric analysis.

Chapter 1. SLOPE STABILITY ANALYSIS AND STABILIZATION METHODS

Introduction

Slope stability is a common problem encountered in the construction of roads, dikes, canals and dams. The disorders caused by the failure of these slopes are generally spectacular, often destructive and sometimes deadly (Khemissa, 2006). Hence the importance of studying their stability. Numerous stability calculation methods (which differ in the hypotheses accepted by their authors) have been proposed. Since slopes are essentially made up of soil, it is very important to understand deeply this complex material. In this chapter, key concepts of soil will be detailly explained, followed by the different methods of slope stability and finally slope stabilization methods.

1.1 Soil

1.1.1 Definition

Different researchers gave several definitions of the word Soil. Thus, according to MISCHERLICH, "soil is a mixture of pulverised solid particles, water and air, which serve as a support for plant nutrients". RAMMAN then presented the Soil as "the upper loose layer of the earth's crust. It consists of more or less chemically transformed rock fragments with the debris of plants and animals that live on them". According to DOKUCHAEV, Soil consists of "the upper horizons of a rock that has undergone a change under the influence of water, air, and different species of living or dead organisms". The definition was later perfected and led to that of JOFFE, for whom the Soil is “a natural body of organic and mineral constituents, differentiated into horizons of varying thickness, which differ from the underlying material by their morphology, physical constitution, chemical and physical properties, and composition of biological characteristics” (Dabin & Segalen, 2000).

Soil can be also considered as the fine earth which covers land surfaces as a result of in situ weathering of rock materials or the accumulation of mineral matter transported by water, wind or ice. The distinctive feature of soil is that to this weathered mineral material is added organic material (Nortcliff, Bisping, Bannick, & Litz, 2006).

1.1.2 Composition of soil

Soil is composed of various factors like air, water, minerals particles and organic matter. In general, minerals particles are about 40-45% of the soil volume, followed by air and water that occupy 25% each finally by organic matter with 5%. The exact composition of soil, however, might vary from place to place with the existing rocks of the area and the climate. Other factors like the quantity of vegetation, soil compaction, and water retention capacity also influence the composition of the soil in particular area. The organic component of soil is called humus, which is made up of living organisms like insects or microorganisms (dead or alive) and dead animals and plants in varying stages of decay.

1.1.3 Formation of soil and Types of soil

Soil formation is a very slow process. It takes thousands of years to form a very thin layer. Soil is formed under specific naturals conditions. The parent rock (metamorphic or sedimentary rock) is subject to climatic actions (rain, wind, runoff, frost, exposure to the sun). It will undergo alterations that will lead to fragmentation (mechanical action) and dissolution (chemical action) of the parent rock. The altered surface will be colonized by so-called pioneer plants (mosses). Their roots will accelerate the alteration of the rock. The decomposition of the leaves will lead to the formation of a fine litter and then humus. As time goes by, we will have germination of new plants, thus deep and continuous alteration of the parent rock. This will lead to the formation of several horizons (litter, humus, accumulation horizon and alteration horizon), which constituted the Soil.

Soil, in general, is classified into four different types depending on its composition and the size of particles. They are:

- *Sandy Soil:* type of soil that contains a higher proportion of sand and less clay. Because of the size of the sand particles, sandy Soil has low water retention capacity and fewer nutriments
- *Clay Soil:* type of soil that is very dense and drains water very slowly. It has higher water retention capacity since the particles size is very small.
- *Silt Soil:* It’s a light soil with particles size that is large than clay but smaller than sand. It holds water better than sand and encourage plant growth since it retains sufficient nutriments.

- *Loam Soil:* It's a mixture of sand, silt and clay soil, therefore combines the properties of all three the of soil.

1.1.4 Properties of soil

The properties of soil are related to its composition and other factors such as (air, water, minerals particles and organic matter). The combination of these components determine the physical and the chemical properties of soil

1.1.4.1 Physical properties

a. Soil texture

Soil texture refers to the size of the soil particles. It is further influenced by the soil porosity, infiltration and water retention capacity. The texture of soil differs with soil type; sandy soil feels gritty, silt feels smooth and clay is sticky and moldable.

b. Soil structure

The soil aggregates further clump together to form "peds". Between peds are cracks called pores through which soil air and water are conducted. The soil structure is commonly described in terms of the shape of the individual peds that occur within a soil horizon. Soil structure types are granular, platy, blocky, prismatic and columnar.

c. Soil density

The average soil particle density ranges from 2,60 to 2,75 grams per cm^3, which usually remains unchanged for a given soil. This density is lower for soil with high organic matter content and higher for soil with mineral content.

d. Soil porosity

Soil porosity is defined by the number of pores present within the soil. It is influenced by the soil's texture and structure. The pore size in soil affects the ability of plants and organisms to access water, oxygen and other gases and minerals.

e. Soil color

Soil color is determined primarily by the organic composition of the soil. Observation of soil color is a qualitative means of measuring the organic, iron oxide and clay content of the soil.

1.1.4.2 Chemical properties

a. Cation exchange capacity

Cation exchange capacity is the maximum amount of total cations that a soil sample is capable of holding at a given pH.it is used as an indicator of soil fertility, nutriments retention and ability of soil to protect groundwater from cation contamination.

b. Soil pH

The soil pH expresses the acidity or the alkalinity of the soil. It is the measure of hydrogen ion concentration in the aqueous solution. Usually, soil with high acidity contains high amount of aluminum and manganese, and soil with high alkalinity contains high amount of sodium carbonate.

1.1.5 Soil stabilization

Soil stabilization is the process of improving certain desired properties in a soil material to make it more stable and useful for a specific purpose. Improvements in the mechanical properties of the soil to make it more stable, promote an increase in soil strength (shear strength), increase in stiffness (resistance to deformation) and durability (resistance to wear), reduction of swelling.

In construction, the slum stabilizers are mainly: gravel, crushed aggregates, sand, Portland cement, lime (quicklime or hydrated lime) and gypsum, pozzolan, asphalt (bitumen). Each stabilizing agent has a specific role in soil stabilization.

1.2 Slope stability concepts

1.2.1 Mode of failure

Slope failures can be due either to a sudden or gradual loss of soil strength, or to an increase in stress (overloading, removal of the toe stop, deforestation, earthquake), or to a change in the mechanical (loss of resistance through reshaping) or hydraulic characteristics (appearance of runoff, snowmelt) of the field (Khemissa, 2006).

Slope failures take different types of shape such as plane, straight, arc of circle, spherical cap, and a combination of several types. In the calculations, the plane failure mode is considered for plane landslides and the cylindrical failure mode for rotational landslides. Planar landslides occur by shear and translation on a more or less regular inclined plane, where the moving soil mass behaves as a monolith with very small and very localized deformations at the fracture surface.

Simple rotational landslides result in a tilting of the mass along a surface (superficial landslides) or at depth (deep landslides) in an isotropic and homogeneous medium; complex rotational landslides result from the interlocking of simple rotational landslides whose overall failure surface is not circular (staircase landslides, regressive landslides, epicycloidal landslides) and which generally evolve in heterogeneous and anisotropic environments. Figures 1.1 & 1.2 illustrate the different modes of landslides

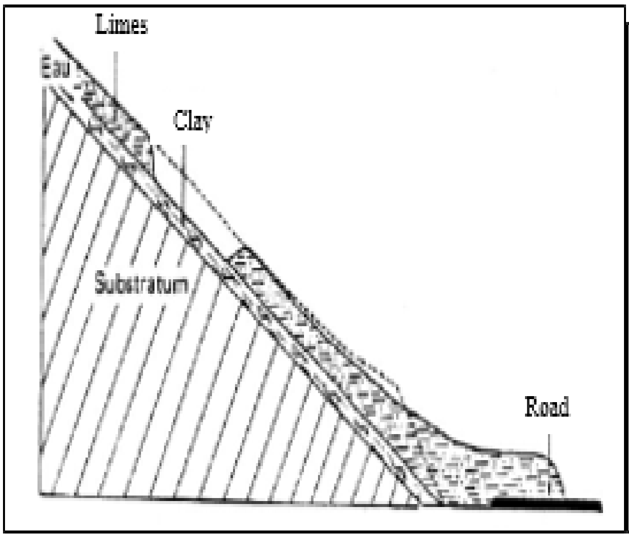

Figure 1.2. Plane landslide **(Reiffsteck)**

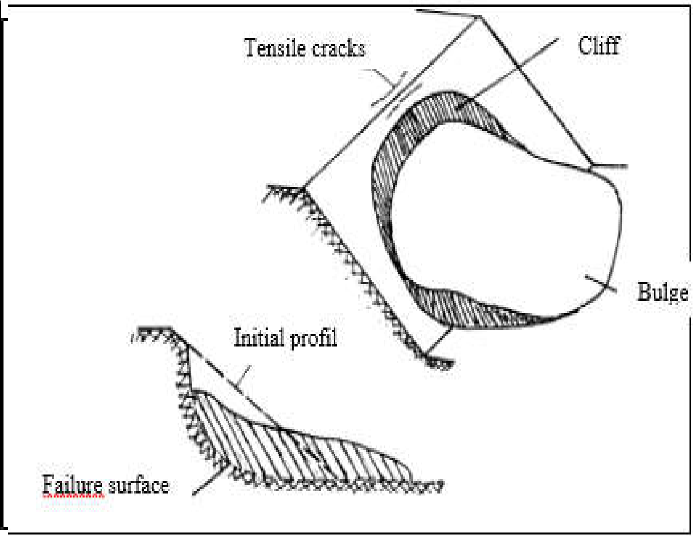

Figure 1.1. Simple rotational landslide **(CHENAFA, 2006)**

1.2.2 Factors of safety

The safety factor FoS is a very important value in the calculation of slope stability. It is used to assess the safety margin against failure. It can be defined in several ways.

If the necessary geotechnical data are defined, the slope stability calculation can be performed using one of the calculation methods available in the technical literature. The calculation principle consists of determining the safety factor FoS by which the resistance of the sliding surface is to be divided so that the potentially stable mass is at the equilibrium limit. For this purpose, the engineer will have to choose between different definitions of the safety factor which can be a ratio of forces (plane slip), moments (rotational slip) or quantities with respect to a limiting quantity as shown in Table 1.1.

Table 1.1. Various examples of definitions of a safety factor. LAMBE [1973] **(MASEKANYA, 2008)**

Définition	**Formula**
Stress ratio	$FoS = \tau_{max}/\tau$
Forces ratio	$FoS = E_{resisting}/E_{driving}$
Ratio of moment	$FoS = M_{resisting}/M_{driving}$

where τ is shear stress, E is Young modulus and M is moment.

After selection and calculation of the safety factor, the decisive fracture surface, found through an iterative procedure, is the one that makes the smallest safety factor appear (if $FoS < 1$ there is a fracture; if $S > 1$, there is no fracture; if $FoS = 1$, there is a limit equilibrium).

It is recommended that the safety factor (non-seismic zones) should be in the range (1.25; 1.5). However, the required safety factor may be greater if there is a high risk of loss of life or uncertainty in the design parameters. Similarly, a lower factor of safety may be used if the engineer is sure of the accuracy of the input data and if the construction is closely monitored (Hai-Sui & Huang, 2018)

1.2.3 Slope stability analysis

1.2.3.1 Limit equilibrium method

All limit equilibrium methods use the Mohr-Coulomb expression to determine the shear strength (τ_f) along the sliding surface. The shear stress at which a soil fails is defined as the shear strength of the soil. It is given by equation 1.1

$$\tau_f = c' + N' \tan \phi' \tag{1.1}$$

where $c'$is the drained cohesion of the soil along the slip surface, $N'$is effective normal force along the slip surface and ϕ' is the friction angle of the soil along the slip surface.

a. Infinite slope analysis

The calculation model is that of an infinite soil mass resting by a flat interface on a bedrock, with flow parallel to the slope. The slope is inclined at an angle with respect to the horizontal, with the height of the element being able to break h as shown in Figure 1.3.

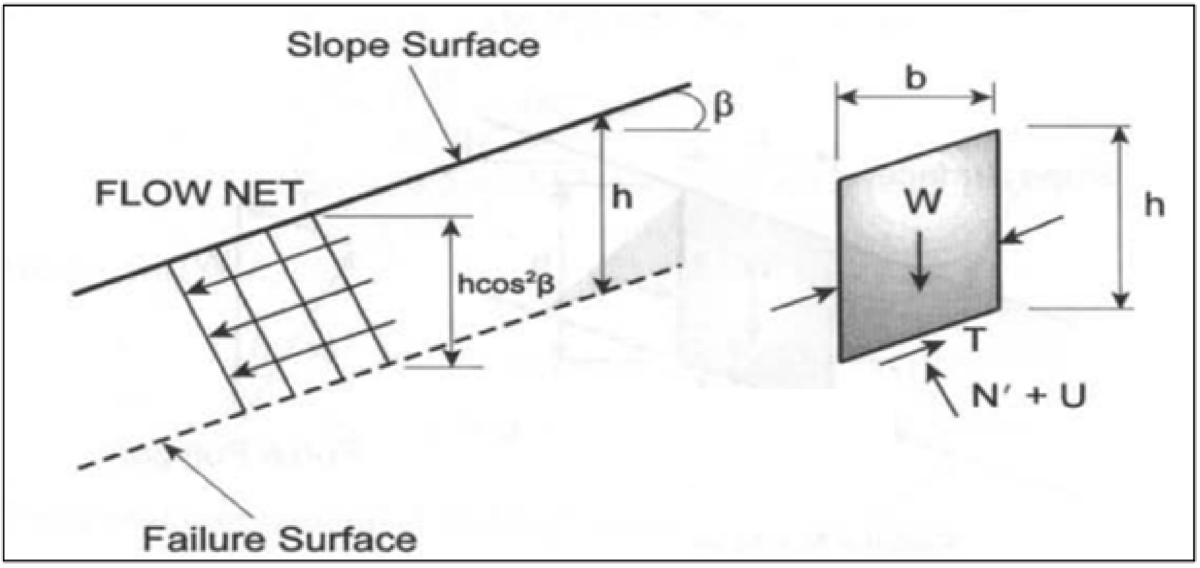

Figure 1.3. Infinite slope failure with parallel seepage (Abramson, Lee, Sharma, & Boyce, 2002)

Let's take a simplified case of the general ; the slope is assumed to be saturated ($h = h_w$). The interstitial force of water acting at the base of the slide is given by the relation

$$U = (\gamma_w\, h\, cos^2\beta)\frac{b}{cos\beta} = \gamma_w\, b\, h\, cos\beta \quad (1.2)$$

The value of the resistance at the plane failure surface depends on ϕ' and the normal force N and is given by:

$$S = c'sec\beta + (N - U)\tan\phi' \quad (1.3)$$

The factor of safety $FoS = \frac{S}{T} = \frac{c'sec\beta + (N-U)\tan\phi'}{T}$ (1.4)

The driving force of sliding is $T = W\, sin\, \beta$, the normal force $N = W\cos\beta$ and the weight of the slice $W = \gamma_{sat}\, b\, h$.

After substitution in equation 1.4, the safety factor is then

$$FoS = \frac{c' + h(\gamma_{sat} - \gamma_w)cos^2\beta\tan\phi'}{\gamma_{sat}\, h\sin\beta\cos\beta} \quad (1.5)$$

b. Planar surface analysis

For a plane failure surface, the limit equilibrium analysis can be solved by two equations. N' is normal to the failure surface, S_r (the required resistance force) is parallel to the failure surface, R_d the weight of the mass that can slip and the angles α_{dn} et α_{ds} are known. The unknowns are N' and FoS. From figure 1.4, the following equations can be deduced:

$$S_r - R_d \sin \alpha_{dn} = 0 \quad (1.6)$$

$$N' - R_d \cos \alpha_{dn} = 0 \quad (1.7)$$

From which $N' = \frac{S_r}{\tan \alpha_{dn}}$ (1.8)

We then obtain

$$FoS = \frac{S_a}{S_r} = \frac{C'_a + N' \tan \phi'_a}{S_r} \quad (1.9)$$

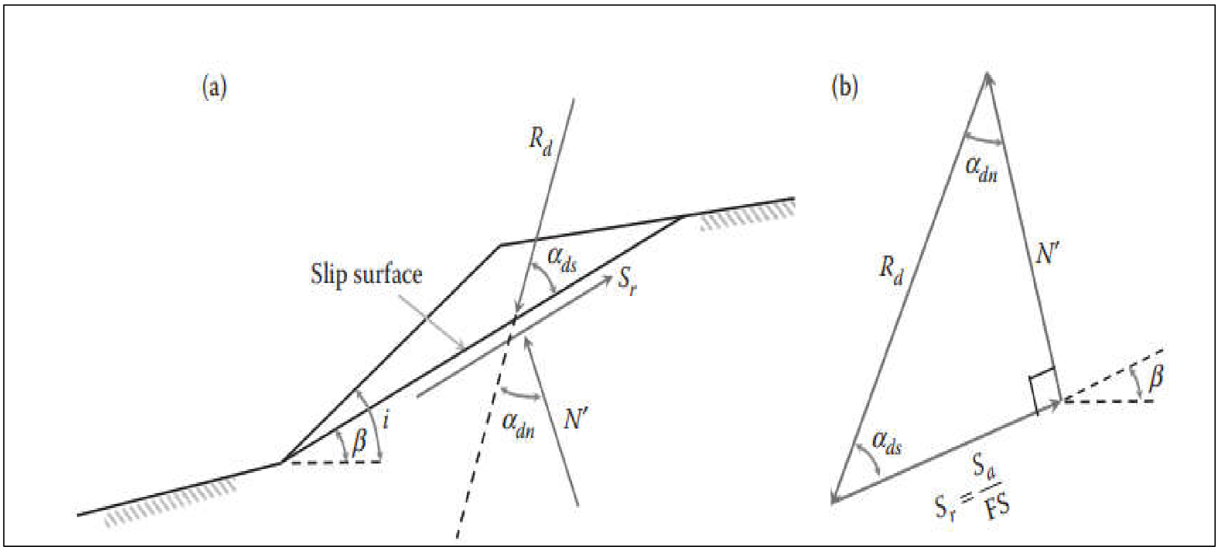

Figure 1.4. Forces acting on a free body with a planar slip surface **(Hai-Sui & Huang, 2018)**

c. Circular surface analysis

For a circular failure surface in a cohesive material under undrained conditions ($\phi = 0$), , there is only one solution. $R_d = W$ the weight of the slidable mass, r_d the distance from the center of the arc to the line of action of R_d, Cu the undrained shear strengh along the slip surface, R the radius of the arc.

The length of the arc L_a is given by equation 1.10

$$L_a = R\theta \quad (1.10)$$

where θ is the angle of the circular slip surface.

The safety factor is calculated using the equilibrium of moments through the formula:

$$FoS = \frac{Cu\, L_a\, R}{W\, r_d} = \frac{Cu\, R^2\, \theta}{W\, r_d} \quad (1.11)$$

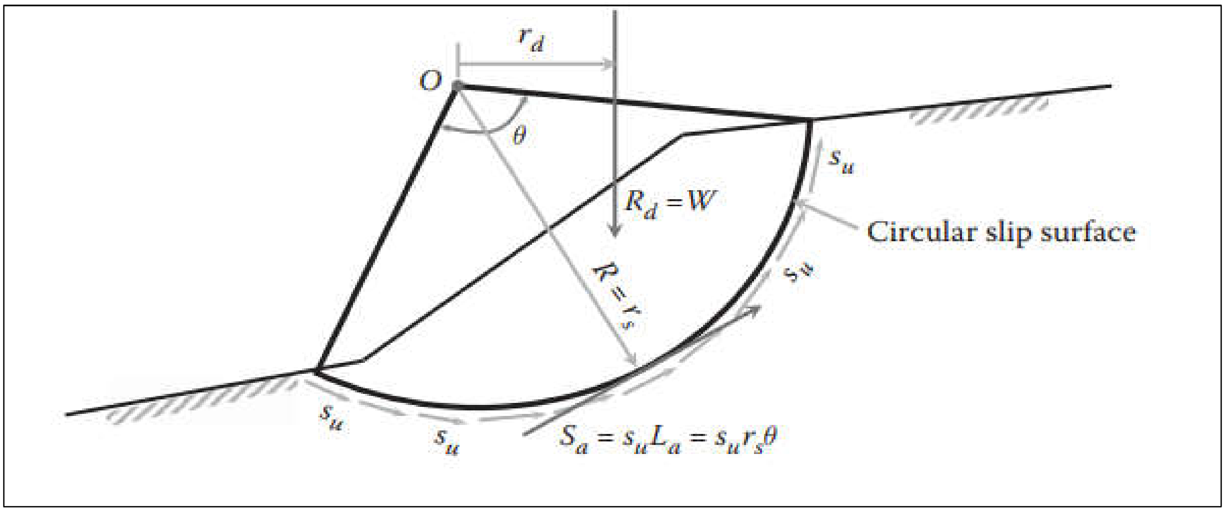

Figure 1.5. Sliding block with a circular slip surface **(Hai-Sui & Huang, 2018)**

d. Method of slices

The method of slices is a versatile and powerful tool under the category of limit equilibrium analysis method for dealing with slopes with an irregular slip surface and in non-homogeneous soils. With the help of computer, this has been widely used in engineering practice. It consists of:

- Divide the sliding block above an assumed slip surface into vertical slices (the width is not uniform) (Figure 1.6)
- The bottom of each slice is simplified as a straight chord
- The properties and the shear strength of the slices can vary

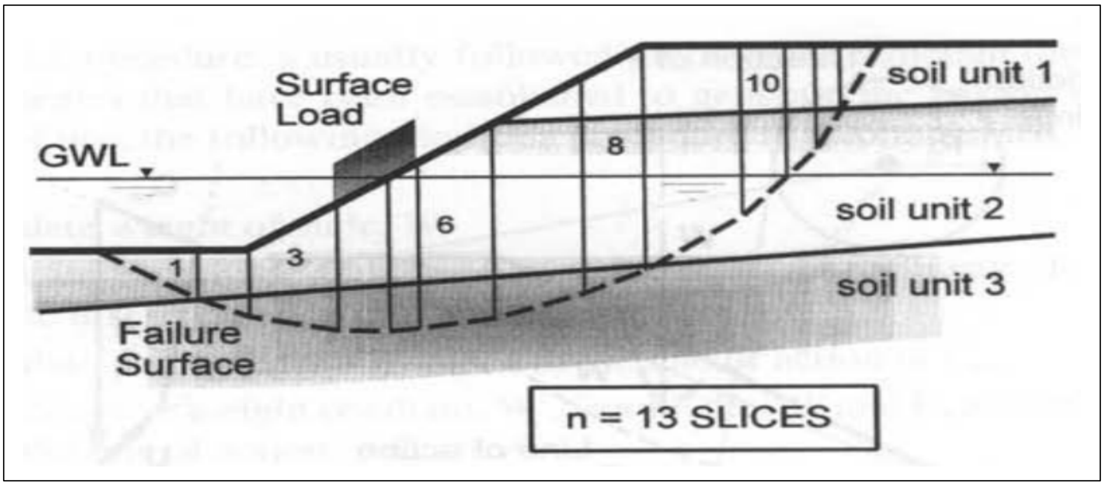

Figure 1.6. Division of potential sliding mass into slices (Abramson, Lee, Sharma, & Boyce, 2002)

Except for the weight of the slice, all other forces are unknown and must be calculated in order to satisfy static equilibrium. Limit equilibrium methods use different assumptions to make the number of equations equal to the number of unknowns.

i. Fellenius method

This method was developed by Fellenius in 1927; it is also known as the Swedish circle or Fellenius method (Hai-Sui & Huang, 2018).

Assumptions

- The method assumes a circular sliding surface and divides the slope into slices
- It neglects the forces between the slices (vertical and horizontal). From figure 1.7, the normal force $\bar{N}$ becomes

$$\begin{cases} \bar{N}\cos\alpha + \bar{T}\sin\alpha = W \\ \bar{T}\cos\alpha - \bar{N}\sin\alpha = 0 \end{cases} \rightarrow \quad \bar{N} = W\cos\alpha \qquad (1.12)$$

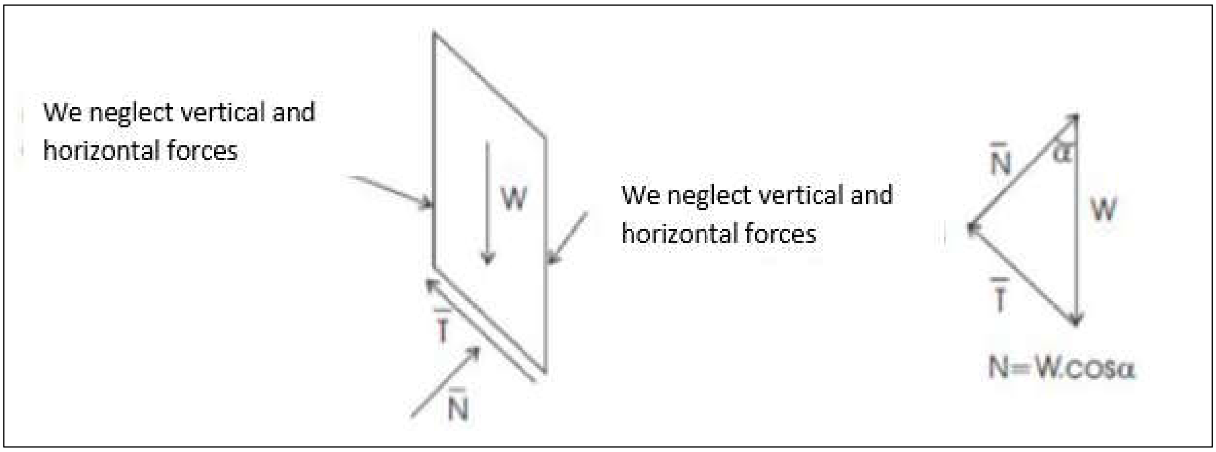

Figure 1.7. Forces apply on the slice with Fellenius method **(MASEKANYA, 2008)**

Fellenius's method checks the global equilibrium of moments while neglecting the forces between slices as we have just seen. By substituting $\bar{N}$ by its value, the safety factor becomes

$$FoS = \frac{\sum c'l + (W\cos\alpha - ul)\tan\phi'}{\sum W\sin\alpha} \qquad (1.13)$$

Where W is the weight of the slice,

$\bar{N}$ and $\bar{T}$ are normal and tangential forces acting on the slice

α is the angle at the base of the slice with the horizontal line

l is the length of the sliding surface of the slice

It is less accurate than other slice methods and is safe for homogeneous soils only (Abderrahmane, 2020) (MASEKANYA, 2008).

ii. Simplified Bishop method

Assumptions

- The method assumes a circular sliding surface
- It neglects the vertical forces between the slide (figure 1.8)
- It considers force equilibrium in vertical direction for each slice

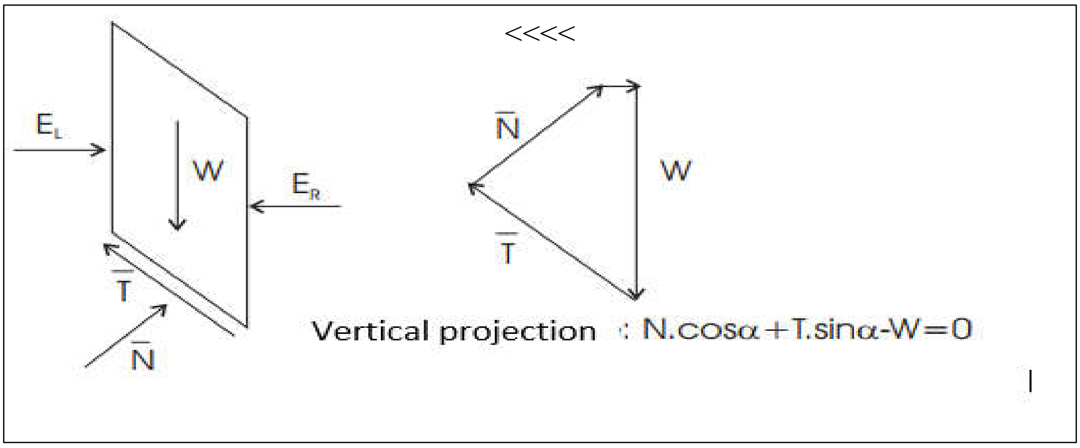

Figure 1.8. Apply forces on a slice with the simplified Bishop method **(Abderrahmane, 2020)**

Bishop's method checks the balance of moments as well as the vertical balance for each slice, but neglects the horizontal balance of forces.

The vertical balance gives the following equation

$$\bar{N}\cos\alpha + \bar{T}\sin\alpha - W = 0 \quad (1.14)$$

We know that $\bar{T} = \frac{1}{FoS}[c'l + (\bar{N} - ul\tan\phi']$ (1.15)

When we substitute $\bar{T}$ by his value in equation 1.15, we obtain

$$\bar{N} = \left[W - \frac{1}{FoS}(c'l\sin\alpha - ul\tan\phi'\sin\alpha)\right]/m_\alpha \quad (1.16)$$

By substituting $\bar{N}$ by it value in the formula $FoS = \frac{\sum[c'l+(\bar{N}-ul)\tan\phi']}{\sum W \sin\alpha}$, we obtained

$$FoS = \frac{\sum[c'l\cos\alpha+(W-ul\cos\alpha)\tan\phi']/m_\alpha}{\sum W\sin\alpha} \quad (1.17)$$

Where $m_\alpha = \cos\alpha(1+\tan\alpha\frac{\tan\phi'}{FoS})$ (1.18)

We find that the safety factor FoS is in two members of the equation, and therefore the resolution is through iterative technics. Usually, the coefficient obtained with the Fellenius method is uesd as the starting point for this iteration. This method is more precise than the Fellenius method and the calculation is particulary well suited to computer prosessing (MASEKANYA, 2008)

iii. Morgenstern and Price method

Assumptions

- It assumes that the forces between slices are parallel (figure 1.9)
- It also assumes that the normal force N act at the center of the base of each slice.

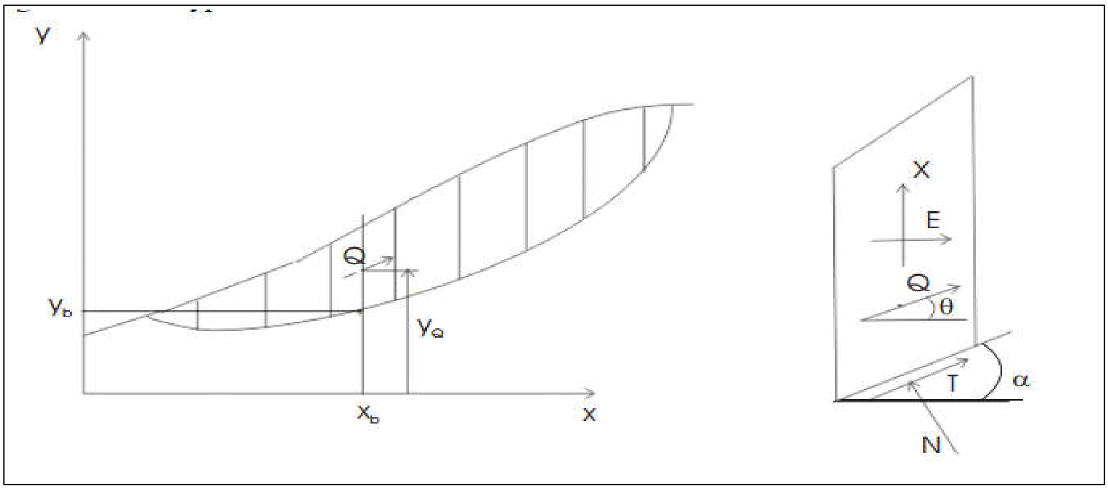

Figure 1.9. Coordinates of the sliding surface with Morgenstern and Price method **(Abderrahmane, 2020)**

This method checks the horizontal and vertical equilibrium of forces, equilibrium of moments at any point. It determines the inclination of the forces between the slices, which gives an additional unknown. This method is precise and can be applied to all geometries all soil types.

The common limitations of limit equilibrium methods are as follows:

- It is assumed that the safety factor is constant along the sliding surface
- The initial distribution of forces along the sliding surface is not explicitly considered
- For complex geometries, there may be a local minimum that remains undetected and complex sliding surface (non-circular) may be difficult to detect
- The calculation process is iterative and in some cases, convergence is difficult
- It is assumed that the soil is rigid and perfectly plastic, so they do not give any information about the movements

1.2.3.2 Other numerical methods

In the previous paragraphs, we have reviewed the different limit equilibrium (LE) methods for slope stability analysis. All these methods are based on an arbitrary selection of a series of sliding surfaces and the definition of the one that gives the minimum value of the coefficient safety. But for some time, we are witnessing the intensive use of numerical analysis methods giving access to stresses and deformations in the soil.

Zienkiewicz et al. (1975) introduced the strength reduction method (SRM) into finite element (FE) analysis. Variations of SRM were implemented in finite elements and finite difference (FD) codes resulting in an extensive comparison of continuum mechanics-based approaches (FE/FD) with LE.

Although LE/FD provides an alternative approach to LE, it has not wholly displaced LE as the major methodology in the design of slopes. Another approach, based on Limit Analysis (LA) were developed. It is more robust than LE as it does not require as many assumptions about formulation of static equilibrium. LA, applied with Discontinuity Layout Optimization (DLO), especially in the context of complex problems, determine the critical failure mechanism based on an assignment of evenly spaced nodes to the soil geometry. LimitState:GEO version 3.5 (LimitState Ltd., 2019) is a software that employs the DLO-LA technique (Leshchinsky, 2015).

1.3 Slope stabilization methods

The cause and nature of a slope failure should be understood before embarking on corrective action. Does the failure involve only the soil above the toe of the slope, or does it extend into the foundation? Was the failure caused by an excessively thick on a weak foundation, by an excessive steep slope, by a rise in the groundwater level; by the blockage of seepage paths, by erosion at the toe, creep and weathering? (Hai-Sui & Huang, 2018). Then, we can choose the appropriate slope stabilization method that ensures the stability of the slope without any risk of increasing the driving forces on the slope, is the least expensive and the most available on the market and must be simple in its realization.

1.3.1 Drainage

Slope failures are very often precipitated by a rise in the groundwater level and increased pore pressures. Therefore, lowering groundwater levels and reducing pore pressure is a logical means of improving stability. As a result, drainage is an often-used method, either alone or in conjunction with other methods. It improves slope stability in two important ways : (Hai-Sui & Huang, 2018)

- It reduces pore pressures within the soil, thereby increasing effective stress and shear strength
- It reduces the driving forces of water pressures in cracks, thereby reducing the shear stress required for equilibrium.

We distinguish many type of drainage:

1.3.1.1 Surface drainage

Preventing water from ponding on the ground surface, and directing surface flow away from the slide area, will help to reduce groundwater levels and pore pressures within the slide mass. Means to improve surface drainage include:

- Establishing lined or paved ditches and swales to convey water away from the site.
- Grading to eliminate low spots where water can pond
- Minimizing infiltration by covering the ground with plastic in short term or through the use of vegetation or paving in long term.

Covering the ground with plastic has some drawbacks. Undulations in the surface of the plastic tend to collect water and because individual sheet of plastic are sealed to each other, water can reach the ground at points of overlap between the sheets. Vegetation increases resistance to erosion by surface runoff and stabilizes the top foot of the soil at the surface of the slope. In long term, evapotranspiration helps to lower the groundwater level. Paving the surface of the slope promotes runoff and impedes infiltration (Duncan, Wright, & Brandon, 2014).

1.3.1.2 Subsurface drainage

The factor of safety against failure on any potential slip surface that passes below the phreatic line can be improved by surface drainage. Methods that can be used to accomplish subsurface drainage are:

- *Drain blankets* When there is a thin layer of poor quality saturated soil at a shallow depth, it may be practical to remove the poor quality layer and replace it with a well-draining soil fill. The bottom of the excavation should be covered with a filter stone layer with a performed pipe embedded in it to capture flow (Abramson, Lee, Sharma, & Boyce, 2002)
- *Trenches:* They should be constructed when subsurface water or soils of questionable strength are found at such great depths that stripping of the soils as discussed above is not practically feasible. They usually are excavated at the steepest stable side slopes for the construction period.
- *Cut-off drains:* at a site where shallow groundwater is encountered, cut-off drains can be used to intercept the groundwater flow. An impermeable zone is used as a cut-off downslope of the drain, and the top zone of the trench is backfilled with impermeable material. Runoff from the upper slope should be collected in drainages channels (Abramson, Lee, Sharma, & Boyce, 2002).
- *Horizontal drains:* Sometime called Hydrauger drains, are perforated pipes inserted in drilled holes in a slope to provide underground drainage. The drains pipes are commonly perforated or slotted PVC and are installed by drilling into the slope using a hollow-stem auger. They are usually installed from points of convenient access for the drill rig, often fanning out to achieve broad coverage of the area.

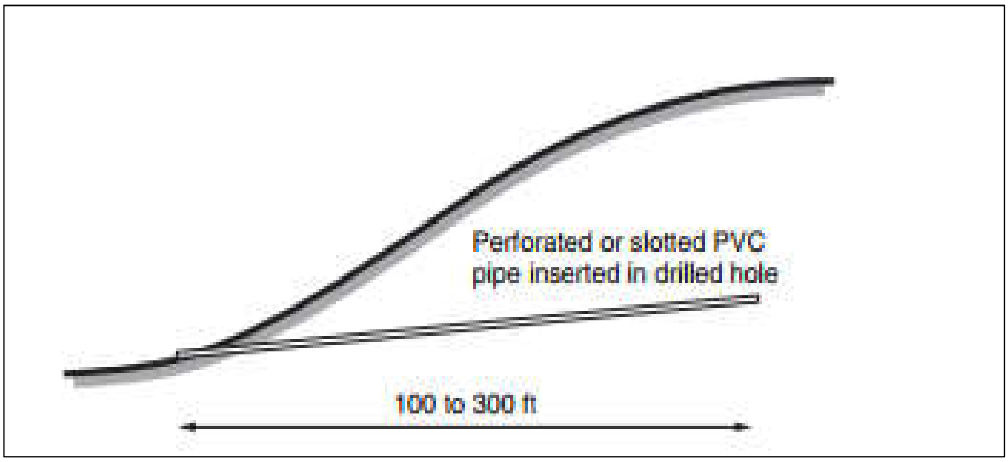

Figure 1.10. Horizontal Drain **(Duncan, Wright, & Brandon, 2014)**

1.3.2 Reinforcement

1.3.2.1 Soil nailing

Soil nailing is a method of in situ reinforcement utilizing passive inclusions that will be mobilized if movement occurs. It can be used to retain excavations and stabilize slopes by creating in situ, reinforced, soil retaining structures. The mains applications are shown in figure 1.11.

In soil nailed excavations, the reinforcement consists of steel bars, metal tubes, or other metals rods that resist tensile stresses, shear stresses and bending moments imposed by the slope movements. The nails generally are not prestressed and are relatively closely spaced. Corrosion protection should always be considered. Soil nailing is composed of nail, facing, connection nail and drainage system. (Abramson, Lee, Sharma, & Boyce, 2002)

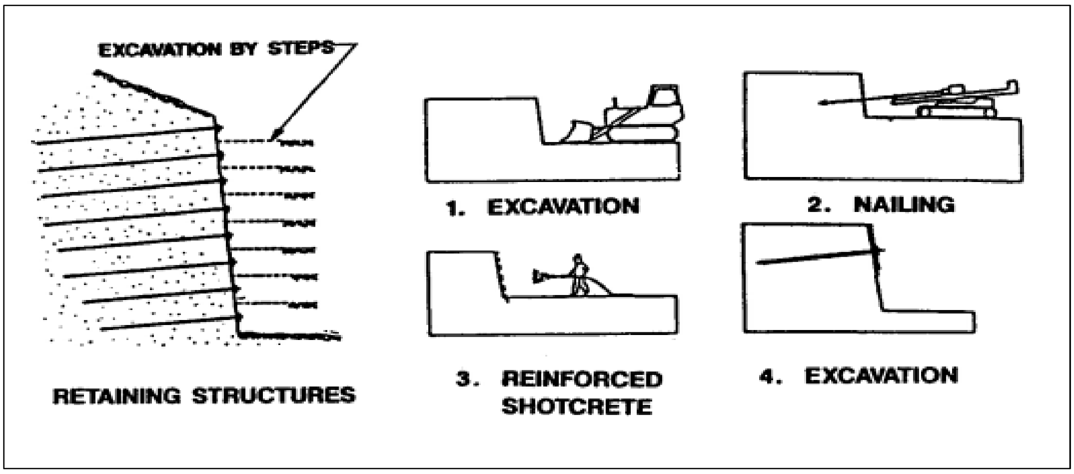

Figure 1.11. Main applications of soil nailing **(Abramson, Lee, Sharma, & Boyce, 2002)**

1.3.2.2 Stone columns

Stone columns can be used to stabilize or prevent landslides. This ground improvement technique increases the average shear resistance of the soil along a potential slip surface by replacing or displacing the in situ soil with a series of closely spaced, large-diameter columns of compacted stone. It also function as an efficient gravel drains by providing a path for the release of pore water pressures, thereby increasing the strength of the surrounding clayey soils. (Abramson, Lee, Sharma, & Boyce, 2002)

Construction of stone columns, shown in figure 1.12, consists of:

- Forming a vertical hole in the underlying material
- Placing a stone in the preformed hole from the ground surface or using a bottom surface
- Compacting the stone by penetration of each lift with the vibroflot, a process that drives the stone laterally to the sidewalls of the hole and thus enlarges the hole.

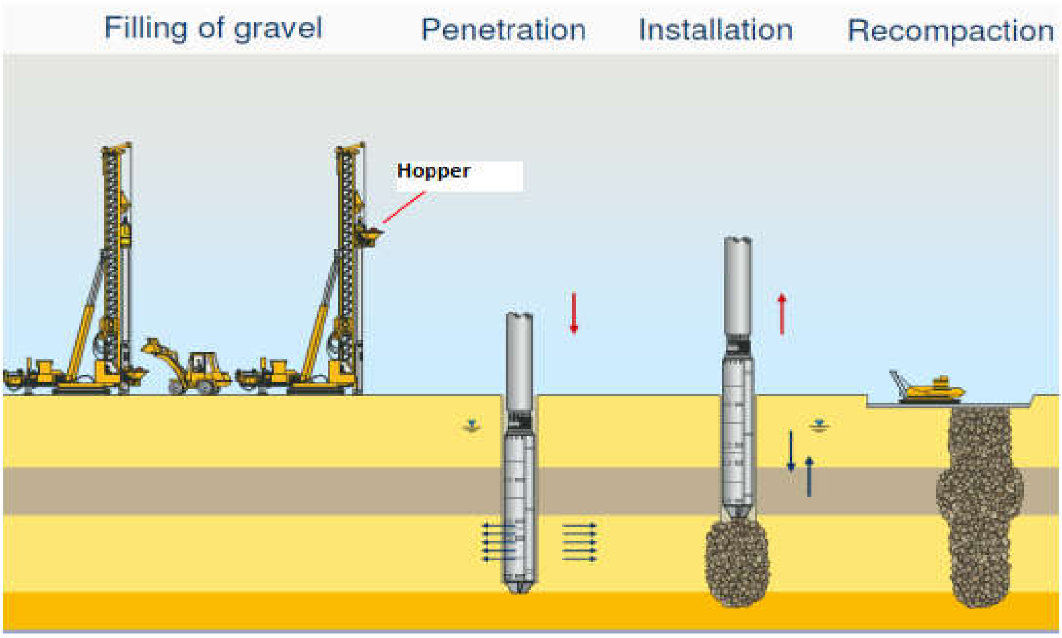

Figure 1.12. Stone column **(Brezzi, 2020)**

1.3.2.3 Geosynthetics

Geosynthetics are man-made materials, made from various types of polymers and used to enhance the environment, transportation and geotechnical engineering construction projects and make possible cost effective. They are used to provide one or more of the following functions like separation, reinforcement, filtration, drainage or liquid barrier.

They are grouped into the following:

- *Geotextiles:* they are permeable fabrics that, when used in association with soil, have the ability to separate, filter, reinforced, protect or drain. They are made from polypropylene or polyester as shown in figure 1.13
- *Geomembranes:* They are flexible, polymeric sheets that have a very low hydraulic conductivity and, consequently, are used as a liquid barrier (figure 1.13).
- *Geogrid:* geosynthetic material used to reinforce soil and similar material.
- *Geonet:* geosynthetic material consisting of integrally connected parallel sets of ribs overlying similar sets at various angles for in plane drainage of liquids.
- *Geocells:* Cellular confinement systems, widely used in construction for erosion control, soil stabilization on flat ground and steep slopes, channel protection.

Figure 1.13. Examples of geosynthetics **(Brezzi, 2020)**

1.3.3 Use of grasses

Grasses on slopes provides protection against erosion and shallow sliding as shown in figure 1.14. Roots reinforce or blind the soil and provide cohesion that improves stability against shallow sliding. In addition, plant roots are believed to reduce pore pressures within slopes by intercepting rainfall (reducing infiltration) and by evapotranspiration. Gray and Sotir (1992) found that living plant material (brush), embedded in horizontal layers at the surface of slopes, provided some reinforcement immediately, and more as the plants began to grow and put out new roots. They suggested that these brush layers also improve stability by intercepting water flowing within the slope and diverting it to the surface, reducing pore pressures in the process (Duncan, Wright, & Brandon, 2014).

Figure 1.14. Slope stabilization with vegetation **(Portar, 2008)**

1.3.4 Retaining wall

The most common use of retaining wall for slope stabilization is when a cut or fill is required and there is not sufficient space available for just the slope itself. The wall should be deep enough so that the critical slip surface passes around it with an adequate factor of safety. In addition, the ability of the retaining wall to perform as a stabilizing mass is a function of how well it will resist overturning moments, sliding forces at or below the base and internal shear forces and the bearing capacity of the soil. There are many type of retaining wall

1.3.4.1 Gravity wall

Gravity wall are the earliest known retaining structures. They are built from solid concrete or rock rubble mortared together (figure 1.15). The lateral forces from backfill is being resisted by the weight of the wall itself, and due to their massive nature, they develop little or no tension. Therefore, they are usually not reinforced with steel (Portar, 2008).

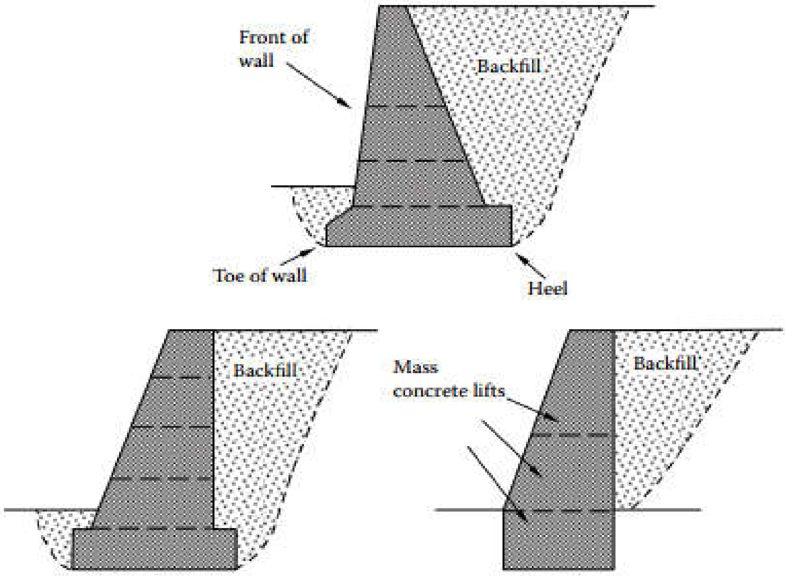

Figure 1.15. Cross section of a gravity wall **(Clayton, Woods, Bond, & Militisky, 2013)**

1.3.4.2 Cantilever wall

Cantilever walls are built of reinforced concrete and are typically composed of horizontal footing and a vertical stem wall (figure 1.16). The weight of the soil mass above the heel helps keep the wall stable.

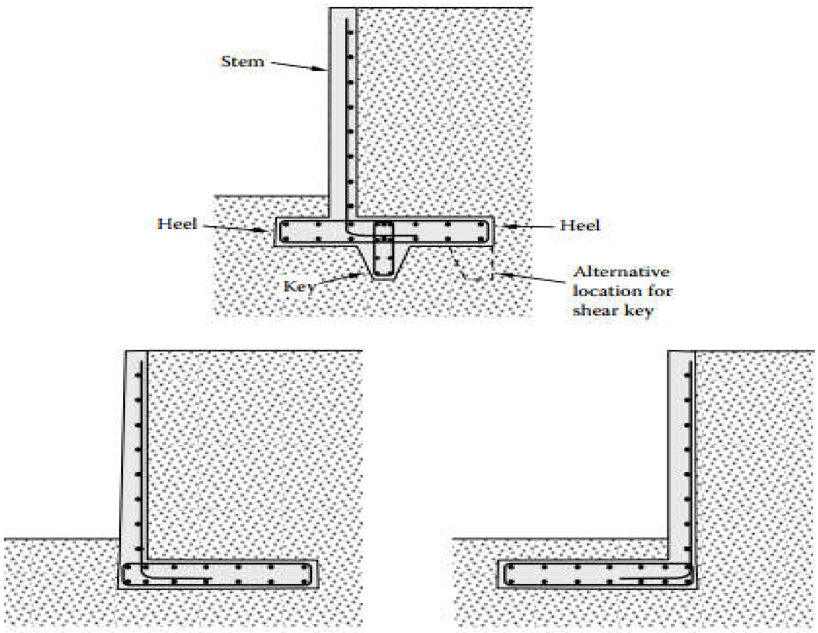

Figure 1.16. Cantilever Wall **(Clayton, Woods, Bond, & Militisky, 2013)**

1.3.4.3 Gabions wall

Gabions are multi-celled wire or rectangular wire mesh boxes, which are then rock-filled, and used for the construction of erosion control structures and to stabilize steep slopes. The advantage of this system is its flexibility; only the mesh needs to be transported to the site, and local labor and materials can be used to complete the structure. Their applications include retaining wall, bridge abutments, wing wall, culvert headwall, outlet aprons.

Conclusion

This chapter aimed at understanding Soil, slope stability analysis and identifying the different methods of slope stabilization. It thus emerges that several methods of slope stability analysis and slope stabilization techniques exist and the cause and nature of a slope failure should be understood before embarking on corrective action. To facilitate the resolution of this problem, several numerical approaches have been developed (LE, FE, FD, LA, DLO). Since our slope stabilization study will be done with a gabion wall, it would be important to understand in detail the use of gabions in order to better apply it.

Chapter 2. GABIONS WALL

Introduction

Gabions is an Italian word that means big cage. When filled with rock or stones at the project site, they form a flexible, permeable, monolithic structures called gabions retaining walls. They are used for the construction of erosion control structures and to stabilize steep slopes. What are the requirements in the construction of gabions wall? In this chapter, we will focus firstly on gabions (use, standards specifications of gabions), secondly on the design of gabion retaining wall, thirdly on some advantages and disadvantages of gabions wall and finally on the failures of gabions.

2.1 Gabions

2.1.1 Shape of Gabions

Gabion walls consist of steel wire baskets filled with rocks and stacked as units to form gravity retaining walls. Figure 2.1 shows some forms of gabions.

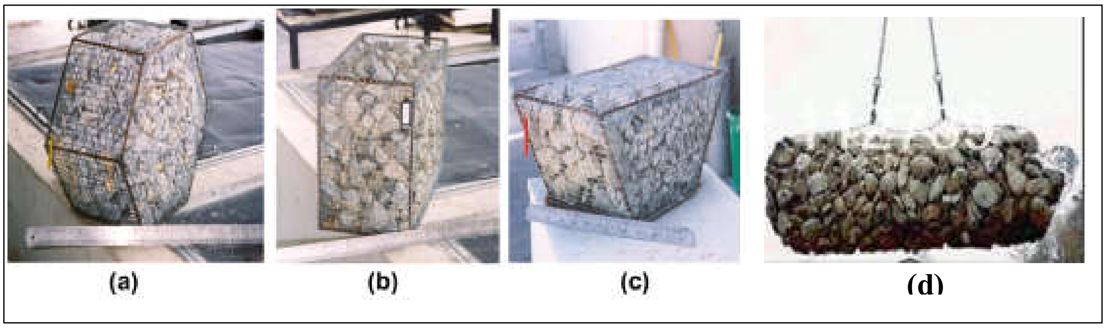

Figure 2.1. Shape of gabions (a) Hexagonal (b) rectangular (c) semi hexagonal (d) cylindrical **(Ramli, Karasu, & Thanon, 2013)**

2.1.2 Types of gabions and their use

The types of gabion to be used in the field determine the requirements of the process in the construction field. It exists many types of gabions commonly used today.

2.1.2.1 Gabion baskets

Net wire mesh is usually produced in box-shaped and in different sizes. Gabion baskets are frequently used in highway, railway works. Providing the filling material from a quarry close to the worksite is an economical alternative. Figure 2.2 shows an example of gabion basket. It can reach a natural green appearance as the plantation between the rocks spread along with the basket.

Figure 2.2. Gabion basket **(Toprak, Sevim, & Kalkan, 2016)**

2.1.2.2 Gabion mattresses

Gabion mattresses are generally used for riverbank, spillway, channel, slope protection or anti-scour protection. They have generally short height in comparison with the width and the length dimensions. Figure 2.3 shows channel coating for preventing erosion with gabion mattresses.

Figure 2.3. Channel coating for preventing erosion with gabion mattresses **(Toprak, Sevim, & Kalkan, 2016)**

2.1.2.3 Gabion sacks

This type of gabions is usually used in hydraulic works in emergencies and in a practical way and they are formed quickly. Gabion sacks have a porous and flexible structure. Figure 2.4 shows how gabion sacks are used in hydraulic works.

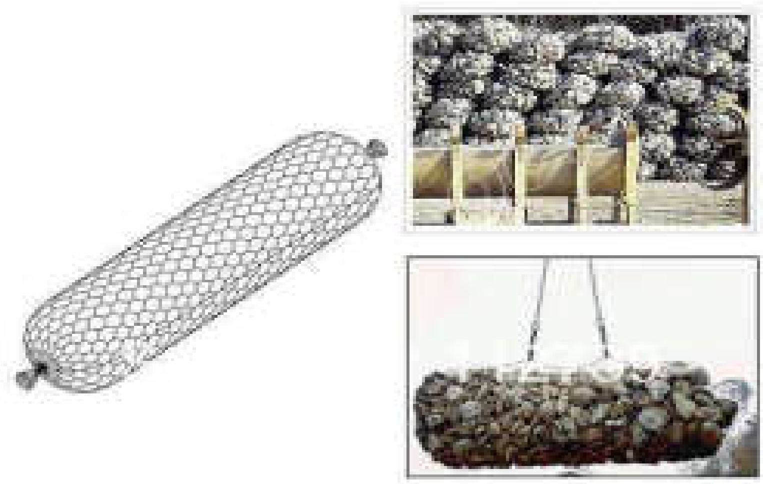

Figure 2.4. Gabion sacks in hydraulic works **(Toprak, Sevim, & Kalkan, 2016)**

2.1.2.4 Gabion wire mesh

Gabion wire mesh is an alternative to rigid retaining structures such as retaining walls. They are usually used for keeping the possible fall rocks and stones on the highway and railway surfaces and for directing the material to toe area to keep the stability of the slope close to the highway and railways. Gabion wire mesh reaches a natural green appearance as the plantation between rocks spread along with the wire mesh and used for anti-erosion to the slope.

Furthermore, a combination of the use on geogrid reinforcing with gabion in the soil embankment provides greater strength of embankment to support the gabion face wall. Figure 2.5 shows a gabion wire mesh in keeping the possible fall of rocks and stones and a combination of geogrid reinforcing with gabions.

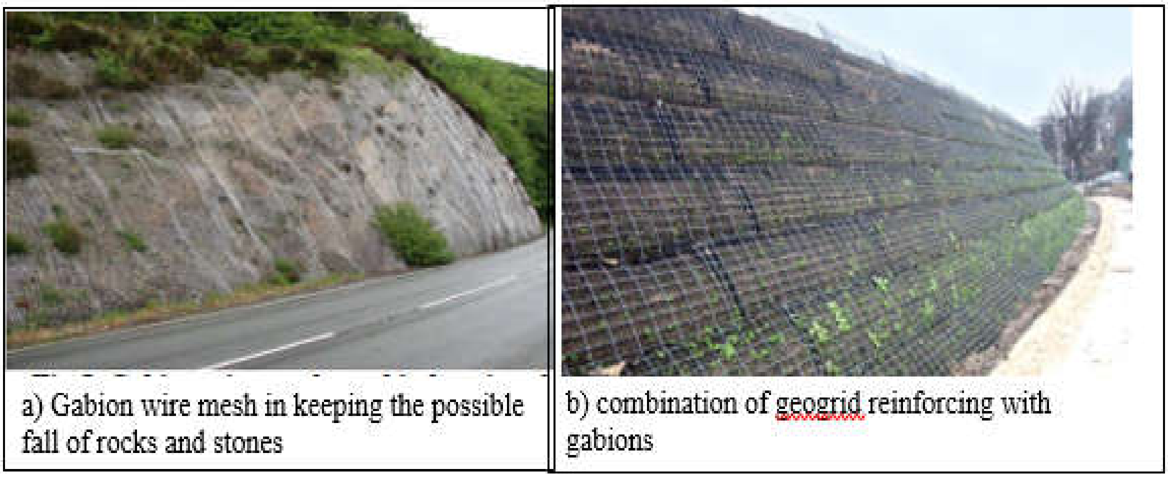

Figure 2.5. Gabion wire mesh **(Toprak, Sevim, & Kalkan, 2016)**

The strength and the stability of a gabion wall depends upon the materials which are used for its constructions. It should respect some standard specifications for gabions.

2.1.3 Standard specifications for gabions

These specifications cover gabions and revet mattresses produced from double-twisted coated wire mesh and welded wire mesh, stiffeners and fasteners used for manufacturing.

2.1.3.1 Mesh characteristics

a. Double twisted wire mesh

Referring from (ASTM A975-97, 2003), the double twisted wire mesh made by twisting continuous pairs of wires through three one half turns (commonly called double-twisted), which are then interconnected to adjacent wires to form hexagonal openings, as shown in figure 2.6

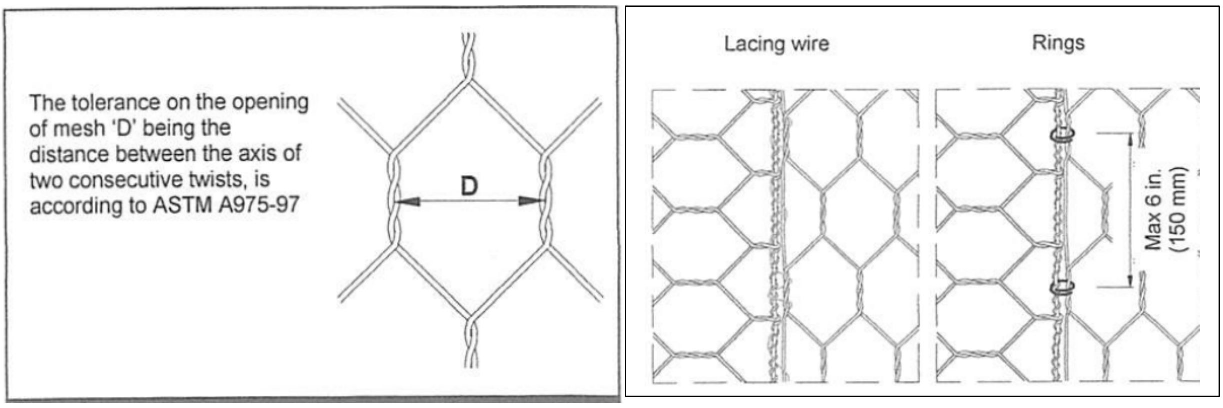

Figure 2.6. Double twisted mesh **(ASTM-A975-97)**

Double twisted wire mesh gabion are classified according to coating, which is applied before manufacturing the mesh. Coating styles are as follows:

i. Coating style

Style 1, consists of a double-twisted wire mesh made from wire which is zinc coated before being double twisted into a mesh. Fasteners, lacing wire and stiffeners are also produced from zinc-coated wire.

Style 2, consists of double-twisted wire mesh made from wire which is coated with

Zn-5Al-MM (zinc- 5% aluminium mischmetal alloy) before being double-twisted into a mesh. Fasteners, lacing wire, and stiffeners are also produced from Zn-5Al-MM coated wire.

Style 3, consists of a double-twisted mesh, lacing wire, and stiffeners as style 1 and overcoated with PVC. Fasteners shall be of stainless steel wire.

Style 4, consists of double-twisted mesh made from wire which is aluminum-coated before being double-twisted into a mesh. Fasteners, lacing wire, and stiffeners are also produced from an aluminum-coated wire.

ii. Physical and chemical properties

Metallic coating

The coating weight shall conform to the requirements of Specifications A641, Class 3 (style 1), for Zinc coating or Specification A856/A856M, Class 3(style 2), for Zn-5Al-MM coating, or Specifications A809 (style 4) for aluminum coating.

PVC for coating

The PVC coating shall show no cracks or breaks after the wires are twisted in the fabrication of the mesh. The initial properties of PVC coating material shall have a demonstrated ability to conform to the following requirements:

- *Specific gravity*. In the range from 1,3 to 1,35 when tested in accordance with test method ASTM D792
- *Tensile strength*. Not less than 20,6 MPa when tested in accordance with the test method ASTM D412

- *Modulus of Elasticity*. Not less than 18,6 MPa when tested in accordance with the test method ASTM D412
- *Hardness*. Between 50 and 60, when tested in accordance with the test method ASTM D2240
- *Brittleness Temperature*. Not higher than -9°C or lower temperature when specified by the purchaser, when tested in accordance with the test method ASTM D746
- *Resistance to Abrasion.* The percentage of the weight loss shall be less than 12%, when tested in accordance with the test method ASTM D1242
- *Salt Spray Exposure and Ultraviolet Light Exposure*.
  The PVC shall show no effect after 3000 hours of salt spray exposure in accordance with test method ASTM B117
  The PVC shall show no effect of exposure to ultraviolet light with test exposure of 3000 hours, using apparatus type E and 63°C, when tested in accordance with practice ASTM D1499 and G23
  After the salt spray test and exposure to ultraviolet light, the PVC coating shall not show cracks nor noticeable change of color, or blisters or splits. in addition, the specific gravity, tensile strength, hardness, and abrasion resistance shall not change more than 6%, 25%, 10% and 10% respectively, from their initial values.
- *Salt Spray Resistance for Fastener.* After testing the fasteners, the selvedge, or mesh wire confined by the fasteners shall show no rusty spots on any part of the surface excluding the cut ends.

The mesh thus formed gabions must respect the dimensions (see table 2.1) and the minimum resistance presented in table 1.2 according to the requirements of ASTM A975-97.

Table 2.1. Mesh characteristics **(ASTM-A975-97)**

Characteristics	Gabion Metallic Coated	Gabion PVC Coated	Revet Mattresses Metallic Coated	Revet Mattresses PVC Coated
Mesh Type	8 by 10		6 by 8	
Mesh Opening	83 by 114 mm [3.25 by 4.5 in.]	83 by 114 mm [3.25 by 4.5 in.]	64 by 83 mm [2.5 by 3.25 in.]	64 by 83 mm 2.5 by 3.25 in.]
Mesh wire	3.05 mm [0.120 in.]	2.7 mm [0.106 in.]	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]
Selvedge wire	3.8 mm [0.150 in.]	3.4 mm [0.134 in.]	2.7 mm [0.105 in.]	2.7 mm [0.105 in.]
Lacing wire	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]
Fasteners	3.0 mm [0.118 in.]	3.0 mm [0.118 in.]	3.0 mm [0.118 in.]	3.0 mm [0.118 in.]
Stiffeners:				
Using lacing wire	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]	2.2 mm [0.087 in.]
Preformed	3.8 mm [0.150 in.]	3.4 mm [0.134 in.]	N/A	N/A
PVC coating thickness:				
Nominal	N/A	0.50 mm [0.02 in.]	N/A	0.50 mm [0.02 in.]
Minimum	N/A	0.38 mm [0.015 in.]	N/A	0.38 mm [0.015 in.]

^All dimensions and wire diameters are minimum nominal values.

Table 2.2. Minimum Strength Requirements of Mesh and Connections **(ASTM-A975-97)**

Test Description	Gabions, Metallic Coated		Gabion, PVC Coated		Revet Mattress Metallic Coated and PVC Coated	
	kN/m	[lbf/ft]	kN/m	[lbf/ft]	kN/m	[lbf/ft]
Parallel to twist	51.1	[3500]	42.3	[2900]	33.6	[2300]
Perpendicular to twist	26.3	[1800]	20.4	[1400]	13.1	[900]
Connection to selvedges	20.4	[1400]	17.5	[1200]	10.2	[700]
Panel to panel connection using lacing wire or fasteners	20.4	[1400]	17.5	[1200]	10.2	[700]
	kN	[lbf]	kN	[lbf]	kN	[lbf]
Punch Test	26.7	[6000]	23.6	[5300]	17.8	[4000]

b. Welded wire mesh

Welded wires are classified according to wire coating styles as follows:

i. Coating style

Style 1, made from wire which is Zinc coated before being welded into the fabric. Spiral binders, lacing wire, and stiffeners are produced from Zinc-coated wire. It's normally recommended for temporary gabion structures, for works in non-aggressive or non-polluted environments.

Style 2, made from uncoated wire and the fabric is subsequently zinc-coated after fabrication. Spiral binders, lacing wire, and stiffeners are produced from the zinc-coated wire. It's normally recommended for permanent gabion structures, for works installed in non-aggressive or non-polluted environments, and this condition remains unchanged over time.

Style 3, made from wire which is coated with Zn-5Al-MM before being welded into the fabric. Spiral binders, lacing wire, and stiffeners are produced from Zn-5Al-MM coated wire. It's normally recommended for both permanent and temporary gabion structures.

Style 4, made from which is aluminum-coated before being welded into fabric. Spiral binders, lacing wire, and stiffeners are produced from aluminum-coated wire. It's very seldom used in the gabion industry. Its life expectancy shall be adequately documented to guarantee its consistency and reliability.

Style 5, welded wire fabric, spiral binders, lacing wire, and stiffeners as style 1,2,3,or 4 and overcoats with PVC. It's normally recommended for both permanent and temporary gabion structures, for works installed in aggressive or polluted environments, or when the aggressiveness of the site is moderately unpredictable or variable from low to high.

ii. Physical and chemical properties

Metallic coating

The coating weight shall conform to the requirements of Specifications A641, Class 3 (style 1), for Zinc coating or Specification A856/A856M, Class 3 (style 2), for Zn-5Al-MM coating, or Specifications A809 (style 4) for aluminum coating.

PVC for Coating (SPECIFICATIONS, 2020)

PVC coating shall show no cracks or breaks after the wires are twisted in the fabrication of the mesh. The initial properties of the PVC coating on the wire and welded wire fabric shall have a demonstrated ability to conform to the following requirements:

- *Adhesion.* The PVC test shall adhere to the wire such that the coating breaks rather than separates from the wire, in accordance with the test method ASTM A974
- *Mandrel Bend.* The PVC-coated wire subjected to a single 360 bend at -18°C around a mandrel ten times the diameter of the wire, shall not exhibit breaks or cracks in the PVC coating.
- *Specific gravity*. In the range from 1,20 to 1,40 when tested in accordance with test method ASTM D792
- *Tensile strength*. Not less than 15,7 MPa when tested in accordance with the test method ASTM D638
- *Modulus of Elasticity*. Not less than 13,7 MPa when tested in accordance with the test method ASTM D638
- *Hardness*. Not less than 75, when tested in accordance with the test method ASTM D2240
- *Brittleness Temperature*. Not higher than -9°C or lower temperature when specified by the purchaser, when tested in accordance with the test method ASTM D746
- *Resistance to Abrasion.* The percentage of the weight loss shall be less than 12%, when tested in accordance with the test method ASTM D1242
- *Salt Spray Exposure and Ultraviolet Light Exposure*. The PVC shall show no effect after 3000 hours of salt spray exposure in accordance with test method ASTM B117. The PVC shall show no effect of exposure to ultraviolet light with test exposure of 3000 hours, using apparatus Spectral Irradiance of Open Flame Carbon with daylight Filters and 63°C, when tested in accordance with practice ASTM D1499 and ASTM G152
- *Evaluation of coating after Salt Spray and Ultraviolet exposure.*
  After the salt spray test and exposure to ultraviolet light, the PVC coating shall not show cracks nor noticeable change of color, or blisters or splits. In addition, the

specific gravity, tensile strength, hardness, and abrasion resistance shall not change more than 6%, 25%, 10% and 10% respectively, from their initial values.

Table 2.3 shows us welded mesh characteristics

Table 2.3. Welded mesh characteristics **(ASTM-A975-97)**

Characteristic		Gabion: Metallic coated	Gabion: PVC coated	Revet mattress: Metallic coated	Revet mattress: PVC coated	Unit
Mesh type		8 x 10		6 x 8		cm
Mesh opening d x h		75x75	75x75	75x75	75 x75	mm
Mesh wire		3.05	2.7	2.2	2.2	
Selvedge wire		3.8	3.4	2.7	2.7	
Lacing wire, ϕ_W		2.2	2.2	2.2	2.2	
Fasteners		3.0	3.0	3.0	3.0	
Stiffeners:	Using lacing wire	2.2	2.2	2.2	2.2	
	Preformed	3.8	3.8	N/A	N/A	
PVC coating thickness	Nominal		0.50		0.50	
	minimum		0.38		0.38	

2.1.3.2 Stone fill

Stones should be hard, durable, angular and well graded with a minimum size of 1,5 times the aperture size and maximum size of one-half the height of the gabion baskets. Naturally, occurring rounded stone or quarried stones are acceptable. Gabion fill is normally a graded fill of between 100 to 200 mm in diameter. The more angular the fill, the better interlock and the less deformation of the face occurs. A quarried stone that is normally angular filled is preferable as the interlock is very good. Construction waste, debris from demolished structures, quarry rejects stones, and industrial waste has been used as gabion filler material. However, care has to be taken to check the durability, strength and chemical properties of the fill materials. Stones with high specific gravity are preferable since the gravity behavior of the structure is predominant. Table 2.4 presents some bulk densities for filling material of gabion.

Table 2.4. Typical bulk densities for filling, as a function of rock type (**Officine Maccaferri SpA, 1987)**

Type of rock used in gabion	Bulk density (kg/m^3)
Basalt	1740–2030
Granite	1560–1820
Hard limestone	1560–1820
Calcareous gravel	1500–1750
Sandstone	1380–1610

A gabion wall used as a retaining structure should be well design.

2.1.4 Gabions advantages and disadvantages

The mains advantages of gabions elements are:

Flexibility: Gabion elements constitute convenient solution for soils with a high settlement and swelling potential. Flexible gabion elements do not crack and are not affected by the earthquake such as gravity retaining walls (Toprak, Sevim, & Kalkan, 2016)

Permeability: Gabion elements do not require an additional drainage system because of the gaps between filling materials.

Economical: Shipping costs of gabion elements are lower due to the ease of packing. Assembling the net mesh does not require qualified labor, and therefore, the labor costs are low. Filling material can easily be provided from a quarry close to the worksite. Maintenance costs of gabion are extremely low. 30 – 50% savings could be obtained by going in for gabion faced gravity wall while the percentage goes up to 60 – 70% for gabion faced reinforced soil walls (Joshi, 2016).

Eco-friendly: Gabion elements are environmentally compatible. The gaps in the soil between filling materials allow the plantation to grow over time. Gabion elements are not affected by natural phenomena.

Aesthetical: In architecture, gabion elements are used for indoor and outdoor arrangements. Gabion elements have a natural outlook.

Gabion also reduces sound pollution by absorbing sounds up to 18 – 28dB and absorbs large vibrations and hence widely used near railway tracks.

However, gabion elements have also disadvantages:

- The wire baskets used for gabions may be subjected to heavy wear and tear to wire abrasion by bedload movement in streams with high-velocity flow
- It can be labor-intensive to fill large gabions by hand
- Gabion walls in river and sea erosion control need to be immediately inspected and evaluated after any storm, which has caused heavier than normal water flow.

2.1.5 Failures of gabions

From an investigation of existing sites, some defects were noted and were classified into five categories:

Bulging (figure 2.8): It can be due to the irregular shape of the stones that cause more voids loads to rearrangement of stones, loose packing of stones which happen when a single size stone is used or the use of small size stones that can split out from mesh opening.

Figure 2.7. Bulging of Gabion mesh **(Chikute & Sonar, 2019)**

Corrosion (figure 2.9): It due to the sharp edges of a stone that may cut gabion mesh wire. In presence of water, the cut mesh wire corrodes and causes a break of the wire.

Figure 2.8. Corrosion of Gabion mesh **(Chikute & Sonar, 2019)**

Erosion of stones (figure 2.10): it is due to soft stones or weathering prone stones that can easily crumble and lead to bulging.

Figure 2.9. Soft rocks fill **(Chikute & Sonar, 2019)**

Cracks in backfill (figure 2.11): It is due to rainfall and drainage. If the wall is located in the rainfall area, with no drainage system there is more possibility of cracks. It can also due to improper backfilling .

Figure 2.10. Crack in backfill **(Chikute & Sonar, 2019)**

Erosion of foundation soil): If the wall is located in the rainfall and mountainous area or near a stream, without foundation protecting blanket, the soil under the foundation gets washout and leads to the collapse of the wall.

Hence the importance of gabion maintenance.

2.1.6 Maintenance of gabions

Gabion should be inspected regularly and after every large storm event. All temporary and permanent erosion and sediment control practices shall be maintained and repaired as needed to assure the continued performance of their intended function. Many things can be done to avoid gabion's failures. To avoid bulging, the stones used in the gabion box should minimizes the voids and increases the self-weight of the gabion. To avoid corrosion of gabion wire, the stones should be placed properly (the shape edge must not be perpendicular) and properly select the wire mesh based on design and coating PVC/Zinc. To avoid erosion of soft stones, stones filled in gabion should be tested not only for strength, but also for weathering action. To avoid erosion of foundation soil, the wall should be properly embedded.

2.2 Design of Gabions Walls

The design begins with the selection of trial dimensions for a typical cross-section through the wall. Forces acting on the wall, sliding check, overturning check, bearing capacity pressure check are the main steps that must be followed to verify the stability of the wall.

2.2.1 Forces acting on the wall

The mains forces acting on the gabion wall are the vertical forces from the weight of the gabions and the lateral earth pressure acting on the back face as shown in figure 2.7. If other forces are encountered, such as vehicular loads or seismic loads, they must also be included in the analysis.

According to Coulomb theory, the total active force of the triangular pressure distribution acting on the wall is:

$$P_a = \frac{1}{2} K_a \gamma_s H^2 + K_a q H \qquad (2.1)$$

Where γ_s is the density of backfill soil

H is the wall height

K_a is the coefficient of active soil pressure

q is the uniformly distributed surcharge on the top of the backfill surface.

The coefficient of active pressure K_a is given by:

$$K_a = \frac{cos^2(\phi - \beta)}{cos^2(\beta)\cos(\delta+\beta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\alpha)}{\cos(\delta+\beta)\cos(\alpha-\beta)}}\right]^2} \qquad (2.2)$$

Where α is the slope of the backfill soil surface

β is the inclination angle of the face of gabion with respect to the vertical

δ is the angle of wall friction

ϕ is the angle of internal friction of backfill soil

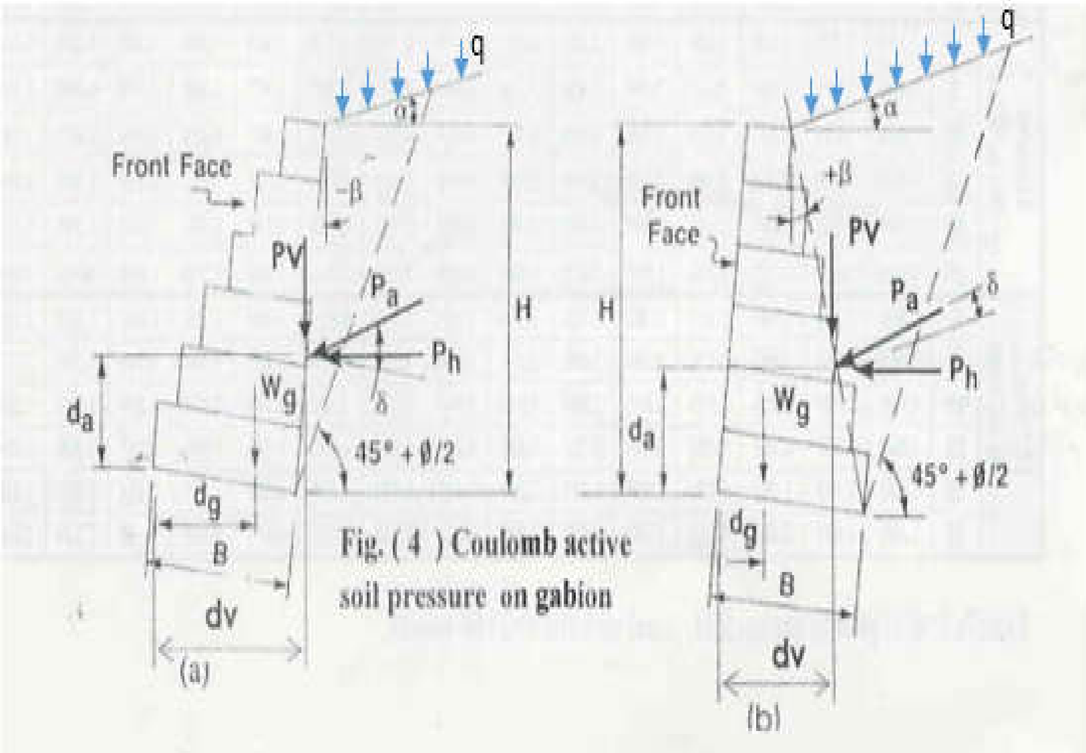

Figure 2.11. Forces acting on a gabion wall **(Mawlood, 2010)**

P_a is inclined to a line normal to the slope of the back face by the angle δ, where δ is usually taken as ϕ for Gabion walls. The horizontal and vertical components of P_a are showed below

$$P_h = P_a \cos(\delta + \beta) \tag{2.3}$$

$$P_v = P_a \sin(\delta + \beta) \tag{2.4}$$

2.2.2 Sliding check

The tendency of the active earth pressure to cause the wall to slide horizontally must be opposed by the frictional resistance at the base of the wall. This may be expressed as:

$$FoS_{slid} = \frac{F_r}{F_s} \tag{2.5}$$

Where F_r is the resisting force

F_s is the sliding force

FoS_{slid} is the safety factor against sliding. Usually, $FoS_{sli} \geq 1{,}5$

The resisting force is:

$$F_r = W_v * \tan\phi' + c'B \tag{2.6}$$

Where tan ϕ' is the coefficient of the sliding friction at the base of the gabion

W_v is the sum of the vertical forces $(W_g + P_v)$ with W_g the weight of the gabion wall

$$F_s = P_h \tag{2.7}$$

2.2.3 Overturning moment check

The active soil pressure forces tend to overturn the wall, and this must be properly balanced by the resisting moment developed from the weight of the wall and other forces. Using basics principles of statics, moments are taken about the toe of the wall to check overturn. This check is expressed as:

$$FoS_{over} = \frac{M_r}{M_o} \tag{2.8}$$

Where M_r is the resisting moment

M_o is the overturning moment

FoS_{over} is the safety factor against overturning. Usually, $FS_{over} \geq 2$

Each moment is obtained by summing the products of each appropriate force times its perpendicular distance to the toe of the wall. Neglecting wall friction, the active earth force acts normal to the back face at a distance $H/3$ above the base. When a surcharge is present, the distance of the total active force above the toe becomes

$$d_a = \frac{H(H+3q/\gamma_s)}{3(H+2q/\gamma_s)} + B\sin\beta \tag{2.9}$$

So, the overturning moment is

$$M_o = d_a * P_h \tag{2.10}$$

The weight of the gabion (W_g) acts vertically through the centroid of its cross section area. The horizontal distance to this point from the toe of the wall (d_g) may be obtained from the statically moment of wall areas. That is, moments of areas about the toe divided by the total area. The resisting moment is the sum of the products of vertical forces and their distance from the toe of the wall, so it's given by:

$$M_r = W_g d_g + P_v d_v \tag{2.11}$$

Where $d_v = B \sin\beta + d_a \sin\beta$ (2.12)

2.2.4 Bearing Pressure check

First check to determine if the vertical resultant of forces (W_v)lies within the middle third of the base. If B denotes the width of the base, the eccentricity (e) of W_v from the mid width of the base is

$$e = \frac{B}{2} - \frac{M_r - M_o}{M_v} \quad (2.13)$$

Where $W_v = W_g + P_v$

For the resultant force to lie in the middle third, check that $e \leq B/6$

Then the maximum pressure under the base σ_{max} is

$$\sigma_{max} = (W_v/B)(1 + 6e/B) \quad (2.14)$$

The maximum pressure must not exceed the allowable soil bearing pressure of base (q_{all}), $\sigma_{max} \leq q_{all}$. Finally, the factor of safety for bearing capacity(FoS_{bear}) is:

$$FoS_{bear} = \frac{q_{all}}{\sigma_{max}} \ . \quad (2.15)$$

Usually, $FoS_{bear} \geq 1$

2.3 Construction of gabion wall

It is done in three phases:

2.3.1 Preparing the ground

Building a gabion wall is simple, especially since it does not necessarily require a foundation like other retaining walls. We need to level our ground, then recover it with a geotextile film that will prevent grasses from invading our wall. Then, lay a layer of sand and gravel on the top of this film, which we spread with a rake, making sure that the soil obtained is straight with the help of a spirit level

2.3.2 Installation of the gabion

The gabion is a sort of wire cage that is filled with a material of our choice (often stones), to form a wall or a low wall. It is available in several panels in the shop. To form the cage, we just need to assemble these panels with a wire, staples or specific spirals. We can first install the bottom of the cage on the ground, then erect the other panels by securing them with the fastening systems supplied with the gabion.

2.3.3 Filling the gabion

Once the base of the gabion is installed on the ground and its structure is solidified, we can fill it with the material we have chosen.

Conclusion

In this part, it was a question of understanding what gabions wall are, the design method and their construction procedure. The use of Gabions walls as retaining structure is good method since it's ecological, flexible and economical. It is therefore important to respect the code when we are designing in order to minimize the different destructives failures that can occur it. Regular checks are important to ensure a correct performance of gabions in serviceability and ultimate conditions.

Chapter 3. METHODOLOGY

Introduction

The previous chapter enabled us to understand the uses of gabions walls, the standard and specifications for gabions and the different failures to which gabions wall could be subjected. This chapter will focus on the description of the methodology of work. The methodology is the part of study that establishes the research procedure after the definition of the problem, so as to achieve the set objectives. It consists of presenting the study site, data collection and the different equations used by analytical and numerical procedures. The modern software used to analyze the model is LimitState:Geo.

3.1 General recognition of the site

The site recognition passes through a documentary research aimed at defining the geographic location, the physical characteristics of the site (geology, relief and soil, climate, hydrology).

3.2 Data acquisition

The data collection consists of obtaining the different characteristics of materials through geotechnical tests (in-situ tests or laboratory tests). Geometric data and geotechnical data of the gabion retaining wall will enable its design by numerical and analytical methods later described in this chapter.

3.2.1 Geometric data

Geometric data of this project were obtained from a study that was already done by Michal Grodecki & Aleksander Urbanski on the same landslide. The data present the various sections of the gabion retaining wall.

3.2.2 Geotechnical data

Same as for geometric data, the geotechnical data were obtained from the study of Michal Grodecki & Aleksander Urbanski. The data present the physical, mechanical and hydraulic characteristics of the materials used in the construction of the gabion retaining wall.

3.3 Stability analysis

To achieve the objectives of this sudy, the software LimitState:GEO helped in designing and analyzing the gabion wall.

3.3.1 Generalities

LimitState was spun out from the University of Shelffield in 2006 to develop and market cutting-edge ultimate analysis and design software for engineering professionals. It comprises a suite of products, which are LimitState:RING, LimitState:GEO, and LimitState:YIELD, with applications in the structural, geotechnical and mechanical engineering sectors. LimitState:GEO will make the subject of our interest.

LimitState:GEO is a general-purpose software program that is designed to rapidly analyse the ultimate limit state (or collapse state) for a wide variety of geotechnical problems. LimitState:GEO utilizes Discontinuity Layout Optimization (DLO) to directly identify the critical collapse mechanism. The Discontinuity Layout Optimization procedure effectively relies on the familiar mechanism method of analysis originally pioneered several centuries ago by workers such as Coulomb (1776), but posed in a modified form to allow modern-day computational power to be applied to the problem of finding the critical solution from billions of possibilities. The solution is presented as an adequacy factor (applied to one or more loads or material strengths in the problem). The solution is also displayed visually as a failure mechanism involving several blocks that will slide and/or rotate relative to one another.

3.3.2 Discontinuity Layout Optimization (DLO)

Discontinuity Layout Optimization procedure was developed at the university of Sheffield and was first described in a paper published in the Proceedings of the Royal Society. It can be used to identify critical translational sliding block failure mechanisms for any geotechnical stability problem.

3.3.2.1 How does DLO works.

DLO involves the use of rigorous mathematical optimization techniques to identify a critical layout of lines of discontinuity that form at failure. These lines of discontinuity are typically 'slip-lines' in planar geotechnical stability problems. To allow a wide range of different failure mechanism to be identified, a large number of potential lines of discontinuity must be considered.

If there are n nodes for example, there are approximately $n(n-1)/2$ possible slip-lines and approximately $2^{n(n-1)/2}$ possible slip-line mechanism. An example involving the bearing capacity of a footing is given in figure 3.1. The thin black line indicates the set of potential discontinuities to be considered and these connect an initial set of nodes. The solution of which identifies the optimal subset of discontinuities that produce a compatible mechanism with the lowest energy dissipation (thick lines in Figure 3.10). the accuracy of the solution obtained depends on the prescribed nodal spacing.

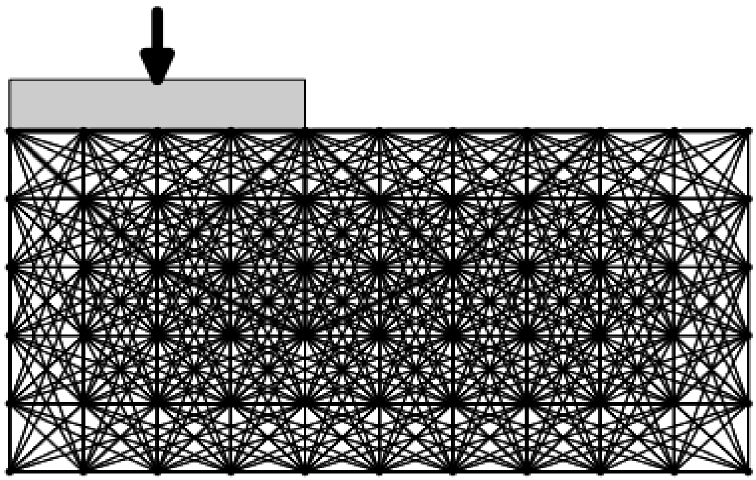

Figure 3.1 . DLO analysis of the undrained stability of a footing **(LimitState Ltd, 2019)**

3.3.2.2 Linear Programing formulation

The kinematic slip-line discontinuity layout optimization formulation for the plane strain analysis of a quasi-statically loaded, perfectly plastic cohesive body discretized using m connections (slip-lines discontinuities), n node and a single load case is defined as follows:

$$\min E = g^T d$$

subject to: $$Bd = u \quad \text{with} \quad d \geq 0 \tag{3.2}$$

where E is the total internal energy dissipated due to shearing along the discontinuities,

$g^T = \{c_1 l_1,\ c_1 l_1, c_2 l_2, c_2 l_2, \dots, c_m l_m\}$, c_i and l_i are respectively the length and associated cohesive shear strength of discontinuity i, B is a $2n \times 2m)$ matrix, $u^T = \{u_1^x, u_1^y, u_2^x, u_2^y, \dots,\ u_n^y\}$ where u_j^x and u_j^y are the x and y components of the displacement imposed at the node, $(j = 1 \dots n)$.

3.3.2.3 Rotational Failure Mechanisms

The current implementation of DLO in LimitState:GEO generates solutions based on translational mechanism. In order to model mechanisms involving rotation of structural elements such as cantilever walls, it is necessary to set the *Model Rotations* value in the project level *Property Editor* to Along Edges. This allows solids to rotate as a rigid bodies and to be modelled along Boundaries. This allows Solids to rotate as a rigid bodies and to transmit these rotations into translational deforming bodies by modelling localized rotational deformations along the boundary. This is an approximation to modeling rotational and translational failure everywhere. Figure 3.2 depicted a series of small rotational elements modeled along the length of the boundary.

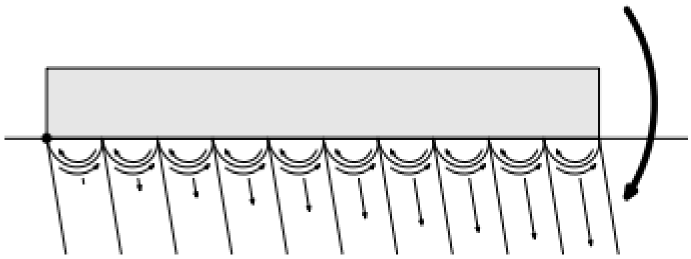

Figure 3.2. Modeling of rotational elements along a rotating boundary **(LimitState Ltd, 2019)**

In the following analysis, c and ϕ may represent either c' and ϕ' for drained parameters or cu for undrained parameters. From the geometry of the log-spiral (figure 3.3), the following expression can be obtained:

$$r_o = \frac{l}{(1+e^{\pi \tan \phi})} \tag{3.3}$$

Where l is the length of the segment along the boundary.

If the boundary rotates by an angle ω, the resultant effect is an equivalent rotation about the segment midpoint by ω, accompanied by an additional dilation:

$$n_\omega = ul\omega \tag{3.4}$$

Where $u = 0{,}5 - \frac{1}{(1+e^{\pi \tan \phi})}$ (3.5)

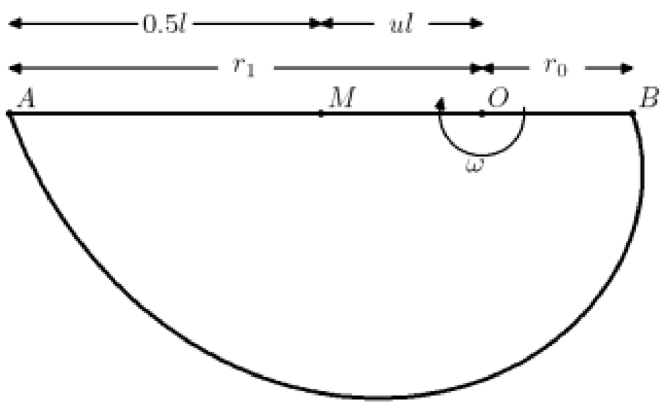

Figure 3.3. Geometry of log-spiral **(LimitState Ltd, 2019)**

Discontinuity Layout Optimization is a powerful new numerical procedure that shows considerable promise. In contrast with finite elements limit analysis, no mesh is required and the underlying formulation is simpler.

3.3.3 Analysis applications

LimitState:GEO can model almost any 2D problems of geometry including :

- ✓ Slope stability,
- ✓ Retaining walls,
- ✓ Embankment stability
- ✓ Foundations on heterogeneous soils,
- ✓ Pipelines,
- ✓ Tunnels,
- ✓ Reinforced earth walls
- ✓ Quay wall
- ✓ Propped excavation

3.3.4 Design methodology

There is a very large number of generic principles in LimiState:GEO for modeling any stability analysis problem. These are

- ✓ *Model definition and Solver*:
- ✓ *Adequacy Factor and Factors of safety*

- ✓ *Partial Factors*
- ✓ *Solution Accuracy*
- ✓ *Adapting Plane Strain Results to 3D*

3.3.4.1 Model definition and Solver

iv. Model definition

Problem geometries are built up using Geometry objects. The two key geometry objects relevant to the model definition are:

- Solid, which is a 2D polygon defining a body of soil or other material.
- Boundary, which is a straight line that defines the edge or boundary of a Solid, or an interface between two Solids.

Generally, the problem will be defined in terms of Solid objects. Boundary objects are automatically generated around Solid objects. Boundary objects are also used to define interface properties and set boundary conditions.

v. Solver Specification

The specifications of the LimitState:GEO solver is as follows:

- The software is designed to generate an optimal layout of slip-lines that make the critical or failure translational sliding block mechanism
- The slip-lines are restricted to those that connect any two nodes within a predefined grid
- Slip-lines are restricted to those that connect nodes within a single solid object
- The solution is given the form of an Adequacy factor

3.3.4.2 Adequacy Factor and Factors of safety

Many different definitions of factors of safety are used in geotechnical engineering. Three common usage are:

- Factor on load
- Factor on material strength

- Factor defined as ratio of resisting forces (or moments) to disturbing forces (or moments)

The calculation process used to determine each of these factors for any given problem will in general result in a different failure mechanism. There are three ways to drive a system to ultimate limit state corresponding to the factor of safety definitions previously mentioned:

- Increasing an existing load in the system
- Reducing the soil strength
- Imposing an additional load in the system.

The Adequacy factor is defined as the factor by which specified loads must be increased, or materials strengths decreased, for the system to reach a collapse state. There are two type of adequacy factor:

- Adequacy factor on load
- Adequacy factor on soil strength

In general, if:

Adequacy factor ≥ 1.0 the problem is safe against collapse

Adequacy factor < 1.0 the problem is unsafe against collapse

3.3.4.3 Partial Factors

LimitState:GEO is designed to work closely with Eurocode 7 approach to Ultimate Limit State design. It has therefore adopted the Eurocode 7 definitions of actions and partials factors. The LimitState:GEO partial factoring system is designed to facilitate input factoring which is typical for material factoring approaches such as Eurocode 7 Design Approach 1 combination 2 and Design approach 3, and adopted for action factoring approaches such Eurocode7 Design approach 1 combination 1. Action and resistance factoring approaches such as Eurocode 7 Design approach 2 will generally adopted output factoring.

In LimitState:GEO , three types of loads are available:

- Permanent
- Variable

- Accidental

The relevance of each action is the nature of the partial factor to be applied to it, with the corresponding value taken from the scenario Manager.

Eurocode 7 also requires that each action is assessed as to its effect on the overall stability calculation. If it contributes to stability then it is Favorable, if it contributes to collapse then it is Unfavorable. The loading type affects the value of the partial factor to be applied to it. The following loading type are available in LimitState: GEO:

- *Favorable*: apply the favorable partial factor to any loads on a boundary or to the self-weight of the materials within a solid
- *Unfavorable*: apply the unfavorable partial factor to any loads on a boundary or to the self-weight of the materials within a solid
- *Neutral:* do not apply any partial factor to any loads on a boundary or to the self-weight of the materials within a solid

3.3.4.4 Anchor reinforcement

In LimitState:GEO, soil reinforcement is typically modelled using the Engineered Element material. The parameters defining the behavior of an Engineered Element are:

- *Pullout factor*, T_c: the contribution of the material cohesion to the resistance of the element
- *Pullout factor,* T_q : the contribution of the overburden to the pullout resistance of the element
- *Lateral factor,* N_c *:* the contribution of the material cohesion to the lateral resistance of the element
- *Lateral factor* N_q : the contribution of the overburden to the lateral resistance of the element
- *Plastic moment* M_p : the plastic moment of resistance of the element. (If rigid joints are modelled then this parameter should be set to 1E+30)
- *Rupture strength* R : the maximum tensile force that can be carried by the material
- *Compression strength, C:* the maximum compressive force that can be carried by the material

For soil anchor, the pullout properties T_c, T_q, N_c and N_q (which are defined by user) are ideally determined by field tests. However estimates using theory may be made. For example if an anchor of diameter D is embedded in the soil of cohesion c', friction angle ϕ', interaction coefficient α, horizontal anchor spacing n, the following is obtained:

The pullout factor $$T_c = \frac{(\pi D)\alpha c'}{n} \quad (3.6)$$

The lateral factor $$T_q = \frac{(\pi D)\alpha \tan\phi'}{n} \quad (3.7)$$

When an anchor passed through m soil layers, the cohesion c' is given by:

$$c' = \frac{c'_1 L_1 + c'_2 L_2 + c'_3 L_3 + \cdots + c'_m L_m}{L_1 + L_2 + L_3 + \cdots + L_m} \quad (3.8)$$

where c'_m is the cohesion of the soil layer n and L_m is the length of the anchor in soil layer m.

In the same order, the friction angle ϕ' is given by

$$\tan\phi' = \frac{\tan\phi'_1 L_1 + \tan\phi'_2 L_2 + \tan\phi'_3 L_3 + \cdots + \tan\phi'_m L_m}{L_1 + L_2 + L_3 + \cdots + L_m} \quad (3.9)$$

where ϕ'_m is the friction angle of the soil layer m

A soil anchor can be:

- *Rigid :* the plastic moment, the rupture and the compression strength are set to be effectively infinite (1E+30kN/m)
- *Flexible:* behaves in the same way as the rigid soil nail except that the plastic moment capacity is set to be zero
- *Can yield:* behaves in the same way as the flexible soil nail except that finite value can be set for rupture and compression strengths

If required, tensile failure of an anchor can be represented using Rupture Strength, bending failure can also be specified using plastic moment. Conventionally, laterals resistances N_c and N_q of the nail are neglected. However, in practice it is preferable to set N_c to a small nominal value (e.g. 1kN/ m^2).

3.4 Eurocode 7 Design Approaches

Eurocode 7 (EC7) help the engineer to design building and other things, with a special focus on structural and geotechnical aspects. In general, the design defines the coefficients by which the strength of the materials has to reduce, the coefficients by which the intensity of the external forces and loads has to be increased and the varieties of external loads needed to be considered. In general, the verifications of the stability of a retaining wall are:

- Overturning (EQU in EC7)
- Sliding (GEO in EC7)
- Bearing capacity (GEO in EC7)
- Global stability (GEO in EC7)

The aim is to verify the relation

$$E_d \leq R_d \tag{3.9}$$

Where E_d is the design value for the action effect and R_d the design value for the resistance.

The previous relation must be verified with different possible combination of partial safety factors:

- For Actions (loads): A_1 and A_2
- For materials characteristics: M_1 and M_2
- For system resistance: R_1, R_2, and R_3

Ultimate Limit State verifications are carried out with three possible Design Approaches

- *Design Approach 1*: it has two combinations which are combination 1(A_1 "+" M_1 "+" R_1) and combination 2 (A_2 "+" M_2 "+" R_1).
- *Design Approach 2*: A_1 "+" M_1 "+" R_2
- *Design Approach 3*: (A_1 or A_2) "+" M_2 "+" R_3

where "+" implies "to be combined with".

Table 3.1 and table 3.2 give us the different partial factors on Actions and Soil parameters for equilibrium (EQU) limit state. table 3.4 illustrate the values of the different partial factors to be used, depending on the Design Approach chosen.

Table 3.1. Partial Factors on actions for EQU limit state **(Eurocode7, 1997-1:2004)**

Action	Symbol	Value
Permanent		
Unfavourable[a]	$\gamma_{G;dst}$	1,1
Favourable[b]	$\gamma_{G;stb}$	0,9
Variable		
Unfavourable[a]	$\gamma_{Q;dst}$	1,5
Favourable[b]	$\gamma_{Q;stb}$	0
[a] Destabilising		
[b] Stabilising		

Table 3.2. Partial Factors on soil parameters for EQU limit state **(Eurocode7, 1997-1:2004)**

Soil parameter	Symbol	Value
Angle of shearing resistance[a]	$\gamma_{\varphi'}$	1,25
Effective cohesion	$\gamma_{c'}$	1,25
Undrained shear strength	γ_{cu}	1,4
Unconfined strength	γ_{qu}	1,4
Weight density	γ_{γ}	1,0

Table 3.3. Partial factors on actions for GEO limit state **(Eurocode7, 1997-1:2004)**

Action		Symbol	Set	
			A1	*A2*
Permanent	Unfavourable	γ_G	1,35	1,0
	Favourable		1,0	1,0
Variable	Unfavourable	γ_Q	1,5	1,3
	Favourable		0	0

Table 3.4. Partial Factors on soil parameters for GEO limit state **(Eurocode7, 1997-1:2004)**

Soil parameter	Symbol	Set	
		M1	*M2*
Angle of shearing resistance[a]	$\gamma_{\varphi'}$	1,0	1,25
Effective cohesion	$\gamma_{c'}$	1,0	1,25
Undrained shear strength	γ_{cu}	1,0	1,4
Unconfined strength	γ_{qu}	1,0	1,4
Weight density	γ_{γ}	1,0	1,0

Table 3.5. Partial resistance factors for retaining structures **(Eurocode7, 1997-1:2004)**

Resistance	Symbol	Set		
		R1	*R2*	*R3*
Bearing capacity	$\gamma_{R;v}$	1,0	1,4	1,0
Sliding resistance	$\gamma_{R;h}$	1,0	1,1	1,0
Earth resistance	$\gamma_{R;e}$	1,0	1,4	1,0

The stability verifications of our gabion retaining wall will be done through the Design Approach 1-combination 2.

.

Conclusion

In this chapter**,** the methodology of this work was discussed. A general description of the site was done through a documentary research, then the data acquisition which was obtained from the study of Michal Grodecki & Aleksander Urbanski. Lastly, a presentation was done on the numerical software LimiState:GEO. LimitState:GEO , utilizing the Discontinuity Layout Optimization procedure, help to directly identify the critical collapse mechanism. The next chapter deals with the design procedure to a practical case, follow by a presentation and interpretation of the results obtained.

Chapter 4. NUMERICAL MODEL OF A GABION WALL AND INTERPRETATION OF RESULTS

Introduction

The study of the stability of a slope is an important aspect of geotechnical engineering. So a numerical software is useful to model a real situation and to have good results. A slope is stabilized through a gabion retaining wall and the stability analysis will be performed using LimitState:Geo software. In this chapter, a presentation of the project is firstly done, followed by the presentation of the design parameters and finally numerical analysis of our model with presentation and interpretation of results.

4.1 Description of the site

4.1.1 Geographic location

Nowy Sacz is located in southern Poland, and is bounded by Nowy Targ County and Limanowa to the west, Brzesko Gorlice County and tarnów to the north and Gorlice county to the east as shown in figure 3.1. Geographycally, it is at latitude 49°37’26’’N and longitude 20°41’50’’E. The study area is located in a sub-mountain, in Nowy Sacz city.

Figure 4.1. Localization of Nowy Sacz **(Google map)**

4.1.2 Geology, relief and soil

The geological basis of Nowy Sacz is Carpathian flysch (an undifferentiated grey-banded sandstone) with alluvial sediment (Miocene, biogonice, zembrzce beds,…) from the Dunajec, Poprad and Kamienica rivers in the valley basin. Figure 3.2 illustrates in detail the geological composition of Nowy Sacz city.

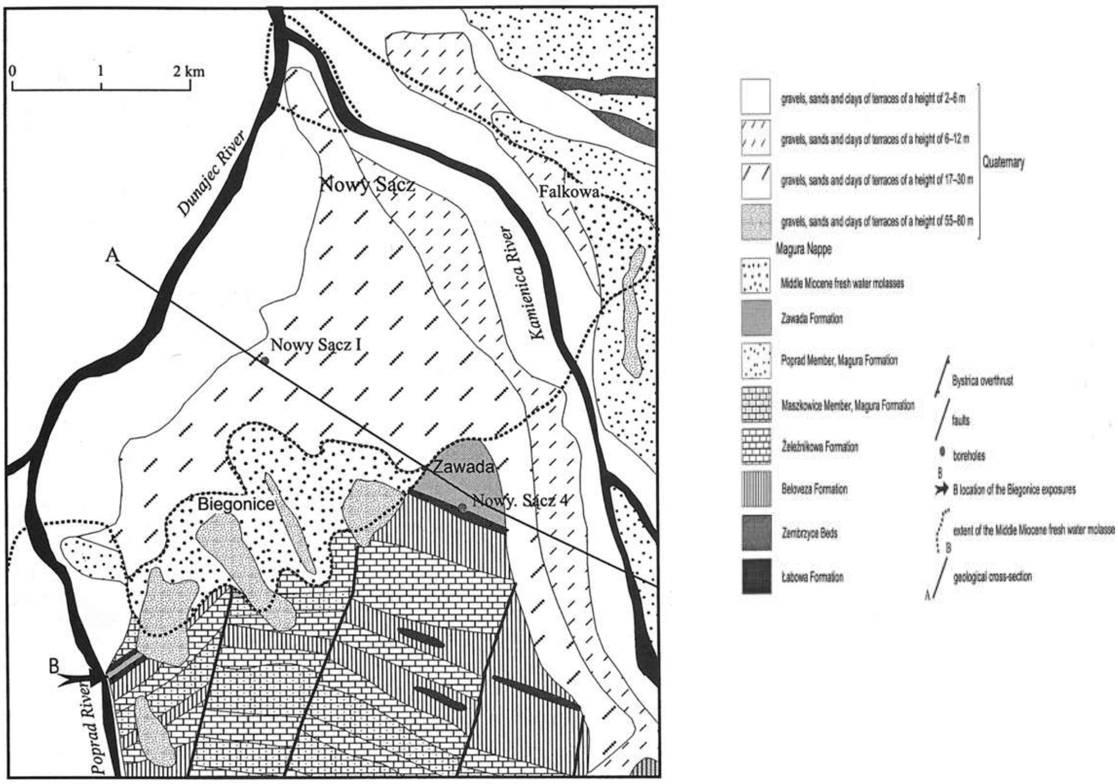

Figure 4.2. Geological map of the Nowy Sacz **(OSCZYPKO & OSCZYPKO-CLOWES, 2002)**

4.1.3 Climate

The climate is temperate, with an average rainfall annual of about 700 millimeters. The temperature oscillates between -6.6 °C and 22.5 °C. From the historical climate data, the months of April, May, June, July, August and September have a high chance of precipitation. The coldest season is the months of January and February and the warmest month is July. Table 3.1 summarizes temperatures, precipitations, humidity and rainy days between 1998 and 2018.

Table 4.1. Nowy Sacz weather by month Climate features for 1998-2018 **(en.climate-data.org)**

	January	February	March	April	May	June	July	August	September	October	November	December
Average temperature (°C)	-3.2	-1.8	2.4	8.5	13.1	16.6	18.3	18	13.3	8.2	3.7	-1.1
Minimum temperature (°C)	-6.6	-5.5	-2	3.1	7.9	11.7	13.5	13.1	9	4.5	0.7	-4
Maximum temperature (°C)	0.1	1.9	6.7	13.3	17.7	20.7	22.5	22.4	17.6	12.4	6.9	1.7
Precipitations/Rainfall (mm)	52	53	58	78	110	112	134	94	83	66	56	52
Humidity (%)	83	81	75	69	73	74	76	75	78	82	85	83
Rainy days (d)	9	9	10	10	12	11	12	10	9	8	8	9

4.1.4 Hydrology

Nowy Sacz is watered by many streams of varying importance. The most important are Dunajec, Poprad and Kamienica rivers. The Dunajec river runs through northeasthern Slovakia and southern Poland. It is a right tributary of the Vistula river and begins in Nowy Targ. It has a length of 249 kilometers. Kamienica and Poprad rivers are tributary of the Dunajec river, respectively, in the Poprad valley basin and near Stary Sacz .

4.1.5 Population and economic activities

With an area of about 57km^2, Nowy Saczis the third most populous city in Malopolska province with a population of around 83,896 inhabitants (at 2021 census). This city is important in the food industry, specializing in processing fruits (apples). Its economy also has an extensive network of services to business that provide a faster flow of information and better accessibility to high quality services.

4.2 Presentation of the project

The landslide which is analyzed is located in sub-mountain area of southern Poland, in the city of Nowy Sacz. It is located at the lower parts of slopes of a V-valley. At the valley floor a stream runs, which normally carries minimal flow (less than 0,5 cubic meters/sec), but it temporarily grows up to several cubic meters per second. Exact hydrological data were not available; however, evidence of such events in the past was visible, along the bank of the stream. Erosion related to this was the most probable cause of landslide activation, endangering the municipal road crossing it. Signs of massive movements were observed, both as the deterioration of the road surface and terrain deterioration. To protect the road, and fix the problem of stream bank strengthening, a gabion retaining wall is proposed, since the height of the slope beneath the road reaches 7.5 m. Ecological and landscape preservation issues were taken into account.

The whole landslide has a length of about 300m, but the length to be protected by the gabion wall is 60m. The remaining parts of the road crossing the landslide were protected by other structural means. According to existing geotechnical evidence, the existing slope is built mostly from weak clays. Compressive rocks are found at about 4m the terrain level. Figure 3.3 gives us a situation of the landslide area.

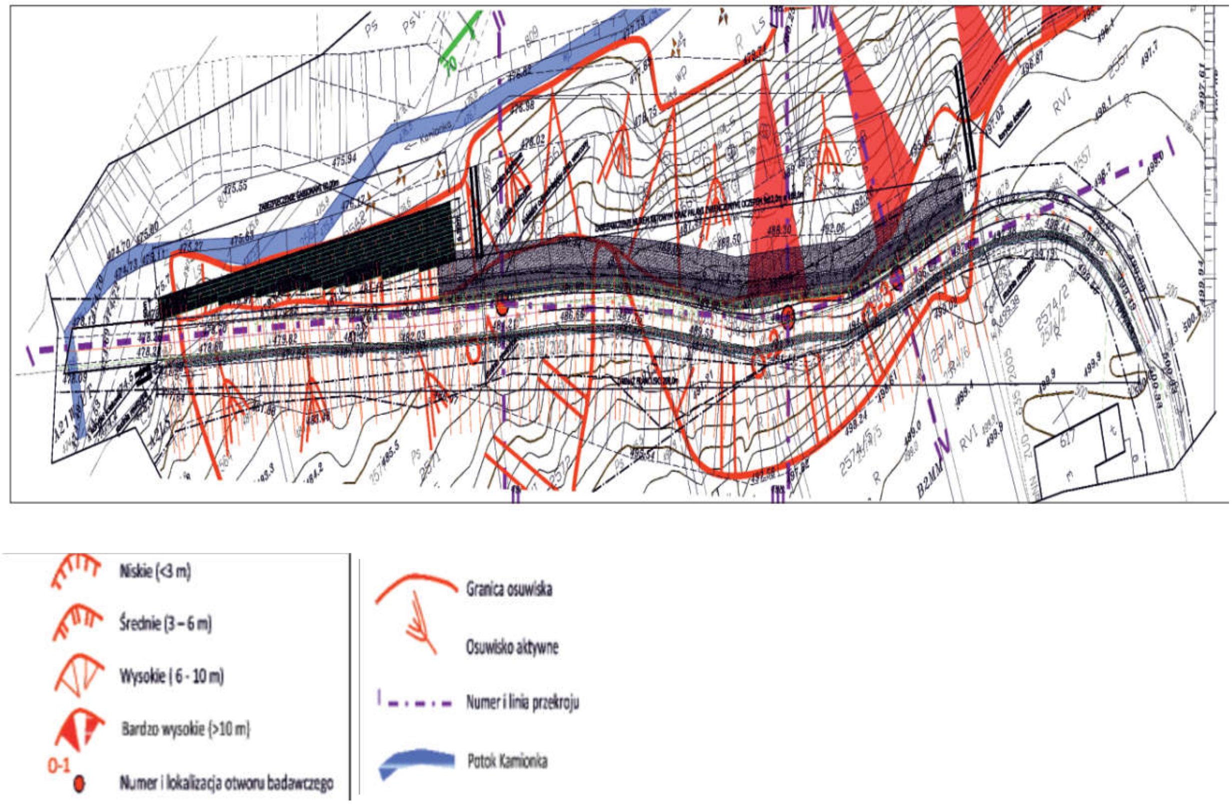

Figure 4.3. Situation of the landslide area (gabion wall at the left, market dark) **(Grodecki & Urbanski, 2018)**

4.3 Design parameters

4.3.1 Soil properties

Soil properties are obtained from tests that were done by Michal Grodecki & Aleksander Urbanski during their study on the same landslide. We noted that the existing slope is built mostly from weak clays (cohesion c about 11 kPa, internal friction angle ϕ about 11°). Rocks with compressive strength f_c about 0.36 MPa are found at about 4 m below the terrain level, stronger rocks (with f_c about 0.78 MPa) are found at about 6 m below the terrain level. Table 4.2 gives us in detail the different types of soil existing in the slope and their properties. the Mohr Coulomb model is used for the different type of soil in this project.

Table 4.2. Soil parameters **(Grodecki & Urbanski, 2018)**

Material	**Cohesion c [kPa]**	**Dry unit weight γ_{dry} [kN/M³]**	**Saturated uint weight γ_{sat} [kN/M³]**	**Friction angle ϕ [°]**	**Type**
I (uncontrolled embankment)	5	16	21	20	MC
II (silty clay + rubble)	11	16	22.5	11	MC
IVa (clay rubble)	12	15	21.5	12	MC
Va (rubble)	57	18	24	55	MC
VI (soft rock)	123	24	24	55	MC
Concrete	-	24	24	-	Rigid

MC: Mohr Coulomb

4.3.2 Gabions parameters

Gabions baskets are made of 60 x 80 x 3.3 mm wire mesh. They are filled with crushed aggregate. The interface elements between gabions and soil and between gabions were used. Table 4.3 & 4.4 give us more information on their parameters.

Table 4.3. Parameters of filing aggregates **(Grodecki & Urbanski, 2018)**

Material	**Cohesion c [kPa]**	**Dry unit weight γ_{dry} [kN/M^3]**	**Saturated unit weight γ_{sat} [kN/M^3]**	**Friction angle ϕ [°]**	**Type**
Gabions h=1m	30.75	18	22	40	Rigid
Gabions h=0.5m	61.5	18	22	40	Rigid

Table 4.4. Interface parameters **(Grodecki & Urbanski, 2018)**

Material	**Parent**	**Type**	**Multiplier on friction angle tan ϕ**	**Multiplier on cohesion c**
Interface Gabion – soil II	II (silty clay + rubble)	Mohr Coulomb derived	0.9	0.9
Interface Gabion – soil IVa	IVa (clay rubble)	Mohr Coulomb derived	0.9	0.9
Interface Gabion – soil Va	Va (rubble)	Mohr Coulomb derived	0.9	0.9
Interface Gabion – soil VI	VI (soft rock)	Mohr Coulomb derived	0.9	0.9

Interface Gabion – Gabion

Type	**Characteristics**
Combined : Mohr Coulomb and Cutoff	Cohesion c : 0 kPa Friction angle ϕ**:** 40° Dry unit weight γ_{dry}: 0 kN/m^3 Saturated unit weight γ_{sat}: 0 kN/m^3 Limiting compressive stress: 500 kPa Limiting tensile stress: 0 kPa

4.3.3 Self-drilling anchor

In order to obtain a satisfactory value of the safety factor, the anchoring of the wall was analyzed. Titan 40/20 type anchors with 90 mm drilling tool diameter were used. An anchoring length of 12 m (including 5 m active zone), distance between anchors of 5 m and pre-stress σ_0 =20 MPa was applied.

Referring to the literature, we were able to find the parameters of the self-drilling anchor Titan 40/20:

- Nominal outside diameter: 40 mm
- Nominal inside diameter: 20 mm
- Ultimate load: 539 kN
- Yield point: 430 kN
- Yield stress TO.2: 590 N/mm^2
- Cross section: 726 mm^2
- Weight: 5.6 kg/m
- Thread left/right hand: left
- Length: 3 m

Referring to the anchor Titan 40/20 parameters and using equations (3.6), (3.7), (3.8) & (3.9), we were able to parametrize the anchor in LimitState:GEO as shown in table 4.5.

Table 4.5. Anchor parameters

Name	**Type**	**Pullout fact. T_c [kPa]**	**Pullout fact. T_q [-]**	**Lateral fact. N_c [kPa]**	**Lateral fact. N_q [-]**	**Moment resis M_p [kNm/m]**	**Rupture stren R [kN/m]**	**Comp stren C [kN/m]**
Anchor plate	Engin. Element	20000	0	1E+30	0	1E+30	1E+30	1E+30
Anchor 1 in soil	Engin. Element	2.067	0,0333	1	0	1E+30	590000	1E+30
Anchor 2 in soil	Engin. Element	1.897	0,0281	1	0	1E+30	590000	1E+30

Anchor 3 in soil	Engin. Element	1.626	0,025	1	0	1E+30	590000	1E+30
Anchor 4 in soil	Engin. Element	1.288	0,021	1	0	1E+30	590000	1E+30

Engin: Engineering fact: factor resis: resistance stren: strength comp: compression

4.3.4 External load

A load of 25 kPa due to road traffic was introduced at the municipal road. It is placed 5 m behind the gabion wall over a length of 4 m.

4.4 Numerical analysis

This study is to investigate the stability characteristics of the landslide in the city of Nowy Sacz (Poland) using an anchored gabion wall. This work was divided into three steps:

- Slope stabilization analysis without gabion wall (without the road load)
- Slope stabilization analysis with gabion wall (without the road load)
- Slope stabilization analysis with anchored gabion wall (with the road load)

4.4.1 Modeling

The different soil layers are represented to obtain the geometry of the landslide. Followed the addition of a gabion wall and finally anchors. Figure 4.4, figure 4.5 and figure 4.6 present the different steps in this study.

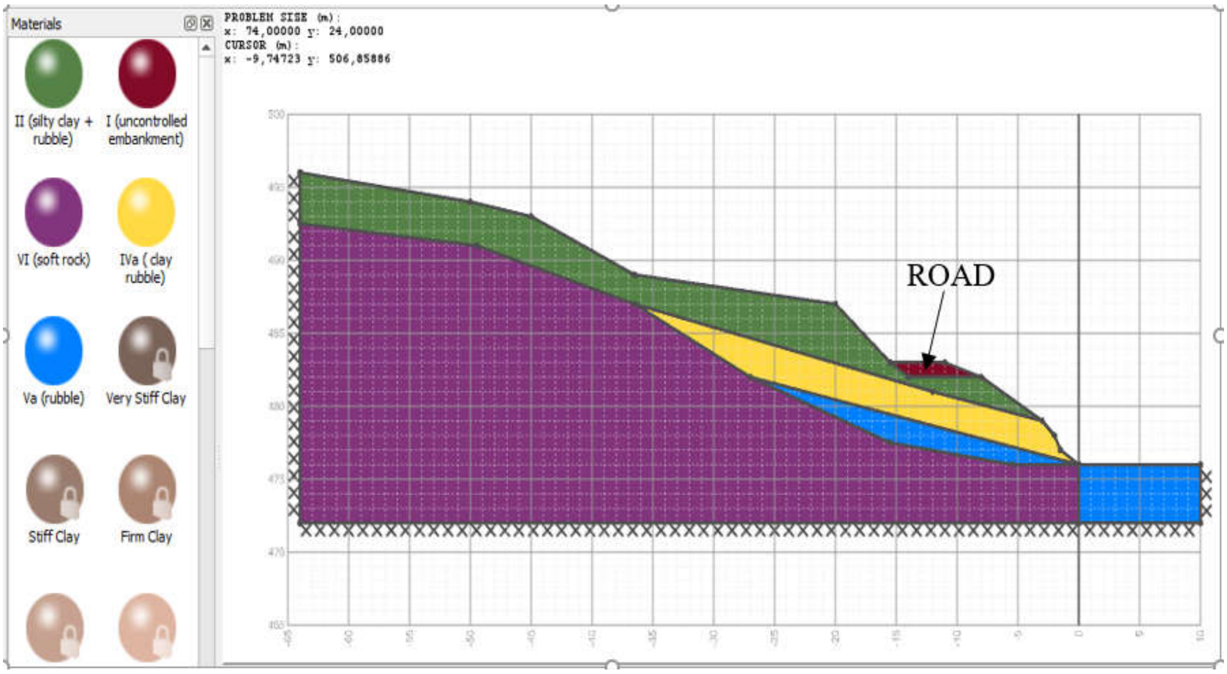

Figure 4.4. Numerical model of the existing landslide (**LimitState GEO 3.5)**

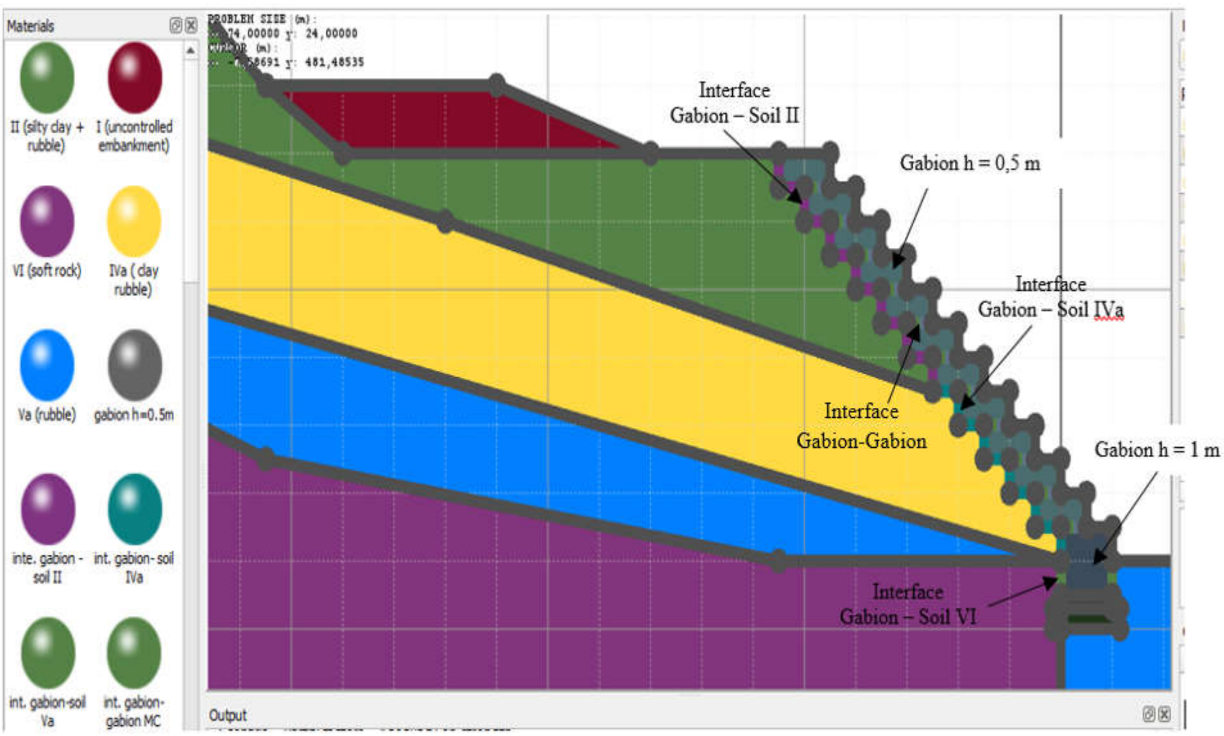

Figure 4.5. Landslide with Gabion wall (**LimitState GEO 3.5**)

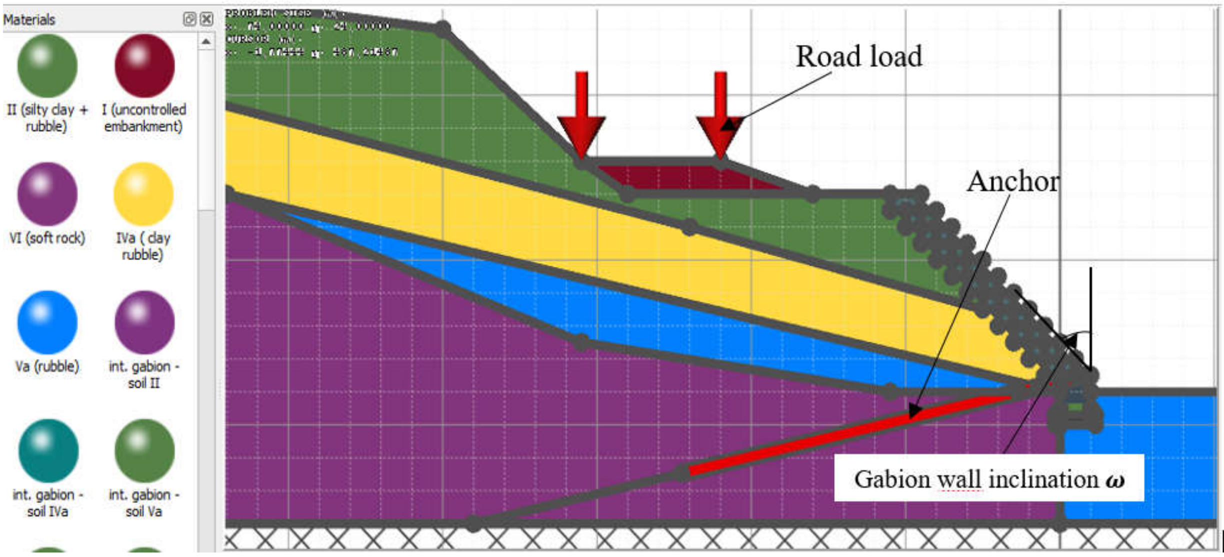

Figure 4.6. Landslide with anchored gabion wall (**LimitState GEO 3.5**)

4.4.2 Assumptions

The assumptions considered for this case are:

- No water table regime
- No seismic zone
- Analysis is done in the long term
- The type loading of soil is neutral
- The type loading of road load is unfavorable
- The road load is variable
- The gabion wall is inclined of 45° with respect to the vertical axis.
- Nodal density: very fine (2000 nodes)
- Factor of safety on soil strength is computed
- Eurocode 7 Design Approach 1 combination 2 is used
- The interaction coefficient $\alpha = 1$
- Horizontal Anchor spacing $n = 5m$
- Anchor diameter $D = 40mm$

4.4.3 Results and interpretation

4.4.3.1 Numerical analysis of the existing landslide

During the performing of the stability analysis of the existing "in situ" state of the slope as shown in figure 4.7, a safety factor of **0.9884** was obtained. This value of the safety factor is less than 1 implying and unstable slope. Therefore, the design of a protecting structure is really necessary and a gabion wall is proposed

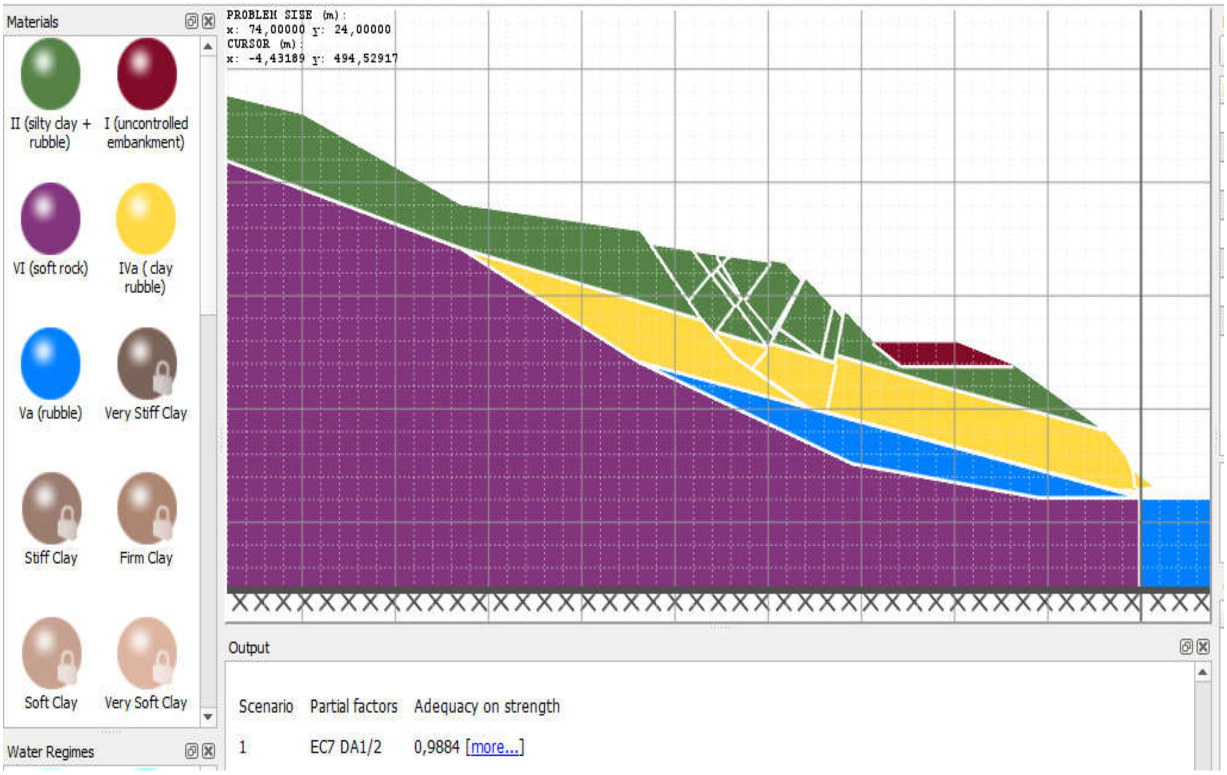

Figure 4.7. Failure mode of the existing landslide (**LimitState GEO 3.5**)

4.4.3.2 Numerical analysis of the landslide with gabion wall

The first variant of the gabion wall consists of one 1m high and eleven 0.5m high gabions as shown in figure 4.5. After numerical simulation, a safety factor **1.082** was obtained as shown in figure 4.8. it is noticed that the construction of the gabion wall improves the stability of the landslide, however it shows that the slope is practically in the limit state of stability. Thus, the use of anchors is proposed in order to increase the safety factor.

Figure 4.8. Failure mode of the landslide supported by gabions (**LimitState GEO 3.5**)

It's worth noticing that the failure mode of this variant includes lowest gabion failure due to sliding.

4.4.3.3 Numerical analysis of the landslide with anchored gabion wall

To obtained a satisfactory value of the safety factor, a series of stability analysis were done on the gabion wall with a variable number of anchors. The following values of safety factor were recorded (table 4.6). It's useful to precise that the road load is taken into account. Figures 4.9, 4.10, 4.11 & 4.12 illustrate the failure mode of the landslide supported by gabions wall with a different number of anchors.

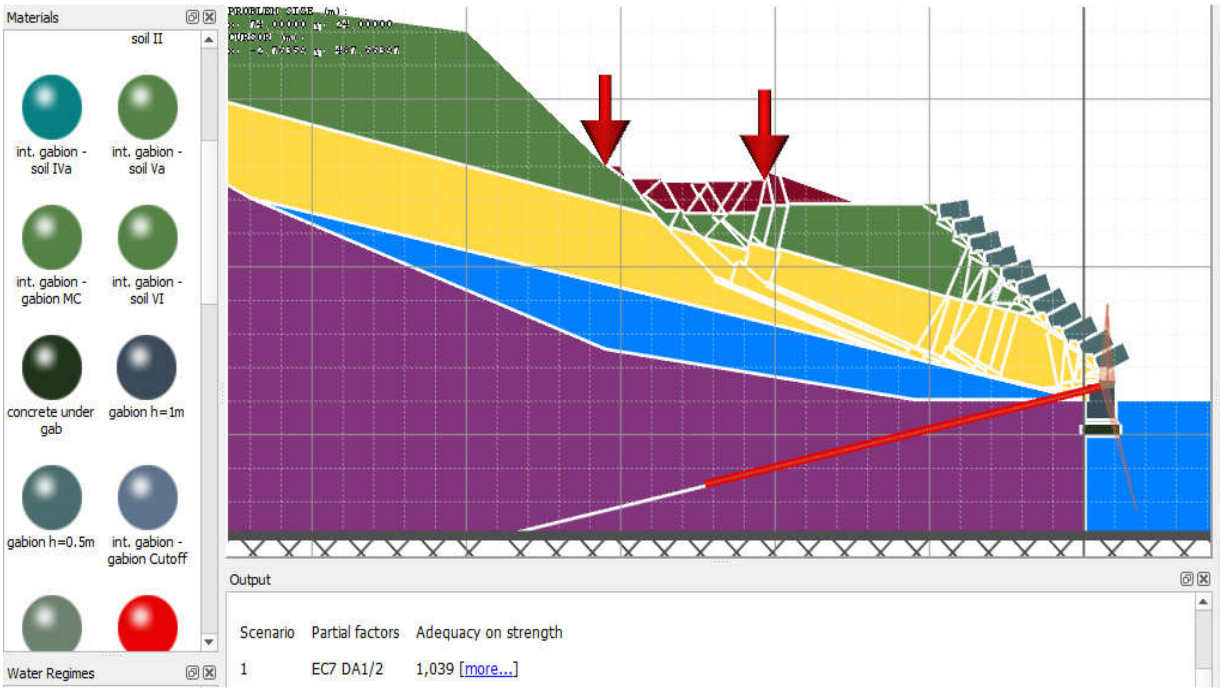

Figure 4.9. Failure mode of the landslide supported by gabions with 1 anchor (**LimitState GEO 3.5**)

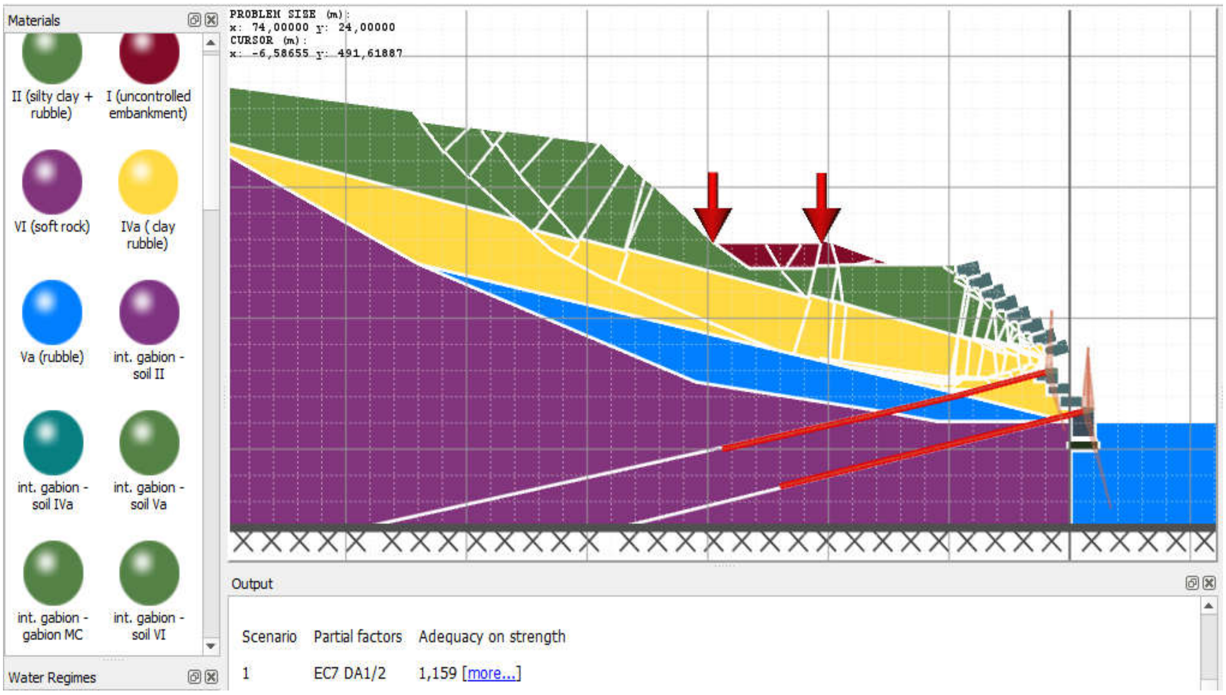

Figure 4.10. Failure mode of the landslide supported by gabions with 2 anchors (**LimitState GEO 3.5**)

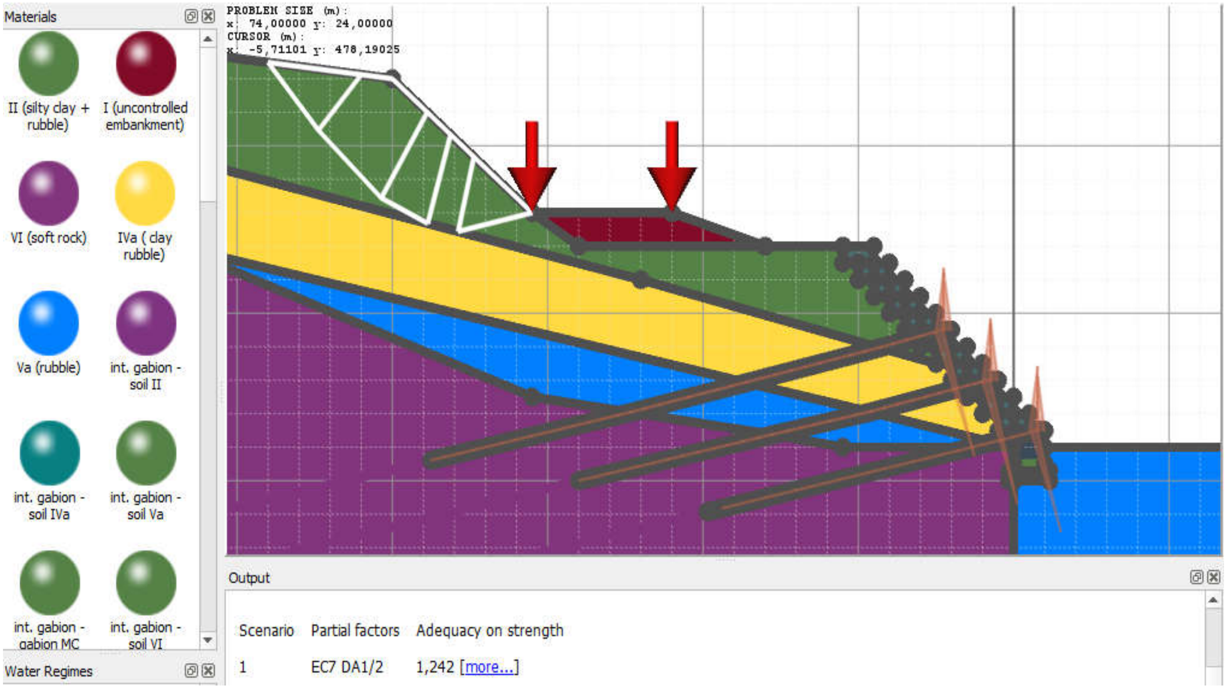

Figure 4.11. Failure mode of the landslide supported by gabions with 3 anchors (**LimitState GEO 3.5**)

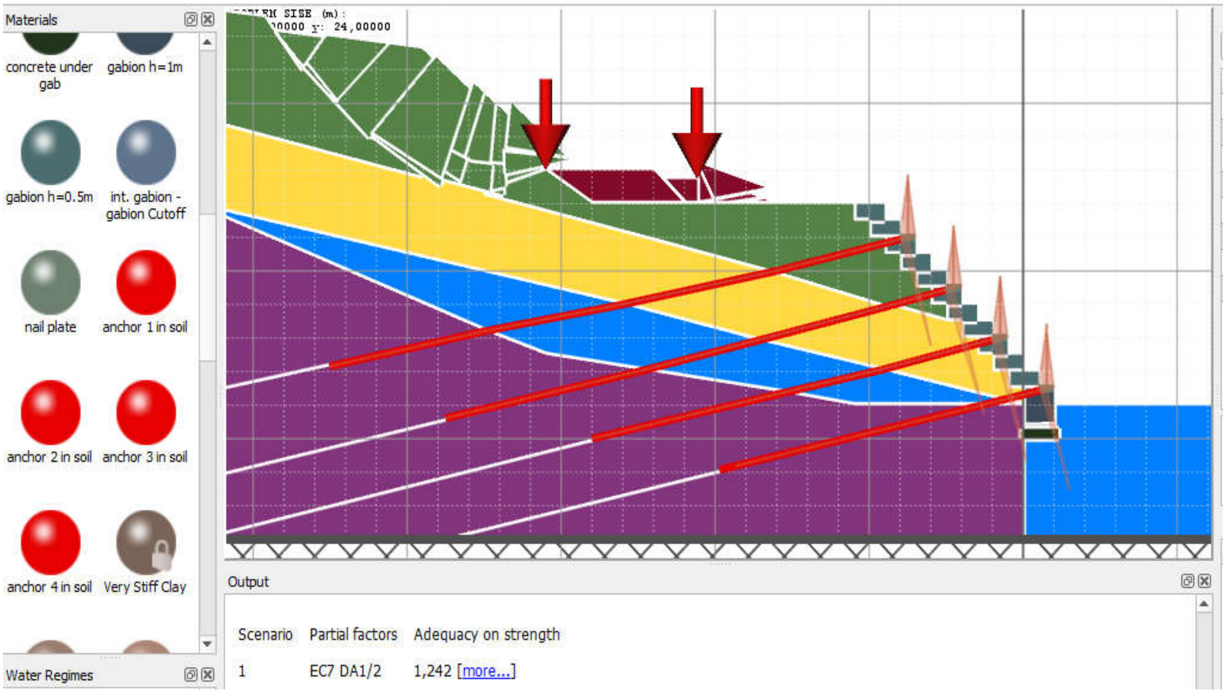

Figure 4.12. Failure mode of the landslide supported by gabions with 4 anchors (**LimitState GEO 3.5**)

Table 4.6. Different values of safety factors

Case	Safety factor
Gabion wall + 1 anchor	1.039
Gabion wall + 2 anchors	1.159
Gabion wall + 3 anchors	1.242
Gabion wall + 4 anchors	1.242

Three different failure mechanisms are observed for the cases with 1, 2 and 3 anchors.

- In the case of the gabion wall with 1 anchor (figure 4.9), the failure occurs on the part of the slope which is below the road. The soil behind the gabion wall pushes against it, causing an internal failure of the gabion wall. But the lowest gabion does not fail as in the previous case.
- In the case of the gabion wall with 2 anchors (figure 4.10), it can be seen that the failure extends over the lower and the upper slopes of the road. An internal failure mechanism of the gabion wall is observed, which onset at the middle height of the gabion wall.
- In the case of the gabion wall with 3 anchors (figure 4.11), only the upper part of the slope is interested in the failure mechanism, while the gabion wall ensures the stability of the lower part of the slope (the one below the road level). This put into light that, if a further increment of the safety factor is needed, a stabilizing intervention on the upper part of the slope should be considered (e.g. reprofiling of the slope above the road level).

4.4.3.4 Summary of safety factor

Table 4.7 gives us a summary of the different value of safety factor that was obtained during our analysis.

Table 4.7. Summary of the value of Safety factor

Case	Safety factor
Landslide without gabion wall(without road load)	0.9884
Landslide with gabion wall(without road load)	1.082
Landslide with gabion wall(with road load)	1.026

Gabion wall + 1 anchor (with road load)	1.039
Gabion wall + 2 anchors(with road load)	1.159
Gabion wall + 3 anchors(with road load)	1.242
Gabion wall + 4 anchors(with road load)	1.242

From table 4.7, we were able to obtain the graph illustrating the variation of the safety factor in function with the number of anchor as shown in figure 4.13.

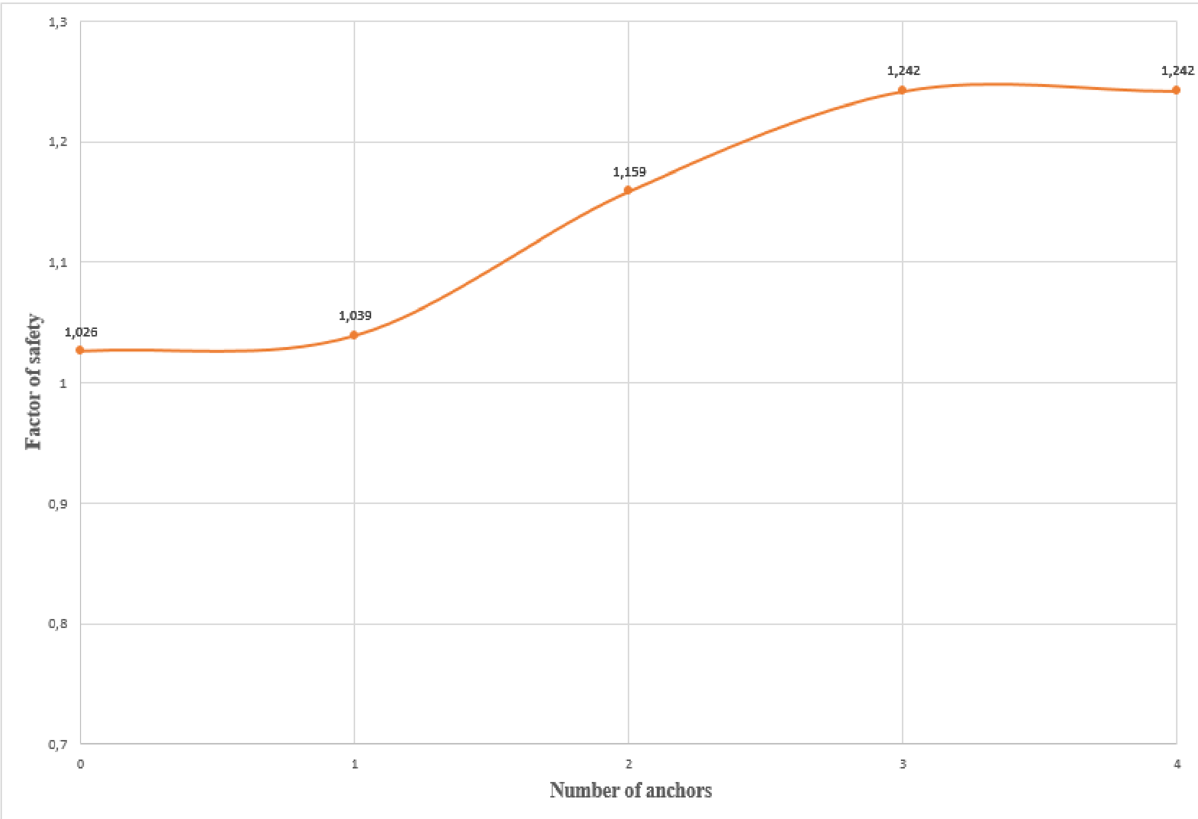

Figure 4.13. Variation of the safety factor in function with the number of anchors

From figure 4.13, we notice that the safety factor is the same for the cases with 3 and 4 anchors as well as the failure modality. This is because the failure surface does not interest the slope gabion wall. To increase safety, it's useful to stabilize the part of the slope that is above the road. Similarly, it can happen in the future that the road is widened and this causes additional loads, therefore this slope can be stabilized using a gabion retaining wall with 4 anchors.

4.4.3.5 Influence of some parameters on the slope stability

a. Influence of friction angle at the interface gabion-gabion (ϕ)

The influence of the friction angle of the interface gabion-gabion on the slope stability in the case of the gabion wall without anchor (without road load) was investigated by considering for seven values of "ϕ" (25°, 30°, 35°, 45°, 50°). The following safety factor value was obtained as shown on table 4.8

Table 4.8. Value of Safety Factor at different friction angle at the interface gabion-gabion

Friction angle [°]	25	30	35	40	45	50
Factor of safety [-]	0.9919	1.034	1.055	1.082	1.119	1.174

From table 4.8, a graph showing the variation of the slope stability in function of the friction angle of the interface gabion-gabion was plotted (figure 4.14).

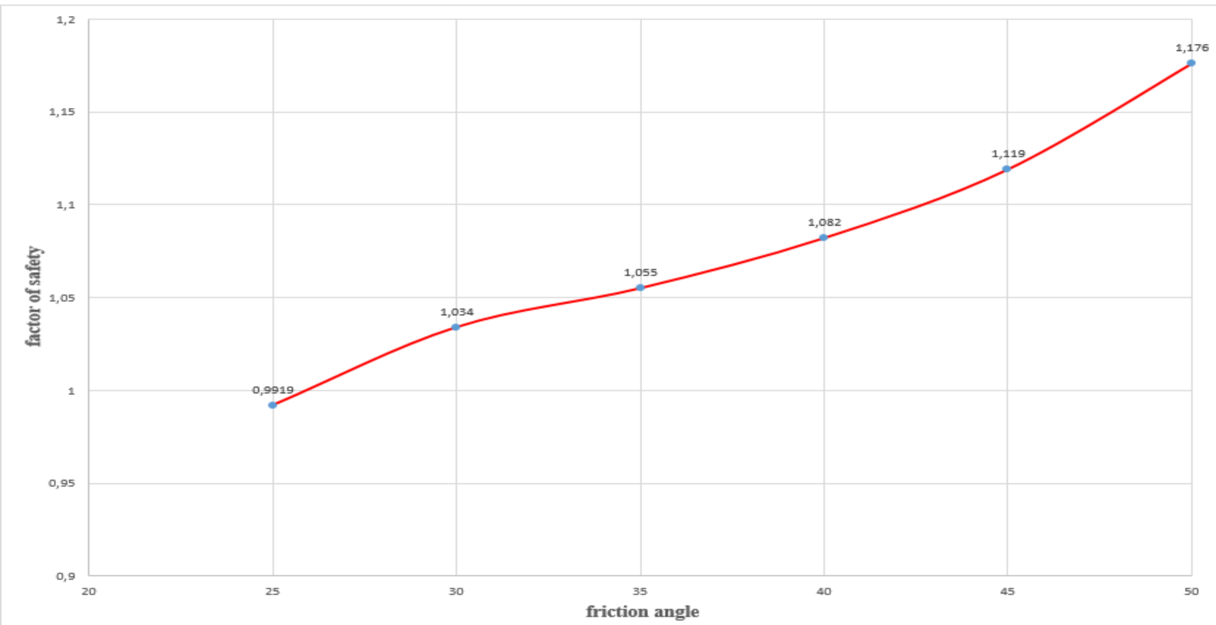

Figure 4.14. Effect of friction angle of interface gabion-gabion on the slope stability

A growth of the curve is observed. It is therefore uncerstood that the more the friction angle of interface gabion-gabion increases, the more the slope stability increases. So the influence of the friction angle of interface gabion-gabion on the safety factor is not negligible.

b. Influence of interface friction coefficient at the interface gabion-soil

An investigation of the influence of the friction angle at the interface gabion-soil on the slope stability was carried out in the case of the gabion wall without anchors (without road load) for a first time, then the cases with 1, 2 &3 anchors (with road load). The interface multipliers that were used are (0.5, 0.6, 0.7, 0.8, 1) gave the following value of the safety factor (Table 4.9 & 4.10).

Table 4.9. Value of safety factor at different friction coefficient at the interface gabion-soil

Interface multiplier	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1**
Gabion wall without anchor	1.075	1.077	1.079	1.08	1.082	1.083

Table 4.10. Value of safety factor at different friction coefficient at the interface gabion-soil

Interface multiplier	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1**
Gabion wall with 1 anchor	1.027	1.077	1.079	1.08	1.082	1.083
Gabion wall with 2 anchors	1.154	1.155	1.157	1.158	1.159	1.159
Gabion wall with 3 anchors	1.24	1.241	1.242	1.242	1.242	1.242

From tables 4.9 & 4.10, we obtained the following graph illustrated by figures 4.15 & 4.16

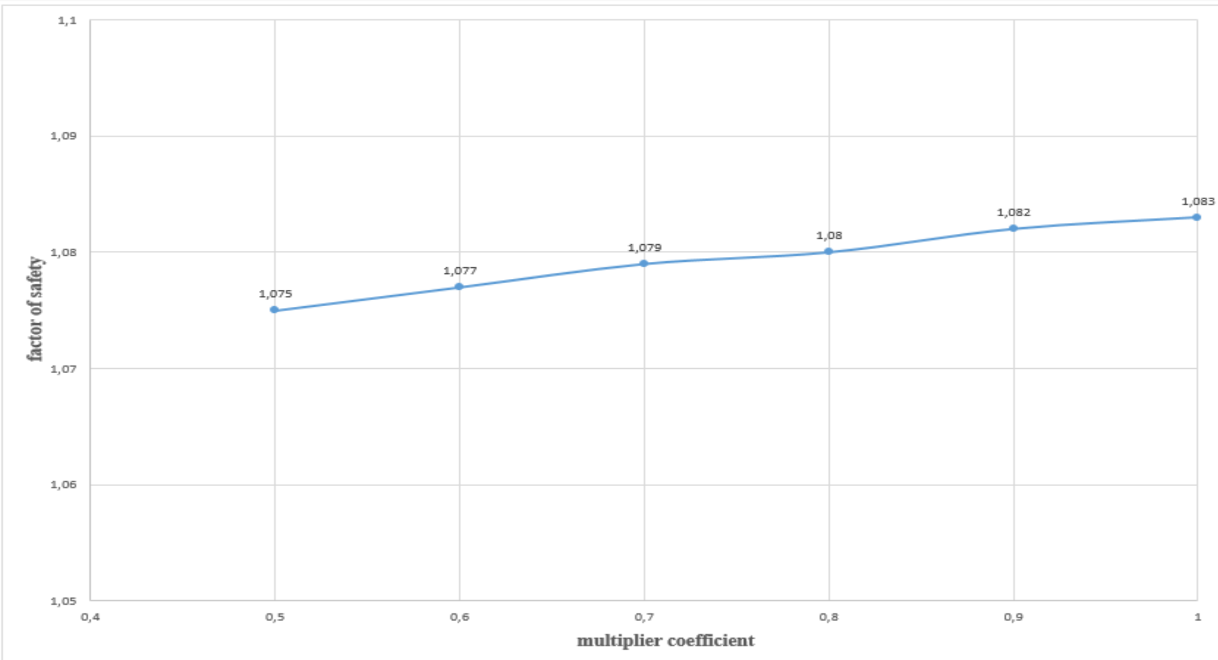

Figure 4.15. Effect of the friction coefficient at the interface gabion-soil on the slope stability (without anchor)

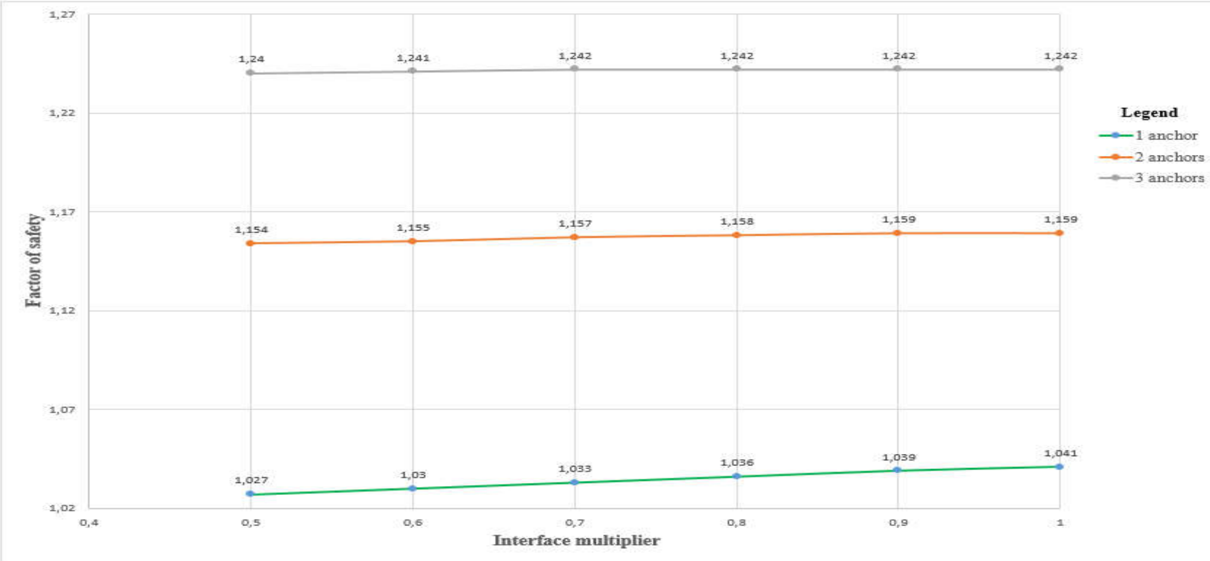

Figure 4.16. Effect of the friction coefficient at the interface gabion-soil on the slope stability (with anchor)

- In the case of gabion wall without anchor (figure 4.15), a growth of the graph is observed. This shows that the greater the friction coefficient at the interface gabion-soil, the greater the safety factor increases, but only slightly. A stress a bit more than the gabion-soil friction has a low effect.
- On the other hand, in figure 4.16, a weak growth of the graph (cases 1 & 2 anchors) is observed and becomes more and more constant when the number of anchor increases. It is understood therefore that the friction coefficient at the interface gabion-soil has an effect on the stability of the gabion wall when failure surface is at the level of the gabion wall, otherwise it has no influence (case with 3 anchors).

c. Influence of the road load

In order to find the maximum road load for which the slope is stable in the case with 4 anchors, a progressive increase was made on the road load. The values (25kPa, 30kPa, 35kPa, 40kPa, 43kPa, 45kPa) was used to obtaining the safety factors contained in table 4.11

Table 4.11. Value of safety factor at different road load

Road load [kPa]	25	30	35	40	43	45
Safety factor [-]	1.242	1.199	1.115	1.046	1.009	0.9868

From table 4.11, we obtained the following graph (figure 4.17)

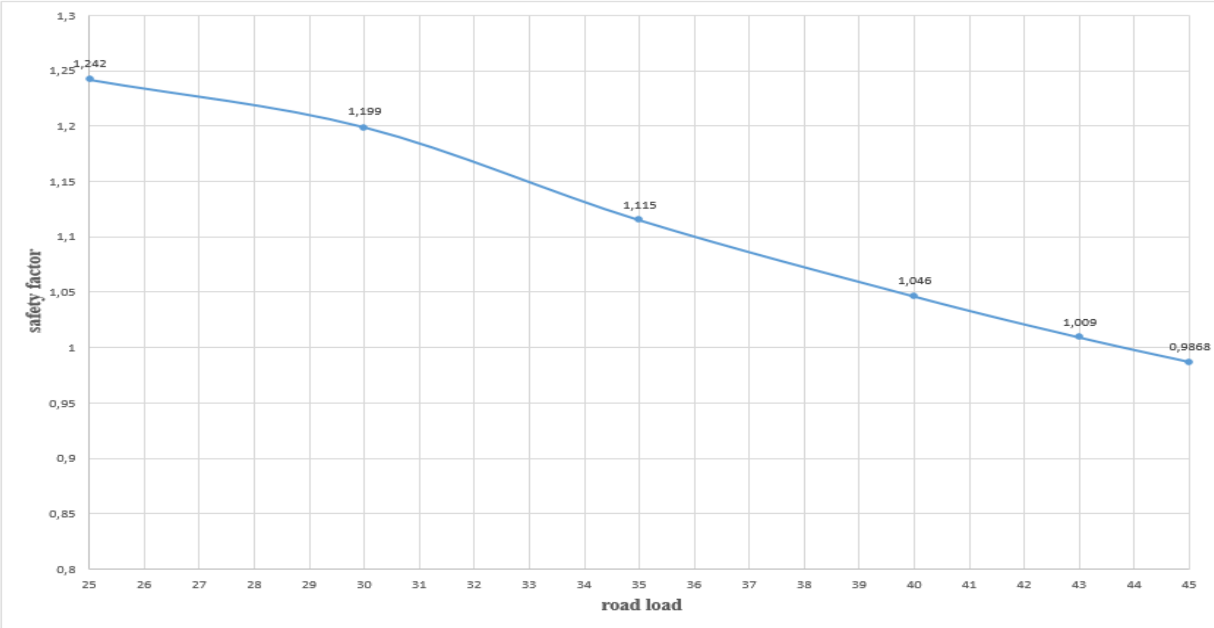

Figure 4.17. Effect of road load on the slope stability

A decrease of the curve is observed, to say that as the road load increases, the stability of the slope decreases. It's therefore deduce that the maximum road load that keeps the slope stable is about 43kPa.

d. Influence of anchors spacing

The impact of the anchors spacing on the stability of the gabion wall is evaluated. In the cases 2, 3 & 4 anchors, with spacing values of 2.5 m, 3.5 m, 7.5 m and 10 m, the following results were obtained as shown in table 4.12

Table 4.12. Value of safety factor at different anchor spacing

Spacing [m] / Number of anchors	**2.5**	**3.5**	**5**	**7.5**	**10**
2	1.159	1.159	1.159	1.151	1.138
3	1.242	1.242	1.242	1.242	1.223
4	1.242	1.242	1.242	1.242	1.239

From table 4.12, we obtained the following graph (figure 4.18)

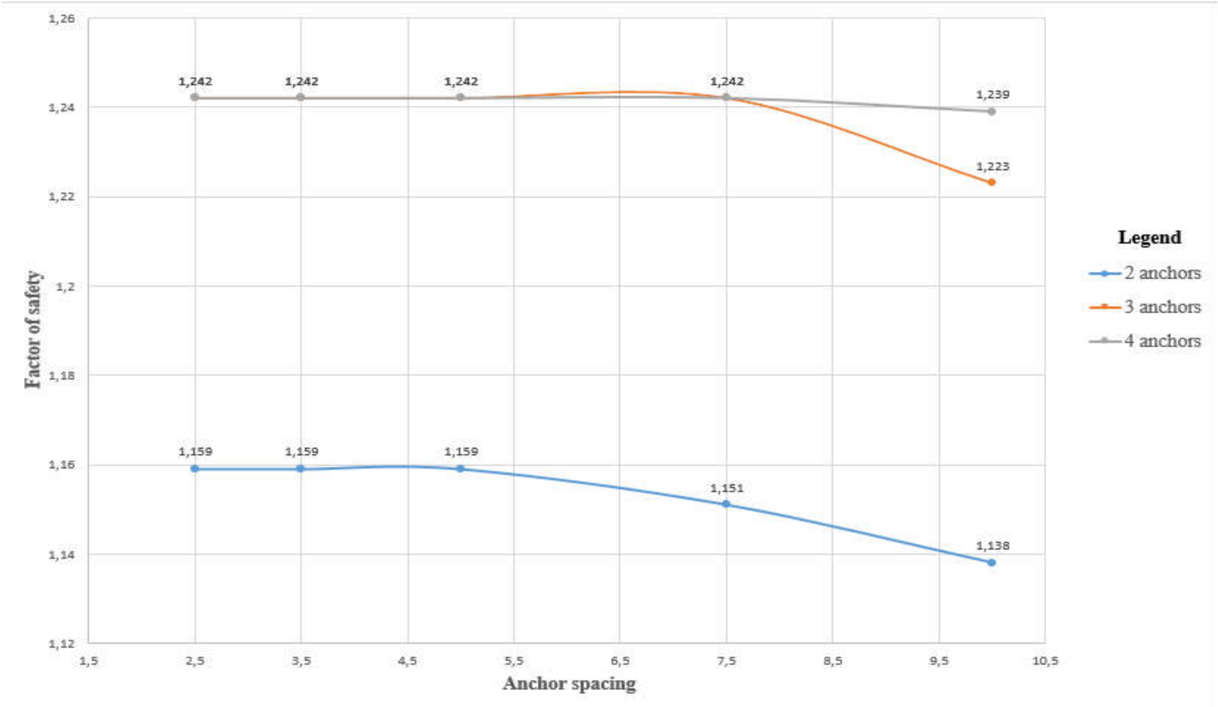

Figure 4.18. Effect of anchor spacing on the slope stability

It can be seen that the greater the spacing between the anchors, the more the safety factor decreases, making the slope more and more unstable. But the speed of decrease of the safety factor is a function of the number of anchors. The more the number of anchors, the smaller the decrease of the safety factor (with an increase of anchor spacing).

e. Influence of the water table

The influence of the presence of water on the stability of the slope (case with 4 anchors) was further studied. For this purpose, a water table with an inclined slope (figure 4.19) was considered and its impact on the slope stability evaluated every 2m. the chosen water table levels are materialized by levels (level 1, level 2, level 3, level 4, level 5) as shown on figure 4.19. The following factors of safety were registered (table 4.13).

Table 4.13. Value of safety factor at different water table level

Level	Level 1	Level 2	Level 3	Level 4	Level 5
Factor of safety	1.055	1.213	1.239	1.242	1.242

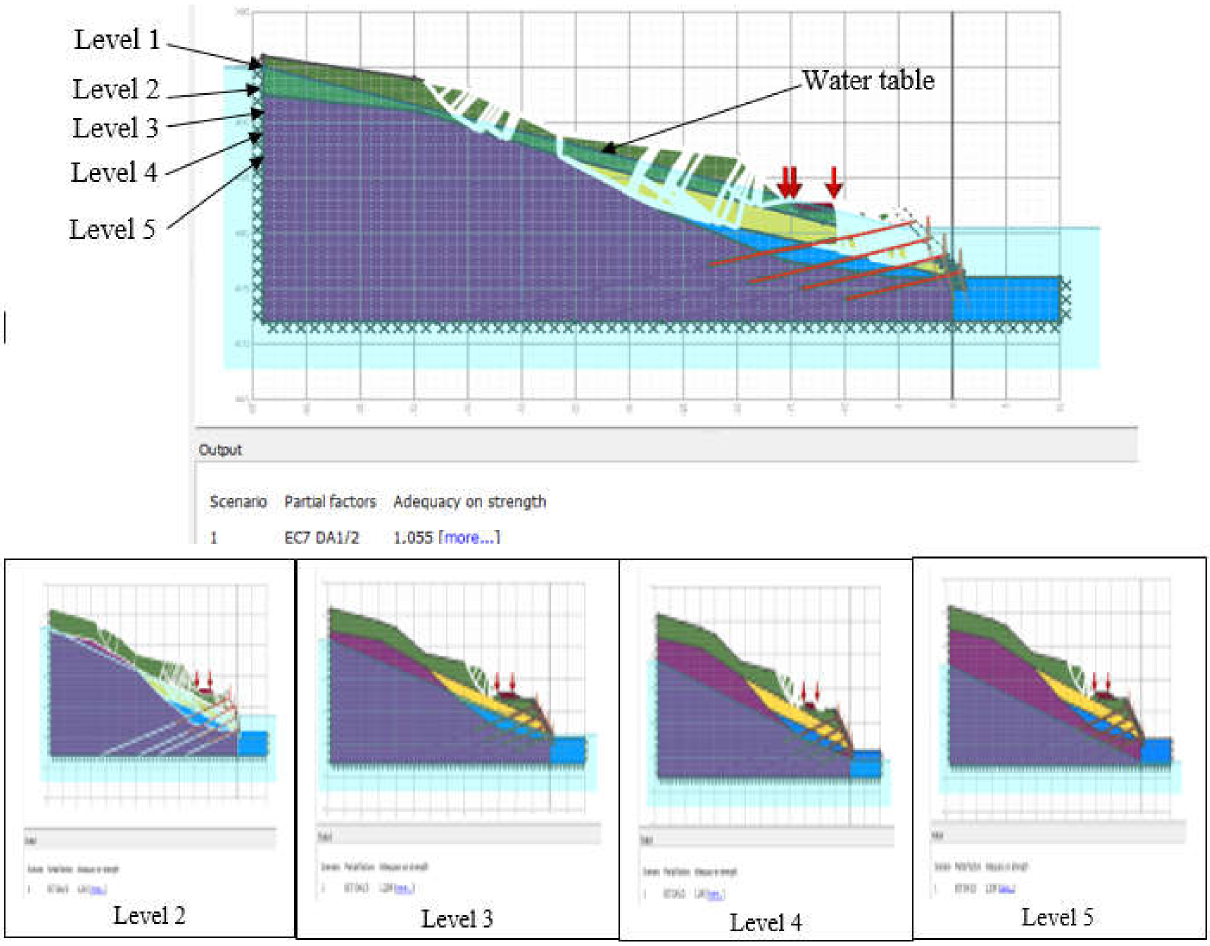

Figure 4.19. Different level of water table (**LimitState GEO 3.5**)

The stability of the slope remained constant (1.242) until level 4 and then starts to decrease. This shows that as soon as the failure surface touches the water table, the safety factor decreases (level 1, level 2), otherwise it remains constant. The presence of water has a negative impact on the slope stability.

Conclusion

This chapter aimed at conducting the stability analysis of a landslide that was stabilized through an anchored gabion wall, located in Nowy Sacz city. A description of the site and a presentation of the project was done, followed by a numerical analysis the slope stability. The stability analysis was conducted using the numerical software LimitState:GEO. To do this, the analysis was partitioned into three steps (Slope stabilization analysis without gabion wall (without the road load), Slope stabilization analysis with gabion wall (without the road load), Slope stabilization analysis with anchored gabion wall (with the road load)). A parametric analysis was performed to point out the role of the main variable of our problem. The different DLO results obtained help to understand the influence of some parameters on the slope stability.

GENERAL CONCLUSION

The main objective of this thesis entitled "Slope stabilization through a gabion wall: case study of a landslide in southern Poland", was to analyze the stability of a slope using a gabion wall.

To attain this objective, a literature review was done, base on slope stability analysis, slope stabilization methods and gabion retaining wall. The methodology used in order to achieve the above objective consisted of documentary research and slope stability analysis using the software LimitState: GEO (presentation of the method use: DLO method, formulation of some parameters). Based on the data collected from the study of Michal Grodecki & Aleksander Urbanski (Grodecki & Urbanski, 2018), a numerical modeling of the landslide, the gabion wall and different simulations were done. The software LimitState: GEO allows us to analyze the stability of our slope. A parametric analysis was performed in order to point out the influence of certain parameters (road load, friction angle at the interface gabion-gabion, friction coefficient at the interface gabion-soil, anchors spacing, level of the water table) on the stability of the slope.

From the results obtained, the following conclusions were obtained:

- To obtain a satisfactory safety factor, the gabion wall should be reinforce with 4 anchors, since the road can have additional loads in the future.
- The safety factor is 1.242 > 1 implying the slope is stable.
- The friction angle at interface gabion-gabion and friction coefficient at the interface gabion-soil have an influence on the safety factor and therefore should not be neglected.
- The maximum road load for which the slope is stable is 43kPa.
- The safety factor decreases with an increase in anchor spacing.
- An increase of the water level has a negative influence on the stability of the slope.

To improve this work,

- further analysis should be conducted considering seismic effect on the stability of the slope;
- a structural study of the metal cage of the gabion should be necessary in order to evaluate the evolution of solicitations in each metal wire to avoid any local breakage of the wires;
- stabilization analysis of the upper part of the slope could be interesting in order to increase the value of the safety factor.

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