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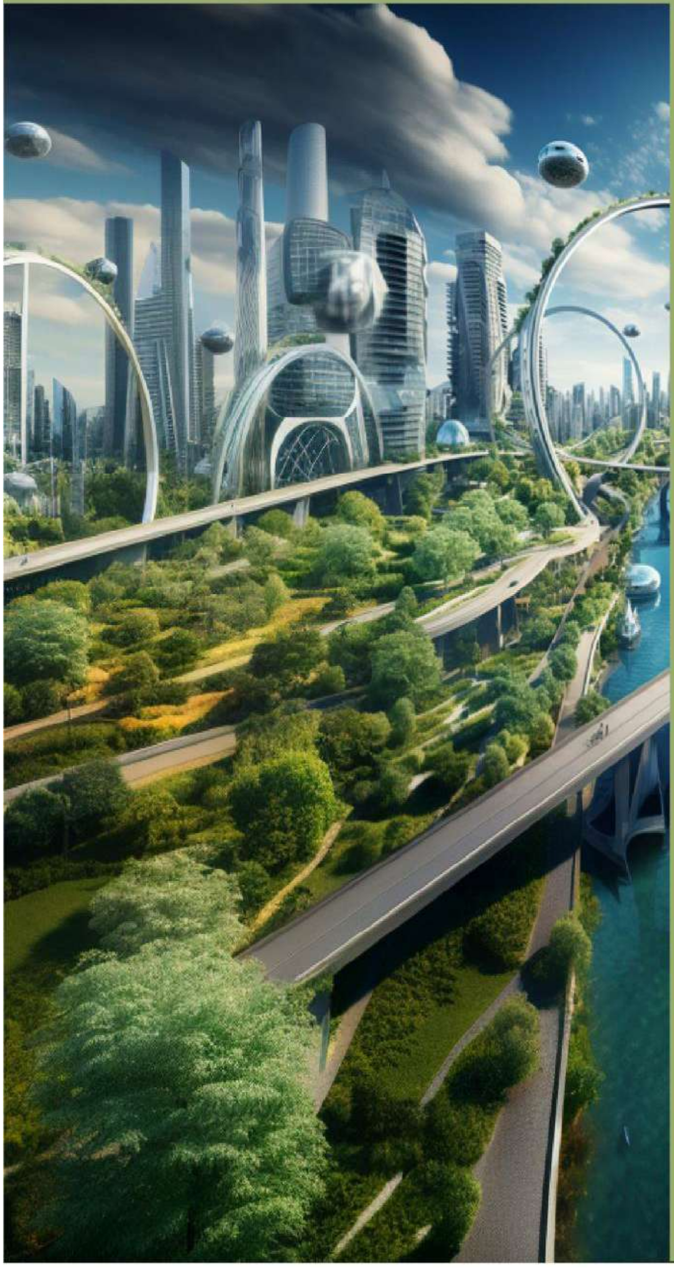

Sustainability and Resilience: Innovation and Solutions

27, 28 e 29 nov

Universidade da Beira Interior

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GLOBAL STABILITY IN GABION WALLS

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ABSTRACT

Gabion walls, composed of double-twisted galvanized steel mesh cages filled with rocks, are widely used in civil engineering as retaining structures for slopes, allowing for steeper inclinations than the natural slope of the soil, optimizing space and ensuring construction safety. This study aimed to analyze the influence of differents soil parameters on the design of these gravity walls, using the Geo5 software (2020) to verify their ultimate limit states according to Eurocode 7, with a focus on global stability through different methods. The analysis involved the variation of the following parameters: foundation soil cohesion, backfill soil friction angle, and backfill soil type. It was found that cohesion and friction angle significantly influence shear strength, with more favorable parameters resulting in greater structural stability. Additionally, it is noted that a higher backfill soil friction angle also affects the foundation by causing variation in the eccentricity values of the reaction force. The Fellenius/Petterson method (1955) proved to be conservative due to its simplifications, yielding significantly lower safety factors compared to other methods. In contrast, the Janbu (1954) and Morgenstern-Price (1965) methods demonstrated more consistent and closely aligned results in all analyses. These findings underscore the importance of a detailed and accurate analysis of soil parameters and proper compaction of the backfill soil to ensure the safety and effectiveness of gabion walls as retaining structures.

Keywords: *Gabion wall; global stability; gravity wall; support structure.*

RESUMO

Muros de gabiões, constituídos por gaiolas de malha de aço galvanizado de dupla torção preenchidas com rochas, são amplamente utilizados na engenharia civil como estruturas de suporte para taludes, possibilitando nestes uma inclinação maior do que a natural do solo, permitindo otimizar o espaço e garantindo segurança em construções. Este estudo teve como objetivo analisar a influência de diferentes parâmetros do solo no dimensionamento desses muros de gravidade, utilizando o programa Geo5 (2020) para realizar a verificação quanto aos estados limites últimos do Eurocode 7, com foco na estabilidade global, a partir de diferentes métodos. A análise envolveu a alteração dos seguintes parâmetros: coesão do solo de fundação, ângulo de atrito do solo de aterro e tipo de solo de aterro. Verificou-se que a coesão e o ângulo de atrito têm uma influência significativa na resistência ao corte, com parâmetros mais favoráveis resultando em maior estabilidade da estrutura. Além disso, nota-se que um maior ângulo de atrito do solo de aterro impacta também na fundação, por gerar variação dos valores de excentricidade da força de reação. O método de Fellenius/Petterson (1955) revelou-se conservador devido às suas simplificações, apresentando fatores de segurança significativamente menores em comparação à outros métodos. Já os métodos de Janbu (1954) e Morgenstern-Price (1965) demonstraram resultados mais consistentes e próximos entre si em todas as análises realizadas. Estes resultados destacam a importância de uma análise detalhada e precisa dos parâmetros do solo e da compactação adequada do solo de aterro para garantir a segurança e a eficácia dos muros de gabiões como estruturas de suporte.

Palavras-chave: *Muro de gabiões; estabilidade global; muro de gravidade; estrutura de suporte.*

1 INTRODUCTION

The need for modifications in space, aiming for its optimization, is recurrent, often leading to large-scale slopes and consequently creating the need for structures to support these conditions. In this context, gabion walls have been developed—composite structures made of steel and rock, consisting of double-twisted galvanized steel cages filled with weather-resistant rocks. These form according to Barros (2021) a monolithic, draining structure and capable of withstanding small plastic deformations due to the steel components.

Designing such a structure requires knowledge of soil parameters, acting loads, and the type of intended use. In this study, the design of a gabion wall for a particular case will be carried out, with variations in soil parameters to assess their respective impacts. The “Slope Stability” and “Gabion Wall” modules of the Geo5 (2020) software will be used, following the guidelines of Eurocode 7 – Geotechnical Design (NP EN 1997-1.2010).

The objective of this work is to analyze the influence of backfill and foundation soil parameters on the ultimate limit states of gabion walls, in accordance with Eurocode 7, with an emphasis on global stability, through the comparison of different verification methods.

2 THEORETICAL FRAME

2.1 Shear strenght and earth pressure

The shear strength of soils is fundamental for understanding the limitations of soil in resisting earth pressure forces, requiring knowledge of the soil's cohesion and internal friction angle. The definition follows the principles presented by the Mohr-Coulomb Failure Criterion (1773), as shown in Equation 1.

$$s = c + \sigma \tan ø \qquad (1)$$

where, s = shear strenght;

c = cohesion

σ = normal stress;

ø = internal friction angle.

The earth pressure can be defined as the force or pressure that the soil exerts on the retaining structure and is categorized into active, passive, and at-rest pressure.

For the calculation of earth pressures, the most commonly used methods are those of Rankine (1857) and Coulomb (1776), the latter being the one used in this work.

The Coulomb method (1776) was defined to find the minimum and maximum limit forces between the soil and the wall, being in some cases subject to an analytical solution.

2.2 Slope stability

The support structure must also be evaluated with respect to slope stability, preventing mass sliding movements that could lead to its overall collapse. Soils that exhibit a certain degree of homogeneity and isotropy tend to display a circular-shaped sliding surface, whereas heterogeneous soils generally exhibit a non-circular sliding surface.

To perform this evaluation, the method of slices can be employed. In this method, starting from a radius (R), the slope is divided into slices, and by summing the forces acting on each slice, the global safety factor is calculated.

Incorporated into the limit equilibrium theory, the method assumes that failure occurs along a circular surface, or any other surface, and the condition of imminent failure is simultaneously reached by all elements along this surface (GERSCOVICH, 2016).

Figure 1 shows the considerations in the formulation of the general model of the slice method.

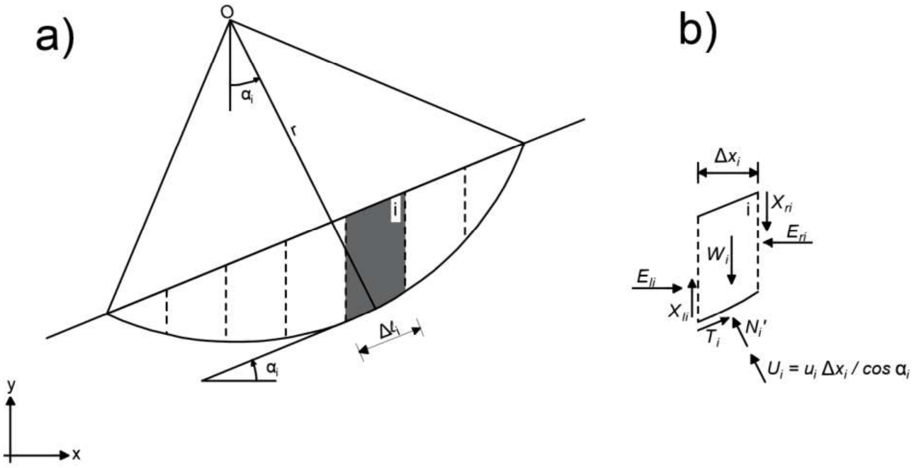


Figure 1 – Method of slices: a) Land mass under analysis; b) Generic slice with applied forces

Source: Adapted from Fernandes (2015)

The general formulation is shown in Equation 2.

$$FoS = \frac{\sum c' \times \Delta l + N' \times \tan\phi'}{\sum W_i \sin\alpha} \tag{7}$$

where, FoS = Safety factor;

W_i = Weight;

N'_i = Resultant of the effective normal stress at the base of slice;

U_i = Resultant of the pore water pressure at the base of the slice;

T_i = Resultant of the mobilized tangencial stress at the base of slice;

E_{li} e X_{li} = The normal and tangencial componentes of the interaction forces on the left face;

E_{ri} e X_{ri} = The normal and tangencial componentes of the interaction forces on the right face.

The challenge of the method of slices lies in its statically indeterminate nature, presenting, according to Fernandes (2015), 3n available equations and 4n-2 unknowns. To render the problem statically determinate, various authors have proposed different assumptions in their calculations. These assumptions are the factors that define the uniqueness of each method, leading to varying values in the calculation of the safety factor.

The Fellenius Method (1936), also known as the Swedish Method, was the first to verify slope stability, bringing significant contributions to this field. It is a conservative method, which is due to its simplifications and the fact that it was a manual calculation method, considering the time when it was conceived.

An improvement to the Fellenius Method (1936) was made by Petterson, with the inclusion of the interference of water effects on slope stability, thereby creating the Fellenius/Petterson Method (1955), which offers greater accuracy in the calculations.

The Bishop Method (1955) disregards the shear forces Xi between slices, and with the factor of safety in both terms of the equation, this value is calculated through iterations. With fewer simplifications than the Fellenius/Petterson Method (1955), this method provides greater accuracy.

One of the challenges to be overcome until then was the simultaneous calculation in terms of force and moment equilibrium, which was achieved in the Spencer Method (1967). In this method, inter-slice forces are considered, with a constant inclination, and the sum of the resulting forces forms a force Q (in terms of total stresses) with an inclination θ, passing through the intersection point of other acting forces (GERSCOVICH,2016).

The Janbu Method (1954) is considered a rigorous and complex method. The planes between the slices are always considered vertical, with the line of action of the block weight (Wi) passing through the center of the sliding surface(M), where the normal force also acts.

Morgenstern & Price (1965) developed a method distinct from the others mentioned, where the inclination of the resultant (θ) is altered by a function along the failure surface, with the forces acting on infinitesimal slices (GERSCOVICH, 2016). The method still treats the slices as vertical.

A relationship between the tangential and normal components of the interaction forces is adopted, based on an arbitrarily chosen function and a dimensionless scale factor (λ), thus having a number of unknowns equal to 3n, the same as the number of available equations, making the problem statically determinate (FERNANDES, 2015).

2.3 Eurocode 7

The Eurocode 7 presents three calculation approaches for verifying actions and resisting forces. In Portugal, the standard is to use Calculation Approach 1. Calculation Approach 1 is distinguished by its verification of safety for Combination 1 and Combination 2, choosing the one that yields the lower value of the factor safety.

Combination 1 increases both permanent and variable actions, while the soil properties remain constant, whereas Combination 2 decreases these properties and increases only the variable actions.

EC 7 (2010) also specifies the need to verify the Ultimate Limit State (ULS) for the following conditions: global failure, foundation failure, sliding, overturning, and internal failure, which are illustrated in Figure 2.

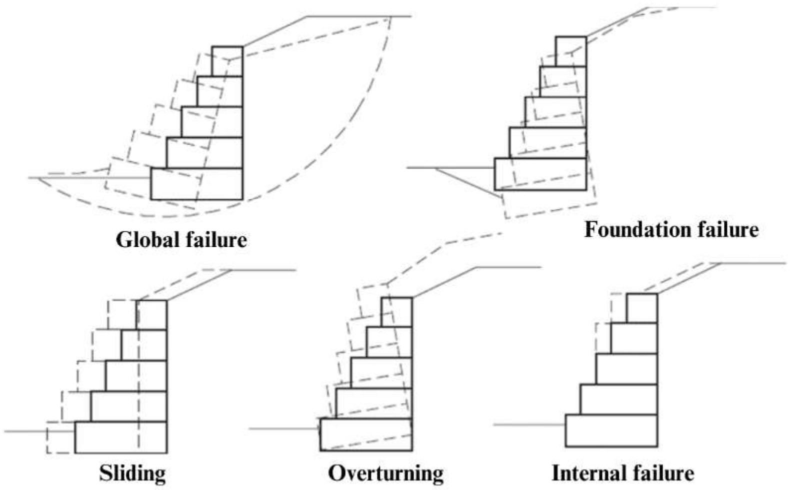


Figure 2 – Gabion wall failures

Fonte: Adapted from Barros (2021)

3 METHOD

The general model to be designed is presented in Figure 3. This figure also shows the values used in the calculations and the illustration of the soil wedges behind the wall.

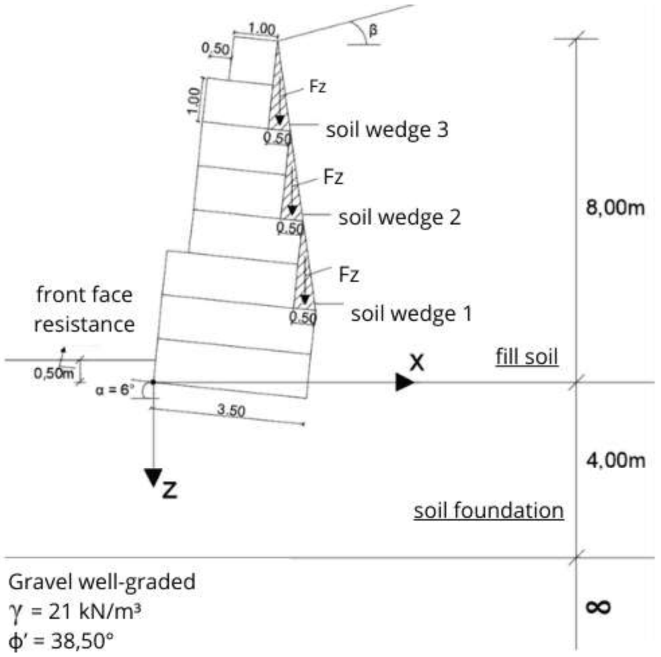


Figura 3 – General model

Source: Author (2024)

The checks regarding earth pressure forces, overturning and sliding, foundation bearing capacity, and, above all, global stability will be presented.

Since the stepped face of the gabion wall is oriented toward the back of the wall, the soil-structure friction angle (δ) was assumed to be equal to the soil's effective friction angle (ϕ'). This is due to the roughness of the structure and the formation of soil wedges along this face.

Because the study focuses on the analysis of soil parameters, the values of the structural parameters and others were standardized and are defined in Table 1.

Table 1 – Standardized parameter values in sizing

Sector	Paramater	Value
Gabion fill	Unit weight (γ)	17 kN/m³
	Internal friction (ϕ)	40°
	Cohesion (c)	0 kPa
Mesh	Mesh tensile strength	40 kN/m
	Connection strength (Rs)	40 kN/m
	Vertical spacing of dividers	1,00 m
Wall inclination	(α)	6°
Foundation bearing capacity	(Rd)	300 kPa
Slope inclination	β	15°

For calculation purposes, it was decided to follow the guidelines set forth in the European Standard EN 1997-1:2010 – Eurocode 7 (EC 7), and the Calculation Approach 1.

Table 2 presents the variation of the soil parameters that were evaluated. In models A.1, A.2, and A.3, the friction angle of the fill soil is varied. In models B.1 and B.2, the cohesion of the foundation soil is altered, and finally, in model C, the granular fill soil is replaced with high or very high plasticity soil, with firm consistency.

Analyses and verifications will be carried out using the "Gabion Wall" and "Slope Stability" modules of the Geo5 (2020) software, modifying the following parameters: backfill soil friction angle, foundation soil cohesion, and changing the backfill soil from granular material to high or highly plastic silt. The earth pressure calculation method used in the Geo5 (2020) software was Coulomb's method (1776).

In the verifications, a minimum safety factor of 1.50 will be considered. The selection of this value must be made carefully by the designer, analyzing the geotechnical characteristics and the specifics of the project. Therefore, in this case, for the verification to be satisfied, the design resistance must be 1.50 times greater than the design load.

For the ultimate limit state verifications, except for slope stability, the coordinate system is defined in Figure 3.

Table 2 – Values of soil parameters used

Model	Fill soil			Foundation soil		
	γ (kN/m^3)	$\phi' = \delta$ (°)	c' (kPa)	γ (kN/m^3)	$\phi'\text{cv} = \delta$ (°)	c' (kPa)
A.1	Granular fill			Clayey sand		
	20	30	0	20	30	10
A.2	Granular fill			Clayey sand		
	20	25	0	20	30	10
A.3	Granular fill			Clayey sand		
	20	35	0	20	30	10
B.1	Granular fill			Clayey sand		
	20	30	0	20	30	20
B.2	Granular fill			Clayey sand		
	20	30	0	20	30	40
C	Silt with high or very high plasticity (firm consistency)			Clayey sand		
	21	17	7	20	30	10

In the verification of the foundation's bearing capacity, the reaction stresses will be considered in a trapezoidal shape, as this offers greater fidelity to reality compared to rectangular ones, which is the other option available in the Geo5 program (2020). The maximum eccentricities must also be satisfied.

The following slice methods for circular surfaces will be used to check slope stability: Bishop (1955), Fellenius/Petterson (1955), Spencer (1967), Janbu (1954), Morgestern-Price (1965). The origin point of the coordinates (X, Z) is at the top rear corner of the gabion wall, with X+ to the right and Z+ upwards.

4 RESULTS

Based on the calculations performed using the Geo5 (2020) software, the results for the weight of the gabion wall, earth pressures, and verification of the ultimate limit states were obtained.

As there was no change in the values of the filling material, the weight and points of application of the gabion wall was constant, according to Table 3.

Table 3 – Weight of the gabion wall

Resultant forces		Coordinates	
Fx (kN/m)	Fz (kN/m)	X (m)	Z (m)
0,00	323,00	1,93	-3,00

The Table 4 shows the result of the earth pressures and the forces due to the soil wedges are shown in Table 5.

Table 4 – Results of earth pressures

Model	Passive earth Pressure				Active earth Pressure			
	Resultant forces		Coordinates		Resultant forces		Coordinates	
	Fx (kN/m)	Fz (kN/m)	X (m)	Z (m)	Fx (kN/m)	Fz (kN/m)	X (m)	Z (m)
A.1	12,23	-4,90	0,02	-0,22	-221,93	177,13	3,36	-2,50
A.2	12,23	-4,90	0,02	-0,22	-287,48	183,61	3,36	-2,50
A.3	12,23	-4,90	0,02	-0,22	-172,13	167,69	3,36	-2,51
B.1	19,40	-8,09	0,02	-0,23	-221,33	176,67	3,36	-2,51
B.2	33,74	-14,48	0,03	-0,24	-221,33	175,76	3,36	-2,51
C	12,23	-4,90	0,02	-0,22	-420,94	164,43	3,42	-2,35

Table 5 - Results of wedge soils

Model	Fz (kN/m)	Soil wedge 1		Soil wedge 2		Soil wedge 3	
		Coordinates		Coordinates		Coordinates	
		X (m)	Z (m)	X (m)	Z (m)	X (m)	Z (m)
A.1 = B.1 = B.2	4,72	3,39	-1,97	3,10	-4,01	2,81	-6,05
A.2	4,88	3,39	-1,98	3,10	-4,02	2,82	-6,06
A.3	4,78	3,39	-1,98	3,10	-4,02	2,82	-6,06
C	7,69	3,41	-2,14	3,12	-4,18	2,83	-6,23

The soil wedges contribute to the stability of the structure, with the force Fz generating moments opposing overturning. The magnitude of the moment depends on the distance from the X-axis at which the force is applied.

The verification of the foundation's bearing capacity is shown in Table 6.

Table 6 - Foundation bearing capacity

Model	Eccentricity		Foundation bearing capacity		Factor of safety (FoS)	Verification
	e_{max}	e	Rd (kPa)	σ (kPa)		Fos ≥ 1,50
A.1	0,333	0,117	300	258,36	1,16	No
A.2	0,333	0,200	300	346,44	0,87	No
A.3	0,333	0,054	300	195,82	1,53	Yes
B.1	0,333	0,114	300	253,27	1,18	No
B.2	0,333	0,107	300	243,05	1,23	No
C	0,333	0,359	300	743,22	0,40	No

Model A.3 was the only one where the FoS was achieved, due to a higher friction angle, which reduced active pressure and eccentricities. Model C, with the same foundation parameters as A.1 but a lower friction angle, exceeded the maximum eccentricity and had the lowest factor of safety (FoS).

Table 7 presents the verification of the overturning and sliding.

Table 7 - Verification of the overturning and sliding

Model	Overturning		Sliding	
	Mres (kNm/m)	Move (kNm/m)	Hres (kN/m)	Hsli (kN/m)
A.1	1262,25	552,45	824,34	155,31
	FoS = 2,28		FoS = 5,42	
A.2	1285,95	714,83	734,35	219,78
	FoS = 1,80		FoS = 3,34	
A.3	1231,23	428,71	921,21	106,75
	FoS = 2,87		FoS = 8,63	
B.1	1260,56	550,84	844,27	147,97
	FoS =2,29		FoS = 5,71	
B.2	1257,20	547,25	848,59	134,47
	FoS =2,30		FoS = 6,31	
C	1257,07	948,88	514,54	353,63
	FoS = 1,28		FoS = 1,46	

Model C was the only one where the required factor of safety was not met; this is due to its lower values for the fill soil parameters.

Table 8 shows the value and verification of the critical circular slip surfaces for the models presented.

Table 8 – Result and verification of the critical circular slip surface

Model	Method	Critical circular slip surface			Factor of safety (FoS)	Verification
		x (m)	z (m)	R (m)		FoS ≥ 1,50
A.1	Bishop	-3,12	4,74	13,37	1,51	Yes
	Fell/Pett	-3,27	4,21	12,92	1,29	No
	Spencer	-3,08	3,7	12,38	1,51	Yes
	Janbu	-3,15	4,6	13,25	1,51	Yes
	Morg-Pric	-3,15	4,6	13,25	1,51	Yes
A.2	Bishop	-3,14	4,55	13,2	1,37	No
	Fell/Pett	-3,09	4,39	13,03	1,18	No
	Spencer	-2,78	6,91	15,39	1,39	No
	Janbu	-2,54	9,51	17,89	1,42	No
	Morg-Pric	-2,54	9,51	17,89	1,42	No
A.3	Bishop	-3,18	3,06	11,79	1,66	Yes
	Fell/Pett	-3,24	2,43	11,23	1,38	No
	Spencer	-3,10	2,67	11,4	1,63	Yes
	Janbu	-3,18	3,06	11,79	1,63	Yes
	Morg-Pric	-3,18	3,06	11,79	1,63	Yes
B.1	Bishop	-2,59	5,16	13,65	1,69	Yes
	Fell/Pett	-2,91	2,76	11,43	1,44	No
	Spencer	-2,58	6,39	14,84	1,70	Yes
	Janbu	-2,21	7,93	16,29	1,70	Yes
	Morg-Pric	-2,19	7,75	16,1	1,70	Yes
B.2	Bishop	-2,52	4,9	13,38	2,01	Yes
	Fell/Pett	-2,12	3,94	12,35	1,72	Yes
	Spencer	-2,13	4,96	13,36	1,98	Yes
	Janbu	-2,25	5,36	13,77	1,98	Yes
	Morg-Pric	-2,57	5,06	13,55	1,99	Yes
C	Bishop	-2,59	5,16	13,65	1,23	No
	Fell/Pett	-2,57	5,06	13,55	1,11	No
	Spencer	-3,33	7,75	16,34	1,27	No
	Janbu	-3,33	7,75	16,34	1,30	No
	Morg-Pric	-3,33	7,75	16,34	1,30	No

The visual comparison of these values, using the optimized method in the program, helps to understand the influence of the parameters. The color scale used represents the factor of safety.

Table 9 presents the figures related to the slope stability verification for models A.1, A.2, and A.3, while Table 10 presents the figures for models B.1, B.2, and C.

Table 9 – Comparison between the figures in models A, A.1 and A.2

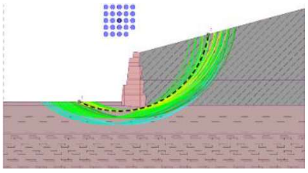
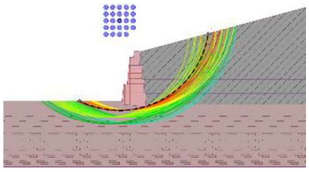
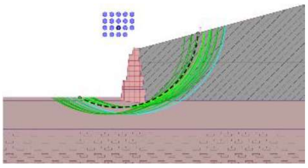
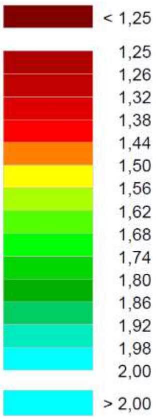
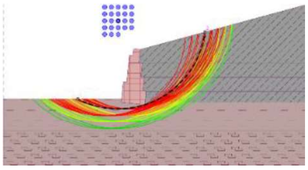
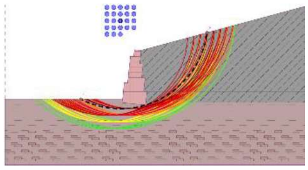
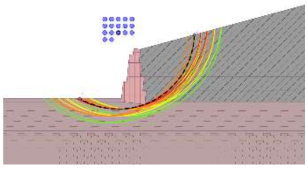
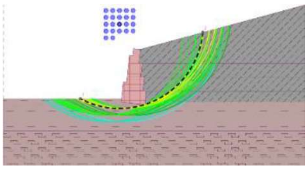
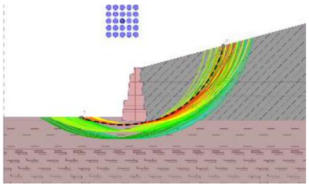
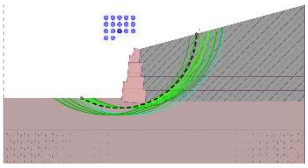
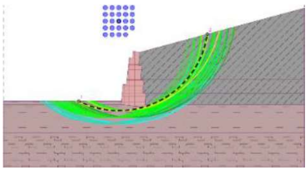
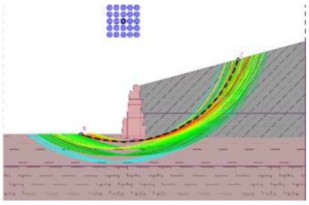
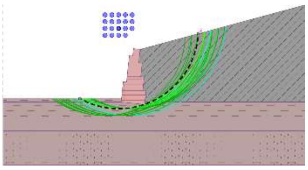
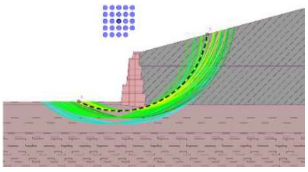
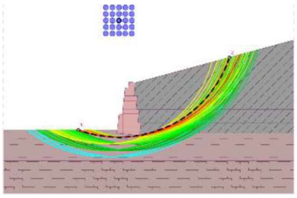
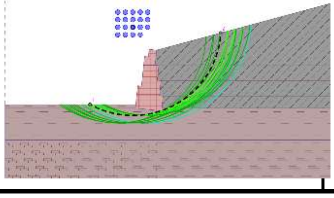
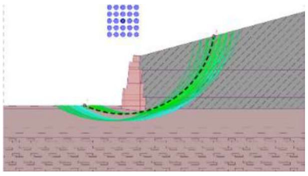
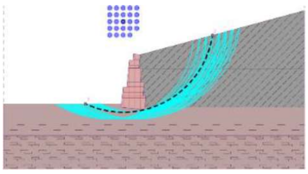
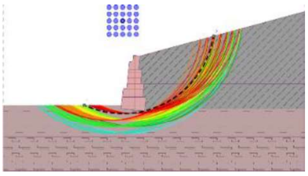
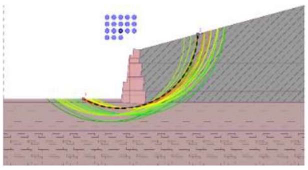
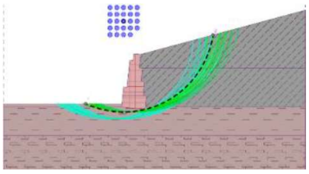
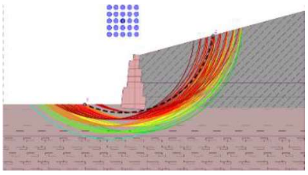
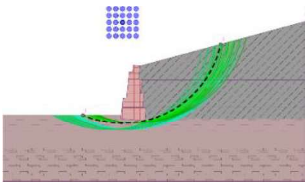
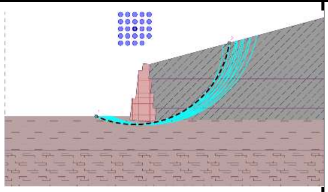
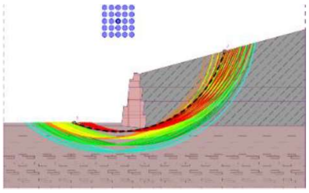
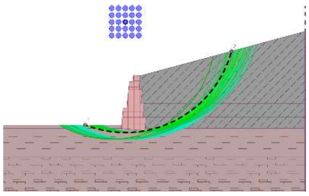
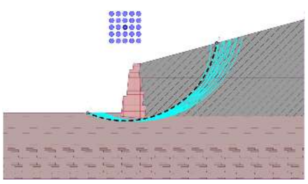
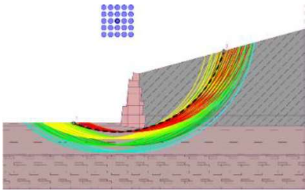
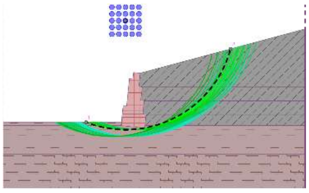
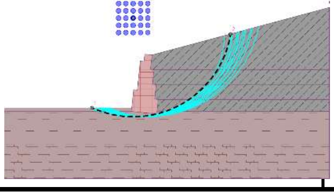
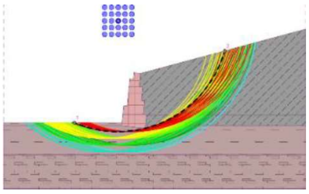
Method	Model			Scale (FoS)
	A.1	A.2	A.3	
Bishop				
Fell/Pett				
Spencer				
Janbu				
Morg. & Price				

Table 10 – Comparison between the figures in models B.1, B.2 and C

Method	Model			Scale (FoS)
	B.1	B.2	C	
Bishop				
Fell/Pett				
Spencer				
Janbu				
Morg. & Price				

It is possible to observe the importance of soil shear strength, as defined by the Mohr-Coulomb failure criterion. In models A, A.1, and A.2, the influence of the soil's friction angle was observed, where higher values of this parameter favor the stability of the structure. This occurs because the higher the friction angle, the less prone the soil is to deformation.

Models B.1 and B.2 present the same value as A.1 for backfill soil parameters, but those models have a higher cohesion value for foundation soil. It is evident that this increase in cohesion enhances stability, which is explained by the Mohr-Coulomb failure criterion.

Model C differs from the others, as in this case, no method met the required FoS (FoS ≥ 1.50). This is due to a lower friction angle in the backfill soil compared to the others, highlighting the relevance of this parameter, which is related not only to soil shear strength but also to the soil-structure friction angle (δ).

Fellenius/Petterson (1955) method proved to be conservative, with safety factors ranging from 10.81% to 20.34% lower than the others. This can be noted in models A.1, A.2, and B.1, where the FoS verification was not achieved only by this method.

5 CONCLUSIONS

From the design process and the presentation of the results, conclusions regarding the backfill and foundation soil parameters in gabion walls can be drawn.

The friction angle of the backfill soil can be considered one of the predominant parameters in gabion walls, influencing both the active earth pressure and the foundation demands.

Soil wedges provide an important connection between the backfill and the structure, with their contribution depending on the geometry adopted in the design.

The factor of safety for sliding was lower than that for overturning in all cases. Therefore, generally, if the structure shows favorable results for overturning, it will also likely meet sliding criteria. However, both checks should always be performed.

In global stability analysis, the Fellenius/Petterson method (1955) consistently yielded lower FoS values, which was expected given its conservative nature. The methods of Spencer (1967), Janbu (1954), and Morgenstern & Price (1965), despite differences in their formulations regarding adopted simplifications, produced similar results.

The foundation soil properties influenced the global stability results, with models featuring superior soil properties showing more satisfactory outcomes.

Proper backfill soil compaction is crucial for the structure's effectiveness, alongside testing to obtain accurate parameter values, ensuring a design that reflects the site conditions.

The objectives were achieved, as the variation in soil parameters allowed the observation of the impacts caused by these changes. Regarding global stability, differences and similarities between the various methods were noted, highlighting which produced the best results.

This study did not include the presence of water or dynamic analysis; therefore, future work should analyze the influence of saturated soils and seismic activity in this context.

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