

Article

Impact of Rock Mass Strength Anisotropy with Depth on Slope Stability Under Excavation Disturbance

Gulseren Dagdelenler

Applied Geology Division, Geological Engineering Department, Faculty of Engineering, Hacettepe University, Beytepe, 06800 Ankara, Türkiye; gulsrn@hacettepe.edu.tr

Abstract: In open-pit excavations, overburden rock mass is disturbed by processes like blasting and mechanical excavation, leading to a reduction in mechanical properties. Accounting for this disturbance is essential for ensuring slope stability, optimizing costs, and maintaining feasibility. The Hoek–Brown failure criterion, a widely used empirical method in rock mechanics, incorporates the disturbance factor to reflect the reduction in rock mass strength after disturbance. This study reviews five approaches from the literature regarding the role of disturbance in rock mechanics, focusing on its impact on the factor of safety and the volume of rock mass above the potential failure surface. Additionally, an “S” shaped decay formulation was proposed as an alternative to existing equations. A key consideration is the transitional disturbance effect, which reflects the gradual change from a fully disturbed rock mass near the excavation surface to an undisturbed rock mass with increasing depth. Among the examined approaches, the “S” shaped decay equation, informed by insights from previous studies, appears to be the most realistic. One approach assumes the disturbance factor is highest at the surface due to the removal of blasted rock, leading to a fully disturbed rock mass in front of the excavation face. The disturbance then decreases with depth, transitioning to an undisturbed condition depending on the excavation method. Even when the rock mass is homogeneous and isotropic in joint properties, excavation induces anisotropy in mass strength, causing overall strength to increase with depth. This study also investigates the effect of anisotropic strength behavior resulting from the disturbance factor. For incorporating transitional disturbance in the design stage, both circular and combined failure mechanisms should be considered for a comprehensive understanding of slope stability.



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Keywords: disturbance; disturbance factor; the Hoek and Brown failure criterion; slope stability; anisotropy

1. Introduction

It has become economically possible to operate deeper ore deposits and coals with open pit mining due to technological advances in the excavation process. Depending on the rock masses characteristics of the geologic unit overly on ore deposits or coals, different excavation methods from mechanical excavation to effective blasting are widely used. After disturbance, while the aperture of existing discontinuities increases, new cracks may also be developed due to the method of excavation. As a result of these inverse changes in rock mass properties due to the excavation, the overall mechanical properties, in terms of strength and deformation, may decrease compared to their undisturbed condition.

As is known, it is almost impossible to prepare undisturbed rock mass samples that preserve their in situ conditions including discontinuity patterns. Even if such undisturbed

samples can be prepared, the equipment to perform the related laboratory investigations is generally not commonly available in conventional rock mechanics laboratories. To overcome these difficulties, researchers have focused on empirical tools for designing engineering projects involving rock masses, especially after the early 1950s [1–5]. Due to the simple and ease accessible input parameters used in the Hoek and Brown empirical strength criterion, it has been preserved its popularity in rock mechanics since it was introduced to the rock mechanics literature in 1980. By their nature, empirical tools are continuously updated and modified by considering new data and new cases. Due to its openness to the improvement of the empirical tools, the disturbance factor ranging a value from undisturbed to completely disturbed rock mass condition was first introduced to the literature by Sonmez and Ulusay [6]. Their approach was subsequently adopted to the criterion by Hoek and co-authors as originators of the criterion who incorporated the disturbance factor into their criterion, with classification ranging from an undisturbed ($D = 0$) to a completely disturbed ($D = 1$) rock mass [4]. Hoek [7] offered a technical note outlining recommendations for selecting and applying the disturbance factor in various projects. The degree of disturbance naturally decreases to the undisturbed rock mass condition from the excavation surface to a certain depth depending on the excavation method. Due to difficulties in collecting real data, especially in terms of failed rock mass cases, there are few studies from the past to the present on the effects of transitional rock mass disturbance from surface to depth on the stability of slope excavation and the location of potential failure surfaces [8–14]. These studies have provided important contributions to the perspectives on the use of transitional rock mass disturbances. On the other hand, even if a rock mass is homogeneous and isotropic in terms of joint properties, it will exhibit anisotropic mass strength after an excavation disturbance, which causes the overall strength to increase from the surface to the depth. An important problem to be overcome for the use of the disturbance factor is determining how to use it from the excavation surface to the depth, considering the method of excavation.

In this study, an “S” shaped decay formulation was proposed to define and describe the reduction in a disturbance in a rock mass. More importantly, this study also aims to address the uncertainties surrounding the use of the transitional rock mass disturbance factor by considering anisotropic strength behavior due to the disturbance effect and offers practical recommendations for its effective application during the design phase.

2. Disturbance Concept in the Hoek and Brown Failure Criterion

The excavatability of a rock mass is closely related to the intact rock strength and the overall rock mass strength together with its discontinuity properties. Many studies in the rock mechanics literature discuss the excavatability of rock masses [8,15–18]. While mechanical excavation methods such as digging with excavators and ripping can be used to excavate weak rock masses, as the strength of the rock material and the quality of the rock mass increases, there is a need to loosen the rock mass in situ and reduce it to smaller block sizes through blasting.

Clearly, the degree of disturbance due to every different technical process, such as digging, ripping, and blasting, can vary in a rock mass. After disturbance of the rock mass, depending on the excavation method, the apertures of the existing discontinuities increase (the loosening of the rock mass) and the block size decreases with the development of new cracks. Dinc et al. [19] schematically illustrated the concept of disturbance together with the effect on rock mass strength as shown in Figure 1a–d.

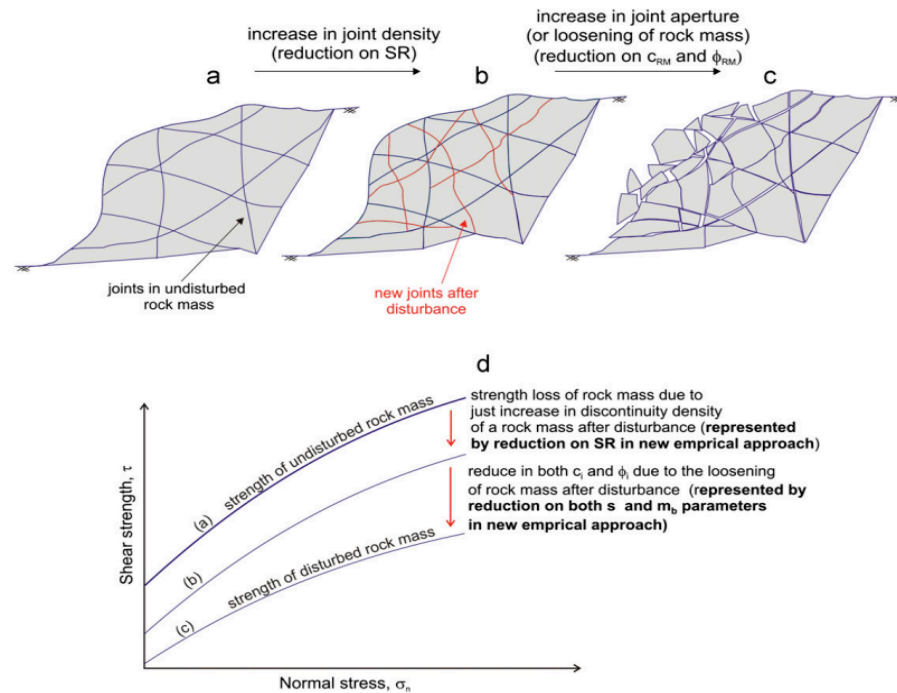


Figure 1. Schematic representation of disturbance effects on rock masses: (a) in situ undisturbed rock mass, (b) illustration of new crack development, (c) losing of rock mass, and (d) illustration of disturbance effects on shear strength envelope [19].

During the period when the Rock Mass Rating (RMR) was used to assess the quality of rock masses within the Hoek–Brown failure criterion, two separate equations were recommended for determining the m and s constants—one for disturbed and one for undisturbed rock masses (Equations (1a)–(1e)). However, this approach posed a limitation in accurately defining varying degrees of disturbance.

$$\sigma_1 = \sigma_3 + \sqrt{m\sigma_3\sigma_{ci} + s\sigma_{ci}^2} \quad (1a)$$

For disturbed rock mass:

$$\frac{m}{m_i} = \exp\left(\frac{RMR - 100}{14}\right) \quad (1b)$$

$$s = \exp\left(\frac{RMR - 100}{6}\right) \quad (1c)$$

For undisturbed rock mass:

$$\frac{m}{m_i} = \exp\left(\frac{RMR - 100}{28}\right) \quad (1d)$$

$$s = \exp\left(\frac{RMR - 100}{9}\right) \quad (1e)$$

The Geological Strength Index (GSI) was subsequently incorporated into the Hoek and Brown failure criterion in place of the Rock Mass Rating (RMR), due to RMR's inefficiency in rating very weak rock masses ($RMR < 25$).

Hoek et al. [20] replaced GSI with RMR_{76} (1976 version of ratings of RMR parameters), which varies between 0 and 100, in their update, without providing any information about the disturbance condition, and used the relations related to the undisturbed rock mass condition from the 1988 version of the criterion.

The concept of degree of disturbance by using a continuous relationship from an undisturbed rock mass to a completely disturbed rock mass was firstly introduced to the Hoek and Brown criterion by Sonmez and Ulusay [6]. To achieve this aim, a reduction factor, namely, a disturbance factor (D_f) ranging from 0.8 for a completely disturbed condition to 1 for an undisturbed condition, was used in the denominator of the m and s equations to join continuously undisturbed to completely disturbed condition, as given in Equations (1b)–(1e). After this modification by Sonmez and Ulusay [6], it became possible to account for different degrees of disturbance, from basic mechanical excavation to uncontrolled blasting.

Similar to the innovative disturbance degree approach proposed by Sonmez and Ulusay [6], a disturbance factor (D) ranging from 0 to 1 for an undisturbed and completely disturbed condition, respectively, was also adopted in the criterion by Hoek et al. [4]. The latest form of the criterion as presented by Hoek et al. [4] is given in Equations (2a)–(2d).

$$\sigma'_1 = \sigma'_3 + \sigma_{ci} \left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (2a)$$

$$m_b = m_i \exp \left(\frac{GSI - 100}{28 - 14D} \right) \quad (2b)$$

$$s = \exp \left(\frac{GSI - 100}{9 - 3D} \right) \quad (2c)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right) \quad (2d)$$

3. Approaches to the Use of Disturbance Factor on Rock Slope Excavations

As mentioned above, technical processes, such as mechanical excavation or blasting, used for the excavation of rock masses cause a decrease in the strength of rock masses compared to in the undisturbed condition due to the increase in discontinuity apertures and the development of new cracks. The latest version of the Hoek and Brown criterion, introduced by Hoek et al. [4], was used in this study.

A practical guideline for the selection of disturbance factor (D) was also defined by Hoek et al. [4]. In the guideline, the lower value of “ D ” for slope case was recommended as 0.7. It means that, except natural slopes, the undisturbed rock mass condition cannot be achieved even if mechanical excavation methods such as digging or ripping are used in slope cases. Probably underlying this suggestion is the foresight that a rock mass can be loosening close to the excavation surface due to the removal of overburden, which may also cause disturbances. Although it is possible that the disturbance factor can reflect the degree of disturbance to the rock mass by using the disturbance factor depending on the excavation method, uncertainty about how the disturbance will change with depth, especially for deep excavation slopes, still remains in the literature.

After the introduction of the disturbance factor to the criterion, only a few studies have addressed the use of the disturbance factor depending on degree of disturbance for especially deep slope excavations. Within the scope of this study, the five approaches given below were taken into consideration. In addition to the recent modification to Rose et al. [10] by Sonmez et al. [14], an “ S ” shaped decay formulation proposed in this study was also applied to the same slope profile, along with a comparison of the five approaches. In the subsequent section, the impact of rock mass strength anisotropy with depth on slope stability under excavation-induced disturbance was assessed through slope stability analysis.

The approaches used in this study, together with an “S” shaped decay equation proposed in this study, are explained as follows.

- i The first approach (AP1): a disturbance factor (D) independent from depth was applied to the entire rock mass of the analyzed slope cross section.
- ii The second approach (AP2): Conditions for blasted rock that can be excavated by digging efficiently; however, the blast-damaged rock mass lies between the digging limit and the insitu rock mass. The properties of this blast damaged rock mass will control the stability of the slope that remains after digging of blasted rock has been completed [8]. Hoek and Karzulovic [8] defined the following approximate relationships which can be used as a starting point in judging the extent (thickness, T) of the blast damaged zone resulting from open-pit mine-production blasting:
 - Large production blast, confined and with little or no control: $T = \text{from } 2 \text{ to } 2.5 H$.
 - Production blast with no control but blasting to a free face: $T = \text{from } 1 \text{ to } 1.5 H$.
 - Production blast, confined but with some control, e.g., one or more buffer rows: $T = \text{from } 1 \text{ to } 1.2 H$.
 - Production blast with some control, e.g., one or more buffer rows, and blasting to a free face: $T = \text{from } 0.5 \text{ to } 1 H$.
 - Carefully controlled production blast with a free face: $T = \text{from } 0.3 \text{ to } 0.5 H$.
- iii The third approach (AP3): Rose et al. [10] proposed a quantified approach for determining transitional rock mass disturbance in open pit slopes related to mining excavation. The researchers accepted that the disturbance gradually decreases from the excavation surface to the depth. In this approach, it is assumed that the rock mass is fully disturbed depending on a specific depth from the surface (FDL), and the degree of disturbance gradually decreases to match the in situ rock mass (undisturbed) condition from the FDL to the depth of the undisturbed level (UDL). In our study a schematic representation of the input parameters given in Figure 2 was prepared to clarify the terminology used by Rose et al. [10]. While the disturbance factor is equal to one ($D = 1$) from the excavated surface to the depth of the FDL, Equations (3a) and (3b) were proposed by Rose et al. [10] for a gradual decrease of D from 1 to zero.

$$D = \frac{\ln(H) - \ln(H_i)}{\text{decay}} \quad (3a)$$

$$\text{decay} = -\left(\frac{FDL}{H}\right) \quad (3b)$$

where D: Hoek et al.'s [4] the disturbance factor, H_i : increase in slope height, H (or UDL): general slope height, FDL: total disturbance depth.

Rose et al. [10] proposed a classification, namely, the Rock Mass Disturbance Rating (DR in percentage), which includes nine input parameters to determine the depth of the fully disturbed zone (FDL) from the excavation surface. The Rock Mass Disturbance Rating proposed by Rose et al. [10] is openly accessible on the web. The FDL can be estimated by multiplying the DR with the UDL. Among the parameters in the DR classification, particularly the structural elements such as “Fore dipping structure”, “Back dipping structure”, and “Perpendicular structure”, it can be argued that using these parameters in the rating aimed at determining the depth of the fully disturbed zone (FDL) may be debatable. This is because these parameters are related to the anisotropic behavior of the rock mass. However, as is well known, the Hoek and Brown criterion is only applicable to the isotropic rock mass condition. Therefore,

in this study, a rating of “0” was preferred for these parameters under the assumption that they are “not present”.

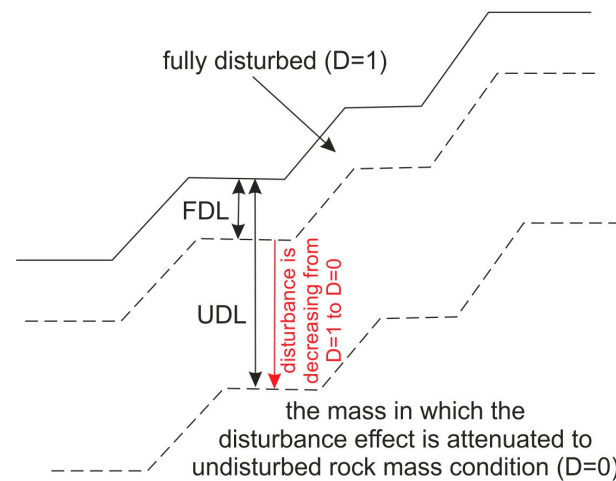


Figure 2. A schematic representation of the input parameters prepared in this study to clarify terminology used in transitional rock mass disturbance.

- iv The fourth approach (AP4): Rose et al.’s [10] approach can be considered as a much logical approach due to considering the attenuation of disturbance with distance of the blasting effect within the rock mass. However, like any empirical approach, surely it is open to improvement depending on new cases and ideas. Although the empirical approaches are very practical tools in engineering applications, they should produce reliable outputs within expert engineering acceptance. It is well-known that when D is equal to one, the Hoek and Brown criterion yields almost cohesionless mechanical properties. By considering this fact, Sonmez et al. [14] recommended a minor improvement to Rose et al.’s [10] approach by combining it with Hoek and Karzulovic’s [8] approach. While Rose et al. [10] proposed the use of a fully disturbance factor as $D = 1$ for FDL zone, Sonmez et al. [14] considered $D = D_f$ due to removal of blasted rock (fully disturbed rock mass) by digging in front of the blasted, damaged rock. In addition, FDL was replaced with D_fL (disturbed rock mass limit) to avoid terminological confusion, as suggested by Sonmez et al. [14].
- v The fifth approach (AP5): Park and Michalowski [21] proposed a continuous a decay equation (Equation (4)) from excavation surface to depth to allow flexibility in describing the rate of disturbance decay.

$$D(l) = \begin{cases} D_0 \left[1 - \left(\frac{l}{t} \right)^k \right] & \text{for } (l \leq t) \\ 0 & \text{for } (l > t) \end{cases} \quad (4)$$

As defined word-for-word in Park and Michalowski [21], where l is the distance into the slope measured orthogonally away from the slope surface, D_0 which is the same as D_f , is the maximum disturbance factor on the slope surface, t is the thickness of the disturbed zone, and k is the power of disturbance decaying rate. When $k = 1$, Equation (4) describes a linear disturbance decrease, whereas exponential-type or logarithmic-type decaying can be considered by adjusting the k value to be larger or smaller than one, respectively.

- vi The sixth approach (AP6): On the other hand, AP3 and AP4 define a maximum disturbance factor from the surface to a certain depth; below this zone, a transitional rock mass disturbance was defined by decreasing degree of damage from surface

to depth. However, it is an acceptable reality that the excavation disturbance of a rock mass decreases gradually from the surface to a depth. Similarly, with this inference, as can be seen from a graphical presentation given by Rose et al. [10], the yielded rock mass in percentage versus depth graph for both the southeast wall of the HVC Lornex Pit and northeast of the HVC Valley Pit exhibit an inverse “S” shaped trend. In this study, an inverse “S” shaped equation was proposed to define the continuous decrease in transitional rock mass disturbance from the surface to a depth (Equation (5)). The structure of Kalender et al.’s [22] equation was used in the derivation of the equation. In the developed inverse “S” shaped equation degree of decreasing in rock mass disturbance with depth is less until DfL (disturbed rock mass limit); after DfL, the decreasing trend of the transitional rock mass disturbance is similar to the exponential trend.

$$D(z_n) = D_f \left(1 - \frac{50}{50 + k^{(100-100z_n)/15}} \right) \left(\frac{100z_n}{100z_n + 1} \right) \quad (5a)$$

$$k = 1.8254e^{2.8249DfL_n} \quad (5b)$$

$$z_n = z/UDL \quad (5c)$$

where D_f is the maximum disturbance factor on the face of the excavation, DfL_n is the DfL (disturbed rock mass limit) normalized to UDL , z is the depth from excavation surface, z_n is the normalized depth (z) to UDL , and the k value determined with DfL is related to the extremum point of the developed equation, which satisfies the yielded rock mass of $\sim 90\% \pm 5$, consistent with the southeast wall of the HVC Lornex Pit and northeast of the HVC Valley Pit cases studied by Rose et al. [10]. In order to obtain reasonable inverse S shaped curves, k should not be taken to be less than 3.

For the approaches listed above, representative graphical presentations depending on the normalized depth to UDL are given in Figure 3 in order to understand them well. The approaches for the use of transitional disturbance factors consider a determination of disturbance factors for every slice above the failure surface by considering a depth of slice from the bottom surface of the slope.

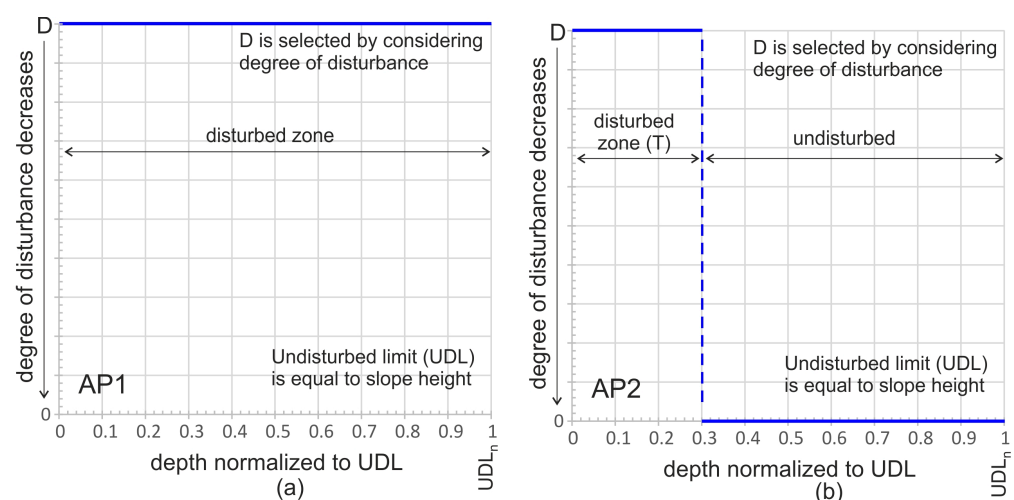


Figure 3. Cont.

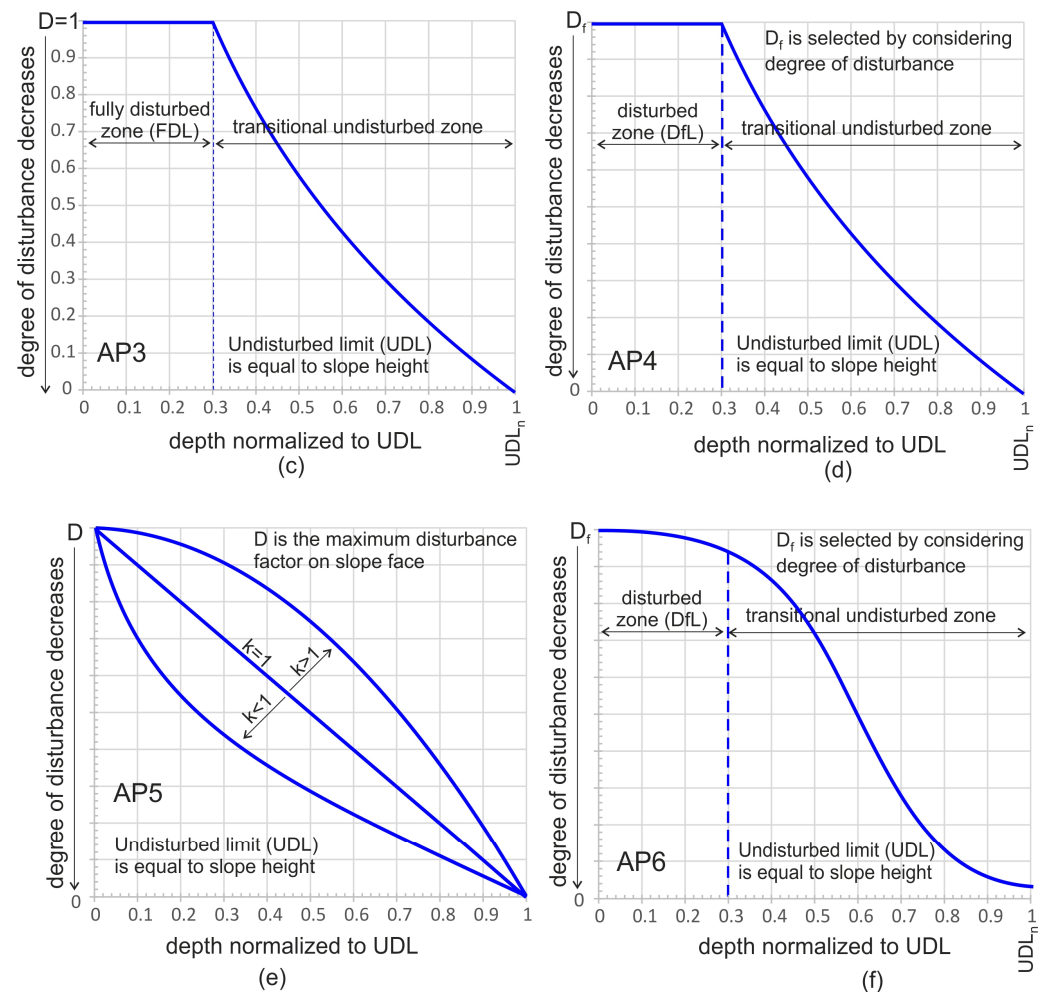


Figure 3. Representative graphical presentations of the approaches (a) AP1, (b) AP2, (c) AP3, (d) AP4, (e) AP5 and (f) AP6 depending on normalized depth to UDL.

4. Slope Stability Evaluations of a Hypothetical Slope Model

As is well known, the relationship between the factor of safety (FOS), slope angle (β), and slope height (H) needs to be investigated to ensure a safe, applicable, and economical slope design. Even if the characterization of a rock mass has been well studied and the necessary input parameters have been obtained with high quality, the strength reduction due to the disturbance effect should be reliably included in the analysis, especially for deep slope excavations. However, as mentioned above, there is no widely accepted approach for the use of disturbance effect in the design of slope excavations. Therefore, the effects of the six approaches discussed on the stability of the hypothetical slope models were investigated using a slope stability analysis for circular-type mass failure.

As commonly observed in Türkiye and similar geological regions, Neogene-aged formations, such as lignite coal and borate mines, may be overlain by jointed rock masses such as limestone, marl, and mudstone (Figure 4). By considering the actual in situ view given in Figure 4, in order to reflect the moderate and weak strength jointed rock masses in the realistic hypothetical slope model, the presence of a 50 m thick mudstone rock mass overlying a limestone rock mass with a 50 m thickness was considered. On the other hand, another advantage of the use of these two rock masses with different rock mass strengths is that they are exposed to different degrees of disturbance depending on their excavatabilities together with the excavation method used. The assumed geomechanical properties of the selected limestone and mudstone rock masses are summarized in Table 1. Additionally,

to minimize the influence of independent variables on slope stability assessment, only a drained slope case was considered in this study.

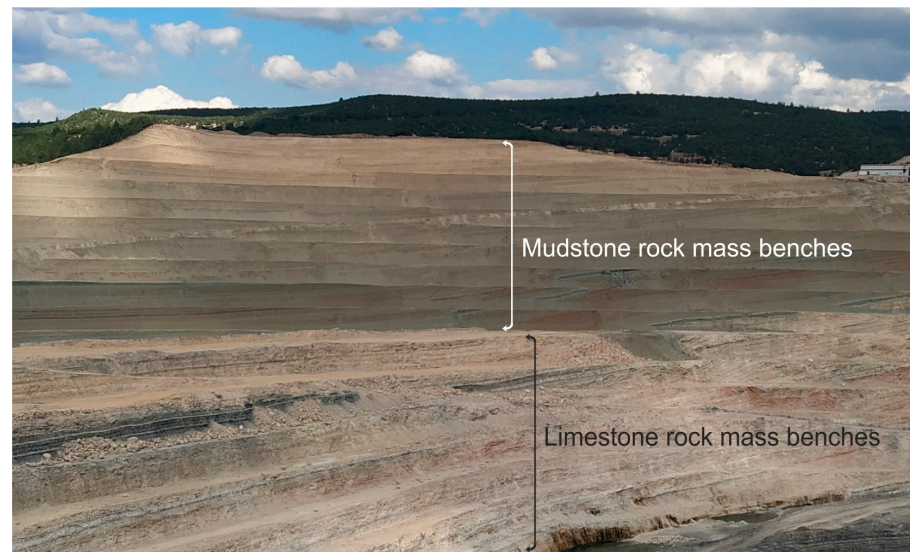


Figure 4. A view from an open pit mine slope benches of mudstone overlies a limestone rock mass (photo taken by H. Sonmez, 13 September 2022).

Table 1. The assumed geomechanical properties of the limestone and mudstone rock masses (the values of m_i were selected from Hoek's recommendations based on type of rock material).

Rock Mass	UCS (MPa)	GSI	m_i	γ (kN/m ³)
Limestone	75	45	10	25
Mudstone	5	30	7	20

As shown in Table 1, while the excavation of overburden could be performed by blasting the limestone rock mass, mechanical excavation with excavators could be sufficient for the mudstone rock mass. By considering this fact, the disturbance factor beyond the digging limit was selected as 0.9 and 0.7 for the limestone and mudstone rock masses, respectively.

The evaluation of disturbance zones from the surface to the undisturbed rock mass condition, based on the four approaches, was considered in this study. For this purpose, a bench angle of 50° and a bench height of 10 m was used, while the overall slope angle was 31.4° for five grouped benches of the two rock masses. A counter-upslope bench width on the boundary between the two rock masses was selected as 20 m, resulting in an overall slope angle of the hypothetical slope model of 28.5° (Figure 5).

In the hypothetical slope model presented in Figure 2, two critical points were defined for use in the slope stability analysis. The first critical point was located at the toe of the overall slope profile on the lower limestone bench, while the second critical point was at the toe of the lower mudstone bench (see Figure 5). In other words, the stability of the hypothetical slope profile was evaluated by considering potential failure circles that pass through only these two points. While the circular type analyses were performed in accordance with the simplified Bishop [23] method, the overall strength of two rock masses was defined according to Hoek et al. [4]. The slope was assumed to be in drained conditions, with no seismic events considered to minimize variable influences.

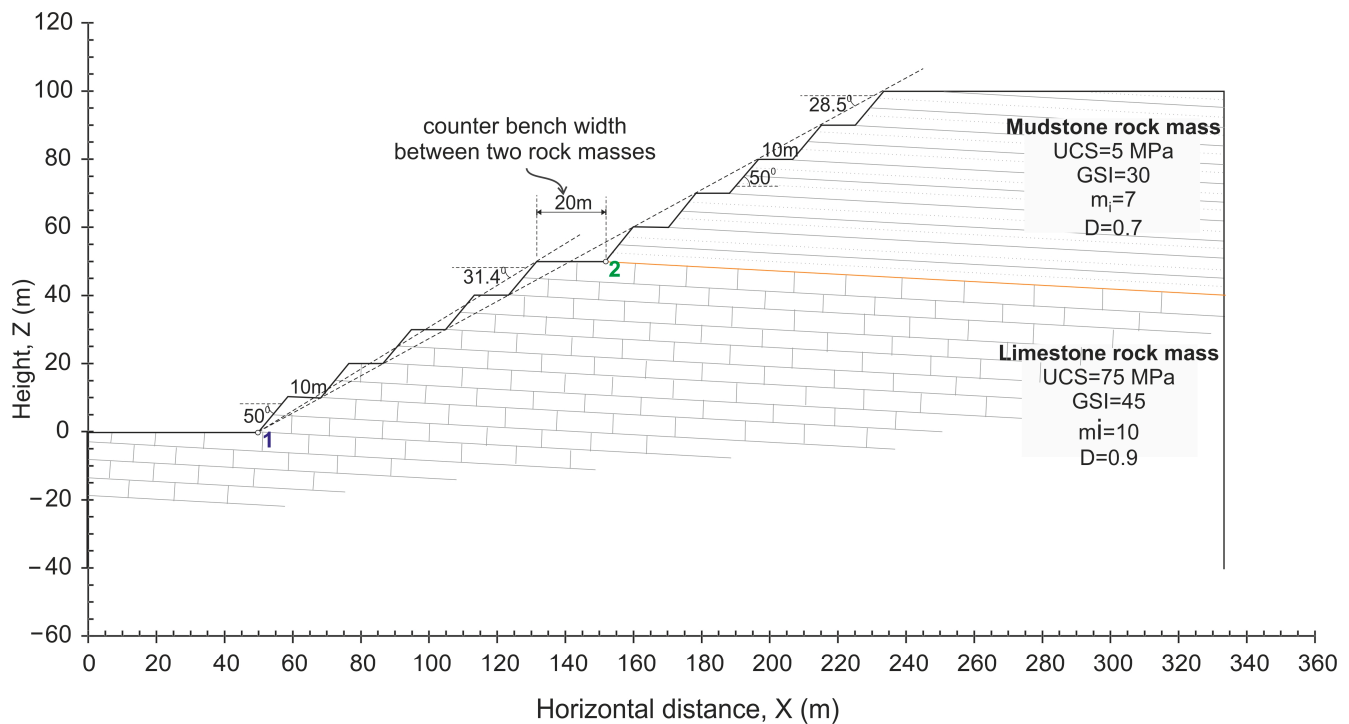


Figure 5. Hypothetical slope model used in the slope stability evaluations.

In the first approach (AP1), the disturbance factors selected for the limestone and mudstone rock masses ($D = 0.9$ and $D = 0.7$, respectively) were applied to the entire rock mass. Hoek and Karzulovic [8] defined three rock mass groups after blasting, considering the degree of disturbance from the surface toward the depth, such as blasted rock, blast-damaged rock, and in situ rock. In the second approach (AP2), only the blast-damaged rock (disturbed rock mass condition, $D = 0.9$ and $D = 0.7$ for limestone and mudstone rock masses, respectively) overlying in situ rock (undisturbed rock mass condition, $D = 0$) was used in the hypothetical slope model. This choice is based on the researchers' suggestion that the blasted rock would be excavated and removed after blasting operations. These operations were considered to lie between “large production blast, confined and with little or no control: $T = 2$ to $2.5 H$ ” and “production blast with no control but blasting to a free face: $T = 1$ to $1.5 H$ ”, according to Hoek and Karzulovic's [8] recommendations. Therefore, the thickness of the blast-damaged (disturbed) limestone rock mass (T) remaining above the undisturbed rock mass after excavation of the blasted rock mass was evaluated as equal to $2 H$. mass, $T = 0.5$ was preferred, considering the ease of excavation by ripper and/or digging. Accordingly, when a bench height equal to 10 m was considered, the T values for limestone and mudstone rock masses were determined to be 20 m and 5 m, respectively.

The transitional disturbance of the rock mass proposed by Rose et al. [10], which is the third approach (AP3), defines a fully disturbed rock mass condition ($D = 1$) to a depth of FDL and assumes that the disturbance effect attenuates to an undisturbed rock mass ($D = 0$) within a depth of UDL. While UDL is assumed to be equal to the height of the overall slope profile, FDL can be calculated as a percentage of the height of the overall slope profile based on the rock mass properties rated in the rock mass disturbance rating (DR) classification proposed by Rose et al. [10]. The DR values for the limestone and mudstone rock masses were 15.5 and 7.5, respectively, according to the rock mass disturbance rating evaluation given in Table 2. While the UDL used was 100 m, considering the height of the overall slope profile, the FDL values for the limestone and mudstone rock masses were determined as 15.5 m and 7.5 m, respectively, by using the DR values and the height of the overall slope profile. The last approach used in this study was a modification of Rose et al.'s [10] method

by Sonmez et al. [14]. The D_f values for the limestone and mudstone rock masses were 0.9 and 0.7, respectively, with the same thicknesses of disturbed rock masses (DFL) from the surface, 15.5 m for limestone and 7.5 m for mudstone, as determined by Rose et al. [10]. As explained above, the value of k as a constant in the equations proposed by Park and Michalowski [21] (AP5), and derived in this study (AP6), is used to control the shape of the decay curve. In AP6, while the k values for limestone and mudstone were determined by using the DFL, which is equal to 15.5 m and 7.5 m, respectively, the UDL was taken as 100 m, the same as with AP3 and AP4. The value of k in AP5 is considered to be 1 and 0.5 for limestone and mudstone after matching the curve, as much as possible, to the one used in AP6.

Table 2. Rock mass disturbance rating evaluation for limestone and mudstone masses (ratings with their descriptions of rock mass in DR classification proposed by Rose et al. [10] are available on the website of <https://www.researchgate.net> (accessed on 1 October 2024).

Parameter	Limestone Rock Mass		Mudstone Rock Mass	
	Description	Rating	Description	Rating
RMR (or GSI)	GSI = 45	5	GSI = 30	10
UCS (MPa)	75	3	5	12.5
Fore dipping structure	Not available	0	Not available	0
Back dipping (toppling) structure	Not available	0	Not available	0
Perpendicular (release) structure	Not available	0	Not available	0
Groundwater pressure	None	0	None	0
Controlled blasting	Not free faced—production	7.5	Mechanical excavation	−15
Confinement	Straight	0	Straight	0
Total rating		15.5		7.5
In situ stress ratio factor (k_0)	Not determined	1		1
Rock mass disturbance rating, DR		15.5		7.5

The factors of safety determined for criteria points 1 and 2 on the slope profile with the potential failure circles passing upwards along the slope profiles were given in Figures 6–11 for the six approaches considered in this study.

The factors of safety determined for the drained condition and the static case by using the four approaches to the potential instability passing through the critical point 1 varies between 2.791 and 3.403 depending on the considered approach (Figure 12a). The factors of safety obtained from AP1 and AP3 were close, at 2.791 and 2.91, respectively, while higher factors of safety, above 3, were determined for AP2, AP4, AP5, and AP6.

When the determined FOS values for critical point 2, which is important in terms of evaluating the stability of the mudstone, are evaluated, it is seen that the results are similar to the results belonging to critical point number 1. However, it is notable that the lowest safety coefficient was obtained with AP3 (Figure 12a). In addition to the drained condition, stability evaluations were also conducted when the pore water pressure ratio (r_u) was set to $r_u = 0.2$ (Figure 12b). Although the factors of safety decreased due to the pore water pressure, the distribution of the factors of safety based on the considered approaches remained similar to those in the drained condition. However, the factors of safety decreased to lower than the limit equilibrium conditions for AP1 and AP3 when critical point 2 was considered.

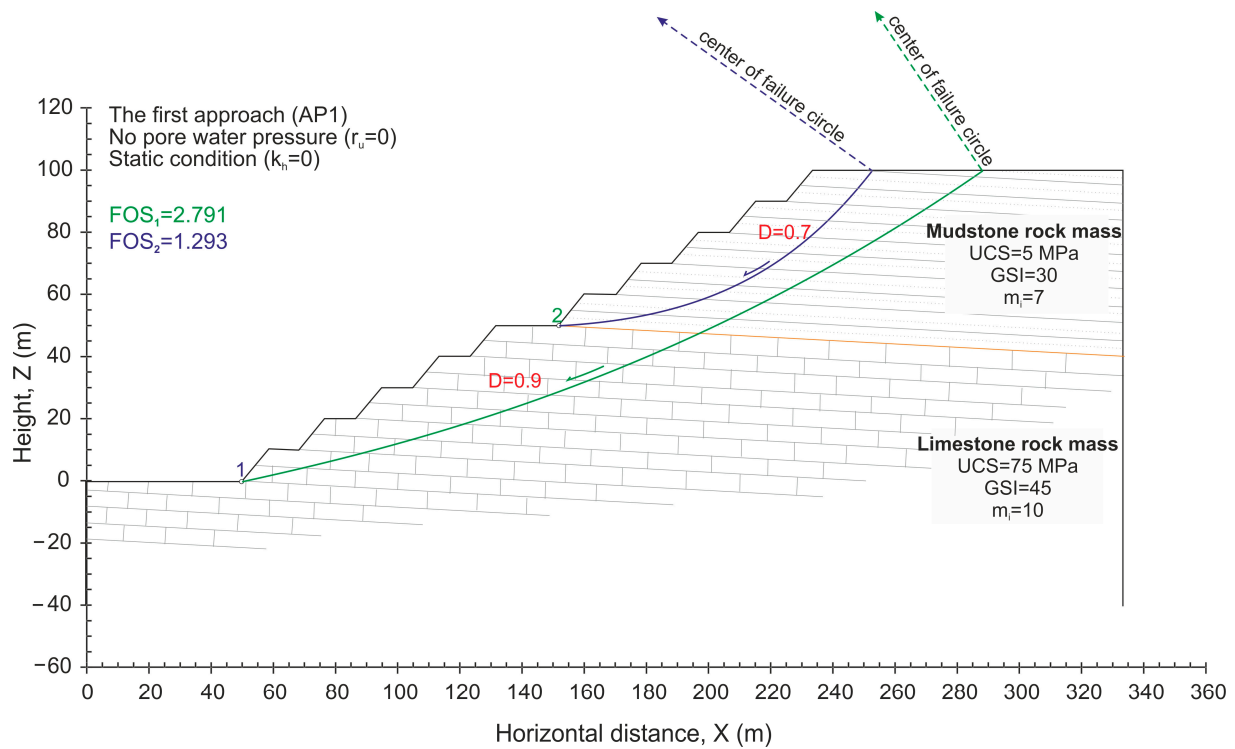


Figure 6. The overall slope stability evaluations by considering the first approach (AP1).

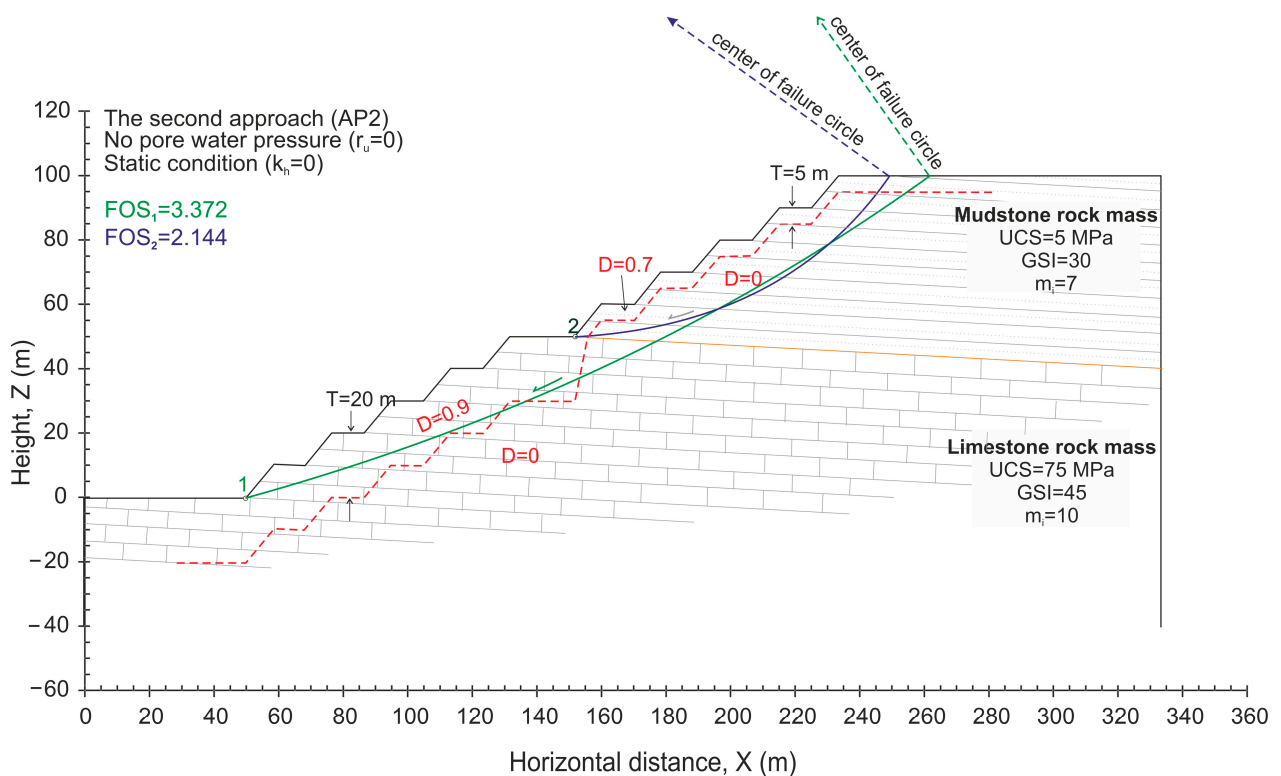


Figure 7. The overall slope stability evaluations by considering the second approach (AP2).

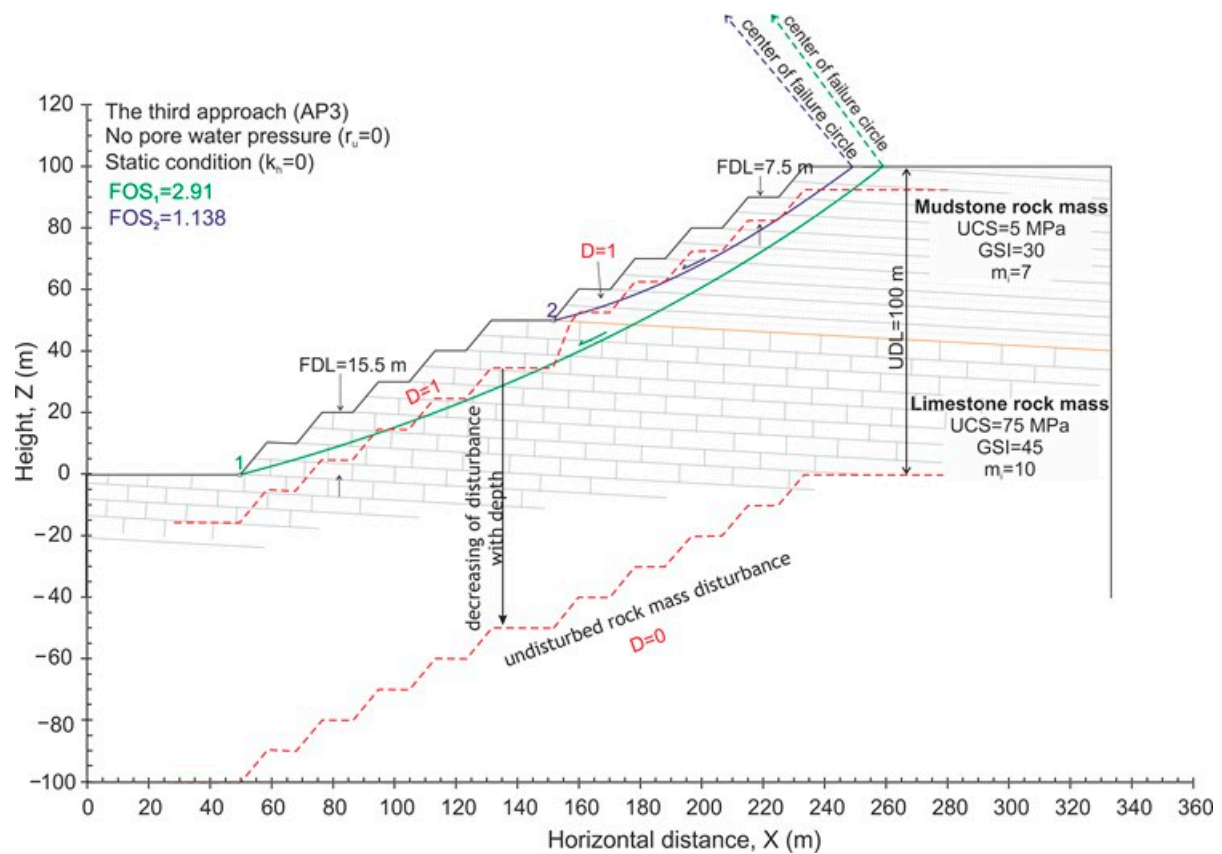


Figure 8. The overall slope stability evaluations by considering the third approach (AP3).

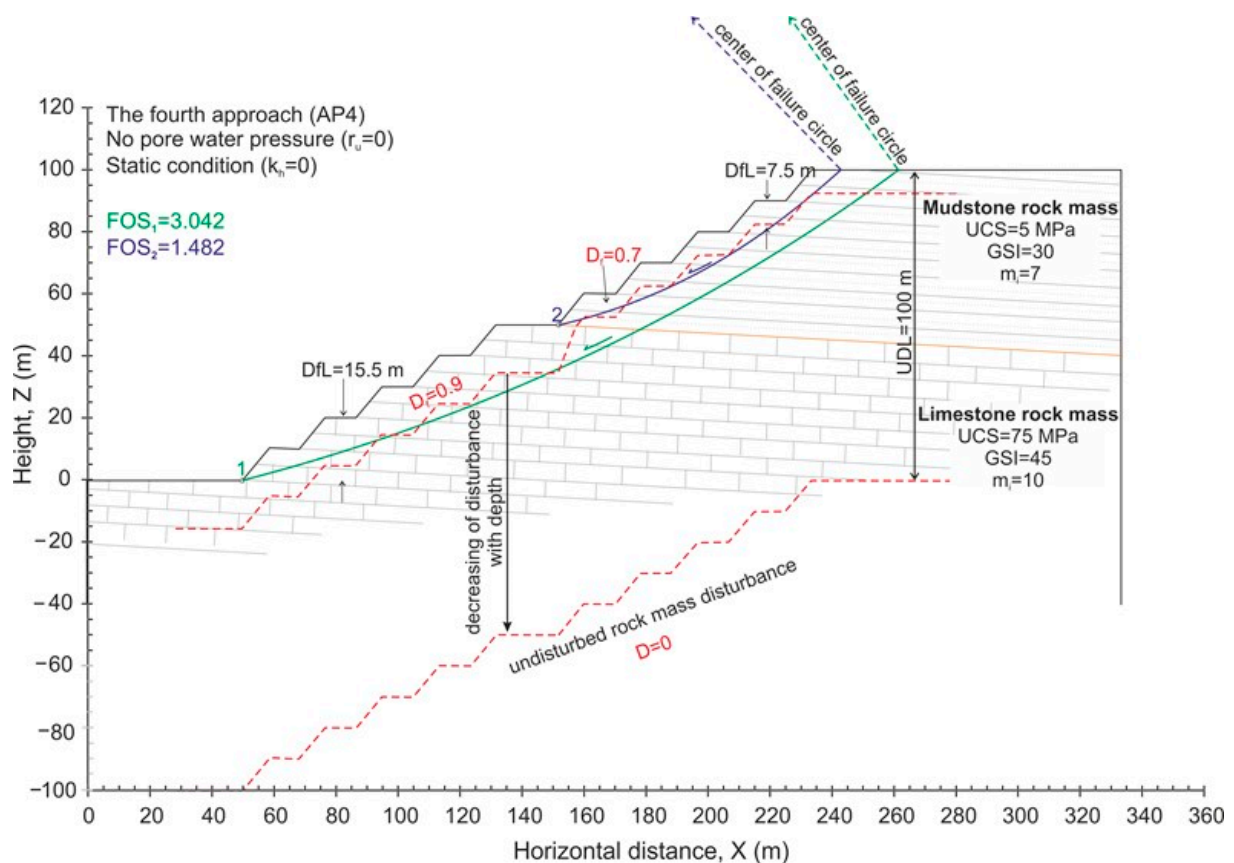


Figure 9. The overall slope stability evaluations by considering the fourth approach (AP4).

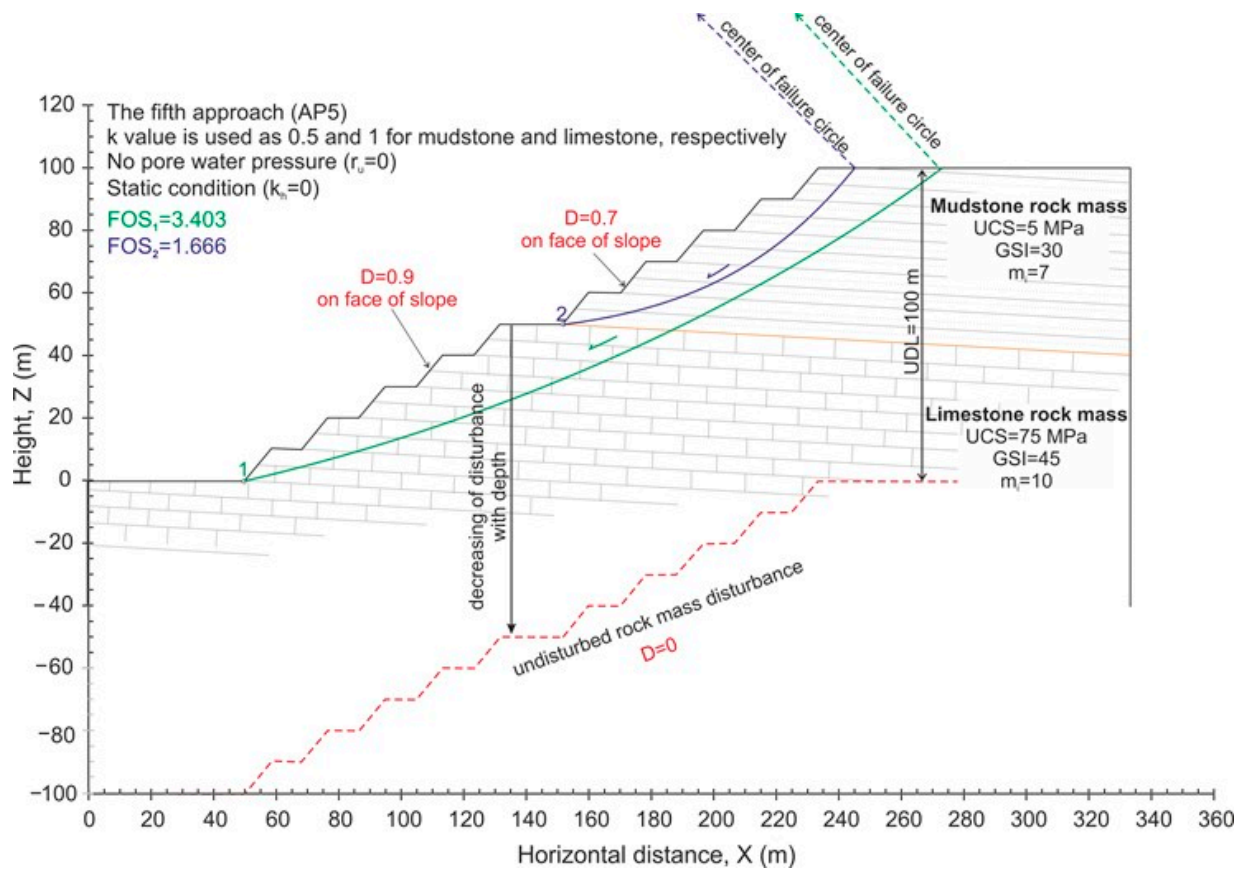


Figure 10. The overall slope stability evaluations by considering the fourth approach (AP5).

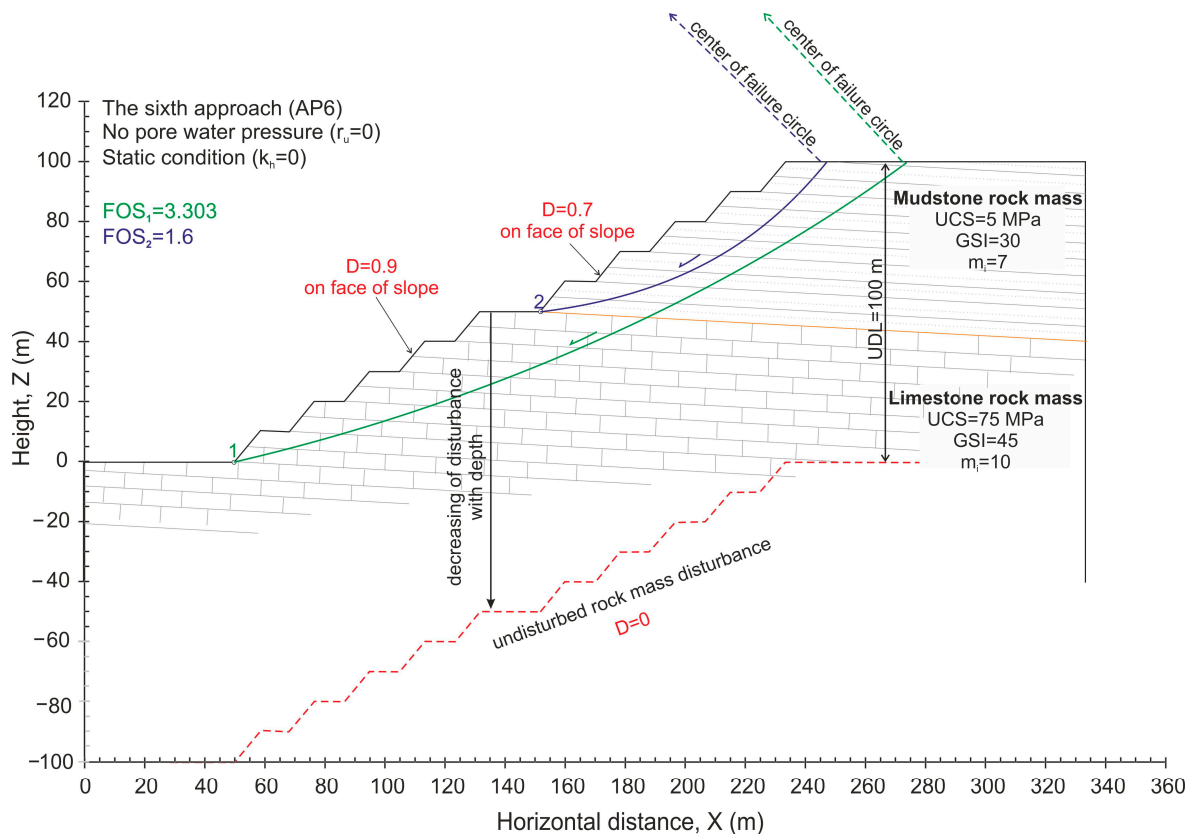


Figure 11. The overall slope stability evaluations by considering the fourth approach (AP6).

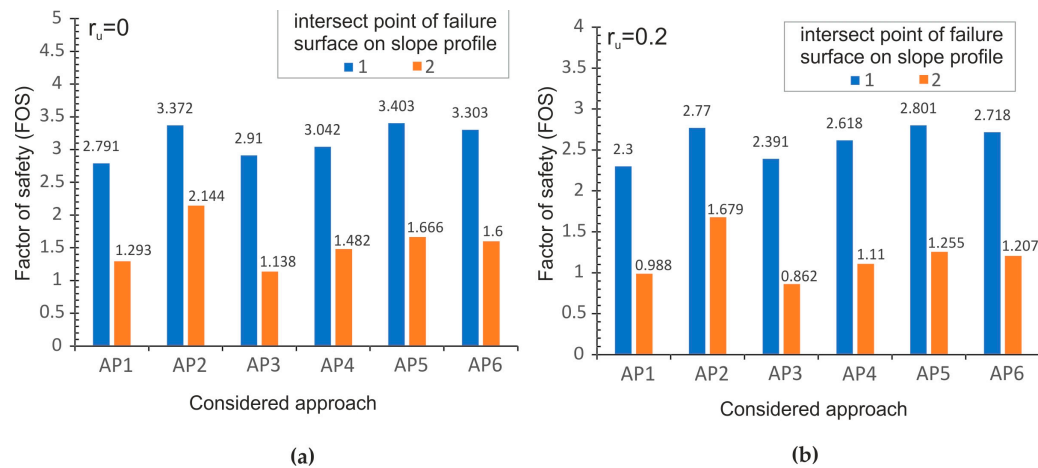


Figure 12. Histogram graphs of factors of safety for the considered approaches: (a) drained and (b) $r_u = 0.2$ conditions.

In addition to the factor of safety against overall slope failure, another important consideration in hazard assessments of a possible slope failure is the volume of sliding material above the potential failure surface. When the results of the stability analysis of the four approaches considered in this study are evaluated from this perspective, it is seen that the highest amount of mass above the potential failure surface was obtained from the stability evaluations based on AP1 (Figure 13). This output is a result of the application of the disturbance effect by considering AP1 without any attenuation from the surface to the depth of the entire rock mass, forming the geological cross-section of the slope model. On the other hand, it is more realistic that the effects of disturbance, such as the generation of new cracks and the increase in the aperture of existing joints, reduce from the surface to the depth. Similar results were also obtained from a recent study performed by Park and Michalowski [13]. The results of AP5 and AP6 in terms of failure surfaces and factor of safeties for critical points 1 and 2 are very close to each other. However, while, in AP6, the values of k for limestone and mudstone were determined by DfL, in PA5, they were determined by matching the curve as much as possible used in AP6.

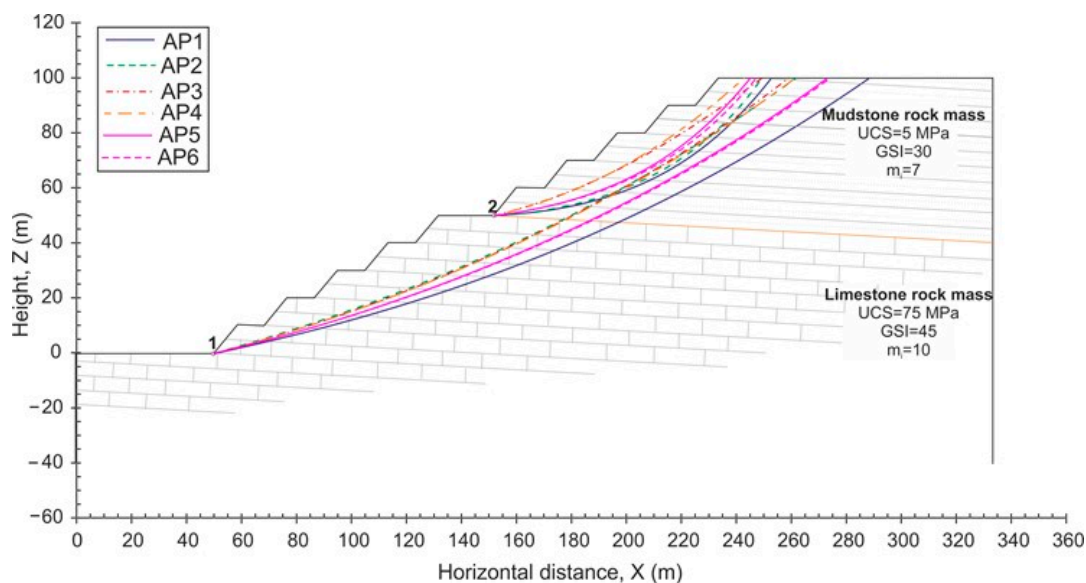


Figure 13. Potential failure surfaces obtained from circular stability analysis by considering the defining of approaches to the disturbance effects.

5. Evaluation of Transitional Rock Mass Strength Anisotropy with Depth on Slope Stability

It is obvious that the disturbance of the rock mass, depending on the excavation method, causes a decreasing degree of disturbance from the surface to the depth. Therefore, even if the rock mass is completely homogeneous and isotropic with its jointing properties, the strength of the rock mass becomes anisotropic after disturbing the rock mass, which causes an increase in the overall strength from the surface to the depth.

All the stability assessments given above were performed by using circular failure surfaces without taking into account the anisotropic mass strength behavior under the excavation disturbance. Therefore, the question of whether circular failure analysis is sufficient to determine the minimum factor of safety and the potential failure surface needs to be addressed. For this aim, the derived equation in this study (AP6) was taken into consideration to repeat the stability analyses considering the combined failure surface by a combination of circular failure surfaces and the surface of the anisotropic strength of the excavated rock mass. For this aim, a plane parallel to the surface of the general slope angle that includes the anisotropic strength in the stability analyses was defined (Figure 14). In this case, stability analyses were performed by using Janbu [24] due to the non-circular failure surface of the combined type of failure surfaces.

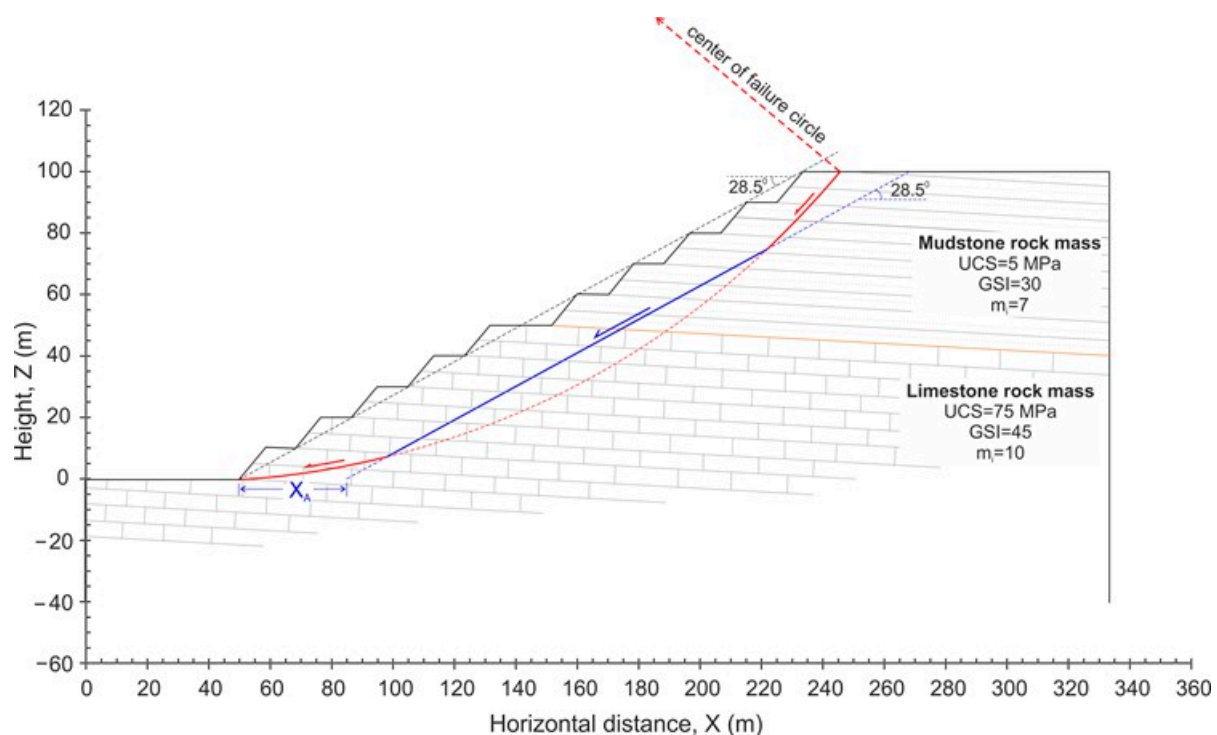


Figure 14. Hypothetical slope model used to evaluate the anisotropic rock mass strength in slope stability analysis.

The analyses by using the Janbu [24] method were performed with an increase in the distance (X_A) from the surface to the toe of the general slope for the linear plane parallel to the general slope surface, i.e., parallel to the distribution of the disturbance zone. The results were then presented as X_A versus FOS graphs, as shown in Figures 15 and 16. In the analyses conducted for potential failure surfaces passing through points (1) and (2), the factors of safety gradually decreased as the X_A distance of the plane defined to account for the anisotropic strength of rock mass increased (Figures 15 and 16). The lowest factor of safety at point (1) was determined as a potential circular failure surface when

the defined plane was sufficiently far away for no intersection with a potential circular failure surface. On the other hand, this result may not be interpreted as just circular analyses being sufficient for stability evaluation because the lowest factor of safety was obtained for a combined potential failure surface when point (2) was considered. However, very little difference was obtained when it is compared with the factor of safety against circular failure.

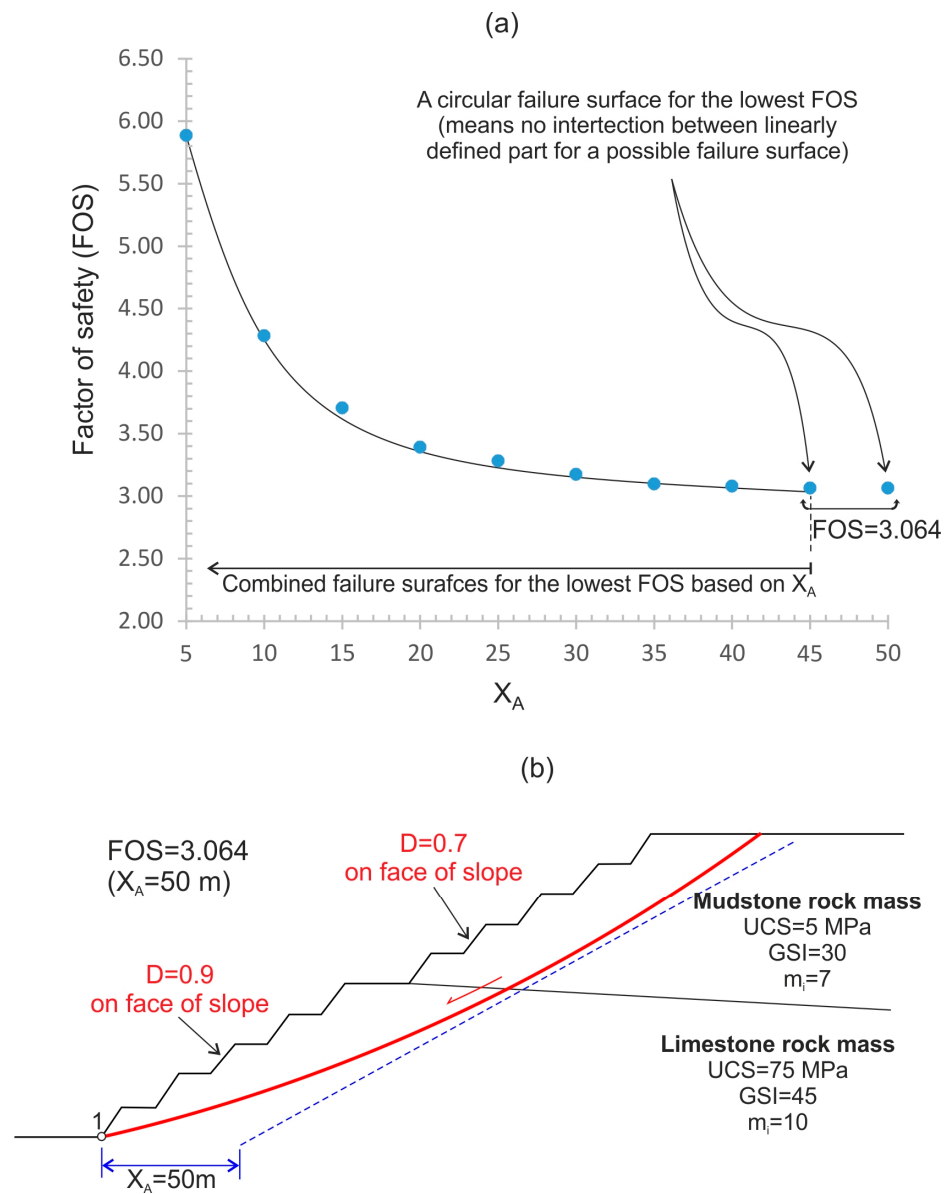


Figure 15. For potential failure surfaces passing through point (1), (a) the X_A distance of the plane defined to account for the anisotropic strength of the rock mass increases versus the FOS graph, and (b) the critical failure surface determined for the lowest factor of safety.

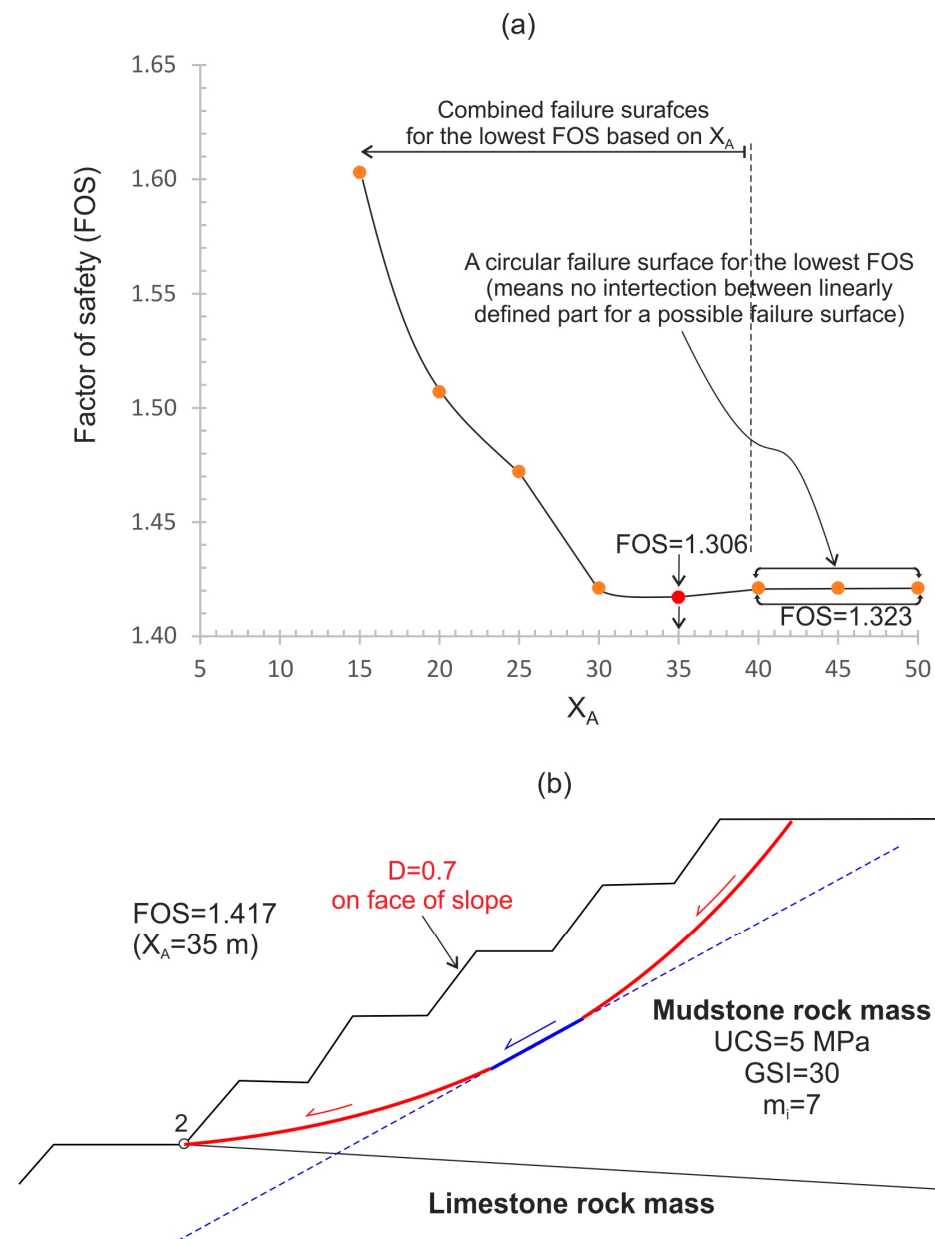


Figure 16. For potential failure surfaces passing through point (2), (a) the X_A distance of the plane defined to account for the anisotropic strength of the rock mass increases versus the FOS graph, and (b) the critical failure surface determined for the lowest factor of safety.

Although the strength of the rock mass decreases toward the excavation surface of the slope profile, the instantaneous internal friction angle increases due to the nonlinear failure envelope as the normal stress becomes lower. For shallow failure surfaces, the factor of safety also increases because the instantaneous internal friction angle is significantly higher than the average slope of the profile. However, shallow sliding surfaces are unlikely to occur, as the interlocking of rock blocks in jointed rock masses typically prevents such failures.

Slope stability is fundamentally defined as the equilibrium between the horizontal and vertical components of the weight of the rock mass along the failure surface projection. Achieving the critical failure surface in jointed rock masses necessitates a specific rock mass weight. However, defining the anisotropic behavior of rock masses remains challenging due to uncertainties in parameters such as the disturbed rock mass limit (DfL), the depth of undisturbed rock mass (UDL), and the formulation for reducing the disturbance factor

with depth. Therefore, innovative studies are essential to better align theoretical models with actual in situ slope conditions.

6. Conclusions

The disturbance factor, introduced by Sonmez and Ulusay [6], was incorporated into the Hoek–Brown failure criterion, which is commonly used to assess the strength of jointed rock masses under an excavation-induced disturbance. This factor was subsequently similarly adopted by the original developers of the criterion. However, there remains some uncertainty about the application of the disturbance factor, particularly in deep slope excavations, and only a limited number of suggestions are available in the literature regarding its use.

In addition to the five approaches from the literature, a new decay formulation is proposed in this study. Initially, the six approaches mentioned in the literature were applied to a hypothetical slope model, and the results were discussed in terms of slope stability and the volume of rock mass above the potential failure surface. As expected, different factors of safety were obtained from each approach considered in this study. However, the use of a single disturbance factor in AP1 for the entire rock mass, from the surface to depth, can be viewed as an indirect conservative contributor to the factor of safety. Furthermore, as a simple assumption, Hoek and Karzulovic [8] divide in situ rock masses into three groups, disregarding the transitional disturbance effects. Their proposal for assessing the possible thickness of disturbed rock mass zones appears to be both reliable and practical for application. Conceptually, the approach proposed by Rose et al. [10] is the first applicable method due to its use of a transitional disturbance zone that extends from the surface to the in situ (undisturbed) rock mass. However, a zone of completely disturbed rock mass ($D = 1$) at the surface overlies a transitional disturbance zone, which eventually leads to the in situ (undisturbed) rock mass ($D = 0$). As noted by Hoek and Karzulovic [8], the completely disturbed rock mass zone can be removed through excavation. It appears that the completely disturbed rock mass zone will be removed during the excavation of benches. Additionally, Sonmez and Ulusay [6] preferred using the completely disturbed rock mass condition ($D = 1$) to define the strength of dump rock masses, such as spoil piles, due to the nearly cohesionless shear failure envelope derived from the Hoek–Brown criterion. The fourth approach, which is a minor improvement to Rose et al.'s [10] approach by combining it with Hoek and Karzulovic's [8] approach, seems more acceptable than AP1, AP2, and AP3. This is because Sonmez et al. [14] considered $D = D_f$ at the surface, reflecting the removal of blasted rock (fully disturbed rock mass) during excavation in front of the blasted damaged rock. On the other hand, AP5 (Park and Michalowski [21]) and AP6 (this study) consider continuous decay equations extending from the surface of the slope to the undisturbed depth limit (UDL). Therefore, conceptually, both AP5 and AP6 are better able to represent on-site conditions. AP6 has an advantage due to the use of D_{fL} for determining the extremum point, which satisfies the yielded rock mass of approximately $90\% \pm 5$, consistent with the southeast wall of the HVC Lornex Pit and the northeast of the HVC Valley Pit cases studied by Rose et al. [10].

The impact of rock mass strength anisotropy with a depth on slope stability under an excavation disturbance was analyzed using a combined failure surface approach. The results of the slope stability analysis indicate that the overall stability can be influenced by both circular and elliptical failure surfaces, owing to the anisotropic nature of rock mass strength under excavation disturbances. As a result, both circular and combined failure surface analyses should be conducted in stability evaluations to more accurately assess the depth of potential failure surfaces. Geology can be complex in nature and, as noted by Liu

and Chen [11], substantial spatial variability may significantly affect the factor of safety and the design of slopes.

There is a clear need for further research, such as performing back-analyses of failed slopes, aligning failure surfaces with analytical findings, and employing geophysical techniques to investigate disturbance variations from surface to depth.

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