Rock slope stability analysis adopting Eurocode 7, a limit state design approach for an open pit

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Abstract. In this paper we compare the partial factors approach of Eurocode 7 with a traditional deterministic limit equilibrium analysis approach and probabilistic approach. The example case is a simplistic model profile of an open pit mine cut slope configuration. The controlling parameters in a structurally dictated failure mechanism were introduced as random variables in the model, to account for the different types of uncertainties arising in rock engineering data. The limit equilibrium method was used in this study to conduct rock slope stability analysis, while the impact of naturally uncertain and variable parameters, among others water pressure and earthquake activities were considered. Different scenarios from the best to the worst-case were analysed and later compared. The sensitivity analysis revealed the importance of the joint failure plane, joint roughness coefficient and joint friction angle and how the actions (groundwater pressure and earthquake action) affect the slope's stability. These results emphasize some advantages of the approach, and the potential further embracement from the rock engineering fraternity.

1. Introduction
In the case of a rock structure in which many ill-defined parameters interact in a complex manner, the calculation of safety is a much less satisfactory process [1]. One possible approach which has been discussed by several authors is the probabilistic analysis of variables leading to a concept of safety in terms of a given probability of failure. Eurocode 7 (EC7), the reference code for geotechnical design of public-funded civil engineering works was implemented within the European Union in 2010. EC7 is a code for the design of foundations on soil yet is considered as being weak with regards to rock engineering design, and in practice incompatible with contemporary rock engineering design. Contemporary rock engineering practice is generally deterministic as stated in [2]. Non 1:1 mapping, of properties that are defined as ordinal data and thus, unsuitable for use in a probabilistic framework are typical. Contemporary rock engineering design is fundamentally incompatible with Limit State Design (LSD) and hence EC7, suggesting for a new start in Rock Engineering. For, many reasons, rock engineering (and, more widely, geotechnical) design formulae often cannot be written in the form \( R > E > 0 \) (R structural resistance, and applied loads (effect of actions E). The uncertainty and variability in both R and E are often both poorly understood and possibly not accurately characterized. The rates of rock engineering failure that society will tolerate is not known, possibly because those failures that do occur are either of no direct consequence to society. For the above-mentioned reasons, partial factor approach seems not to be well suited to rock engineering.
What seems to be a widely held view amongst geotechnical designers is that EC7 would be more applicable to rock engineering if only the Code included the necessary partial factors. However, we see that the root of the difficulty is much more fundamental: simply put, rock engineering design has developed in such a way that much of it is incompatible with the principles of LSD, and the problems involved in rock engineering design are not amenable to the partial factor approach. Mainly because the kernel of LSD is probabilistic principles, a key concept of LSD is that safety factor does not exist. To further understand the inherent problems in this paper we use the example of an open pit slope profile to examine the results from both the traditional and partial factors approach and further results on the benefits and the limitations. Slope failures in mining environments must be manageable from bench to overall pit slope scales to safeguard personnel, equipment, and the continued mining operation. The main objective for a slope design is to identify the major parameters that influence the stability of the slope under consideration, the specific condition of the rock, discontinuities, groundwater, earthquakes action, etc. The definition of these key parameters is also a crucial step in the process that requires the availability of qualitative and quantitative data. From there, an assessment can be executed to define a design criterion according to experience and judgment with the focus on maximizing the overall slope angle and reduce the waste ore stripping (guideline for open pit). Together with ensuring safety, [3] also highlight the need for slopes to be steep enough to warrant maximum ore recovery with minimum waste stripping.

Mines develop acceptability criteria to be used as a standard value to quantify the performance of slopes. The performance can be described in terms of the factor of safety (Fs) or probability of failure (Pf). The Fs refers to the ratio of the ultimate shear strength and the mobilized shear stress at incipient failure [4], whereas the Pf pertains to the probability of the Fs being one or less [3]. The importance of the slope in question gives an indication of the level of acceptance where critical slopes with vital facilities such as ramps are designed according to higher Fs and lower Pf [3]. Typical acceptance criteria are shown in table 1.

Table 1. Typical Fs and Pf acceptance criteria values [3].

| Slope scale | Consequence of failure | F_s (min) (static) | F_s (min) (Dynamic) | Pf |
|-------------|------------------------|--------------------|---------------------|----|
| Bench       | Low-high \(^b\)        | 1.10               | 0.067               | 25-50\% |
| Inter-ramp  | Low                    | 1.15-20            |                     | 25\% |
|             | Moderate               | 1.20               |                     | 20\% |
|             | High                   | 1.20-1.30          |                     | 10\% |
| Overall \(^a\) | Low                  | 1.2-1.3            | 0.600               | 10-20\% |
|             | Moderate               | 1.3                |                     | 10\% |
|             | High                   | 1.3-1.5            |                     | 5\% |

\(^a\): Needs to meet all acceptance criteria.  
\(^b\): Semi-quantitatively evaluated; b: Semi-qualitatively evaluated.

2. Main input parameters for limit equilibrium analysis

This study uses a selected profile from a cut wall in an open pit mine, to illustrate the deterministic, partial factors of EC7, and probabilistic approach. Similar examples were generally used in rock engineering literature to illustrate the above-mentioned approaches of rock slope design [5,6,7,8].

A typical rock slope stability analysis in an open pit, proceeds in three scales: the bench, inter-ramp, and overall slope scale. In this example, a stack of four 10m benches configuration was considered, and overall slope initially. The case represents a profile in an open pit mine where recurrent failures occurred at bench and inter-ramp level, under a structurally controlled failure mode of planar and wedge type of failure. A rock slope stability analysis is based on different process
2.1. Influence of geometry

The bedrock in this case example is tonalite gneiss intruded by metagabbro that makes up the wall rock. Numerous subvertical dolerite dykes, and brittle shear zones are exposed in the specific cut wall. The kinematic analysis identified a critical zone for plane failure, and wedge failure. The slope design comprised a stack of four, 10m vertical benches with 2.5m step-offs between each bench and a 15m wide catchment berm between each stack of four benches. A stack angle of 76° and overall slope angle of roughly 58° was produced from the 40 m high stacks and 200 m deep pit (figure 1). The ramp width is 25 m. The slope angle represents the inclination of the slope which was specified as either 58° or 76°. The slope height is specified to be 200 m and 40 m. The inclination of potential sliding plane is 44° refers to the dip of the sliding plane or the critical joint set angle identified through kinematic analysis whereas the upper face angle is the dip of the upper face of the slope which was estimated to be flat hence using 0°. The geometry and acting forces for the studied profile at stack of benches, and over all slope, is height $H = 45m$, 200m, $\psi_f = 76^\circ$, 58°, inclination of potential sliding plane $\psi_p = 44^\circ$ based on the mean value of the window dip range for the selected main discontinuity set. The specific gravity of rock = 27kN/m$^3$ and that of water $\gamma_w = 9.81$kN/m$^3$.

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure1.png}
\caption{Pit slope geometry depicting key components of slope design [9].}
\end{figure}

The acting forces are as follows:

\[ W = \text{weight of potential slide block} \]
\[ = \left(\frac{\gamma H^2}{2}\right) \times \left(\frac{1}{\tan \psi_p} - \frac{1}{\tan \psi_f}\right) \quad (1) \]

$U$ water pressure (kN/m$^2$)

$A$ seismic acceleration as a fraction of g (m/s$^2$)

$F_a = ma$ seismic action (kN/m) \quad (2)

2.2. Influence of Shear strength

The shear strength of discontinuities and the associated controlling factors are important in the analysis of structurally controlled rock slope failures because discontinuity surface roughness increases its shear strength and could play a part in stabilizing rock slopes [10]. In [11], proposed the following empirical equation to estimate the peak shear strength of rough discontinuities ($\tau$):

\[ \tau = \sigma_n \tan (\varphi_r + JRC_n \log \frac{JCS_n}{\sigma_n}) = \sigma_n (\tan \varphi_r + i) \quad (3) \]
where $\sigma_n$ is the normal stress, $\phi_r$ is residual friction angle of the discontinuity surface, $\text{JRC}_n$ is the scale dependent joint roughness coefficient on a scale of 1 fro the smoothest to 20 for the roughest surfaces, $\text{JCS}_n$ the scale dependent joint wall compressive strength, [12]. In that case the active friction angle $\phi_a$ or total friction angle ($\phi_r$ and $i$) on the potential sliding plane is strongly dependent on the normal stress the residual friction angle. The mean basic friction angle adopted based on shear box tests was 37.1$^\circ$ and back analysis resulted on a friction angle of 42$^\circ$.

The waviness (undulations) of the failure plane surface, observed over distances on the order of 1 m to 10m [13], and has the effect of increasing the shear strength of the failure plane, was considered in the analysis. The waviness angle is equal to the average dip of the failure plane, minus the minimum dip of the failure plane.

The waviness angle can be determined by taking several failure plane dip measurements, calculating the average dip of the failure plane, and subtracting the minimum dip. In this example the waviness angle was 14$^\circ$. The persistence was also considered in the analysis with a mean value of 10m. In the current case example $\text{JRC}_n$ and $\text{JCS}_n$ values were determined based on laboratory test results which were downgraded for in situ discontinuity of 10m length using the equation (4) and (5) [12].

$$\text{JRC}_n = \text{JRC}_0 \left[ \frac{L_n}{L_0} \right]^{0.02 \text{JRC}_0}$$  \hspace{1cm} (4)

$$\text{JCS}_n = \text{JCS}_0 \left[ \frac{L_n}{L_0} \right]^{0.02 \text{JCS}_0}$$  \hspace{1cm} (5)

where $\text{JRC}_0$, $\text{JCS}_0$, $L_0$ (length) refer to 100mm laboratory scale samples, and $\text{JRC}_n$, $\text{JCS}_n$ and $L_n$, refer to in situ block sizes. In the current case $\text{JRC} = 6$ and $\text{JCS} = 90$ as many discontinuities have a planar or undulating profile on a macro scale. These values were verified using back analysis on pre-existing plane failure at the specific profile.

The normal stress is given below by equation (6).

$$\left( W \cos \psi_p - U - F_a \sin \psi_p \right) / \left( \frac{H}{\sin \psi_p} \right)$$  \hspace{1cm} (6)

2.3. Influence of water pressure

The consideration of the influence of water pressure upon the stability of a slope is of major importance. Although, a precise calculation of the influence of water pressure upon slope stability is not possible, it is reasonable to base the calculation upon the worst set of conditions which can occur and to use the results of these calculations as an aid to judging the consequences of probable groundwater conditions in the rock mass under consideration. The common approach as worst case scenario is water pressure builds up hydrostatically to a maximum value at the middle of the plane, and drains freely t the bottom of the slope, [1]. In that case $U$ is

$$U_{\text{max}} = \gamma_w H^2$$  \hspace{1cm} (7)

As indicated in [8], the persistence of the discontinuities has a significant effect on the reduction of the real pore water pressure.

According to the results of the analysis, the presence of a tension crack does not significantly reduce the stability of the slope if there is no water present. When the tension crack becomes water-filled under conditions of heavy rain or due to poor control of surface drainage, a significant increase in shear strength is required to maintain stability. The most severe conditions which could occur in heavy and prolonged precipitation which could result in the slope becoming completely saturated would almost certainly produce failure in this slope.

2.4. Seismicity

Seismic activity has an important influence of slope stability, which often results in rock fall events, or shallow failure events. The pseudo – static simplified principle is used [1], and the seismic load is
included in the calculation as an equivalent horizontal load, Equation (2). In this case example a value was set to 0.12g.

3. Main input parameters for limit equilibrium analysis

3.1. Deterministic analysis
The well-established method of limit equilibrium (deterministic approach) defines the safety factor Fs under gravity, as the ratio of the stabilizing forces (F_{stab}) over the destabilizing forces (F_D). In the studied example

$$F_{stab} = (W \cos \psi_p - U - F_a \sin \psi_p) \tan \varphi_a$$  \hspace{1cm} (8)

$$F_D = (W \sin \psi_p + F_a \cos \psi_p)$$ \hspace{1cm} (9)

$$F_s = \frac{F_{stab}}{F_D}$$ \hspace{1cm} (10)

A slope is stable when Fs > 1, yet a higher value of safety factor is required, in accordance with safety acceptance criteria, limits are Fs > 1.3 and Fs >1.5 for short term and long-term stability, to account for low to high consequences at failure (table 1). It is essential to perform a sensitivity analysis that identifies the effect of the considered parameters, in the calculation of safety factor.

![Figure 2. Sensitivity analysis results.](image)

3.2. Partial factor method of EC7
To satisfy the ultimate limit state design, the sum of the applied actions (loads) F_d, must be less or equal to the available resistance R_d to those actions. Defining the actions and the material properties,
the criterion that must be satisfied, in design may be expressed \( F_d \leq R_d \). Potential variability in shear strength parameters from those derived in the laboratory and the accurate determination of applied loads is necessary. Partial factors of safety, are used to modify the terms of actions, reactions, and material properties, to satisfy the design equation:

\[
\sum \gamma_G F_d \leq \frac{R_d}{\gamma_R}
\]  \((11)\)

In the case that is presented in this paper the respective partial factors adopted for the ultimate limit state according to Eurocode 7, [14], followed design approach 1, combination 2. For W, U: \( \gamma_G \):1.0 (permanent action). For Fa \( \gamma_G \):1.3 (unfavorable variable action), has been used according to table NA.A1.2 ( C) in CEN, (2002). [15], in clause 4/4.1(2) states that “Certain actions such as seismic actions and snow loads, may be considered as either accidental and/or variable actions”. For \( \tan \phi \gamma_R = 1.25 \). There is a dilemma the effect of the weight of the potentially unstable sliding rock block and the relevant partial factors. The weight is the cause of loss of stability, but also contribute to the resistance, [16]. The results of the slope stability analysis for both methods are summarized in table 2. below, including the “best case” scenario, no earthquake action and no effect of water pressure, earthquake action only, water pressure only, and earthquake action and effect of water pressure, “worst case”. The analysis was repeated for the overall slope with a height equal to 200m and a slope angle of 58° and for a slope of 40m heigh and an average slope angle of 76°, simulating a stack of four benches 10m height each, with and without tension crack.

**Table 2.** Factor of safety (Fs) for the best to worst case scenario for deterministic approach (A) and the partial factor approach (B) for \( \phi = 42° \).

| Scenario | Best case | Earthquake / no water | Water / no earthquake | Worst case |
|----------|-----------|------------------------|-----------------------|------------|
|          | A         | B                      |                       |            |
| Slope H = 200m, \( \psi_p 58° \) | | | | |
| \( \sigma_n \) (MN/m²) | 4.23 | 9.44 | 2.80 | 4.16 | 1.74 | 2.75 | 0.52 | 1.24 |
| \( \sum R_d \) (MN/m) | 5.50 | 9.99 | 3.60 | 4.62 | 2.52 | 4.23 | 0.77 | 1.61 |
| \( \sum F_d \) (MN/m) | 4.44 | 9.78 | 3.76 | 5.71 | 2.32 | 5.57 | 1.02 | 2.67 |
| \( F_s \) | 1.23 | 1.02 | 0.95 | 0.80 | 1.08 | 0.75 | 0.75 | 0.60 |
| \( P_f \) | 0 % | 0 % | 0.02 % | 3.87 % | 0 % | 0.35 % | 1.42 % | 20.29 % |
|            | A         | B                      |                       |            |
| Slope H = 40m, \( \psi_p 76° \) without tension crack | | | | |
| \( \sigma_n \) (MN/m²) | 24.66 | 22.69 | 10.56 | 10.67 | 13.81 | 6.04 | 14.0 | 16.99 |
| \( \sum R_d \) (MN/m) | 30.96 | 22.95 | 12.80 | 12.73 | 17.08 | 6.48 | 18.54 | 17.11 |
| \( \sum F_d \) (MN/m) | 27.21 | 25.21 | 14.37 | 17.16 | 16.41 | 7.11 | 21.38 | 22.45 |
| \( F_s \) | 1.13 | 0.91 | 0.89 | 0.74 | 1.04 | 0.91 | 0.86 | 0.76 |
| \( P_f \) | 0 % | 0.05 % | 0.21 % | 9.75 % | 0 % | 0.05 % | 0.2 % | 9.76 % |
|            | A         | B                      |                       |            |
| Slope H = 40m, \( \psi_p 76° \) with tension crack 83° | | | | |
| \( \sigma_n \) (MN/m²) | 66.28 | 66.28 | 58.60 | 58.60 | 13.70 | 13.70 | 11.74 | 11.74 |
| \( \sum R_d \) (MN/m) | 97.75 | 81.51 | 87.25 | 72.72 | 21.15 | 17.60 | 18.35 | 15.26 |
| \( \sum F_d \) (MN/m) | 64.01 | 64.01 | 71.96 | 71.96 | 18.79 | 18.79 | 20.82 | 20.82 |
| \( F_s \) | 1.52 | 1.27 | 1.21 | 1.01 | 1.12 | 0.93 | 0.88 | 0.73 |
| \( P_f \) | 0 % | 0.95 % | 4.06 % | 44.64 % | 15.59 % | 70.75 % | 84.67 % | 99.35 % |
4. Discussion and conclusions

In this paper a simplified example of planar failure in an open pit, was used to present the application of the so called traditional deterministic slope stability analysis method, and the partial factor approach. Both methods generate results that should be further evaluated in practice for a specific project. A factor of safety equal to 1.2 is estimated for a friction angle of 42°, indicating a marginally stable slope without considering the effect of earthquake action and water pressure. The application of the partial factors method marginally verifies the stability of the slope even under the “best case” scenario, agrees, with the multiple failures at bench level that occurred on site. The probability of failure analysis indicates a dramatic increase of the probability of failure in the “worst case” scenario. A comparison between the results for a friction angle of 37° and those of 42° indicates a dramatic decrease in the probability of failure as the friction angle is increased by 5°.

As can be seen from the table rock slope stability according to the partial factor principle appears to be unsatisfactory, for the cases with earthquake, with water and with earthquake and water (“worst case”). The results in figure 3 based on the results of limit equilibrium analysis [17] based on the old deterministic approach versus the new principle with partial factors are not very different. It is evident that calculations according to the partial factors are more conservative than the deterministic approach. Although these values themselves may not be accurate, their difference and the understanding of the mechanism which leads to this difference is important and this analysis could enable the designers to implement measures to prevent the recurrence of unfavorable conditions.

![Figure 3](image)

**Figure 3.** Comparison between results from the two approaches, deterministic (in blue) and partial factors (in red), a) 58° 200m, b) 76° 40m without tension crack, for $\phi_a = 42^\circ$.

In conclusion, there is uncertainty related to the role of weight of the unstable rock block, and the suggested by Eurocode partial factor of 1. The quantification of shear strength is complex and affects the magnitude $\phi_a$. A pure probabilistic approach through reliability-based design, suggesting the calculation of results through the probability of failure seems to be the most suitable solution towards addressing the problem and yielding more reliable results.

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