

Doctoral Thesis

Procedures for assessing the hazard posed by rock slopes

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Dissertation

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Affidavit

I declare in lieu of oath, that this thesis and the work presented in it are my own and have been generated by me as the result of my own research, using only literature cited in this work. Where I have quoted text passages from the work of others, the source is always given and clearly attributed.

I confirm that this work is original and has not been submitted elsewhere for any examination, nor is it currently under consideration for a thesis elsewhere.

Mariella Illeditsch

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*„Research is to see what everyone has already seen
and to think what no one has thought yet.“*

Albert Szent-Györgyi

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English Abstract

Rock slopes can pose various hazards due to their natural characteristics, geological conditions, and external factors. This work focuses on gravitational hazards posed by rock slopes, such as rock fall or rock mass fall due to slope failure.

The hazard posed by a process such as rock fall is determined by its intensity on the one hand and by its probability of occurrence on the other. Due to the high velocities that falling blocks can reach, the intensity – even of small blocks – can be very high. For persons, therefore, the smallest falling block can pose a major hazard. For hazard assessment, not only the intensity and probability, but also the propagation and bounce height of those events may be crucial. The kinetic energies, bounce heights, and propagation probabilities are strongly dependent on the modelled block size(s).

This work includes a critical review of the Austrian guideline ONR 24810 for technical protection against rockfall. One of its central elements is the evaluation of a so-called design block size. It is derived as a percentile (P95-P98, depending on event frequency) from a block size distribution. We have analysed the effect of a block size distribution (BSD) versus a design block P95 and P98 on rock fall calculations. The entire BSD should be simulated for most realistic distributions of kinetic energy, bounce height and propagation. This is even more true for hazard zoning, considering that the many smaller blocks may have a large effect on the propagation probabilities and bounce heights. Therefore, most realistic modelling requires most accurate detachment volumes and block size distributions.

The general aim of this work is to understand the hazard posed by rock slopes (in terms of rock fall) in a more holistic way and to show how detachment areas and volumes (block size(s)) can be determined based on profound analyses.

A discontinuum mechanical approach would be the best way to analyse the detachment volume and discrete block sizes for rock fall processes. However, often continuum mechanical approaches are required due to the shorter computing time. This way the individual block sizes for rock fall analyses need to be determined in a second step. The detachment process and the fall process are linked processes. Current process models for rock fall are limited to modelling individual trajectories. Therefore, they require the potential detachment areas and block size(s) as input. These must be determined using other methods.

We chose to determine in-situ block size distributions (IBSDs) from Synthetic Rock Mass Models (SRMMs) and fit them to probability density functions.

The contribution of this work is to reduce uncertainties in the determination of detachment areas, volumes, and block sizes. This is achieved firstly by determining HB material parameters at limit equilibrium for prognostic calculations. These parameters could also be applied to large-scale areas to identify rock slope instabilities (i.e., areas and volumes). Second, the reduction in uncertainties is achieved by improving the reliability and quality of block size distributions. This was accomplished by creating SRMMs, performing block surveys, and fitting them to probability density functions (pdfs). We found a pdf that appears to be universally applicable to the creation of IBSDs. Investigation of many more sites of different lithologies using this method could lead to a catalogue recommending a range of scale and shape parameters for certain lithologies. This could allow more realistic calculations of kinetic energies, bounce heights and propagation – and thus hazard – for better rock fall protection in the future.

Deutsche Kurzfassung

Felsböschungen können aufgrund ihrer natürlichen Eigenschaften, ihrer geologischen Bedingungen und externen Faktoren verschiedene Gefahren bergen. Diese Arbeit konzentriert sich auf die von Felsböschungen ausgehenden gravitativen Gefahren, wie Steinschlag und Felssturz aufgrund von Böschungsversagen.

Die vom Prozess Steinschlag ausgehende Gefahr wird zum einen durch seine Intensität und zum anderen durch seine Eintrittswahrscheinlichkeit bestimmt. Aufgrund der hohen Geschwindigkeiten, die fallende Felsblöcke erreichen können, kann die Intensität – selbst bei kleinen Blöcken – sehr hoch sein. Für Personen kann daher der kleinste herabfallende Block eine große Gefahr darstellen. Zur Beurteilung der Gefahr können nicht nur die Intensität und die Eintrittswahrscheinlichkeit, sondern auch die Ausbreitung und die Sprunghöhe eine wichtige Rolle spielen. Die kinetischen Energien, Sprunghöhen und Ausbreitungswahrscheinlichkeiten sind stark von den modellierten Blockgrößen abhängig.

Diese Arbeit beinhaltet eine kritische Überprüfung der österreichischen Richtlinie ONR 24810 zum technischen Steinschlagschutz. Eines ihrer zentralen Elemente ist die Bestimmung einer sogenannten Bemessungsblockgröße. Diese wird als Perzentil (P95-P98, je nach Ereignishäufigkeit) aus einer Blockgrößenverteilung abgeleitet. Wir haben die Auswirkung einer Blockgrößenverteilung (BSD) gegenüber einem Bemessungsblock P95 und P98 auf Steinschlagberechnungen analysiert. Für realistische Verteilungen von kinetischer Energie, Sprunghöhe und Ausbreitung sollte die gesamte BSD simuliert werden. Dies gilt umso mehr für die Gefahrenzonierung, wenn man bedenkt, dass die vielen kleineren Blöcke einen großen Einfluss auf die Ausbreitungswahrscheinlichkeiten und die Sprunghöhen haben können. Eine möglichst realistische Modellierung erfordert daher möglichst genaue Ablösevolumina und Blockgrößenverteilungen.

Das allgemeine Ziel dieser Arbeit ist es, die von Felsböschungen ausgehende Gefahr (im Sinne von Steinschlag) ganzheitlicher zu verstehen und zu zeigen, wie Ablöseflächen und Volumina (Blockgrößen) auf Grundlage fundierter Analysen bestimmt werden können.

Ein diskontinuumsmechanischer Ansatz wäre der beste Weg, um das Ablösevolumen sowie die diskreten Blockgrößen für Steinschlagprozesse zu analysieren. Oft müssen jedoch kontinuumsmechanische Ansätze (aufgrund ihrer kürzeren Rechenzeit) eingesetzt werden. In diesem Fall müssen die einzelnen Blockgrößen für Steinschlaganalysen in einem zweiten

Schritt bestimmt werden. Der Ablöseprozess und der Sturzprozess sind miteinander verbundene Prozesse. Aktuelle Prozessmodelle für Steinschlag sind auf die Modellierung einzelner Trajektorien beschränkt. Sie benötigen daher die potenziellen Ablösepunkte und Blockgrößen als Input. Diese müssen mit anderen Methoden bestimmt werden. Wir haben In-situ-Blockgrößenverteilungen (IBSDs) aus synthetischen Gebirgsmodellen (SRMMs) bestimmt und versucht, passende Wahrscheinlichkeitsdichtefunktionen zu finden.

Der wissenschaftliche Beitrag dieser Arbeit besteht darin, die Unsicherheiten bei der Bestimmung von Ablöseflächen, Volumina und Blockgrößen zu verringern. Dies wird zum einen durch die Bestimmung von HB-Materialparametern im Grenzgleichgewicht für prognostische Berechnungen erreicht. Diese Parameter könnten auch im großen Maßstab angewandt werden, um instabile Felsböschungen zu ermitteln. Zum Zweiten werden Unsicherheiten durch die Verbesserung der Zuverlässigkeit und Qualität von Blockgrößenverteilungen verringert. Dies wurde durch die Erstellung von SRMMs, die Ermittlung von Blockgrößenverteilungen daraus sowie die Anpassung von Wahrscheinlichkeitsdichtefunktionen (pdfs) daran erreicht. Wir fanden eine pdf, die für die Erstellung von IBSDs allgemein anwendbar zu sein scheint. Die Untersuchung vieler weiterer Felsböschungen unterschiedlicher Lithologien mit dieser Methode könnte zu einem Katalog führen, in dem eine Reihe von Skalierungs- und Formparameter für bestimmte Lithologien empfohlen werden. Dies könnte realistischere Berechnungen der kinetischen Energien, der Sprunghöhen und der Ausbreitung – und damit der Gefahr – für einen besseren Steinschlagschutz in der Zukunft ermöglichen.

1 Introduction and Motivation

Rock slopes can pose various hazards due to their natural characteristics, geological conditions, and external factors. This work focuses on gravitational hazards posed by rock slopes, such as rock fall or rock mass fall due to slope failure.

Rock fall is the sliding, falling, bouncing, and rolling of individual, independent rock fragments (Kienholz et al. 1998). The dominant state of motion is highly dependent on the nature of the slope (fall height, angle, length, morphology, surface) and the rock material. Usually, the prevailing motion state is bouncing, i.e., the interaction of a rock fragment with the underground by episodic impacts (Bourrier and Hungr 2013; Evans and Hungr 1993; Hungr et al. 2014). The other motion states (sliding, falling, and rolling) are usually observed at the beginning and at the end of a trajectory. According to the size of the fragments, a distinction can be made between rock fall (mean diameter ≤ 0.5 m) and block fall (mean diameter > 0.5 m). There is a general observation, that large blocks roll more than they bounce, compared to smaller blocks. Fall tests and back-calculations of rock fall events have shown that the maximum process velocities are between 30 and 35 m/s (Dorren et al. 2006; Smith and Duffy 1990; Usiro et al. 2006). Occasionally, velocities of more than 45 m/s have been observed or back-calculated (Pamminger 2019; Rauscher 2018). We speak of rock fall (or block fall) up to a detachment volume of 100 m^3 .

Rock mass fall is the detachment of a more or less coherent rock mass between 100 m^3 and 1 Mio m^3 , which fragments into rocks and boulders. They fall as more or less individual fragments, without 'significant influence' on the mobility of the falling mass. The fragments of a rock mass fall behave according to the rock fall process (Preh et al. 2020).

Falling masses $> 1 \text{ Mio m}^3$ are highly mobile processes referred to as rock avalanches. Here, the interaction between the fragments has 'significant influence' on the mobility of the mass. It behaves similarly to an avalanche and is not part of this work.

A hazard exists if personal injury and/or property damage may result from a process (ÖGG 2014). The hazard H posed by a process such as rock fall is determined by its intensity I on the one hand and by its probability of occurrence P on the other (see Eq. 1.1).

$$H = I \cdot P \quad (1.1)$$

Due to the high velocities that falling blocks can reach, the intensity – even of small blocks – can be very high. For persons, therefore, the smallest falling block can pose a major hazard.

For hazard assessment, not only the intensity and probability of events, but also the propagation and bounce height of those events may be crucial.

According to the Matterock method introduced by Rouiller and Marro (1997), the hazard H at a given point x for a given kinetic energy E can be calculated as the product of the event frequency λ_f and the propagation probability P_p of that given kinetic energy E at that given point x (Eq. 1.2 and Figure 1.1).

$$H(E, x) = \lambda_f \cdot P_p(E, x) \quad (1.2)$$

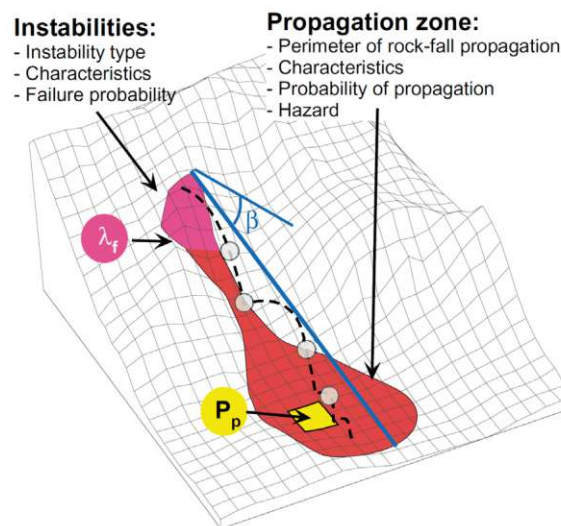


Fig. 1.1 Rockfall process and hazard evaluation. (Jaboyedoff et al. 2005)

The event frequency λ_f must be estimated, which is often very uncertain (Preh et al. 2019; Preh et al. 2020). The kinetic energies and propagation probabilities can be determined by process models, e.g., Throw (Preh et al. 2018). Process models for the simulation of rock fall have existed since the mid-1970s (e.g. Azimi and Desvarreux (1977); Piteau and Clayton (1977)). Today, many rock fall models are available for both two and three dimensions. For an overview refer to Volkwein et al. (2011) or Preh et al. (2020).

This work includes a critical review (Illeditsch and Preh 2020) of the Austrian guideline ONR 24810 for technical protection against rockfall (ON 2021). One of its central elements is the evaluation of a so-called design block size. It is derived as a percentile (P95-P98, depending on event frequency) from a block size distribution. We have analysed the effect of

a block size distribution (BSD) versus a design block P95 and P98 on rock fall calculations. Modelling a single 'constant' design block may result in trajectories far away from reality. Reducing input parameters at the beginning of the design approach results in apparent characteristic energy levels and bounce heights, which are not characteristic, as defined by the Eurocode 7. By disregarding all small and the biggest blocks, valuable information of maximum energy levels and bounce heights is lost. The entire BSD should be simulated for most realistic distributions of kinetic energy, bounce height and propagation. This is even more true for hazard zoning, considering that the many smaller blocks may have a large effect on the propagation probabilities and bounce heights. Therefore, most realistic modelling and hazard assessment requires most accurate detachment volumes and block size distributions.

The general aim of this work is to understand the hazard posed by rock slopes (in terms of rock fall) in a more holistic way and to show how detachment areas and volumes (block size(s)) can be determined based on profound analyses.

Verification of slope stability (or slope failure) by numerical models is mostly solved using the Strength Reduction Method (SRM). The question often arises whether a continuum mechanical (smeared) or discontinuum mechanical (discrete) approach is more appropriate (Preh et al. 2022). A discontinuum mechanical approach would be the best way to analyse the detachment volume and discrete block sizes for rock fall processes (Froschauer 2023; Moser 2020). In most cases, the fracture network is too complex, and the rock slope is too large for discontinuum mechanical calculations. Then, continuum mechanical approaches are required due to the shorter computing time (Illeditsch et al. 2022). However, this way the individual block sizes for rock fall analyses need to be determined in a second step.

The detachment process and the fall process are linked processes. There are no universal models that can satisfactorily simulate both processes. Current process models for rock fall are limited to modelling individual trajectories. The initial detachment needs to be represented by the initial conditions (initial position, initial velocities, and detachment volume). Therefore, all rock fall models require the potential detachment areas and block size(s) as input. These must be determined using other methods. We chose to determine in-situ block size distributions (IBSDs) from Synthetic Rock Mass Models (SRMMs) and fit them to probability density functions (Illeditsch and Preh 2023 (under review)).

2 Methods

Because the calculated kinetic energies, bounce heights and propagation probabilities – and therefore the rock fall hazard – are strongly dependent on the modelled block size(s), it is crucial to be able to estimate the location, extent, and behaviour of the potentially unstable rock mass.

A simple way to identify potential detachment areas is by a critical slope angle. Depending on the resolution of the Digital Terrain Model (DTM) grid, a critical slope angle is calculated. All points, that have a steeper angle than the critical slope angle, are selected as detachment points. This approach is also applicable on a very large scale. It may include false positives. Alternatively, potential rock slope instabilities can be identified by the integration of radar satellite analysis. These can be verified with numerical models. This multidisciplinary approach enables comprehensive monitoring of potential rock slope instabilities. This approach has already been used in the slate belt of Taiwan (Lin and Lin 2023). It is also applicable on very large scale and a promising alternative to the concept of the critical slope angle. It is state of the art to assess slope stability with numerical models. When discontinuum mechanical models are not feasible or only feasible with strong simplifications, a continuum mechanical model may be more appropriate. In continuum mechanical models, it is state of the art to describe heterogeneous rock masses using the Mohr-Coulomb (MC) or the Hoek-Brown (HB) criterion. It has already been known that especially for steep rock slopes the use of equivalent MC parameters with numerical codes and the SRM lead to poor estimates of safety factors (Li et al. 2007).

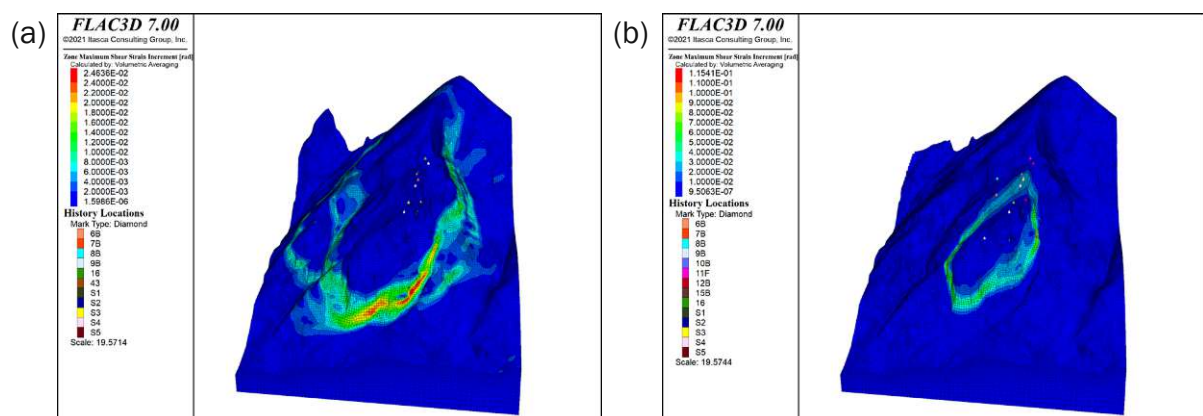


Fig. 2.1 Contour plot of the shear strain increment (predicted detachment area) using (a) MC material ($F = 1.47$) vs (b) HB material ($F = 1.21$). (Illeditsch et al. 2022)

Figure 2.1 illustrates the predicted detachment volumes on the example of a valley flank in Vals/Austria using the SRM with (a) MC material ($F = 1.47$) versus (b) HB material ($F = 1.21$). The predicted detachment volume with HB material correlates with the observed moving area, whereas the predicted detachment volume with MC material is overestimated.

However, when using HB material with the SRM, the resulting reduction factor F cannot be applied to the original HB parameters. To overcome this problem, Illeditsch et al. (2022) show how to determine a HB material equivalent to the HB envelope at limit equilibrium state (i.e., reduced by F). This is achieved by adjusting the geological strength index GSI and/or the disturbance Factor D . It is plausible to adjust those parameters due to the following considerations: (1) Based on the statement in Hoek's GSI chart (Marinos and Hoek 2000), it is more realistic to consider a range of the GSI, rather than a single value; and (2) since it can be assumed that the rock mass may have been disturbed by past impacts causing stress relief, the disturbance factor may be increased. The laboratory parameters m_i , σ_{ci} , and E_i are kept constant.

Figure 2.2 shows the HB envelopes for the in-situ state (green), for the limit equilibrium state (green dot), and for the HB envelope adjusted to the limit equilibrium state by GSI and/or D (yellow).

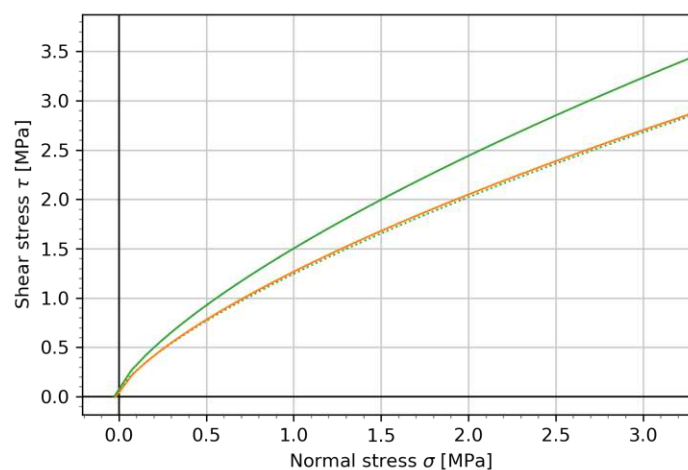


Fig. 2.2 HB failure envelopes in normal-shear stress space; green: in-situ state, green dot: limit equilibrium state, yellow: adjusted to the limit equilibrium state by reducing GSI and/or increasing D . (Illeditsch et al. 2022)

Since this is an approximation, the derived values of GSI and D should be optimized. For more information on the model calibration via concordance check after Mergili et al. (2017)

please refer to the case study of the Vals valley flank presented in Illieditsch et al. (2022). With this approach it is possible to derive the respective HB parameters at limit equilibrium. It helps to make more realistic predictions about the detachment area, mechanism, and volume. The derived HB parameters could also be applied to very large-scale areas to identify unstable rock slopes.

Having determined a potential detachment volume via continuum mechanics requires a further step of deriving block sizes. They are required as input for rock fall models. According to the ONR, one design block size is specified as a certain percentile (P95-P98, depending on event frequency) of a recorded block size distribution (BSD). BSDs may be determined from the detachment area (in the following referred to as in-situ block size distribution IBSD) and/or from the deposition area (in the following referred to as rock fall block size distribution RBSD). The ONR 24810 proposes various methods to determine BSDs. In the detachment area, these include estimation by visual assessment, statistical IBSD, and discrete explicit block measurement. In the deposition area, these include estimation, random axis measurements, the line counting method, the area method, sieve analysis or photosieving by software.

Deposition areas, if available, are generally accessible and measurable without technical aids. However, most measuring methods are subjective, uncertain, not verifiable, or inaccurate. Blocks smaller than a fist, are usually not measured. Sometimes, block volumes smaller than 0.125 m^3 are neglected. This affects the percentiles of the BSD. Neglecting many small blocks in a BSD, results in bigger P95-P98 blocks. In hazard analyses, smaller blocks may play an important role, depending on the protection target. For most realistic distributions of kinetic energy, bounce height, and propagation, the entire BSD should be considered in rock fall models (Illieditsch and Preh 2020).

Also, rock fall blocks are often fragmented due to the preceding fall process. The IBSD is (also) required for meaningful rock fall modelling. The statistical method seems the most efficient and cost-effective method to determine IBSDs with many blocks within the full range of block sizes.

The distribution of in-situ block sizes (IBSD) is based on the knowledge that fractures (bedding, foliation, and joints) are commonly closely spaced and have low persistence. Wide spacing and high persistence fractures are less common. Consequently, for rock blocks formed by fractures, small blocks are more common than large blocks.

To reduce uncertainties in the determination of BSDs we show a new approach (Illeditsch and Preh 2023 (under review)) to determine IBSDs. In recent years, the Synthetic Rock Mass Model (SRMM) has proven to be a useful method to simulate the real behaviour of a rock mass (Turichshev and Hadjigeorgiou 2017). In SRMMs, a Discrete Fracture Network (DFN) is intersected with a volume model to simulate the rock mass. The central element here is the DFN. It results in a collection of disk-shaped fractures whose geometric properties, such as location, orientation, density/intensity, and fracture size distribution are subject to a probability distribution. Thus, the fractures created in this way do not represent the actual fractures in the rock mass. Nevertheless, in this way it is possible to model the fracture system very realistically. This is illustrated by the density plots in Figure 2.3, comparing the (a) measured fracture sets (poles) from the rock face to the (b) created fracture sets in Tiefenbach via SRMM.

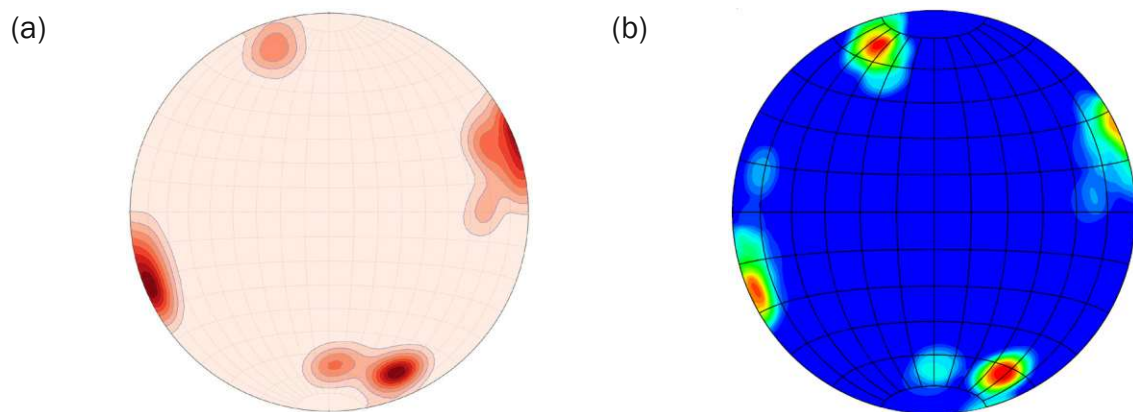


Fig. 2.3 Equal-area lower hemisphere density plots of the (a) measured fracture poles and the (b) created fracture poles in the SRMM for the rock face in Tiefenbach. (Illeditsch and Preh 2023 (under review))

By intersecting the DEM with the DFN, we created the SRMM. With the help of this model, we performed an exact rock mass block survey and determined a holistic IBSD. We used Python to check our derived IBSDs against 79¹ (Christopher 2017) probability density functions (pdfs). We applied the described method to three different rock faces in three different areas within Lower Austria: Tiefenbach, Spitz, and Greifenstein.

¹ norm, alpha, anglit, arcsine, beta, betaprime, bradford, burr, cauchy, chi, chi2, cosine, dgamma, dweibull, erlang, expon, exponweib, exponpow, fatiguelife, foldcauchy, f, fisk, foldnorm, gamma, gausshyper, genexpon, genextreme, gengamma, genlogistic, genpareto, genhalflogistic, gilbrat, gompertz, gumbel_l, gumbel_r, halfcauchy, halflogistic, halfnorm, hypsecant, invgamma, invweibull, johnsonsb, johnsonsu, laplace, logistic, loggamma, loglaplace, lognorm, lomax, maxwell, mielke, nakagami, ncx2, ncf, nct, norm, pareto, powerlaw, powerlognorm, powernorm, rdist, reciprocal, rayleigh, rice, recipinvgauss, semicircular, t, triang, truncexpon, truncnorm, tukeylambda, uniform, vonmises, wald, weibull_min, weibull_max, wrapcauchy, ksone, kstwobign.

We show that the generalized exponential function best describes IBSDs across various lithologies, when compared to 78 other pdfs.

Figure 2.4 illustrates how well the generalized exponential function fits the IBSD of Tiefenbach, compared to the lognormal and exponential pdfs. This is true for all three investigated rock slopes.

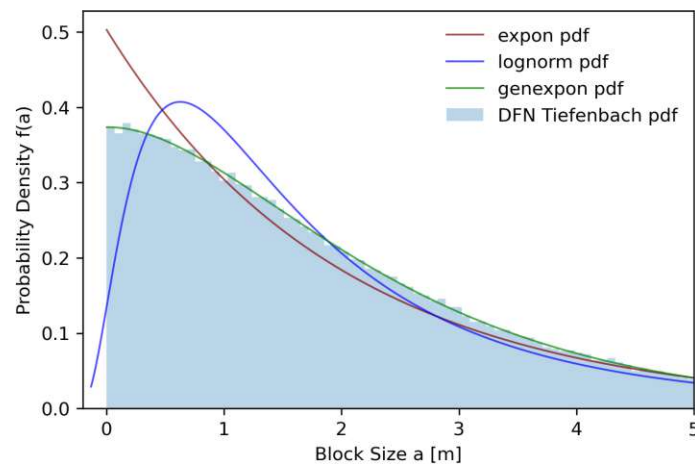


Fig. 2.4 In-situ block size distribution (IBSD) of Tiefenbach [m] (blue bars) with the fitted exponential (maroon), lognormal (blue) and generalized exponential (green) probability density functions (pdfs). (Illeditsch and Preh 2023 (under review))

Describing IBSDs by pdfs is a promising approach to be generally valid for all kinds of lithologies. Further investigations about the sensitivity of the scale and shape parameters on kinetic energies and runout are required. This could allow for more certain, accurate, verifiable, holistic, and objective hazard assessment.

3 Summary of the Scientific Articles

Steinschlag und Felssturz – Aktueller Wissensstand zu Extremereignissen alpiner Naturgefahren in Österreich / Rock fall and Rock mass fall – Current state of the art on extreme events of alpine natural hazards in Austria

Rock fall and rock mass fall are processes emanating from rock slopes. They endanger people, property, and infrastructure. In Austria, the Guideline for ‘Technical Protection against Rock Fall’ ONR 24810 (ON 2021) standardizes the design, construction, and maintenance of rock fall protection measures. It covers everything from investigating relevant process parameters to determining calculation parameters and characteristic resistance of protection systems. In Austria, extreme rock fall events are defined by the ‘design event’ according to the ONR 24810 (ON 2021). Events (magnitudes) larger than the design event are considered extreme.

Beyond the definition of extreme events, we define the terms ‘rock fall’ and ‘rock mass fall’, point out uncertainties with respect to the survey of the process-relevant parameters and uncertainties related to process modelling.

Regarding the imminent climate change with an accumulation of extreme weather phenomena, an increase of extreme events – also in the case of fall processes – can be assumed with high probability. Considering the character of the ‘design event’ approach, which in turn has a defined return period, it must be pointed out that significant changes in the magnitude and frequency of events can only be addressed to a very limited extent. Therefore, it might be necessary to rethink the ‘design event’ to maintain the currently existing safety level in the medium to long term.

Key statements

- Rock fall is the falling, bouncing, and rolling of isolated rocks or blocks.
- Rock mass fall is the detachment of a coherent rock mass fragmenting into rocks and blocks and subsequently moving downwards as individual fragments.
- Extreme events are defined by their intensity (magnitude) and probability of occurrence. Events with higher magnitudes than the defined ‘design event’ according to the national guideline ONR 24810 (overload case), can be defined as extreme events.

- Extreme situations are situations with extremely high damage potential. Crucial factors are the exposure and vulnerability of people, property, and infrastructure.
- In extreme situations, an extreme rock fall or rock mass fall event may lead to a catastrophe (according to T-KMG (2006)).
- Current rock fall programs can model extreme events.
- In most cases, even with extensive rock fall protection measures, it's not possible to fully mitigate the risks due to extreme events.

The concept of design block size – A critical review of ONR 24810 ‘Technical Protection against Rock Fall’

In Austria, the Guideline for ‘Technical Protection against Rock Fall’, named ONR 24810 (ON 2021) provides guidance to assess rock fall hazard (at object scale) by determining a so-called design block size. The aim of this article is to critically review the concept of design block size and to suggest alternatives. The design block size is derived from a block size distribution and event frequency, both of which may be very uncertain. Modelling a single ‘constant’ design block may result in trajectories far away from reality. The design approaches of the ONR 24810 and the Eurocode 7 (EC 7) (ON 2009), (ON 2021) are compared. A case study is presented and discussed.

Key statements and outlook

- Reducing input parameters at the beginning of the design approach results in apparent characteristic energy levels and bounce heights, which are not characteristic, as defined by EC 7 (ON 2009), (ON 2021).
- By disregarding all small and the biggest blocks, valuable information of maximum energy levels and bounce heights is lost.
- The entire block size distribution should be used for rock fall simulations for most realistic distributions of kinetic energy, bounce height and runout, rather than a single block size. Outliers may be disregarded at the end of the simulation process, as it is already specified in the ONR 24810 (ON 2021).
- Maximum block sizes should not be disregarded prior to modelling. This allows to model most realistic trajectories. Otherwise, results pretend apparent safety, as design values end up below characteristic levels (compare to EC 7).
- Fragmentation should be considered, if applicable.

Challenges Assessing Rock Slope Stability Using the Strength Reduction Method with the Hoek-Brown Criterion on the Example of Vals (Tyrol/Austria)

To estimate the hazard posed by rock slopes, it is essential to determine the overall stability and potential detachment volume. This is mostly solved using numerical methods together with the strength reduction method (SRM). Many calculation programs do not provide a direct implementation of the Hoek–Brown (HB) criterion. Equivalent Mohr–Coulomb (MC) parameters are often used. Especially for steep rock slopes, the use of equivalent MC parameters with numerical codes and the SRM lead to poor estimates of safety factors (Li et al. 2007). The problem lies in the required and often difficult estimation of a suitable range of minor principal stresses over a ‘slope height’. In the example of the stability analysis of the rock slope Vals in Tyrol/Austria, we show the differences between the application of equivalent MC parameters and a direct application of the HB criterion with apparent MC parameters. The detachment volume and stability are overestimated when applying equivalent MC parameters, as confirmed by calculations with the continuum mechanics code FLAC3D (Itasca 2021). However, the SRM with HB material (i.e., apparent MC parameters) results in a safety factor that cannot be applied to HB parameters. To date, it has not been possible to determine the HB parameters for limit equilibrium via the SRM. This challenge was overcome by fitting an HB envelope to the original HB shear envelope reduced by the safety factor. The envelope is adjusted by two HB variables: GSI and D. This allows to determine the HB parameters at limit equilibrium. It helps to make more realistic predictions about the detachment mechanism and volume.

Key statements

- The presented method eliminates the difficulty of estimating a suitable range for smaller principal stresses (σ_3) via the ‘slope height’.
- Verification of the most failure-prone area based on field mapping was only possible using apparent MC parameters. Using equivalent MC parameters results in a failure mechanism, which cannot be observed in nature, and in a higher safety factor!
- The present investigations clearly show that an application of equivalent MC parameters can lead to a wrong estimation of the overall stability. This was confirmed by Li et al. (2007).
- The presented method for calculating HB parameters from the distribution of apparent MC parameters by manipulating the GSI and D is a promising approach. This

enables true limit equilibrium studies and prognostic calculations using the HB criterion with continuum mechanical numerical methods.

Determination of Meaningful Block Sizes for Rock Fall Modelling

The determination of the so-called design block is one of the central elements of the Austrian Guideline for rock fall protection ONR 24810 (ON 2021). It is specified as a certain percentile (P95-P98, depending on event frequency) of a recorded block size distribution (BSD). BSDs may be determined from the detachment area (IBSD) and/or from the deposition area (RBSD). Deposition areas, if available, are generally accessible and measurable without technical aids. However, most measuring methods are subjective, uncertain, not verifiable, or inaccurate. Also, rock fall blocks are often fragmented due to the preceding fall process. The IBSD is (also) required for meaningful rock fall modelling. The statistical method seems the most efficient and cost-effective method to determine IBSDs with many blocks within the full range of block sizes. In current literature, fracture properties are often described by the lognormal and exponential functions. Today, we can model Synthetic Rock Masses (SRM) based on Discrete Fracture Networks (DFN). DFNs statistically describe the geometric properties of the fracture sets. This way, we can carry out exact rock mass block surveys and determine IBSDs.

In recent years, the SRMM has proven to be a useful method to simulate the real behaviour of a rock mass (Turichshev and Hadjigeorgiou 2017). One of the major advantages of this is being able to represent the properties of the rock mass at a true scale. This is possible because the individual blocks and fractures are modelled in a lifelike and discrete manner. With the help of a SRMM, the effects of existing fractures and the formation of new fractures can be simulated and investigated. Thus, the characteristic features of a rock mass are explicitly represented in the model and can be reproduced numerically.

We asked whether IBSDs derived from SRM models can be described by probability density functions and if yes, how well. We fitted various probability density functions (pdfs) to three determined IBSDs of different lithologies within Lower Austria. We compared their correlations using the KS test and the MSE method. We show that the generalized exponential function best describes IBSDs across various lithologies, when compared to 78 other pdfs. This could allow for more certain, accurate, verifiable, holistic, and objective results. Further investigations are required.

Key statements and outlook

- The generalized exponential function seems to describe IBSDs very well. It may be universally applicable to the creation of BSDs. Further investigations are required on the sensitivity of the scale and shape parameters.
- The lognormal and the exponential function do not describe IBSDs well.
- Describing IBSDs by probability density functions (pdfs) could allow for more certain, accurate, verifiable, holistic, and objective results.
- With the presented method, it is possible to determine IBSDs based on photogrammetry and SRM models. The investigation of many more sites of different lithologies using this method could result in a catalogue recommending a range of scale and shape parameters for certain lithologies in the future.
- The effect on rock fall modelling should be further investigated, e.g., comparing the runout, kinetic energies, and bounce heights of modelling whole IBSDs vs genexpon pdfs vs design blocks.
- Utilizing a pdf together with a catalogue of suitable fitting parameters may offer the benefit of enabling the determination of an IBSD and, consequently, a meaningful assessment of a design block, even with only a limited number of block size measurements (silent witnesses).

4 Scientific Contribution of the Dissertation

As we see repeatedly in the media, falling rocks can pose a serious hazard to people and property. To quantify the hazard, rock fall models can be used to simulate the process. However, the models require the potential detachment areas and volumes as input. The calculated kinetic energies, bounce heights and propagation probabilities – and therefore the hazard – are strongly dependent on the modelled block size(s).

This work shows that the use of the current guideline ONR 24810 for ‘rock fall protection’ (ON 2021) may be problematic, especially for hazard zoning, because it considers one single (design) block size and neglects all other. Also, the determination of block size distributions (and consequently the design block size) may be very uncertain, due to subjective measuring methods and lack of data.

One scientific contribution of this work is to reduce uncertainties in the determination of detachment areas and volumes by prognostic calculations using HB material parameters at limit equilibrium. They can be derived from the new approach of adjusting the GSI and D. Also, large-scale areas could be examined for rock slope instabilities by applying these HB parameters.

Another scientific contribution of this work is to reduce uncertainties in the determination of block size distributions (BSDs) by improving the reliability and quality of block size distributions. This was achieved by creating SRMMs, performing block surveys and fitting them to probability density functions (pdfs). We found a pdf that appears to be universally applicable to the creation of IBSDs. Investigation of many more sites of different lithologies using this method could lead to a catalogue recommending a range of scale and shape parameters for certain lithologies. This could allow for more realistic calculations of kinetic energies, bounce heights and propagation – and thus hazard – for better rock fall protection in the future.

*“Nature is inexorable and unchanging,
and it doesn’t care whether the hidden reasons and ways of its action
are understandable to man or not.”*

Galileo Galilei

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Appendix

Scientific Articles

Preh A, Mölk M, Illeditsch M (2020) *Steinschlag und Felssturz*. In: Glade T, Mergili M, Sattler K (eds) *ExtremA 2019. Aktueller Wissensstand zu Extremereignissen alpiner Naturgefahren in Österreich*, vol 1. V&R unipress, Vandenhoeck & Ruprecht Verlage, Göttingen, pp 425-459.

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Teil II: Extremereignisse einzelner Prozesse und Prozessbereiche

17 Steinschlag und Felssturz

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Zusammenfassung

Steinschlag und Felssturz sind von Felsböschungen ausgehende Prozesse, welche relativ häufig Siedlungsräume, Verkehrswege und andere Infrastrukturen gefährden. Die Planung, Ausführung und Instandhaltung von Steinschlagschutzmaßnahmen ist ausgehend von der Erhebung der prozessrelevanten Parameter über die Festlegung der maßgeblichen Bemessungsgrößen hin zu den charakteristischen Widerständen diverser Schutzsysteme in Österreich durch die Richtlinie für den technischen Steinschlagschutz (ONR 24810:2017) umfassend standardisiert.

Extremereignisse bei Steinschlag definieren sich in Österreich daher durch das entsprechend der ONR 24810:2017 (2017) festzulegende Bemessungsereignis. Treten Magnituden (Einwirkungen) größer als das Bemessungsereignis auf (der Überlastfall), sind diese als extrem zu betrachten bzw. die Folge eines Extremereignisses.

Bei Felsstürzen kommt es infolge des Abbruchs und der Fragmentierung größerer zusammenhängender Felsmassen zu zeitgleichen Ereignissen (Steinschlag) und folglich sind zeitnahe Einwirkungen auf Personen und Infrastruktur/Schutzmaßnahmen möglich. Darüber hinaus entspricht die Definition von Extremereignissen jener von Steinschlag. Über die Definition der Extremereignisse hinaus werden das von Steinschlag ausgehende Risiko betrachtet, Fallbeispiele und die Entwicklung der Extremereignisse im alpinen Raum diskutiert und Perspektiven für Handlungsoptionen aufgezeigt.

Abstract

Rock fall and rock mass fall are processes emanating from rock slopes, which often put settlement areas, roads and other infrastructure at risk. In Austria, the design, construction and maintenance of rock fall protection measures is standardised by the Guideline for technical protection against rock fall (ONR 24810:2017) comprehensively, originating from the investigation of relevant process parameters across to the determination of significant calculation parameters to the point of the characteristic resistance of various protection systems.

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Therefore, in Austria, extreme rock fall events are defined by the design event, which is determined according to the ONR 24810:2017 (2017). Magnitudes (impacts) greater the design event (overload case) can be considered extreme or rather the result of an extreme event.

The detachment of a bigger coherent rock mass and its fragmentation leads to contemporaneous events (rock fall), which may be followed by contemporary impacts on people and infrastructure/protection measures.

Beyond the definition of extreme events, the risk emanating from rock fall is considered, case studies and the development of extreme events in the alpine region are discussed and prospects for recommended procedures are identified.

Kernaussagen

- Unter Steinschlag versteht man das Fallen, Springen und Rollen von isolierten Steinen und Blöcken.
- Bei Felsstürzen brechen zusammenhängende Felsmassen ab, die während des Abbruchs in Steine und Blöcke fraktionieren, welche sich anschließend als individuelle Fragmente talwärts bewegen.
- Extremereignisse definieren sich über ihre Magnitude und ihre Ereignisfrequenz. Ereignisse mit Magnituden, welche das in der nationalen Richtlinie ONR 24810:2017 definierte Bemessungsereignis übersteigen (Überlastfälle), sind als Extremereignisse anzusehen.
- Extremsituationen sind Situationen mit einem extrem hohen Schadenspotential. Ausschlaggebend sind hier die Ausgesetztheit sowie die Vulnerabilität von Personen und Infrastruktur.
- Extremereignisse in Extremsituationen können auch bei den Prozessen Steinschlag und Felssturz zu einer Katastrophe (entsprechend T-KMG, 2006) führen.
- Die derzeit vorhandenen (gängigen) Steinschlagmodelle sind in der Lage Extremereignisse abzubilden.
- In den allermeisten Fällen wäre es auch mit aufwändigen Schutzmaßnahmen nicht möglich, eine vollständige Minderung/Reduktion der Risiken infolge von Extremereignissen zu erreichen.

Key Points

- Rock fall is the falling, jumping and rolling of isolated rocks or blocks.
- Rock mass fall is the detachment of a coherent rock mass fragmenting into rocks and blocks and subsequently moving downwards as individual fragments.
- Extreme events are defined by their magnitude and probability. Events with higher magnitudes than the defined design event according to the national guideline ONR24810:2017 (overload case), can be defined as extreme events.
- Extreme situations are situations with extremely high damage potential. Crucial factors are the exposure and vulnerability of people and infrastructure.
- In extreme situations, an extreme rock fall or rock mass fall event may lead to a catastrophe (according to T-KMG, 2006).
- Presently available (current) rock fall programs are able to model extreme events.
- In most cases, even with extensive rock fall protection measures, it's not possible to fully mitigate the risks due to extreme events.

17.1 Einleitung

In vielen gebirgigen Regionen weltweit sind von Felsböschungen ausgehende Sturzprozesse eine häufig auftretende Prozessgruppe, welche Siedlungsräume, Verkehrswege und andere Infrastrukturen gefährdet. Diese Prozessgruppe lässt sich in Einzelabstürze (Steinschlag, Blockschlag) und Massenstürze (Felssturz, Felslawine) untergliedern. Das wesentliche Unterscheidungskriterium zwischen einem Einzelabsturz und einem Massensturz ist, dass bei Massenstürzen mehr oder weniger kompakte Felsmassen abbrechen, die während des Abbruchs (entsprechend des vorherrschenden Versagensmechanismus) in Blöcke und Steine fragmentieren, während sich bei Einzelabstürzen lediglich einzelne Kluffkörper aus dem Gefüge der Felswand lösen.

Massenstürze werden entsprechend ihres Abbruchvolumens und ihres Prozessverhaltens in Felsstürze und Felslawinen untergliedert. Hat die Interaktion der Komponenten untereinander einen maßgeblichen Einfluss auf die Mobilität der Sturzmasse, dann verhält sich diese mechanisch ähnlich einer Lawine (»Sturzstrom«; Heim, 1932). Diese hochmobilen Prozesse werden in der aktuellen internationalen Literatur daher zumeist als Felslawine (rock avalanche) oder bei Sturzmassen $> 1 \text{ Mio m}^3$ in der deutschsprachigen Literatur oft als Bergsturz angesprochen. Diese Massenstürze werden im Beitrag 16 zu *Felsgleitung, Felslawine und Erd-/Schuttstrom* detailliert behandelt.

Der vorliegende Beitrag konzentriert sich auf die von Felsböschungen ausgehenden Prozesse Steinschlag, Blockschlag und Felssturz, wobei unter einem Felssturz in diesem Zusammenhang ein Sturzprozess verstanden wird, bei dem die Interaktion der Komponenten untereinander »keinen maßgeblichen Einfluss« auf die Ausbreitung der Sturzmasse hat.

17.1.1 Definition Steinschlag

Evans und Hungr (1993) beschreiben den Prozess Steinschlag als die Bewegung individueller, voneinander unabhängiger Felsfragmente, die mittels episodischer Einschläge mit dem Untergrund interagieren. Die österreichische Richtlinie zum Technischen Steinschlagschutz (ONR 24810:2017) beschreibt Steinschlag gemäß der Definition nach Kienholz et al. (1998) als das Fallen, Springen und Rollen von isolierten Steinen und Blöcken.

Entsprechend den gängigen Definitionen kann der Prozess Steinschlag als die Bewegung individueller, voneinander unabhängiger Felsfragmente (Steine und Blöcke) beschrieben werden, wobei nach Bozzolo (1987) vier Bewegungszustände der Fragmente zu unterscheiden sind: Fallen, Springen, Gleiten und Rollen. Entsprechend vieler Beobachtungen ist der dominante Bewegungszu-

stand jedoch zumeist das Springen, also die Interaktion des Felsfragments mit dem Untergrund mittels episodischer Einschläge. Die übrigen Bewegungszustände (Fallen, Gleiten und, Rollen) beobachtet man vor allem am Beginn und am Ende einer Trajektorie.

Oftmals wird entsprechend der Größe der Fragmente zwischen Steinschlag und Blockschlag unterschieden. Kienholz et al. (1998) definiert den Sturzprozess von isolierten Felsfragmenten mit einem mittleren Durchmesser ≤ 0.5 m als Steinschlag und > 0.5 m als Blockschlag. Diese Unterscheidung findet in der aktuellen Literatur und in der Praxis jedoch nur mehr geringe Beachtung, da sie sich nicht auf den Prozess, sondern lediglich auf die Größe des Fragments bezieht. Als prozessbedingtes Unterscheidungsmerkmal können die vorherrschenden Bewegungszustände während des Sturzprozesses dienen, wobei der dominante Bewegungszustand (zumeist Rollen oder Springen) in hohem Maße von der Fallhöhe, der Länge und der Morphologie (Hangneigung etc.) der Sturzbahn, der Beschaffenheit der Oberfläche (Oberflächenrauheit und Dämpfung) und den Materialeigenschaften des vorherrschenden Gesteins (z. B. Einfluss durch Fragmentierung) abhängig ist. Es gibt aber die generelle Beobachtung, dass große Blöcke im Vergleich zu kleineren Blöcken mehr Rollen als Springen.

Sturzversuche an bewaldeten und nicht bewaldeten natürlichen Hängen (Smith und Duffy, 1990; Dorren et al., 2006; Usiro et al., 2006; Pamminer 2018; Rauscher, 2018) und Rückrechnungen von Steinschlagereignissen haben gezeigt, dass die maximal auftretenden Prozessgeschwindigkeiten im Allgemeinen bei 30 bis 35 m/s liegen. Vereinzelt wurden jedoch auch schon Geschwindigkeiten von über 45 m/s beobachtet bzw. rückgerechnet (z. B. Liechtensteinklamm, siehe Abbildung 01).

17.1.2 Definition Felssturz

Unter dem Prozess Felssturz versteht man den Abbruch (Disposition) einer größeren mehr oder weniger kompakten Felsmasse, die während des Abbruchs – entsprechend dem vorherrschenden Versagensmechanismus (Poisel und Preh, 2004) – in Steine und Blöcke fragmentiert, die in weiterer Folge als mehr oder weniger individuelle Fragmente talwärts stürzen. Bei diesem Prozessstyp hat die Interaktion der Fragmente untereinander während des dem Abbruch folgenden Sturzprozesses »keinen maßgeblichen Einfluss« auf die Mobilität der Sturzmasse. Die Fragmente der Sturzmasse (Steine und Blöcke) verhalten sich entsprechend dem im vorhergehenden Kapitel definierten und beschriebenen Prozess Steinschlag.

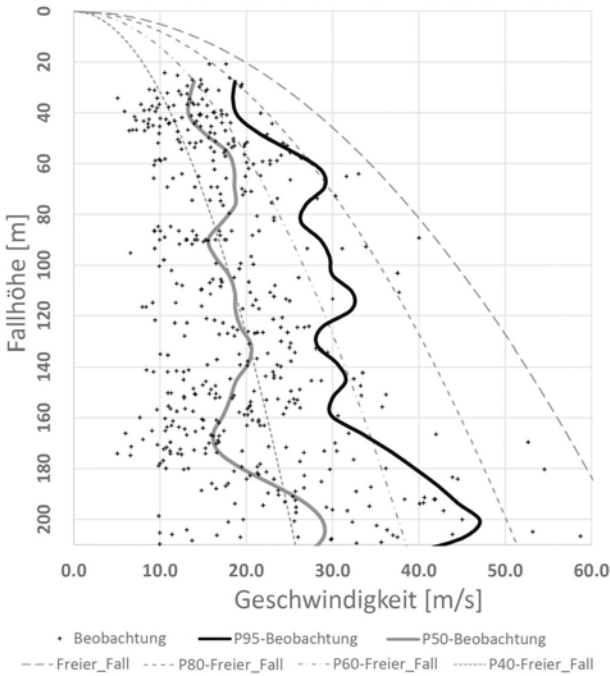


Abbildung 01: Beobachtete Geschwindigkeiten entlang der vertikalen Erstreckung der Versuchsböschung Liechtensteinklamm im Vergleich zur Geschwindigkeit des Freien Falls. Anmerkung: Pxx = 0,xx Perzentil der Geschwindigkeit des freien Falls.

Figure 01: Observed velocities along the vertical section of the test slope Liechtensteinklamm, compared to free fall velocity. Note: Pxx = 0,xx velocity percentile of free fall.

Quellen: Pamninger, 2018; Rauscher, 2018

Gemäß der Definition von BRP et al. (1997) wird bis zu einem Abbruchvolumen von 100 m^3 von Blockschlag gesprochen und bei größeren Volumina von Felssturz. Nach dem heutigen Verständnis können Stein- sowie Blockschlag aus einem Felssturz hervorgehen, wie oben beschrieben.

Abbruch und Sturzprozess sind miteinander gekoppelte Prozesse (Preh, 2016). Ein ideales Modell (Universalmodell) müsste in der Lage sein, diese beiden teilweise sehr unterschiedlichen Prozesse abzubilden. Da wir aber noch über kein zufriedenstellendes Universalmodell verfügen, müssen diese Prozesse rechnerisch derzeit noch getrennt behandelt werden.

17.2 Vom Prozess Steinschlag ausgehende Gefahr und ausgehendes Risiko und deren Beurteilung in Österreich und international (z. B. in der Schweiz)

Gefahr besteht dann, wenn aus einem Prozess Schaden für Personen und/oder Sachgüter entstehen kann (ÖGG, 2014). Die von einem Prozess wie z. B. Steinschlag ausgehende Gefahr wird einerseits durch seine Intensität und andererseits durch seine Eintrittswahrscheinlichkeit bestimmt (siehe Gleichung 17.1). Die Kombination von Intensität (I , intensity) und Eintrittswahrscheinlichkeit (P , probability) zur objektivierten Beurteilung der Gefahr (H , hazard) macht insofern Sinn, da im Allgemeinen lediglich ein möglicher großer Schaden mit einer subjektiv als hoch empfundenen Eintrittswahrscheinlichkeit als Gefahr wahrgenommen wird. Ein unwahrscheinliches Eintreten eines großen Schadens wird hingegen im Allgemeinen kaum als große Gefahr empfunden. Umgekehrt wird es auch nicht als große Gefahr empfunden, eine Geldmünze zu verlieren, obwohl bei diesem Szenario die Eintrittswahrscheinlichkeit eher hoch ist.

$$H = I * P \quad (17.1)$$

Durch die hohen Geschwindigkeiten, die herabfallende Blöcke erreichen können, kann die Intensität – selbst von kleinen Steinen – sehr hoch sein. Für Personen kann daher der kleinste stürzende Stein eine große Gefahr darstellen, umso mehr, weil die Eintrittswahrscheinlichkeit für das Fallen kleiner Steine im Allgemeinen höher ist als für große. Für Infrastrukturen besteht erst ab einer gewissen Intensität Gefahr.

Das von Scheinschlag ausgehende Risiko (Corominas, 2005, 2012) berücksichtigt den von Steinschlag verursachbaren Schaden für Personen und Infrastrukturen. Das von Steinschlag ausgehende Risiko (R) wird durch das Produkt der von Steinschlag ausgehenden Gefahr (H), dem der Gefahr ausgesetzten Personen und Infrastrukturen (E , exposure), der Vulnerabilität der ausgesetzten Personen und Infrastrukturen (V) und den Kosten der gefährdeten Subjekte/Objekte (C , cost) ermittelt:

$$R = H * E * V * C \quad (17.2)$$

17.2.1 Österreich

Im Rahmen der Partnerschaft der Österreichischen Raumordnungskonferenz (ÖROK) »Risikomanagement für gravitative Naturgefahren in der Raumpla-

nung« wurden fachliche Empfehlungen und ein Materialienband erarbeitet (Bäk et al., 2015), in dem eine standardisierte Vorgangsweise zur Abgrenzung von Steinschlaggefährdungsbereichen (Gefahrenzonierung) beschrieben wird. Bei den Empfehlungen werden für die Gefahrenzonierung von Sturzprozessen drei Maßstabebenen unterschieden: regionale Ebene, kommunale Ebene und Objektbene. Folgend wird der derzeitige Stand in Österreich im Kontext mit den Empfehlungen der ÖROK skizziert.

Einige Bundesländer verfügen über regionale Gefahrenhinweiskarten ($M=1:25.000-50.000$), welche auf durch Steinschlag gefährdete Bereiche hinweisen (z. B. NÖ Atlas, Land Niederösterreich, 2018). Es handelt sich hierbei lediglich um die Ausweisung von Gebieten, die von Steinschlag betroffen sein können. Die betroffenen Gebiete werden in dieser Maßstabebene zumeist mithilfe des Pauschalwinkelansatzes »Geometrisches Gefälle« (Hsü, 1975; Grundner, 1984; Domaas, 1985; Evans und Hungr, 1993) ermittelt. Gefahrenhinweiskarten beinhalten keine Angaben über Intensitäten oder Wahrscheinlichkeiten. In der derzeitigen Gefahrenzonenplanung der Wildbach- und Lawinerverbauung (gemäß Forstgesetz 1975 (§11) bzw. Gefahrenzonenplanverordnung BGBL 436/1976) ist ebenfalls keine differenzierte Abgrenzung von Gefahrenzonen für Steinschlag vorgesehen. Auch hier erfolgt lediglich die Ausweisung von »Braunen Hinweisbereichen«, die möglicherweise von Rutschungen oder Steinschlag betroffen sein können.

Auf der kommunalen Ebene arbeiten derzeit mehrere Bundesländer an der Erstellung von Gefahrenkarten ($M=1:10.000-25.000$) für einzelne Gemeinden (z. B. Forschungsprojekt NÖ-Talus). Gefahrenkarten geben an, welche raumrelevanten Bereiche und wie stark diese von Steinschlag betroffen sein können. Die Abschätzung der Reichweite und der Intensität erfolgt in dieser Maßstabebene zumeist mithilfe von Prozessmodellen (verfügbare Modelle siehe Volkwein et al., 2011; Preh und Mölk, 2018). Die Prozessintensität wird hier zumeist entsprechend der berechneten kinetischen Energien beurteilt.

Basierend auf den Empfehlungen der ÖROK (Bäk et al., 2015) wird zurzeit von der Wildbach- und Lawinerverbauung (WLV) eine Methodik entwickelt und installiert (Mölk und Rieder, 2017), mit deren Hilfe Gefährdungsbereiche nachvollziehbar identifiziert und dargestellt werden können. Dabei ist eine Differenzierung der Gefährdungslage entsprechend der mithilfe von Prozessmodellen berechneten kinetischen Energien erforderlich. Bei dem von Mölk und Rieder (2017) präsentierten Vorschlag wird der Bemessungsblock zur Berechnung der energie-kodierten Trajektorien (siehe Abbildung 02) gemäß der Richtlinie für Technischen Steinschlagschutz (ONR 24810:2017) bestimmt. Daher sind bei dieser Vorgangsweise Wahrscheinlichkeiten indirekt über die Bestimmung des Bemessungsblocks enthalten.

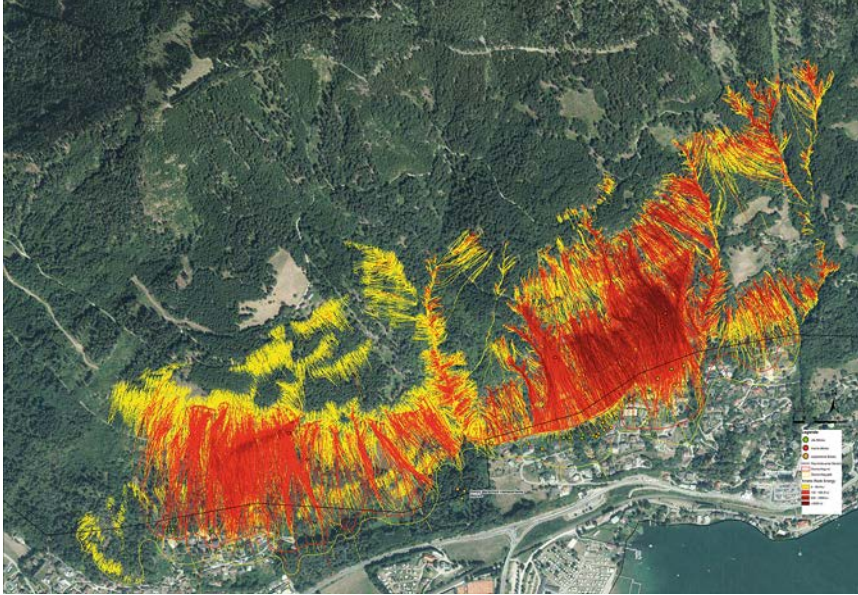


Abbildung 02: Energie-kodierte Trajektorien für eine Gefahrenhinweiskarte/einen Gefahrenzonenplan; gelb: ≤ 100 kJ, rot: > 100 kJ.

Figure 02: Energy-encoded trajectories for hazard indication maps/hazard zone plans; yellow: ≤ 100 kJ, red: > 100 kJ.

Quelle: Preh und Mölk, 2018

Die Beurteilung des von Steinschlag ausgehenden Risikos sowie die Bemessung von entsprechenden technischen Schutzmaßnahmen auf der Objektebene ($M \geq 1:5.000$) erfolgt in Österreich mithilfe der ONR 24810:2017 Technischer Steinschlagschutz. Das zentrale Element dieser Richtlinie ist die Dimensionierung einer sogenannten Bemessungsblockgröße. Dabei werden über die Ereignisfrequenzklassen die Ereigniswahrscheinlichkeit (P), über die Schadensfolgekassen der potentielle Schaden ($E * V * C$, siehe Gleichung 17.2) und über den Bemessungsblock die Intensität (I) berücksichtigt. Die Abschätzung der Reichweite, der Sprunghöhen und der Intensitäten erfolgt dabei zwingend mit Prozessmodellen (verfügbare Modelle siehe Volkwein et al., 2011; Preh und Mölk, 2018). Zur Bemessung von Schutzbauwerken werden sowohl Einwirkung als auch Widerstand mit Teilsicherheitsbeiwerten in Abhängigkeit der Schadensfolgekasse beaufschlagt.

Zur Ermittlung der Bemessungsblockgröße sind in Abhängigkeit von der Schadensfolgekasse CC (siehe Tabelle 01) bzw. der Anzahl der potentiellen Sturzblöcke zwei verschiedene Varianten zulässig.

Tabelle 01: Schadensfolgeklassen CC 1 bis CC 3.**Table 01:** Consequence classes CC 1 to CC 3.

Quelle: ONR 24810:2017, 2017

Schadensfolgekategorie CC	Merkmale	Beispiele
CC 1	Geringe Folgen für Menschenleben und kleine oder vernachlässigbare wirtschaftliche, soziale oder umweltbeeinträchtigende Folgen	Landwirtschaftliche Gebäude ohne regelmäßigen Personenverkehr, sowie Scheunen, Gewächshäuser, Forst- und Güterwege mit geringer Verkehrsfrequenz
CC 2	Mittlere Folgen für Menschenleben, beträchtliche wirtschaftliche, soziale oder umweltbeeinträchtigende Folgen	Wohn- und Bürogebäude, öffentliche Gebäude mit mittleren Versagensfolgen, wie Bürogebäude, regionale und überregionale Verkehrswege mit $DTV \leq 50.000$ KFZ
CC 3	Schwerwiegende Folgen für Menschenleben oder sehr große wirtschaftliche oder umweltbeeinträchtigende Folgen	Tribünen, öffentliche Gebäude mit hohen Versagensfolgen, wie Konzerthallen, regionale und Überregionale Verkehrswege mit $DTV > 50.000$ KFZ

Die Schadensfolgen werden in drei Klassen unterschieden, mithilfe derer die verschiedenen möglichen Folgen für Personenleben, Wirtschaft oder Umwelt als gering, mittel oder schwerwiegend eingeteilt werden.

Für die Schadensfolgeklassen CC 2 und CC 3 oder für den Fall, dass mehr als 100 potentielle Sturzblöcke vorliegen, ist eine statistische Erfassung der Bemessungsblockgröße im Ablöse- und/oder Ablagerungsbereich durchzuführen. Bei dieser (ersten) Variante basiert die Wahl der Bemessungsblockgröße auf der im Zuge der Grundlagen- und Geländeerhebung erstellten Blockgrößenverteilung sowie auf einer geschätzten Ereignisfrequenz (siehe Tabelle 02, Abbildung 03).

Die Ereignisfrequenz wird in vier Klassen unterschieden, mithilfe derer die Häufigkeit von Steinschlag zwischen mehr als zehn Ereignissen pro Jahr und weniger als einem Ereignis alle 30 Jahre eingeteilt wird. Entsprechend dieser Klassen wird die Bemessungsblockgröße durch ein Volumens-Perzentil der Blockgrößenverteilung zwischen 98 % und 95 % festgelegt.

Die (zweite) vereinfachte Variante einer gutachterlichen Festlegung ist dann zulässig, wenn höchstens 100 Klufkörper in den potentiellen Ablösebereichen dokumentierbar sind, wenn lediglich Schadensfolgekategorie CC 1 potentiell betroffen ist oder bei Ereignisfrequenzklasse EF 1 und EF 2.

Tabelle 02: Ereignishäufigkeiten und zugeordnetes Volumens-Perzentil der Blockgrößenverteilung.

Table 02: Event frequency classes and assigned volume percentiles of the block size distribution.

Quelle: ONR 24810:2017, 2017

Ereignisfrequenzklasse	Ereignishäufigkeit n (1/a)	Fraktile für die Bemessungsblockgröße
EF 4 (sehr hoch)	$n \geq 10$ (≥ 10 Ereignisse/Jahr)	V_{98}
EF 3 (hoch)	$1 \leq n < 10$ (1 bis 10 Ereignisse/Jahr)	V_{97}
EF 2 (gering)	$0,03 \leq n < 1$ (1 Ereignis/1 bis 30 Jahre)	V_{96}
EF 1 (selten)	$n < 0,03$ (< 1 Ereignis/30 Jahre)	V_{95}

Zur Ermittlung der Bemessungswerte für die Einwirkungen auf ein technisches Schutzbauwerk werden das 99 %-Perzentil der Einwirkungs-Energie-Verteilung sowie das 95 %-Perzentil der Sprunghöhenverteilung des Bemessungsblocks aus der Prozess-Modellierung mit Teilsicherheitsbeiwerten beaufschlagt, die durch die Schadensfolgeklasse CC bestimmt werden.

Zur Bemessung des Steinschlagschutznetzes, also auf der Widerstandsseite, werden der Nennwert der Energieklasse gemäß ETAG sowie die zugelassene Netzhöhe durch Teilsicherheitsbeiwerte, die ebenfalls durch die Schadensfolgeklasse bestimmt werden, abgemindert.

Bei Steinsschlagschutzdämmen werden beim geotechnischen Nachweis der Tragfähigkeit auf der Widerstandsseite die Materialfestigkeiten mithilfe von Teilsicherheitsbeiwerten abgemindert, bei der Einwirkung der Sprunghöhe kommt bei Dämmen ein Sicherheitszuschlag in Form eines Freibords zur Anwendung, welcher sicherstellt, dass das Bauwerk vom Sturzblock nicht überrollt wird. Der Freibord wird gem. ONR 24810:2017 in Abhängigkeit von der Bauweise (Erddamm, Steinschlichtung, bewehrte Erde) und von der Böschungsneigung festgelegt.

Starre Wände (z. B. Holzbohlenwände) haben i. d. R. keine definierte Energieaufnahmekapazität (keine Europäische Technische Bewertung – CE-Kennzeichnung – wie z. B. Schutznetze) bzw. eine vergleichsweise kurze Lebensdauer und werden daher üblicherweise nicht empfohlen. Stahlbetonwände können für relativ geringe Energie-Einwirkungen (< 300 kJ) eingesetzt werden, gem. Egli (2005) können bewehrte Betonwände mit einer Stärke von 30 cm bis zu 100 kJ aufnehmen, mit 50 cm bis zu 300 kJ. Eine standardisierte Vorgangsweise für die

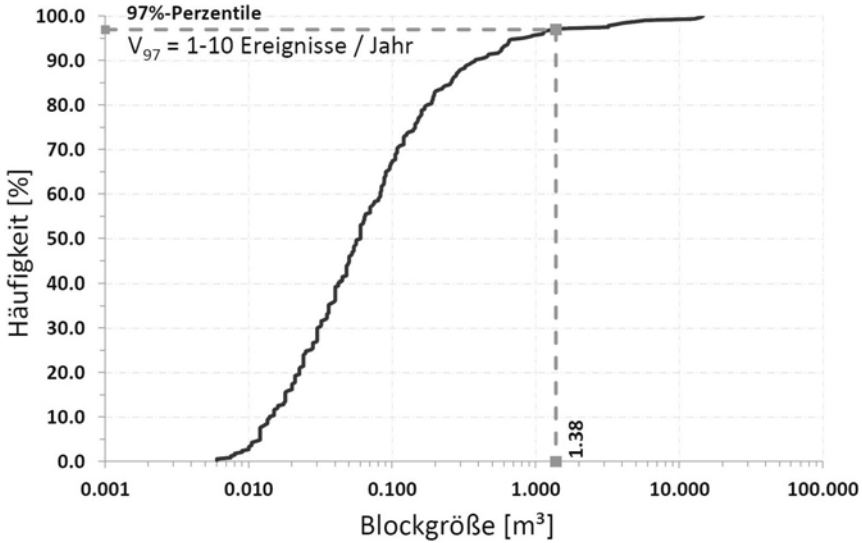


Abbildung 03: Bestimmung des Bemessungsblock-Volumens aus statistischen Aufnahmen einer Schutthalde mittels Linienzählverfahren.

Figure 03: Determination of design block volume from statistic talus investigations using the so-called »line counting approach«.

Quelle: Preh und Mölk, 2018

Bewertung von dynamischen Punktlasten auf starre Konstruktionen existiert derzeit nicht.

17.2.2 Schweiz

Die Schweiz verfolgt die Strategie, dass Risiken nur dann vermieden werden können, wenn sie als solche erkannt werden. Gefahrenhinweis- und Gefahrenkarten dienen dabei als wichtige Grundlage. Die Schweiz verfügt über eine flächendeckende Gefahrenkarte, die eine detaillierte Übersicht über die regionale und lokale Gefährdungssituation in fünf Gefahrenstufen gibt: rot – erhebliche Gefährdung, blau – mittlere Gefährdung, gelb – geringe Gefährdung, gelb-weiß gestreift: Restgefährdung, weiß – keine oder vernachlässigbare Gefährdung (ARE, BWG und BUWAL, 2005).

Die Gefahrenkarte bildet die Grundlage für die Nutzungsplanung (z. B. Ausscheidung von Gefahrenzonen), zur Formulierung von Bauauflagen sowie für die Planung von Schutzmaßnahmen. Sie enthält, zusammen mit den dazugehörigen technischen Berichten, detaillierte Angaben über Ursachen, Ablauf,

räumliche Ausdehnung, Intensität und Eintrittswahrscheinlichkeit von Gefahrenprozessen.

Die Gefahrenstufen zeigen den Grad der Gefährdung von Personen, Infrastrukturen und Sachwerten auf. Sie ergeben sich aus der Intensität und der Eintrittswahrscheinlichkeit der einzelnen Gefahrenprozesse, gemäß dem sog. Intensitäts-Wahrscheinlichkeits-Diagramm (siehe Abbildung 04). Diese Parameter müssen somit für sämtliche maßgebliche Gefahrenprozesse festgelegt werden.

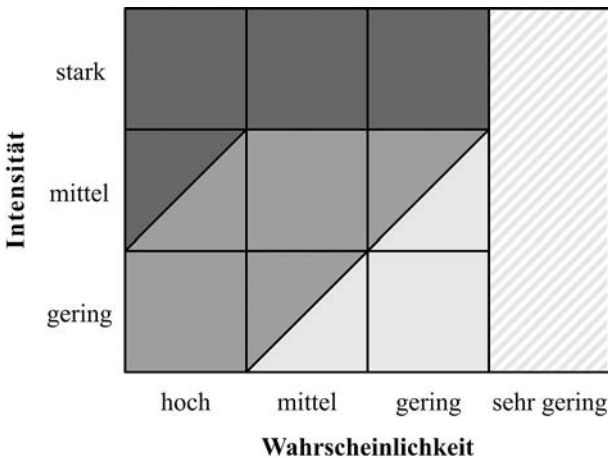


Abbildung 04: Gefahrenstufen: Ergebnis von Intensität und Wahrscheinlichkeit.

Figure 04: Hazard levels: result of intensity and probability.

Quelle: ARE, BWG und BUWAL, 2005, S. 16

Bei identifizierten Schutzdefiziten werden Schutzmaßnahmen vorgenommen, wobei danach die Gefahrenkarte der neuen Gefährdungssituation angepasst wird (»Gefahrenkarte nach Maßnahmen«).

In der Schweiz wird der Bevölkerung sehr offen kommuniziert, dass trotz Schutzmaßnahmen keine absolute Sicherheit gewährleistet werden kann. Um das Ausmaß des verbleibenden Restrisikos abschätzen zu können, werden die Auswirkungen von Extremereignissen überlegt. Mithilfe von Notfallplanung wird versucht, das verbleibende Risiko auf ein akzeptables Maß zu reduzieren.

17.3 Was ist ein Extremereignis?

17.3.1 Allgemein

Ein Extremereignis ist ein Ereignis, das auf den ersten Blick unmöglich erscheint. Es scheint deswegen unmöglich, weil es einerseits ausreichend selten (> 100 Jahre) auftritt, sodass man es »noch nie gesehen« hat und es daher unerwartet war, und weil andererseits die Magnitude des Ereignisses so groß ist, dass enormer Schaden entstehen kann. Extremereignisse können katastrophale Auswirkungen auf Personen, Infrastruktur, Umwelt und/oder Sachwerte haben.

Das Tiroler Katastrophenmanagementgesetz definiert Katastrophen folgendermaßen: »Katastrophen sind durch elementare oder technische Vorgänge oder von Personen ausgelöste Ereignisse, die in großem Umfang das Leben oder die Gesundheit von Personen, die Umwelt, das Eigentum oder die lebensnotwendige Versorgung der Bevölkerung gefährden oder schädigen.« (T-KMG, 2006)

Das Katastrophenschutzgesetz Berlin definiert Katastrophen als »Großschadenereignisse, die [...] von den für die Gefahrenabwehr zuständigen Behörden mit eigenen Kräften und Mitteln nicht angemessen bewältigt werden können.« (KatSG, 1999)

Interpol definiert eine Katastrophe als »unerwartetes Ereignis, bei dem zahlreiche Personen getötet oder verletzt werden« (Interpol, 2009).

Häufig sind durch Extremereignisse die Infrastruktur (Straßen, Brücken, Energie-/Wasserversorgung) und/oder die örtlichen Hilfskräfte und Hilfsressourcen (Polizei, Feuerwehr, Krankenhäuser) selbst beeinträchtigt oder geschädigt.

17.3.2 In Bezug auf Steinschlag

Extremereignisse bei Steinschlag können in Bezug auf die Magnitude und die Ereigniswahrscheinlichkeit definiert werden, Extremsituationen in Bezug auf die Ausgesetzttheit (exposure) und die Vulnerabilität. Treten Extremereignisse in Extremsituationen auf, kann man von einer Katastrophe sprechen (siehe Definition Interpol).

Extremereignisse – in Bezug auf die Magnitude – sind jene Ereignisse, die größer sind, als das Bemessungsereignis (Überlastfall). In Österreich erfolgt die Festlegung des Bemessungsereignisses gemäß der Richtlinie für den Technischen Steinschlagschutz (ONR 24810:2017, 2017) anhand der Einwirkung eines Bemessungsblocks. Die Festlegung des Bemessungsblocks erfolgt, wie in Kapitel 17.2.1 beschrieben, mit einer statistischen Repräsentanz in Abhängigkeit

der Ereignisfrequenz. Alle erfassten Blöcke größer dem festgelegten Bemessungsblock werden als extreme Einwirkungen (Extremereignisse) in Kauf genommen. Diese extremen Einwirkungen (Magnituden) werden gemäß ONR bewusst nicht als Grundlage einer Maßnahmendimensionierung herangezogen, da eine solche Vorgangsweise häufig extrem kostspielig bzw. technisch nicht möglich ist. Eine vollkommene Sicherheit von Personen und Infrastruktur kann nicht gewährleistet werden. Daher muss ein akzeptiertes Restrisiko (akzeptierte Todesfallwahrscheinlichkeit; ÖGG, 2014) festgelegt werden.

Die Vorgangsweise in der Schweiz ist bei der Festlegung von Schutzmaßnahmen auf der Objektebene mit jener in Österreich identisch.

Extremsituationen sind Situationen mit extrem hohem Schadenspotential. Darunter sind ungewöhnlich hohe Aufenthaltswahrscheinlichkeiten und Vulnerabilitäten zu verstehen, also jene von Steinschlag betroffenen Orte, an denen die Ausgesetztheit (exposure) und Vulnerabilität von Personen und Infrastruktur sehr hoch sind. Dort können selbst kleinste Magnituden große Konsequenzen haben (z. B. Besucherstrom in der Liechtensteinklamm, Salzburg).

17.3.3 In Bezug auf Felssturz

Infolge des Abbruchs und der Fragmentierung größerer Felsmassen kommt es zu zeitgleichen Ereignissen (Steinschlag und Blockschlag) und in weiterer Folge zu zeitnahen Einwirkungen auf Personen und Infrastruktur/Schutzmaßnahmen. Diese zeitnahen Einwirkungen stellen per se ein Extremereignis durch Steinschlag/Blockschlag dar (siehe Fallbeispiel Hinterhornbach 2011 in Kapitel 17.5.2).

Für die aus einem Felssturz resultierenden Einzelereignisse (Steinschlag/Blockschlag) gelten bezüglich der Definition von Extremereignissen und Extremsituationen die beim Steinschlag getroffenen Aussagen.

17.4 Entwicklung von Extremereignissen im alpinen Raum

17.4.1 Aufzeichnungen

Seit Ende des 19. Jahrhunderts gibt es Aufzeichnungen von Steinschlag- und Felssturz-Ereignissen in Österreich (Sass und Oberlechner, 2012). Es ist anzunehmen, dass dokumentierte Ereignisse im 19. und 20. Jahrhundert eher größere Auswirkungen auf Personen und/oder Infrastruktur hatten, weil man sich andernfalls nicht die Mühe der damals vergleichsweise aufwendigen Dokumentation gemacht hätte. Seit dem 21. Jahrhundert gibt es aufgrund der ver-

mehrten Nutzung von Computern und dem Internet sowie der zunehmenden Besiedelung von gebirgigen Regionen jährlich mehr dokumentierte Steinschlag- und Felssturz-Ereignisse. Die Auswirkungen der dokumentierten Steinschlag- und Felssturz-Ereignisse auf Personen und Infrastruktur wurden im Zuge dieser Arbeit nicht analysiert; daher können an dieser Stelle keine direkten Aussagen über die Entwicklung von Extremereignissen gemacht werden. Jedenfalls sind die vorhandenen Aufzeichnungen lückenhaft und nicht repräsentativ. Jede Schlussfolgerung aus diesen Daten muss mit Vorsicht behandelt werden.

In den letzten Jahren werden in wissenschaftlichen und nichtwissenschaftlichen Medien vermehrt Bedenken einer höheren Gefahr durch (extreme) Naturereignisse in Folge des Klimawandels geäußert. Sass und Oberlechner (2012) haben die in Österreich vorhandenen Aufzeichnungen verglichen und untersucht, um einen allumfassenden Trend durch Klimawandel festzustellen. Dabei wurden keine mikroklimatischen Situationen (wie Frost-Tau-Wechsel oder Porenwasserdruck) berücksichtigt.

Die Analyse von 252 Ereignissen in Österreich zwischen 1900 und 2010 mit einem Abbruchvolumen zwischen 10^2 und 10^6 m³ hat gezeigt, dass nur 9 % der Ereignisse in Höhenlagen über 2.100 m (wo Permafrost möglich wäre) auftreten. Mit wärmeren Sommern kann es vermehrt zu Ereignissen in zurückweichenden Permafrost-Regionen kommen. Es gibt aber derzeit keinen Hinweis darauf, dass die Ereignisse unter 2.100 m (außerhalb der Permafrost-Zone) wegen des Klimawandels zunehmen. Eine Untersuchung in der Schweiz ergab das Gleiche (Gruner, 2008). Im jährlichen Vergleich mit Niederschlag und Temperatur kann keine Korrelation zwischen Erwärmung und Ereignissen gefunden werden. Bestenfalls ist eine Tendenz zu mehr Ereignissen in kalten Jahren, besonders nach kalten Wintern (ev. wegen Frostsprengung), festzustellen. Die meisten Ereignisse finden im Frühling statt, vermutlich wegen vermehrter Wasserzufuhr durch Schmelzwasser und Regen (Porenwasserdruck) sowie durch Frost-Tau-Wechsel. Die Auswertungen von Niederschlagsdaten bis 2006 zeigen keine Zunahme an extremen Niederschlägen in Österreich (Rudel 2008). Aufgrund von wärmeren Wintern könnte es eine saisonale Verschiebung der Ereignis-Häufigkeit von Frühling auf Sommer geben, aber auf eine allgemeine Ereignis-Zunahme gibt es keine Hinweise (Sass und Oberlechner, 2012).

17.4.2 Schutzmaßnahmen

Seit Beginn der Umsetzung von Schutzbauwerken gegen Steinschlag in der Mitte des 20. Jahrhunderts kam es immer wieder zum Versagen solcher Systeme. Das Versagen hatte durchaus unterschiedliche Ursachen. Diese können einerseits in einer unzureichenden Bemessung der aufzunehmenden Einwirkungen und/

oder andererseits in einer vom Bauwerkstyp oder hinsichtlich ihres Widerstandes ungeeigneten Schutzmaßnahme liegen. Grundsätzlich ist jedoch festzuhalten, dass das Versagen von Schutzmaßnahmen sehr häufig aus einer Überlastsituation resultiert (siehe Kapitel 17.5). Das heißt, dass eine Maßnahme für eine bestimmte Ereignismagnitudo und in Abhängigkeit von deren Häufigkeit in Hinblick auf Einwirkung und Widerstand ausgelegt ist, die jedoch nicht das größtmögliche Lastbild darstellt.

Es gibt jährlich mehr Fallbeispiele von durch Extremereignissen zerstörten Schutzbauwerken. Wie weit dies jedoch auf eine zunehmende Anzahl von Extremereignissen und/oder auf eine zunehmende Anzahl von Schutzbauwerken zurückzuführen ist, ist anhand der heutigen Datenlage nicht verifizierbar.

17.5 Fallbeispiele

Im Folgenden werden einige Fallbeispiele in Österreich aufgezeigt, die gemäß den in Kapitel 17.3 festgelegten Definitionen als Extremereignisse bezeichnet werden können. Die meisten in Österreich beobachteten und dokumentierten Extremereignisse stehen im Zusammenhang mit dem Versagen von Schutzsystemen (sogenannten Überlastfällen, siehe Kapitel 17.3.2 oder Beitrag 13 zu *Überlastfälle*).

17.5.1 Einzelereignisse/Steinschlag

17.5.1.1 Blöslahna/Huben 2013 (5 m³ Block)

Bei diesem Steinschlagereignis kam es zur Ablösung eines mehrere 10er m³ großen Felsbereiches im Oberhang. Die abgelösten Felspartien stürzten relativ konzentriert über eine steile Rinne mit geringer Bodenbedeckung und daher relativ ungünstigen Dämpfungseigenschaften in Richtung Unterhang. Innerhalb der Rinne befanden sich mehrere Reihen von Lawinenverbauungen (Stahlschneebrücken), die von den Sturzblöcken durchschlagen und weitgehend zerstört wurden (siehe Abbildung 05a). Ein beträchtlicher Teil der Sturzmasse kam im Bereich der Lawinenverbauungen jedoch auch zur Ablagerung. Der im Bereich des talseitigen Rinnenausgangs situierte Waldbestand wurde teilweise zerstört bzw. schwer beschädigt. Bergseits einer hier befindlichen Landesstraße bzw. einer darüber gelegenen Hofzufahrt befand sich ein Steinschlagschutznetz mit einer Energieaufnahme-Kapazität von 5.000 kJ und einer Bauwerkshöhe von 6 m. Der am weitesten in den Unterhang vordringende Sturzblock wies eine Kubatur von ca. 5 m³ auf. Dieser Block beaufschlagte das Randfeld des Schutz-

netzes, das aufgrund der Randposition in der Netzreihe eine reduzierte Energieaufnahmekapazität aufweist und den Sturzblock dennoch schadlos aufnehmen konnte (siehe Abbildung 05b). Die Energieaufnahmekapazität der Hauptfelder des Netzes von 5.000 kJ würden einer Translationsgeschwindigkeit eines 5 m³ Blockes mit einer Masse von ca. 14 Tonnen von ca. 27 m/s entsprechen. Dies ist ein Wert, der für die vorliegenden Hangneigungen tendenziell gering ist.

Damit handelte es sich bei dem gegenständlichen Ereignis tendenziell um einen Überlastfall, der Bemessungsblock für das Schutznetz war < 5 m³, die Schutzmaßnahme hat dennoch den Sturzblock aufgehalten und keine signifikanten strukturellen Schäden erlitten.

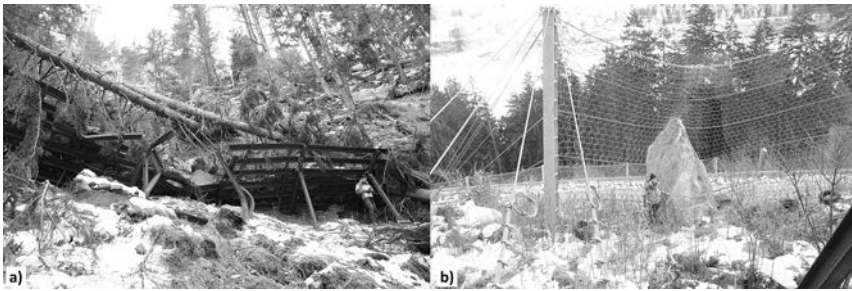


Abbildung 05: Blöslahna/Huben: durch Blockschlag zerstörte Stahlschneebrücke in der Sturzbahn (a); im Steinschlagschutznetz aufgefangener Großblock mit ca. 5 m³ bzw. 14 Tonnen (b).

Figure 05: Blöslahna/Huben: steel snow bridge along the rock fall path, demolished by block impact (a); 5 m³ large resp. 14 ton heavy block, held by rock fall protection net (b).

Fotos: WLV Geologie

17.5.1.2 See im Paznaun 1996 (30 m³ Block)

Ein Blockschlag-Ereignis mit einem ca. 30 m³ großen Felsblock schlägt in die bergseitige Wand eines Wohnhauses ein und beschädigt es so schwer, dass es nicht mehr bewohnbar ist (siehe Abbildung 06). Aufgrund möglicher Folgeereignisse und den stark eingeschränkten Platzverhältnissen wird das schwer beschädigte Gebäude geräumt und abgetragen. An seiner Stelle wird ein Steinschlagschutzdamm errichtet, der seither die darunterliegenden Gebäude vor Steinschlag schützt.



Abbildung 06: See im Paznaun: Großblock mit ca. 30 m^3 beschädigt ein Wohnhaus so stark, dass es nicht mehr bewohnbar ist.

Figure 06: See im Paznaun: 30 m^3 large block demolishes a house, leaving it uninhabitable.
Foto: WLW Geologie

17.5.2 Felsstürze

17.5.2.1 Hinterhornbach 2011 (Abbruchvolumen einige 10 m^3)

Hier kam es 2011 zu einem kleinen Felssturz mit einigen 10 m^3 Gesamtsturzmasse. Dabei wurde das auf einen Einzelblock mit einer Kubatur von $0,5 \text{ m}^3$ ausgelegte Steinschlagschutznetz im Unterhang von mehreren Sturzblöcken, die vermutlich zeitnah das Netz erreichten, überwunden. Das Netz wurde im Bereich der Belastung weitgehend zerstört (Seilrisse, geknickte Stützen, siehe Abbildung 07a).

17.5.2.2 Holderbach 2006 (Abbruchvolumen einige 100 m³)

Im Holderbach wurde in einer Steinschlagrinne als Steinschlagschutzmaßnahme einer dort befindlichen Wildbachbaustelle ein Steinschlagschutznetz in einer Felsrinne errichtet. 2006 kam es zu einem Felssturz mit einigen 100 m³. Dabei wurde das Netz vollständig überlastet, die Stützen geknickt und die Seilanker ausgezogen (siehe Abbildung 07b). Dabei handelte es sich um einen klassischen Überlastfall, auf den das Netz nicht ausgelegt war, Dennoch wurde der überwiegende Teil der Sturzblöcke hinter dem Netz abgelagert.

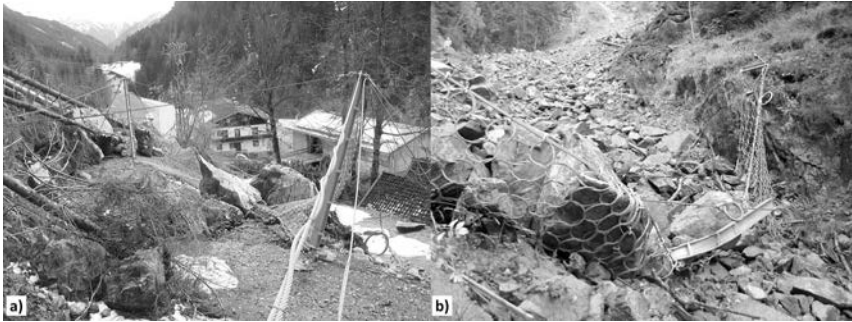


Abbildung 07: Kleiner Felssturz in Hinterhornbach mit einigen 10er m³ Sturzmasse, Überlastfall für Schutzmaßnahmen (dimensioniert auf 500 kJ) (a); Felssturz im Holderbach mit einigen 100er m³ Sturzmasse, Überlastfall für Schutzmaßnahmen (b).

Figure 07: Small rockfall of some 10 m³ falling mass in Hinterhornbach, overload case for protective measures (dimensioned at 500 kJ) (a); Rockfall of some 100 m³ falling mass in Holderbach, overload case for protective measures. (b).

Fotos: WLV Geologie

17.5.2.3 Vals 2017 (Abbruchvolumen ca. 117.000 m³)

Der Felssturz/die Felslawine in Vals (siehe Abbildung 8) ereignete sich am 24. Dezember 2017 und wurde als Massensturz nicht vorhergesehen, obwohl es mehrfach detaillierte geologische Aufnahmen im gegenständlichen Hangabschnitt gegeben hat. In der Hauptsturzbahn befanden sich neben der Landesstraße lediglich landwirtschaftliche Flächen. Gebäude waren vom Ereignis nicht bzw. nicht direkt betroffen, wenn auch nur wenige Meter bis 10er Meter von den Ausläufern der Sturzmasse entfernt.

Geologisch befindet sich der Abbruch des Felssturzes Vals in der Schieferhülle des Tauernfensters und die anstehenden Gesteine sind Kalkschiefer mit Bereichen mit Graphitphylliten.

Aus den Differenzen der mithilfe von mehreren Lidar-Kampagnen für die Situation vor und nach dem Felssturz ermittelten digitalen Höhenmodelle

konnte das Abbruchvolumen von ca. 117.000 m³ und das Ablagerungsvolumen von rund 130.600 m³ bestimmt werden. Die Volumszunahme infolge Erosion und Auflockerung der stark fragmentierten Sturzmasse beträgt somit ca. 11,5 %.

Der hohe Fragmentierungsgrad der Ablagerungen (vor allem im Zentrum, siehe Abbildung 9a) des Massensturzes weist eindeutig darauf hin, dass eine starke Interaktion der Komponenten der Sturzmasse beim Massensturz stattgefunden haben muss und dass man beim Massensturz Vals eher von einer Felslawine als von einem Felssturz sprechen muss.



Abbildung 08: Felssturz bzw. Felslawine Vals: Abbruchvolumen ca. 117.000 m³, Überlastfall für die Schutzmaßnahmen im Randbereich (dunkler Kreis).

Figure 08: Rock fall/rock avalanche of 117.000 m³ falling mass in Vals, overload case for protective measures at the border area (dark circle).

Foto: WLV Geologie

Wie bei vielen Felsstürzen/Felslawinen dieser Größenordnung ereigneten sich auch beim Massensturz Vals mehrere (sekundäre) Stein- und Blockschlagereignisse an den seitlichen Rändern der Sturzbahn und des Ablagerungsbereichs. Ein Großblock zerstörte dabei mehrere Felder eines Steinschlagschutznetzes (siehe Abbildung 9b). Der Großblock stellte mit seinem Volumen von ca. 5 m³ für das für Einzelblöcke bis zu 1 m³ dimensionierte Netz einen Überlastfall, d. h. ein Extremereignis dar.



Abbildung 09: Felssturz bzw. Felslawine Vals: Überlastfall für die Schutzmaßnahmen im Randbereich (dunkler Kreis) (a). Blockschlagereignis am seitlichen Rand des Ablagerungsbereichs zerstört mehrere Felder eines Steinschlagschutznetzes. Die Blockgröße von ca. 5 m^3 stellt im Sinne des Bemessungsblockes (1 m^3) ein Extremereignis dar (b).

Figure 09: Rock fall/rock avalanche Vals: overload case for protective measures at the border area (dark circle) (a). Block impact at the border area demolishes several rockfall protection net segments. The 5 m^3 block size represents an extreme event, with respect to the 1 m^3 design block (b).

Fotos: WLV Geologie

17.6 Einschätzung des Sachstandes und der Unsicherheiten

17.6.1 Stand der Technik

Der Themenbereich Steinschlag ist ausgehend von der Erhebung der prozessrelevanten Parameter über die Festlegung der maßgeblichen Bemessungsgrößen bis zur Kenntnis der charakteristischen Widerstände der Schutzsysteme in Österreich bzw. der EU umfassend genormt und standardisiert (vgl. ONR 24810:2017 Technischer Steinschlagschutz; ETAG027 European Technical Assessment of Rock-Fall Protection Kits).

Für Felsstürze und Felslawinen fehlt derzeit in Österreich eine systematische Erfassung von potentiellen Gefahrenbereichen und den daraus resultierenden Wirkungsräumen bzw. potentiellen Konflikten mit Schutzgütern.

Nachfolgend wird der Stand der Technik für einzelne Aspekte der Prozessbewertung dargestellt:

- Standards für die Kartierung und Erhebung der prozessrelevanten Daten sind in der ONR 24810:2017 beschrieben. Darin ist z. B. festgelegt, dass folgende Inhalte zu erheben und zu bewerten sind:
 - a) die potentiellen Gefahrenbereiche sind flächig zu kartieren,
 - b) die Ablösebereiche sind in Hinblick auf ihre Verbandsfestigkeit und Kluftkörper-größen zu bewerten,

- c) die Transitbereiche sind in Hinblick auf Rauheit und Dämpfung zu bewerten etc.,
- die Modellierung der Wirkungsbereiche (Reichweite und laterale Ausbreitung) wird für Maßnahmendimensionierungen überwiegend mittels physikalisch basierten 2D bzw. 3D Modellen durchgeführt,
 - die Abschätzung von Eintrittswahrscheinlichkeiten erfolgt auf Basis der Beurteilung der vorliegenden Sturzblöcke (Alter und Häufigkeit des Auftretens),
 - die Abschätzung des Risikos wird in Sonderfällen durchgeführt, wenn die Notwendigkeit einer Schutzmaßnahme strittig ist oder die ausreichende Schutzwirkung der Maßnahme bei hohen Restgefährdungen nachvollziehbar dargestellt werden soll,
 - die Bemessung und Erhaltung von Schutzsystemen (Primärsicherungen, Netze, Dämme, Galerien, Überwachung und Instandhaltung) ist in der ONR 24810:2017 ausführlich dargelegt; diese stellt den Stand der Technik dar. Im Bereich von Monitoring-Systemen sind derzeit keine normativen Regelungen in Österreich vorhanden,
 - die Ausführung von Schutzsystemen (starre Wände, Netze, Dämme, Galerien, Monitoring) mit Ausnahme von starren Wänden (diese werde aufgrund ihrer vergleichweisen kurzen Lebensdauer nicht mehr empfohlen) und des Monitorings ist ebenfalls in der ONR 24810:2017 geregelt,
 - die Dauerhaftigkeit von Schutzsystemen ist einerseits durch eindeutig geregelte Überwachungs- und Inspektionsstandards für verschiedene Schutzsysteme bzw. Schadensfolgekassen (Qualität der Schutzgüter) gewährleistet. Andererseits garantieren die Hersteller von Steinschlagschutznetzen bei normalen Umweltbedingungen eine Lebensdauer von 25 Jahren. Nach Ablauf dieser Frist sind jedenfalls Prüfungen zum Nachweis der aufrechten Schutzwirkung der Bauwerke gem. ONR 24810:2017 (2017) vorgesehen. Andere Bauwerke, wie z. B. Schutzdämme, weisen eine i. w. unbeschränkte Lebensdauer auf, die nur durch Beschädigungen im Lastfall oder durch ungeplante Lastfälle beeinträchtigt werden kann. Eine solche Beeinträchtigung wird jedoch wiederum im Rahmen der gem. ONR 24810:2017 vorgesehenen Inspektionen sichtbar und kann korrigiert werden,
 - Vorgangsweise bei veralteten Schutzsystemen: Veraltete Schutzsysteme sollten in Abhängigkeit von der Gefährdung (Qualität der Schutzgüter, unterstellte Ereignisfrequenz) einerseits in Hinblick auf den Bemessungsvorgang – der mit hoher Wahrscheinlichkeit ebenfalls nicht zeitgemäß war – und andererseits in Hinblick auf den Stand der Technik bei den Maßnahmen selbst neu bemessen werden,
 - raumplanerische Maßnahmen und Frühwarnsysteme: diese stellen häufig nachhaltige und kostengünstige Maßnahmen zur Gefahrenprävention dar.

Raumplanerische Aspekte beim Umgang mit gravitativen Naturgefahren sind im Materialienband der ÖREK (Bäk et al., 2015) dargestellt. Standards zum Komplex der Frühwarnsysteme sind derzeit nicht verfügbar. Es sollte jedoch darauf Bedacht genommen werden, dass zumindest zwei voneinander unabhängige Messgrößen erfasst werden, die ungeachtet von Umweltbedingungen (Sichtverhältnisse, Zugänglichkeit etc.) in Echtzeit in ausreichend kurzen Messintervallen erfasst werden, um eine ausreichende Vorwarnzeit vor dem Eintritt des Schadereignisses sicherzustellen. Weiters ist in diesem Fall ein Katastrophenschutzplan mit klaren Handlungsanweisungen für alle unterstellten Szenarien zu erstellen. Standards für solche Katastrophenschutzpläne existieren derzeit nur für Rutschungen (ISO/DIS 22327:2017 Draft: Security and resilience – Emergency management – Guidelines for implementation of a community-based landslide early warning system).

17.6.1.1 Steinschlagmodelle

Besondere Bedeutung bei der Bewertung der vom Prozess Steinschlag ausgehende Gefahr kommt den Steinschlagmodellen (Prozessmodellen) zu, da nur mit Ihrer Hilfe die Reichweite und Intensität einer potentiellen Steinschlaggefahr – auch von Extremereignissen – prognostiziert werden können.

Prozessmodelle zur Simulation von Steinschlag (Steinschlagmodelle) existieren seit Mitte der 70er Jahre (z. B. Azimi und Desvarreux, 1977; Piteau und Clayton, 1977). Gegenwärtig steht eine große Anzahl von Modellen sowohl für zwei als auch für drei Dimensionen zur Verfügung (für einen aktuellen Vergleich siehe Tabelle 03 nach Volkwein et al., 2011). Alle gegenwärtigen Modelle beschränken sich auf die Modellierung des eigentlichen Sturzprozesses, der initiale Abbruch (Disposition) wird in den Modellen nicht simuliert, sondern lediglich durch die Anfangsbedingungen (Anfangsposition und Anfangsgeschwindigkeiten) abgebildet. Daher benötigen alle Modelle als Eingabe die potentiellen Ablösebereiche welche mit Hilfe anderer Methoden zu ermitteln sind.

Arten von Steinschlagmodellen

Die Prozessmodelle zur Berechnung bzw. Abschätzung der Reichweite und der Ausbreitung von Steinschlag werden unter anderem in 2D- und 3D-Modelle unterschieden.

Mittels 2D-Modellen können entlang eines Hangprofils (Bemessungsprofils) die Trajektorien von Sturzblöcken berechnet werden. 2D-Berechnungen (Energien, Reichweite, Sprungweite und Sprunghöhen) werden zumeist für die Planung und Dimensionierung sekundärer Schutzmaßnahmen (Netze, Dämme) auf der Hangskala (Objektebene) verwendet. 2D-Modelle können jedoch auch für die Ausweisung von Gefahrenzonen für größere Hangabschnitte (kommu-

nale Ebene, örtliche Raumplanung) eingesetzt werden. Hierbei werden mehrere repräsentative Hangprofile ausgewählt und die Simulationsergebnisse (z. B. der Verlauf der kinetischen Energien) von diesen Hangquerschnitten mittels Expertenwissen interpoliert (Melzner und Preh, 2012). Schwierigkeiten bei der Anwendung dieses Modelltyps ergeben sich vorwiegend bei der Festlegung des repräsentativen Hangprofils bzw. des adäquaten Abstands von mehreren Hangprofilen zueinander. 2D-Analysen haben darüber hinaus auch eine Bedeutung bei der Überprüfung/Verifikation von 3D-Analysen.

Mit Hilfe von 3D-Modellen können flächendeckende Aussagen über die potentiellen Wirkungsbereiche von Steinschlagprozessen getroffen werden (siehe Abbildung 10). Nachteilig bei der Anwendung von 3D-Modellen ist zum einen der erheblich erhöhte Aufwand für die flächendeckende Datenerhebung und Aufbereitung und zum anderen die Berechnungsdauer, die je nach Modellauflösung bzw. dem Modell zugrunde gelegten Algorithmen hoch sein kann.

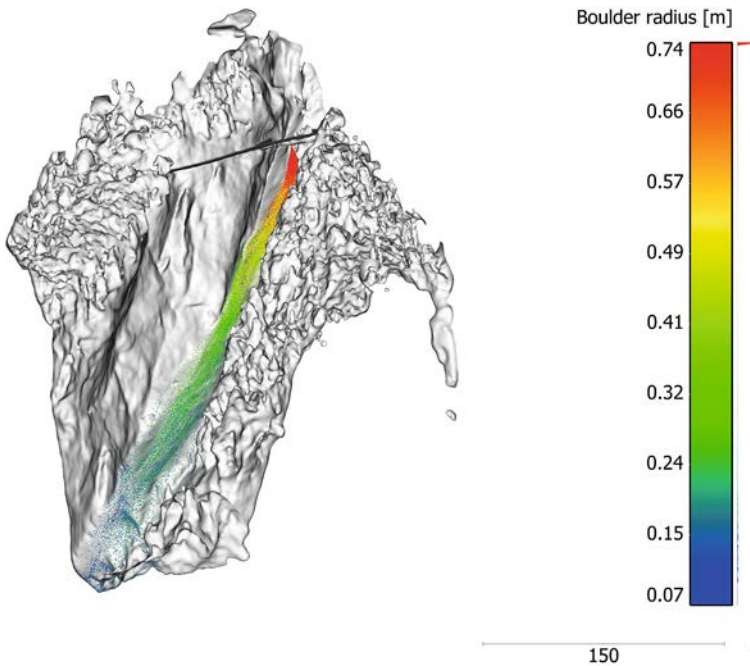


Abbildung 10: Anwendungsbeispiel eines 3D-Modells: Berechnung der Sturzbahnen und der Blockfragmentierung mithilfe des Programms WURF (Fleris und Preh, 2016), simulierte Trajektorien eingefärbt nach der im jeweiligen Bereich auftretenden Blockgröße.

Figure 10: Example of a 3D-model: calculation of trajectories and block fragmentation using the program WURF (Fleris und Preh, 2016), colored trajectories according to the respective block size.

Quelle: Fleris und Preh, 2016

Berechnungsansätze

Ein weiteres signifikantes Unterscheidungskriterium zwischen den unterschiedlichen Steinschlagmodellen ist der Berechnungsansatz, d. h. die mathematische Formulierung und programmtechnische Implementierung des Sturzblocks, des Rückpralls (Dämpfung bzw. Restitution), der Oberflächenrauheit, des Rollwiderstands, etc.

Nach Ihrem Berechnungsansatz werden die Modelle grob in:

- 1) Massenpunktmodelle (»lumped mass approach«),
- 2) Starrkörpermodelle (»rigid body approach«) und in
- 3) hybride Modelle (»hybrid approach«) unterschieden.

Bei den Massenpunktmodellen wird der Block als form- und dimensionsloser Massenpunkt (»lumped mass«) betrachtet. Im Gegensatz dazu wird bei den Starrkörpermodellen der Sturzblock als komplexer Starrkörper (»rigid body«) modelliert. Eine Mischform zwischen diesen beiden Ansätzen stellt der sogenannte hybride Ansatz dar, bei dem der Sturzblock als dimensionsloser Block für die Simulation der Flugbahn, bei der Berechnung des Rückpralls jedoch als Starrkörper (Berücksichtigung von Blockgröße und Form) betrachtet wird.

Der aktuelle Trend in der Entwicklung geht in Richtung komplexer Starrkörpermodelle, bei denen versucht wird, die möglichen Blockformen so naturgetreu wie möglich abzubilden (Andrew et al., 2012; Glover et al., 2012; Vijayakumar et al., 2012). Weitere wesentliche Unterschiede ergeben sich bei den Modellen beispielsweise in der Art, wie wichtige Hangparameter (z. B. Rauheit, Dämpfungseigenschaften) in das Modell integriert sind und ob und wie der Effekt von Bäumen und sekundären Schutzmaßnahmen auf den Sturzprozess berücksichtigt wird. Einige Modelle verwenden einen probabilistischen Ansatz, um die Unsicherheiten in der Qualität der Eingangsparameter besser zu berücksichtigen. Tabelle 03 zeigt eine Auswahl vorhandener Steinschlagmodelle mit ihrem zugehörigen Berechnungsansatz.

Tabelle 03: Auswahl vorhandener Modelle.

Table 03: Selection of existing models.

Quelle: verändert nach Volkwein et al., 2011

Modell/ Programm Name	Referenz	2D/3D	Ansatz	Probabi- listisch	Wald
CRSP (4.0)	Pfeiffer und Bowen, 1989	2D (Profil)	Hybrid	Ja	Nein
Rockfall 7.1 (2008)	Spang und Sönser, 1995	2D (Profil)	Starr- körper	Ja	Ja
ROFMOD 4.1	Zinggeler et al., 1990	2D (Profil)	Hybrid	Ja	Ja

((Fortsetzung))

Modell/ Programm Name	Referenz	2D/3D	Ansatz	Probabi- listisch	Wald
3D-GEOTEST-Zinggeler	Krummenacher et al., 2008	3D (x,y,z)	Hybrid	Ja	Ja
RocFall 6.0 (2016)	Stevens, 1998	2D (Profil)	Massen- punkt, Starr- körper	Ja	Ja
Sturzeschwindigkeit	Meißl, 1998	2D (x,y)	Massen- punkt	Nein	Nein
STONE	Guzzetti et al., 2002	3D (x,y,z)	Massen- punkt	Ja	Nein
Rockyfor 3D 5.2 (2014)	Dorren et al., 2006	3D (x,y,z)	Hybrid	Ja	Ja
RAMMS::Rockfall	Christen et al., 2007	3D (x,y,z)	Starr- körper (reale Form)	Ja	Ja
THROW	Preh, 2015	2D (Profil)	Hybrid	Ja	Ja
WURF	Fleris und Preh, 2016	3D (x,y,z)	Hybrid	Ja	Ja
Pierre	Mitchel und Hungur, 2017	2D (Profil)	Massen- punkt	Ja	Nein
Pierre3D	Gischig et al., 2015	3D (x,y,z)	Massen- punkt	Ja	Nein

Modellierung von Felsstürzen

Der Prozess Felssturz kann mechanisch in den Abbruch und den daraus folgenden Sturzprozess untergliedert werden. Der Abbruch, die Anfangsphase eines Felssturzes, ist maßgeblich von der Fragmentierung der Felsmasse und von den Interaktionen der Komponenten untereinander (entsprechend dem vorherrschenden Versagensmechanismus) geprägt. Der folgende Sturzprozess besteht aus zeitgleichen Steinschlagereignissen, wobei Interaktionen zwischen den einzelnen Sturzblöcken nur einen geringen Einfluss auf den gesamten Sturzprozess ausüben.

Aufgrund ihrer Spezialisierung auf die Modellierung einzelner unabhängiger Felsfragmente können mit klassischen Steinschlagmodellen (entsprechend Tabelle 03) Felssturzprozesse in Ihrer Gesamtheit nicht exakt abgebildet werden. Dies gilt vor allem für die Anfangsphase eines Felssturzereignisses. Solange jedoch Interaktionen zwischen den einzelnen Komponenten während des Sturzprozesses nur im geringen Maße auftreten, kann der dem Abbruch fol-

gende Sturzprozess mit klassischen Steinschlagmodellen ausreichend genau modelliert werden. Dazu ist jedoch eine Abschätzung der durch den Abbruch (Anfangsphase) bestimmten Anfangsbedingungen mit entsprechendem Expertenwissen erforderlich.

Hat das Abbruchereignis einen dominanten Einfluss und/oder spielt die Interaktion der einzelnen stürzenden Fragmente eine wichtige Rolle, dann müssen andere Berechnungsansätze, die Block-Block-Interaktionen und Fragmentierung berücksichtigen können, z. B. die Diskrete Elemente Methode (Cundall und Strack, 1979), eingesetzt werden.

17.6.2 Unsicherheiten

Unsicherheiten ergeben sich aus:

- 1) Fehlender systematischer Erfassung potentieller Gefährdungsbereiche. In der Regel werden Schutzmaßnahmen als Reaktion auf stattgefundenere Ereignisse geplant und umgesetzt. Erst seit 2014 wird in Österreich systematisch an einer Erfassung von Steinschlaggefahren und potentiellen Konflikten mit dem Siedlungs- bzw. Nutzungsraum gearbeitet (Bäk et al. 2015),
- 2) Veralteten Schutzsystemen (sowohl Bemessung als auch technische Lösung),
- 3) Überlastfällen infolge von Extremereignissen,
- 4) Subjektiver Beurteilung einzelner Parameter durch den jeweiligen Sachbearbeiter,
- 5) Modellunschärfen bei der Ermittlung der charakteristischen Einwirkungen,
- 6) Mangelnder Ergebnisvalidierung bei Bemessung (Modellierung),
- 7) Idealierten Bedingungen bei der Charakterisierung des Widerstandes von Schutzsystemen im Rahmen von Bewertungstests (günstige Blockform ohne scharfe Kanten, zentrale Trefferposition),
- 8) Unschärfen bei der Errichtung von Schutzmaßnahmen (z. B. Fundierung von Netzen in engräumig stark variierenden Baugründen etc.).

17.6.2.1 Unsicherheiten in Bezug auf die Kartierung bzw. Erhebung der prozessrelevanten Parameter

Die Unsicherheiten in Bezug auf die Kartierung bzw. Erhebung der prozessrelevanten Parameter können für die erforderlichen Arbeitsschritte wie folgt beschrieben werden:

- 1) Identifikation von stummen Zeugen und Ablösebereichen: stumme Zeugen von Steinschlagprozessen stellen i. d. R. Sturzblöcke am Einhang dar. Die Lage und die Abmessungen dieser Blöcke zur Kalibrierung von Modellen kann mit einfachen Mitteln (GPS, Maßband) gut dokumentiert werden. Um eine Er-

eignisfrequenz/Ereignishäufigkeit abzuleiten, sind die Blockalter grob gem. Tabelle 02 zu klassifizieren. Diese Alterseinstufung ist aufgrund von meist fehlenden analytischen Methoden mit Unsicherheiten aus der gutachterlichen subjektiven Beurteilung behaftet (siehe Abbildung 11).



Abbildung 11: Stumme Zeugen verschiedenen Alters: frisch (a), angewittert (b), alt (starker Bewuchs) (c).

Figure 11: Silent witnesses at different ages: fresh (a), weathered (b), old (heavy natural vegetal cover) (c).

Fotos: WLV Geologie

- 2) Festlegung der Homogenbereiche (Eigenschaften der Sturzbahn): eine Standardisierung ist schwierig, hier sind entsprechende Erfahrungen mit den verschiedenen am Markt erhältlichen Computermodellen in Hinblick auf die Sensitivität der Ergebnisse auf die Auflösung bei der Abgrenzung von verschiedenen Homogenbereichen erforderlich.
- 3) Festlegung des Bemessungsereignisses/der Bemessungsblöcke: Diese ist in der ONR 24810:2017 (2017) klar geregelt. Unsicherheiten und Unschärfen ergeben sich aus der subjektiven Auswahl von Blöcken, die für die statistische Analyse der Blockgrößen herangezogen werden. Dabei können signifikante Differenzen zwischen verschiedenen Bearbeitern beobachtet werden. Abhilfe schaffen hier – geeignete Verhältnisse wie z. B. eine ausreichende Anzahl von am Einhang vorliegenden Blöcken vorausgesetzt – statistische Verfahren wie z. B. Linien- oder Flächenzählverfahren.
- 4) Festlegung des Bemessungszeitraums (relevant bei der Erhebung der stummen Zeugen und daraus resultierend bei der Festlegung des Bemessungsblocks): Die Festlegung des Bemessungszeitraumes ist i. w. eine technisch/gesellschaftliche Konvention. In Anlehnung an die für Hochwasser, Wildbäche und Lawinen gewählten Bemessungszeiträume wird ein Zeitraum von ca. 150 Jahren als sinnvoll angesehen.
- 5) Bestimmung von Frequenzen/Eintrittswahrscheinlichkeiten: siehe Punktation 1). Ergänzend sind hierfür möglichst vollständige Ereignischroniken hilfreich. Diese liegen aber nur in den seltensten Fällen vor.

17.6.2.2 Unsicherheiten in Bezug auf Prozessmodellierung

Aus derzeitiger Sicht besteht kein Grund zur Annahme, dass die am Markt etablierten physikalisch basierten 2D und 3D Modelle Extremereignisse in Hinblick auf die Blockgröße nicht abbilden können. Grundsätzlich gäbe es mehrere Faktoren, die bei Extremereignissen zu Problemen bei der naturgetreuen Abbildung der in der Natur stattfindenden Prozesse führen könnten. Diese sind:

- 1) die Größe der Blöcke. Da für größere Blöcke die Rauheit des Untergrundes geringere Auswirkungen hat als für kleinere, ist eine korrekte Darstellung der tatsächlichen Blockdimensionen bei der Interaktion Boden/Sturzblock entscheidend.
- 2) die Blockgröße in Hinblick auf die Energie, die auf den Boden einwirkt und hier zu entsprechenden elasto-plastischen Verformungen führt. Diese sind abhängig von der Energie.
- 3) Die Interaktion Block/Wald: da diese Interaktion bei jenen Modellen, die eine solche Interaktion abbilden können, i. d. R. energieabhängig bewertet wird, führen extreme Energien bzw. Blockgrößen hier bei der Betrachtung von Einzelblöcken nicht zu Problemen.

Moderne Modelle bilden für Einzelblockstürze die Bodenkontakte und die dabei auftretenden Dämpfungsprozesse physikalisch weitgehend korrekt ab und können daher ohne signifikante Einschränkungen auch Extremereignisse darstellen. Problematisch ist hierbei lediglich der Absturz eines Blockverbandes (Felssturz) zu werten. Im Zuge des Prozesses kann es zu Veränderungen der Topographie (Einschlagtrichter,...) und zu starken Funktionseinbußen des Waldes kommen. Diese könnten in der Modellierung nicht korrekt abgebildet werden.

Kalibrierung der Steinschlagmodelle: In der Regel werden historische Ereignisse in Form von noch vorhandenen stummen Zeugen oder von dokumentierten Ereignissen (Ereignischronik) für die Kalibrierung der Modelle herangezogen. Wenn diese Daten fehlen ist eine Kalibrierung schwierig. Im Umkehrschluss bedeutet das Fehlen von stummen Zeugen aber meist, dass die Ereignisfrequenz gering ist und daher auch die Gefährdung tendenziell gering ist. Teilweise kann hier eine Simulation mit mehreren verschiedenen Modellen mit gut abgesicherten Parametersets Abhilfe schaffen.

Einfluss/Berücksichtigung des Modellfaktors: dem Umstand, dass unterschiedliche Modelle unterschiedliche Ergebnisse liefern, kann nur mit entsprechend großer Erfahrung und einer Plausibilisierung von Modellergebnissen begegnet werden. So existieren zahlreiche Datensätze von 1:1 Sturzversuchen mit einer guten Dokumentation der Geschwindigkeit und Sprunghöhen (Smith

und Duffy, 1990), die dabei für verschiedenen Untergründe und Hangneigungen beobachtet wurden. Die Modellergebnisse sind daher jedenfalls mit solchen gemessenen Daten in Hinblick auf ihre Plausibilität zu prüfen. Grundsätzlich ist jedoch davon auszugehen, dass die Variabilität der Modellergebnisse weniger vom Modell selbst als vom Anwender abhängt (Berger und Dorren, 2007).

Behandlung von Ausreißern der Modellierung bei der Gefahrenzonen-Abgrenzung: Für die Ausscheidung von Gefahrenzonen oder qualitativen Gefahren-Hinweisbereichen sind die Modellergebnisse entsprechend zu bewerten. Bei einer ausreichenden Anzahl von modellierten Blöcken kann i. d. R. ein konvergentes Ergebnis erzielt werden, das auch reproduzierbar ist. Gibt es bei so einem Ergebnis Ausreißer, die sehr untypische Reichweiten zeigen, ist es vertretbar, solche Modellergebnisse zu eliminieren. Es ist zwar durchaus möglich, dass die Modelle auch solche extremen Ereignisse korrekt darstellen, die Eintrittswahrscheinlichkeit solcher extremen Ergebnisse, die meist in Kaskaden von ungünstigen Parameterkombinationen begründet sind, ist jedoch sehr gering und daher nicht zwingend relevant für die Darstellung von Gefahrenzonen.

17.6.3 Ausblick auf mögliche zukünftige Entwicklungen

In Hinblick auf einen bevorstehenden bzw. bereits erkennbaren Klimawandel mit einer Häufung von extremen Wetterphänomenen kann mit hoher Wahrscheinlichkeit auch von einem Anstieg von Extremereignissen – auch bei Sturzprozessen – ausgegangen werden. In Anbetracht des Charakters der Bemessung von Schutzbauwerken auf ein »Bemessungsereignis«, das wiederum eine definierte Wiederkehrperiode aufweist, muss darauf hingewiesen werden, dass mit dem retrospektiven Charakter dieser Festlegung nur sehr bedingt auf signifikante Änderungen der Magnitude und Frequenz von Schadensereignissen eingegangen werden kann. Daher wäre gegebenenfalls ein Überdenken des »Bemessungsereignisses« notwendig, um mittel- bis langfristig das derzeit existierende Sicherheitsniveau zu halten.

Ähnliche Einschränkungen sind in Hinblick auf eine progressive Siedlungsentwicklung gültig. Hier kann über einen risikobasierten Ansatz eine Lösung angeboten werden. Das heißt, dass die errichteten Schutzmaßnahmen zu einem definierten (akzeptablen) Restrisiko führen (vgl. ÖGG, 2014), das auf das bestehende Schadenspotential abgestellt ist. Wird durch die Siedlungsentwicklung das Schadenspotential erhöht, müssen bei der Überschreitung des akzeptablen Restrisikos die bestehenden Schutzmaßnahmen entsprechend nachgezogen und verbessert werden. Hierzu wäre jedoch bei allen Schutzprojekten eine Ermittlung des nach Realisierung der Maßnahmen bestehenden Restrisikos durchzuführen. Dies ist derzeit in Österreich nur in Ausnahmefällen gegeben.

Zu einem zukünftigen veränderten Freizeitverhalten mit einer erhöhten Risikobereitschaft ist festzuhalten, dass es sich hierbei um ein freiwillig eingegangenes, individuelles Risiko handelt. Die Entscheidung hierüber ist in der Regel von Einzelpersonen zu treffen und lässt sich nur in Ausnahmefällen institutionell regeln. Häufen sich jedoch Fälle von extremen Risiken in diesem Bereich, kann es dennoch zu normierten Vorgangsweisen beim Umgang mit diesen Naturgefahren kommen (vgl. Lawinenwarnstufen mit entsprechenden Verhaltensempfehlungen für Skitouristen, Wanderwege-Tool Tirol für steinschlagexponierte Wanderwege mit einem abgestuften akzeptablen Restrisiko in Abhängigkeit von der Schwierigkeit des betroffenen Wanderweges – vgl. LOLA, 2018 etc.).

17.7 Perspektiven für Handlungsoptionen

In den allermeisten Fällen ist es auch mit aufwändigen Schutzmaßnahmen nicht möglich, eine vollständige Abdeckung von Risiken aus Steinschlag- und Felssturzprozessen zu erreichen. Neben der oft nicht gegebenen Machbarkeit sind hierbei vor allem Überlegungen zur Wirtschaftlichkeit der Schutzmaßnahmen und die Relation von Kosten und Restrisiken bei verschiedenen Maßnahmenvarianten maßgeblich (EconoMe 4.0, o. J.). Aus diesen Gründen sieht z. B. auch der Eurocode 7 (ÖNORM EN 1997-1:2014) vor, dass in Abhängigkeit vom Schadenspotential beim Versagen eines Bauwerks (im Überlastfall) eine bestimmte Zuverlässigkeitsklasse des Bauwerks anzustreben ist. Dies bedeutet, dass die akzeptable Versagenswahrscheinlichkeit eines Bauwerkes – und hier werden auch Hochbauten wie z. B. Wohnhäuser betrachtet – mit steigendem Schadenspotential geringer werden sollte. Diese in Ingenieurskreisen durchaus übliche Denkweise ist jedoch häufig schwierig zu kommunizieren, vor allem, wenn es zu Schadensfällen mit Betroffenen kommt. Dann ist jeder Sachschaden inakzeptabel und jedes Menschenleben unersetzlich. Es gilt daher, in der Bevölkerung ein besseres Bewusstsein für solche bestehenden Restrisiken zu schaffen und diese auch klar und deutlich zu kommunizieren.

Häufig ist es hilfreich, Vergleiche zu Risiken in einem anderen Kontext zu ziehen. So ist die Teilnahme am Straßenverkehr als eine der gefährlichsten Aktivitäten, die man in Mitteleuropa ausüben kann, einzustufen. Dabei werden Personenschaden-Risiken in Kauf genommen, die oft mehrere Zehnerpotenzen größer sind als jene, die Personen im Kontext mit Naturgefahren als inakzeptabel einstufen.

Um eine klarere Vorstellung von den Unsicherheiten und Unschärfen, die bei der Bewertung von Naturprozessen unvermeidbar sind, zu bekommen und auch für alle Nachnutzer einer Expertise zugänglich zu machen, sollte besonderes

Augenmerk auf eine nachvollziehbare Vorgangsweise bei den Erhebungs- und Bemessungsprozessen gelegt werden. Im Wesentlichen ist dies bei Beachtung der in der ONR 24810:2017 beschriebenen Vorgangsweise gegeben. So sollte z. B. eindeutig beschrieben und verortet sein, wo die Blöcke vermessen wurden, die für die Bestimmung der Bemessungsblockgröße herangezogen wurden. Die Bestimmung der Ereignisfrequenz sollte ebenso eindeutig über Chronik- oder Geländedaten dokumentiert werden, wie die Dokumentation der Lage von den für die Kalibrierung des Modells bzw. der Validierung der Ergebnisse herangezogenen Blöcke.

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The concept of design block size – A critical review of ONR 24810 “Technical Protection against Rockfall”

In Austria, the Guideline for “Technical Protection against Rockfall”, named ONR 24810 provides guidance to assess rockfall hazard (at object scale) by determining a so-called design block size. The aim of this article is to critically review the concept of design block size and to suggest alternatives. The design block size is derived from a block size distribution and event frequency, both of which may be very uncertain. Modelling a single “constant” design block may result in trajectories far away from reality. The design approaches of ONR 24810 and Eurocode 7 are compared. A case study is presented and discussed. Reducing input parameters at the beginning of the design approach results in apparent characteristic energy levels and bounce heights, which are not characteristic, as defined by EC7. By disregarding all small and the biggest blocks, valuable information of maximum energy levels and bounce heights is lost. The entire block size distribution should be used for rockfall simulations, rather than a single block size. Fragmentation should be considered, if applicable.

Keywords rockfall; design block; hazard; risk; protection; safety; design approach; overload; scale

1 Introduction

Awareness of rockfall hazard has been increasing in the past years. Enhanced tourism and settlement in mountain regions increase general exposure to rockfall. Communes are encountered with the zoning of potentially hazardous regions. They need to identify potential conflicts between process area and land use. A conflict would be an overlap of rockfall impact area with used or planned settlement area. The need of standardized hazard zone plans arises. Proper design and evaluation of rockfall protection measures is getting more important.

In Austria, the Guideline for “Technical Protection against rockfall”, ONR 24810 [1], provides guidance to assess rockfall hazard by determining a so-called design block size. [1] was developed for assessments at object scale, where energy and bounce height are the main design factors. However, [1] is increasingly used for hazard assessments at regional scale, where a third factor, the runout, is relevant.

Das Konzept des Bemessungsblocks – Eine kritische Betrachtung der Richtlinie ONR 24810 „Technischer Steinschlagschutz“

In Österreich regelt derzeit die Richtlinie ONR 24810 den technischen Steinschlagschutz. Sie gibt vor, Steinschlagschutzmaßnahmen (auf Hangskala) auf den sog. Bemessungsblock zu dimensionieren. Das Ziel dieses Artikels ist es, das Konzept des Bemessungsblocks kritisch zu hinterfragen und Alternativen vorzuschlagen. Der Bemessungsblock ergibt sich aus einer Blockgrößenverteilung und einer Ereignishäufigkeit. Beide können sehr unsicher sein. Die Modellierung eines einzigen „konstanten“ Bemessungsblocks kann in Trajektorien fern der Realität resultieren. Der Bemessungsweg der ONR 24810 wird mit jenem des Eurocode 7 verglichen. Eine Fallstudie wird präsentiert und diskutiert. Reduzierte Eingangswerte zu Beginn der Bemessung resultieren in scheinbar charakteristischen Energien und Sprunghöhen, die nicht charakteristisch im Sinne des Eurocodes sind. Durch die Vernachlässigung aller kleinen und der größten Blöcke gehen wertvolle Informationen über maximale Energien und Sprunghöhen verloren. Gesamte Blockgrößenverteilungen sollten simuliert werden, statt ein einziger Bemessungsblock. Fragmentierung sollte berücksichtigt werden, falls vorhanden.

Keywords Steinschlag; Bemessungsblock; Gefahr; Risiko; Schutz; Sicherheit; Bemessung; Extremfall; Maßstab

The aim of this article is to critically review the concept of design block size. Comparative calculations have been carried out in the course of a case study at the Liechtensteinklamm. Results are discussed, problems are illustrated and recommendations are given.

2 Design approach of ONR 24810, compared to Eurocode 7

ONR 24810 [1] provides guidance for the design of appropriate technical protection measures (at object scale: $M \geq 1:5,000$). One of the central elements of [1] is the evaluation of a so-called design block size. It is defined as the cubature of a rock in m^3 , which is relevant for the respective dimensioning of a rockfall protection measure. Its evaluation considers

- Event probability (P), using event frequency classes (EF),
- Potential damage, using consequence of loss classes (CC) and
- A block size diagram.

Tab. 1 Consequence classes CC1 to CC3 [1] (Tab. 1)
Schadensfolgeklassen CC1 bis CC3 [1] (Tab. 1)

Consequence Class	Characteristic	Examples
CC1	Low consequences for people and little or negligible economic, social or environmental consequences.	Agricultural buildings without regular passenger traffic, barns, greenhouses, forest and goods roads with low traffic frequency.
CC2	Medium consequences for people, considerable economic, social or environmental consequences.	Residential and office buildings, public buildings with medium consequences of collapse, like office buildings, regional and national traffic routes with mean traffic volumes $\leq 50,000$ vehicles.
CC3	Serious consequences for people or very high economic or environmental consequences.	Grandstands, public buildings with high consequences of collapse, like concert halls, regional and national traffic routes with mean traffic volumes $> 50,000$ vehicles.

Tab. 2 Event frequencies and corresponding design block fractile [1] (Tab. 4)
Ereignishäufigkeiten und zugehörige Bemessungsblockfraktile [1] (Tab. 4)

Event frequency class	Event frequency	Volume percentile for design block size
EF4 (very often)	$n \geq 10$ (≥ 10 events/year)	V98
EF3 (often)	$1 \leq n < 10$ (1 to 10 events/year)	V97
EF2 (rare)	$0.03 \leq n < 1$ (1 event/1 to 30 years)	V96
EF1 (very rare)	$n < 0.03$ (< 1 event/30 years)	V95

By using consequence of loss classes, which consider potential damage, [1] indirectly considers risk.

There are two versions for the evaluation of design block size. The choice depends on the consequence of loss class and the number of potentially falling blocks. For “Baseline survey for planning rockfall protection measures” refer to Chapter 5 of [1]. The consequences of loss are classified in three classes. They distinguish little, medium and severe consequences for people, economy, society or environment (Table 1).

At CC2 and CC3 or in case of more than 100 potential falling blocks, a statistic evaluation of the design block size is required. A block size distribution has to be derived from field mapping of the detachment and deposit areas. The design block size is evaluated based on that block size distribution and an estimated event frequency (Figure 1, Table 2).

Event frequencies are classified in four classes. They distinguish rockfall frequencies between less than one event in 30 years and more than ten events per year. According to the event frequency class, the design block size is determined between the 95th (V95) and the 98th (V98) volume percentile (Table 2).

Figure 2 shows a flow chart summarizing the design approach of [1], compared to the partial factors used in Eurocode 7 (EC7) [2] [3]. The design block size is an input parameter for the rockfall simulation model. Together with other input parameters (geometry, ground parameters, restitution), it affects the characteristic values of actions, F_k [2] (2.4.2). The reduction of the maximum block size in [1] (Tab. 4) can be compared to the partial factors for geological parameters, γ_M , in [2] (Tab. A.4) and [3]

(Tab. 16). The reduction factor for the maximum block size in [1] depends on event frequency and is less than 1.0 (see Table 2) [1] (Tab. 4). The reduction factor for geological parameters in EC7, γ_M , also depends on event frequency (design situation DS) and is greater or equal to 1.0 [2] (Tab. A.4), [3] (Tab. 16). In EC7, the partial factor for specific weight, γ_p , is equal to 1.0, for all design approaches and all design situations (representing different annualities) [2] (Tab. A.4), [3] (Tab. 19).

According to EC7, factor ψ (≤ 1) may be applied to characteristic values of actions, F_k [2] (2.4.6.1, 2.1b). This means, that not characteristic, but “representative” values, F_{rep} , are used for the design. Factor ψ is used to convert character-

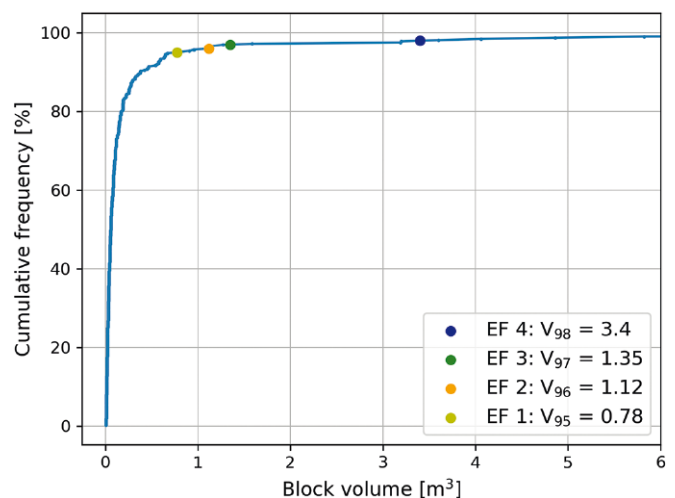


Fig. 1 Determination of design block volume from statistic deposit investigations of a rock mass fall, using the line-counting-method (LCM) [1] (Tab. 3)
Ermittlung des Bemessungsblockvolumens durch statische Untersuchung eines Felssturz-Ablagerungsbereichs mittels Linienzählverfahren (LZV) [1] (Tab. 3)

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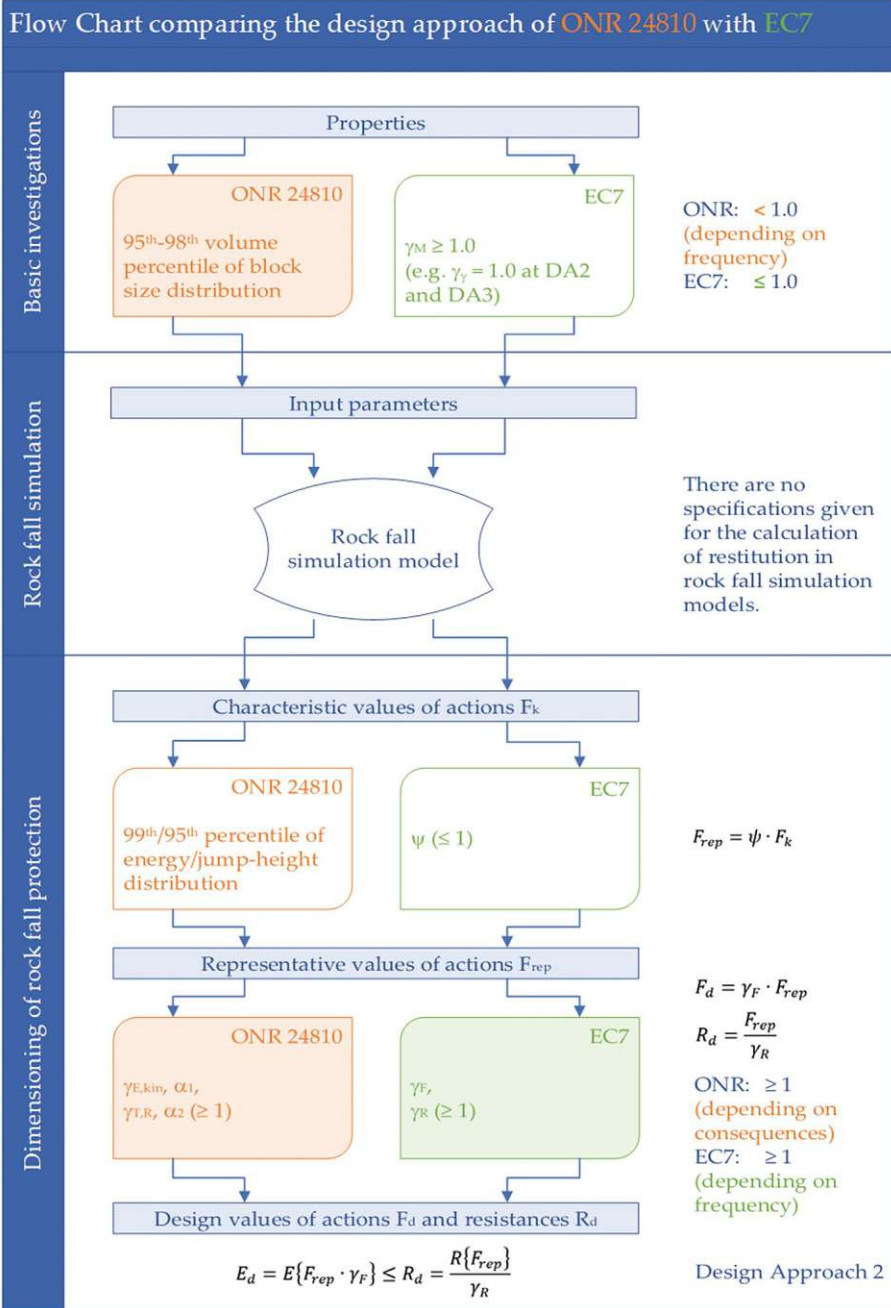


Fig. 2 Flow chart comparing the design approach of ONR 24810 to that of EC7 [2] [3].
Flow Chart zum Vergleich des Bemessungswegs der ONR 24810 mit jenem des EC7 [2] [3].

istic into representative values. This is comparable to [1] (5.2.5.4 and 5.2.5.5), where the 99th percentile of the calculated impact energy distribution (P99) and the 95th percentile of the vertical bounce height distribution (P95) are used to convert characteristic into representative values.

The design values F_d and R_d are calculated by imposition of partial factors. They are to be imposed on both sides, action and resistance. Multiplication by increase factors $\gamma_{E,kin}$ and α_1 in [1] (Tab. 5, Tab. 6) is comparable to multiplication by γ_F in [2] (2.4.7.3.2) and [3] (Tab A.1, Tab A.2). Division by reduction factors $\gamma_{T,R}$ and α_2 in [1] (Tab. 8, Tab. 9) is comparable to division by γ_R in [2] (2.4.7.3.3) and [3] (Tab. 17). The design approach of is [1] most comparable to design approach DA2 of [2] (2.4.7.3, App. A), simplified as:

$$E_d = E\{F_{rep} \cdot \gamma_F\} \leq R_d = \frac{R\{F_{rep}\}}{\gamma_R} \tag{1}$$

Where

- E_d design value of the effect of actions,
- E value of the effect of action,
- γ_F partial factor for an action,
- R value of the resistance to an action,
- γ_R partial factor for a resistance,
- F_{rep} representative value of an action.

In [1] (Tab. 5, Tab. 6, Tab. 8, Tab. 9), the partial factors for F_d and R_d are defined by consequence classes. In [2] (2.4.7.3.2, 2.4.7.3.3) and [3] (Tab A.1, Tab. A.2, Tab. 17),

the partial factors for F_d and R_d are defined by event frequency (design situation DS).

The main difference in the design approach of [1] compared to [2] [3] is, that [1] reduces input parameters at the beginning of the design approach, resulting in apparent characteristic values of actions, F_k . They are apparent, because they are not characteristic values, as defined by [2] (2.4.5).

3 Influence of event frequency class and consequence class on calculated actions at the example of Liechtensteinklamm

The Liechtensteinklamm is a gorge in Salzburg/Austria, in the district of St. Johann im Pongau. In May 2017, a rock mass of approx. 100 to 150 m³ detached and resulted in a rock mass fall and several rockfall events. Due to this unexpected rock mass fall in a sensitive area of the visitor walkway, the gorge was closed to the public until further notice. A profound and detailed rockfall hazard analysis was carried out for the entire gorge. Drop tests were conducted, filmed and evaluated. The data was used to investigate the influence of event frequency class and consequence class on calculated actions.

3.1 Methodology

Comparative calculations have been carried out to investigate the influence of estimated event frequency and

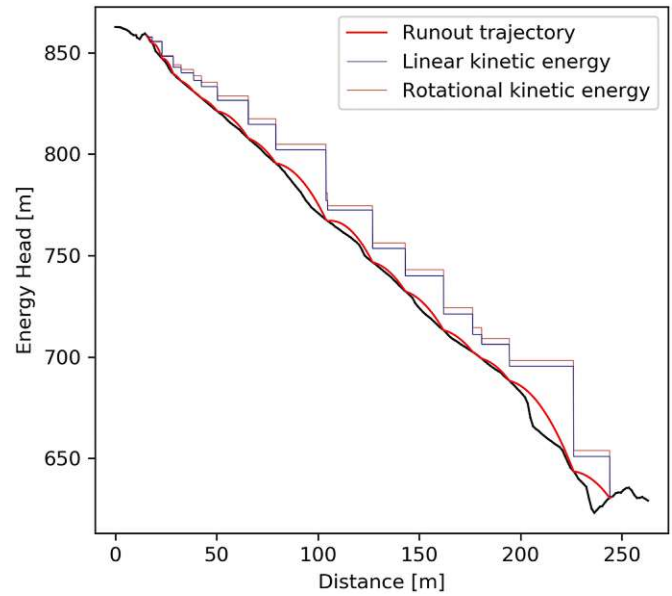


Fig. 4 Energy head [m] of an example-trajectory (red) along the slope profile “Tiefblick” at Liechtensteinklamm. Energielinie [m] einer Beispiel-Trajektorie (rot) entlang des Hangprofils „Tiefblick“ in der Liechtensteinklamm.

consequences on calculated actions, like total kinetic energy and bounce height. The slope profile at the area of “Tiefblick” of Liechtensteinklamm was chosen for this study (Figure 3). This is the same area, where the rockfall tests had been carried out for model calibration. Bounce heights and velocities could be determined very well from video analyses. For rockfall simulations, the 2D rockfall simulation program THROW [7] [8] [9] was used. Runout relevant parameters, as damping parameters and surface

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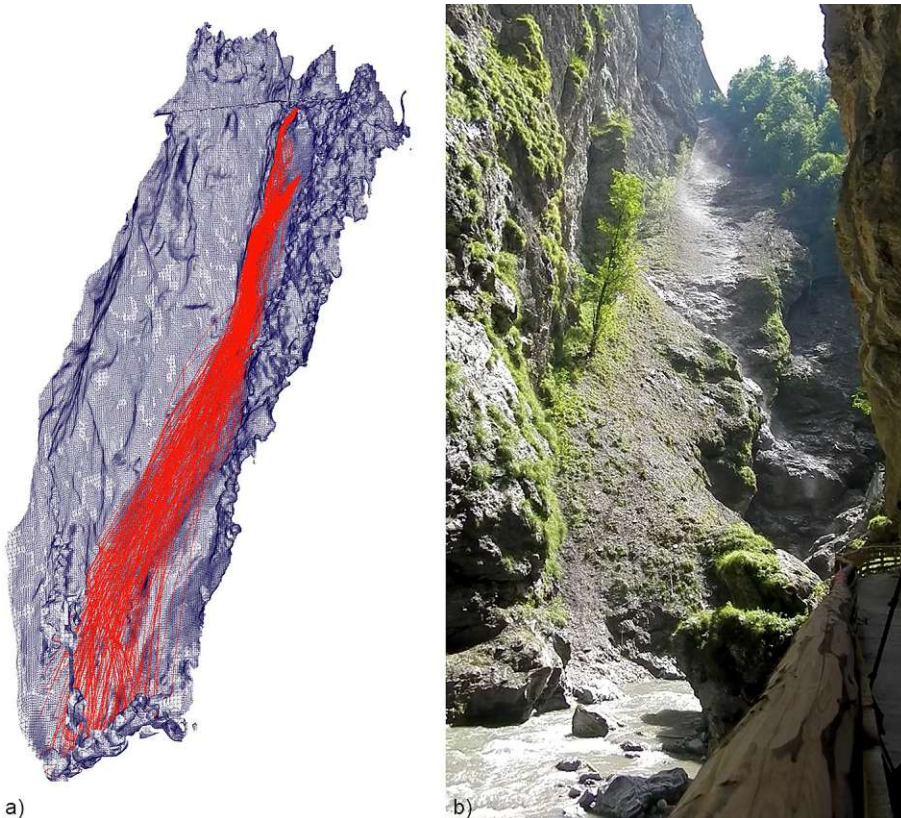


Fig. 3 a) Rockfall simulation with Wurf [5]; b) Photo at the area of the high-lying bridge along “Großarler Landesstraße” named “Tiefblick”. Source: [6] a) Steinschlagsimulation mit Wurf [5]; b) Foto des Bereichs der hochgelegenen Brücke entlang der Großarler Landesstraße namens Tiefblick. Quelle: [6]

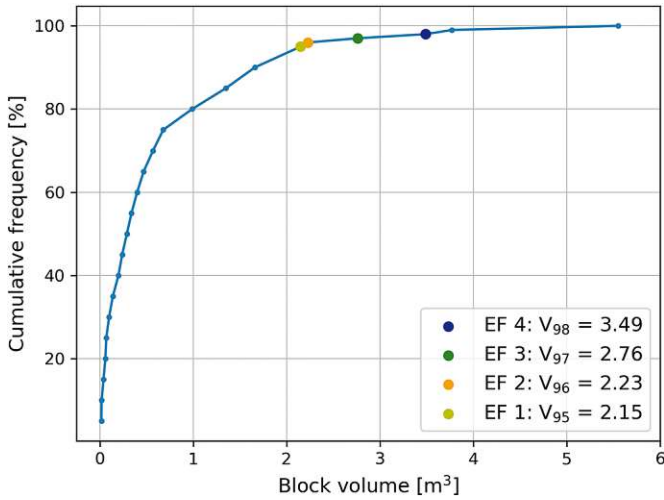


Fig. 5 Cumulative frequency of block sizes (= block size distribution) at the area of “Großarler Landesstraße bis Wandmitte” of Liechtensteinklamm. Source: [6]
 Kumulierte Häufigkeit von Blockgrößen (= Blockgrößenverteilung) im Bereich „Großarler Landesstraße bis Wandmitte“ in der Liechtensteinklamm. Quelle: [6]

roughness, from back analysis have been used for the calculations with THROW. It turned out, that the observed bounce heights could not be modelled without considering all smaller blocks of the block size distribution. Figure 4 shows the energy head [m] of a simulated trajectory along the slope profile “Tiefblick” at Liechtensteinklamm.

Rockfall simulations of the design block at EF1 (V95), the design block at EF4 (V98) and a block size distribution (V5 to V100) are compared. For simulations of EF1 and EF4 design blocks, 100 trajectories of 2.15 m³ (very rare: V95) and 3.49 m³ (very often: V98) blocks each have been calculated (Figure 5). For the simulation of the distribution, block sizes from 5 to 100% of the maximum block size (in 5% steps) have been simulated. The number of trajectories for each block size corresponds to the actual number of blocks from the block size distribution (1,002 blocks total). This means, that the occurrence probability of each block size (= event frequency) is already included in the calculated distributions (and thus, in the percentiles of the distributions).

3.2 Results

Simulation results are compared in Figures 6 and 7. The red dashed lines in Figures 6 and 7 represent the characteristic kinetic energy/vertical bounce height of the block size distribution. Those lines have been calculated simulating the whole block size distribution (see Figure 5). They represent most realistic distributions of kinetic energy and bounce height.

The red solid lines in Figures 6 and 7 represent the 99th/95th percentile of the kinetic energy/vertical bounce height of the block size distribution, multiplied by the respective partial safety factor at CC3 [1] (Tab. 5, Tab. 6). The characteristic distribution energy line (red dashed)

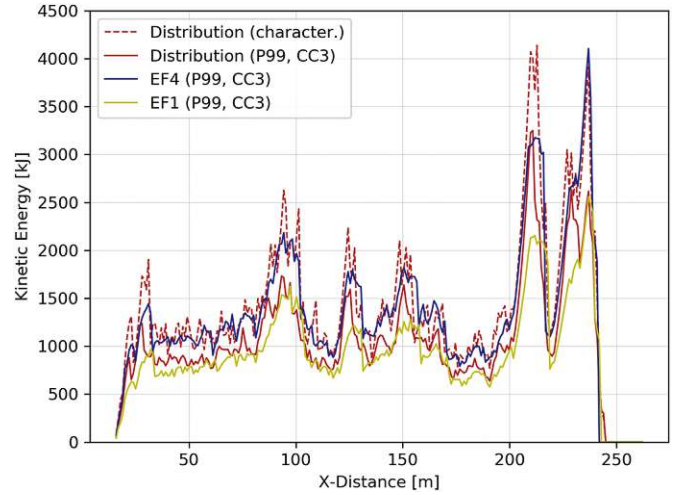


Fig. 6 Comparison of kinetic energies [kJ] of rockfall simulations; red: values of the block size distribution, blue: design values of the design block at EF4, yellow: design values of the design block at EF1
 Vergleich der kinetischen Energien [kJ] der Steinschlagsimulationen; rot: Werte der Blockgrößenverteilung, blau: Bemessungswerte des Bemessungsblocks bei EF4, gelb: Bemessungswerte des Bemessungsblocks bei EF1

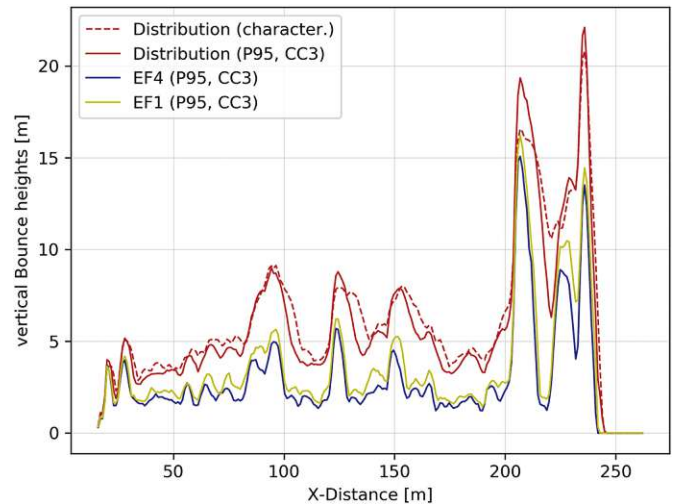


Fig. 7 Comparison of vertical bounce heights [m] of rockfall simulations; red: values of the block size distribution, blue: design values of the design block at EF4, yellow: design values of the design block at EF1
 Vergleich der vertikalen Sprunghöhen [m] der Steinschlagsimulationen; rot: Werte der Blockgrößenverteilung, blau: Bemessungswerte des Bemessungsblocks bei EF4, gelb: Bemessungswerte des Bemessungsblocks bei EF1

(and even the P99 distribution line (red)) in Figure 6 shows on average high energy peaks, compared to the EF1 (yellow) and EF4 (blue) energy lines. The EF4 energy line is close to the characteristic distribution energy line, but peaks are not well represented. The EF1 energy line is always below the characteristic distribution (and mostly even below the P99 distribution) energy line, even in CC3, where a partial safety factor of $\gamma_{E,kin} = 1.15$ [1] (Tab. 5) has been applied.

In Figure 7, the characteristic distribution of the bounce height line (red dashed) shows the bounce heights of smaller blocks, which are disregarded at the EF1 and EF4 simulations. EF1 and EF4 bounce height lines are always below

the characteristic distribution bounce height line (red dashed) and even below the P95 distribution bounce height line (red). This is true even at CC3, where a partial safety factor of $\bar{\alpha}_1 = 1.30$ [1] (Tab. 6) has been applied to the EF1, EF4 and to the P95 distribution bounce height line.

4 Discussion, problems and recommendations

4.1 Apparent safety

The concept implemented in [1] pretends an apparent sense of safety to the user of the guideline. The wording “design block” suggests a certain safety. It is not called “characteristic” or “representative block”. The “design block”, according to [1], is even reduced. By increasing representative values through the application of partial safety factors, the user assumes to be on the safe side. The user trusts that design values end up higher than characteristic values. However, the results of comparative calculations show, that design values of calculated kinetic energy are mostly below characteristic values (see Figure 6).

It is generally observed that smaller blocks bounce higher than bigger blocks [9]. That's because restitution is indirectly proportional to both, mass and velocity. By modelling a single “constant” design block, vertical bounce heights are underestimated (see Figure 7). Again, the partial safety factor in [1] pretends apparent safety. On the other hand, most rockfall simulation programs overestimate bounce heights. There is need for further research. However, the damage potential of small blocks is too high to be fully disregarded.

Characteristic and representative values hardly differ for design block simulations, since there is no distribution behind the calculations. Figure 8 shows three different percentiles (characteristic, P99 and P50) each, for the block size distribution (red) and for the EF4 design block (blue). The characteristic EF4 energy line (blue dashed) is nearly equal to the representative EF4 energy line (blue). On the other hand, the characteristic distribution energy line (red dashed) is much higher than the representative distribution energy line (red). The reason for this difference is the inclusion of all (many smaller) blocks in the distribution lines (red). Comparison of the red lines allows to develop a feeling for the most realistic situation. This is important for decision-making, especially in terms of accepted hazard. With the knowledge of maximum energy levels and realistic percentiles of energy distributions, rockfall protection measures can be designed for most realistic characteristic or representative (P99) energy levels, depending on the requirements.

Situations of low rockfall frequency should not automatically denote smaller block volumes (e.g. EF1: V95, see Table 2). A low number of (big) blocks should not lead to their early disregard. Actual block size distributions should be simulated to achieve most realistic distribu-

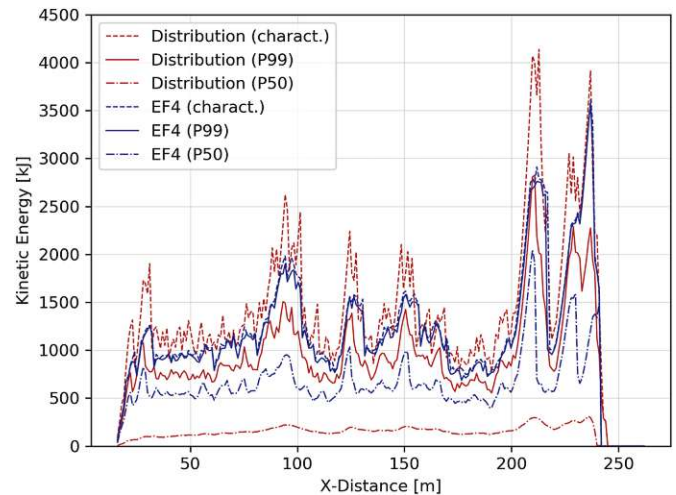


Fig. 8 Comparison of kinetic energies [kJ] of rockfall simulations; red: values of the block size distribution, blue: values of the design block at EF4

Vergleich der kinetischen Energien [kJ] der Steinschlagsimulationen; rot: Werte der Blockgrößenverteilung, blau: Werte des Bemessungsblocks bei EF4

tions of energy levels and bounce heights. The problem of overestimated bounce heights could be managed by defining appropriate representative values (percentiles), as is already being done in [1] ($F_{rep} = P95$ for bounce heights) [1] (5.2.5.4 and 5.2.5.5).

4.2 Vague methods, fragmentation and position of protection measures

The work area to establish a block size distribution is the whole rockfall impact area. It comprises of detachment as well as deposit areas. Often, the origin (detachment area) of silent witnesses is unclear. Often, silent witnesses had been removed or moved. This leaves a wide scope of possible work areas.

From

$$T_{kin,trans} = \frac{m \cdot v^2}{2}, \quad (2)$$

velocity has a much higher impact on translational kinetic energy. However, in the simulations, mass influences restitution. Thus, mass affects velocity, bounce height and runout. If rock mass is significantly decreasing along the slope due to fragmentation, energy levels may be lower down the slope, compared to the design block energy level. Thus, the position of a planned protection measure along the slope is important for its proper design.

At areas of high fragmentation, the determination of the design block is unclear. Considering the case study Liechtensteinklamm, most blocks end up at the bottom of the gorge, carried along by the river. Block sizes had to be derived from detachment areas. In such cases, considering energy levels of V95 to V98 block sizes may result in too high energy levels at the bottom of slopes.

If applicable, detachment and deposit areas should result in two different block size distributions. In case of significant size differences, fragmentation should be considered. If fragmentation plays a role, modelling a single “constant” design block results in trajectories far away from reality. Present rockfall simulation programs are not able to model fragmentation, realistically. Further investigations should be done to develop rockfall simulation programs with the aim of considering rock fragmentation, in future.

4.3 Block size distributions and event frequencies – scenarios

Given less blocks of bigger size, block size distributions are often parabolic (see Figures 1 and 5) [11]. In such cases, the 98th volume percentile block size (V98: EF4) could be twice as big as the 97th volume percentile block size (V97: EF3). This is similar for comparisons of V97 with V96 block sizes and of V96 with V95 block sizes.

According to Table 2, the lower the frequency, the smaller the design block size; and vice versa. [1] always expects higher block volumes at locations, where many rockfall events happen. The possibility of high block volumes at locations of few rockfall events (high annualities) is not considered in [1], and vice versa. Assuming a situation of very rare rockfall events (EF1), the design block size (V95), according to [1], would underestimate energies, even if partial safety factors for CC3 (serious consequences) are applied (see Figure 6). [1] accepts higher hazard at locations of rare rockfall events, independent of consequences. Event frequencies are often rough estimates with great influence on resulting energies (see Table 2 and Figure 6).

Situations of few rockfall events should not automatically indicate smaller block volumes, because this approach accepts higher hazard, independent of consequences. If sufficient information about event frequencies is available, scenarios of different annualities with corresponding block size distributions may be built. Otherwise, one scenario (of high annuality) should be considered. Actual characteristic values should be calculated, using the entire block size distribution. Then, protective structures should be dimensioned considering both, event frequency and consequences.

5 Conclusions and outlook

Field survey and research should result in block size distributions within certain annualities, if applicable (scenarios). The entire block size distribution should be simu-

lated for most realistic distributions of kinetic energy, bounce height and runout (of a certain annuality). Maximum block sizes should not be disregarded prior to modelling. This allows to model most realistic trajectories. Otherwise, results pretend apparent safety, as design values end up below characteristic levels.

Today’s rockfall simulation programs are able to model a range of block sizes, rather than a single one. Outliers (trajectories of very low probabilities) may be disregarded at the end of the simulation process, as it is already transparently specified in [1] (5.2.5.4 and 5.2.5.5) (e.g. $F_{rep} = P95$ for bounce heights).

Fragmentation should be considered, if applicable. The influence of fragmentation on calculated energy levels, bounce heights and runout should be further investigated.

Bounce height and runout should be modelled more accurately (incl. small blocks) for both, designs at object scale and hazard analyses at communal and regional scale. The influence of event frequency class and consequence class on calculated runout should be further investigated.

Different rockfall simulation models could have a high impact on resulting energies, bounce heights and runout. The influence of the so-called model-factor should be further investigated. [1] should not neglect the model-factor, but address rockfall simulation models and their calculation approaches, especially important ground strength parameters, like roughness and damping. Partial factors for input parameters should be provided. This may be done based on design approach 3 of EC7 [2] (2.4.7.3.4.4, App. A.3) [3] (App. A).

For numerical procedures, the national annex of EC7 [3] (App. A) recommends using design approach 3, even if design approach 2 is specified for conventional procedures (e.g. retaining structures). In design approach 2, partial factors are applied to actions (or to the effects of actions) and to ground resistances [2] (2.4.7.3.4.3). In design approach 3, partial factors are applied to actions (or to the effects of actions) from the structure and to ground strength parameters [2] (2.4.7.3.4.4). Using design approach 3 for numerical rockfall modelling is problematic, because there are many different rockfall simulation programs on the market, using different calculation approaches.

If there is no information about event frequencies (annualities), only one scenario (of high annuality) should be modelled. This results in conservative runout, energy level and bounce height distributions. For hazard zoning, the propagation probability of trajectories may be used for hazard zone classification [12].

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Article

Challenges Assessing Rock Slope Stability Using the Strength Reduction Method with the Hoek–Brown Criterion on the Example of Vals (Tyrol/Austria)

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Abstract: To estimate the hazard posed by rock slopes, it is essential to determine the overall stability and potential detachment volume. This is mostly solved using numerical methods together with the strength reduction method (SRM). Many calculation programs do not provide a direct implementation of the Hoek–Brown (HB) criterion. Equivalent Mohr–Coulomb (MC) parameters are often used. Especially for steep rock slopes, the use of equivalent MC parameters with numerical codes and the SRM lead to poor estimates of safety factors. The problem lies in the required and often difficult estimation of a suitable range of minor principal stresses over a ‘slope height’. In the example of the stability analysis of the rock slope Vals in Tyrol/Austria, we show the differences between the application of equivalent MC parameters and a direct application of the HB criterion with apparent MC parameters. The detachment volume and stability are overestimated when applying equivalent MC parameters, as confirmed by calculations with the continuum mechanics code FLAC3D (Itasca Consulting Group). However, the SRM with HB material (i.e., apparent MC parameters) results in a safety factor that cannot be applied to HB parameters. To date, it has not been possible to determine the HB parameters for limit equilibrium via the SRM. This challenge was overcome by fitting an HB envelope to the original HB shear envelope reduced by the safety factor. The envelope is adjusted by two HB variables: GSI and D. This allows to determine the HB parameters at limit equilibrium. It helps to make more realistic predictions about the detachment mechanism and volume.

Keywords: rock slopes; rock slope stability; slope stability analysis; strength reduction method; Hoek–Brown criterion; equivalent Mohr–Coulomb parameters

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1. Introduction

In Vals (Tyrol/Austria), a rock fall with a detachment volume of approximately 117,000 m³ occurred on 24th December 2017. It destroyed parts of the Valser county road L230 and just missed nearby houses. Mapping and measurements by the Austrian Torrent and Avalanche Control (TAC) after the incident indicate a potentially unstable slope area (Figure 1). This needs to be investigated to predict a possible detachment process and volume in the future.



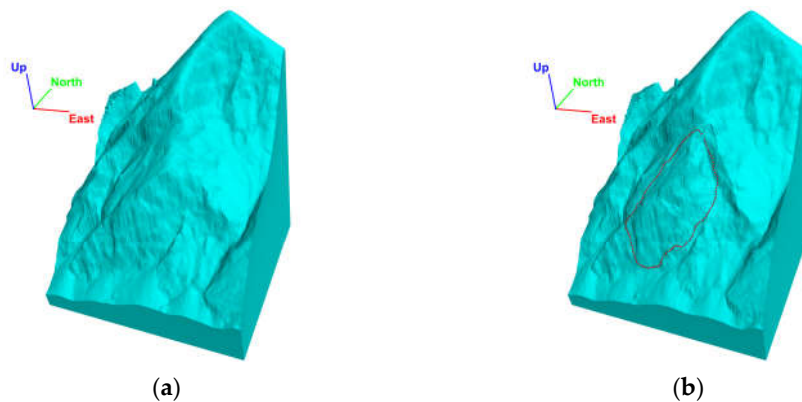
(a)

(b)

Figure 1. (a) Existing scarp and tension crack (red line); and (b) tension crack at the top of the potentially unstable area.

This was achieved by performing the numerical analysis of the valley flank. Laboratory testing results from the nearby Brenner Basis Tunnel [1] are available. The tunnel is not far from the Vals valley flank and explores the same geological rock units. Thus, the test results are representative for the Vals valley flank. These results and the mapping data (GSI, joint structure) from the TAC are the basis for our numerical analysis.

We tried to back-calculate the initial wedge failure of the 2017 rock fall event (Figure 2) using the Discrete Element Method DEM (3DEC) and the finite difference method (FDM) (FLAC3D) together with the strength reduction method (SRM) [2]. Different material models were applied. To date, we were not able to recreate this singular event with the models.



(a)

(b)

Figure 2. FLAC3D model of the Vals valley flank: (a) before and (b) after the 2017 rock fall event; red: outline of observed moving area.

We used the SRM and Hoek–Brown (HB) material with both equivalent and apparent Mohr–Coulomb (MC) parameters. Our limit equilibrium studies ended up in very different results (refer to Section 6). We describe the implications of these different results and point out the challenge they pose. We propose how to overcome this challenge in this article.

2. Geological Settings

The rock fall event in the valley of Vals (Figure 3) is situated at the Western end of the tectonic Tauern Window. Therein, Penninic metamorphic rock nappe units are currently exposed on the surface. These so-called Bündner Schists build up the valley flanks of Vals [3]. They are composed of graphitic phyllites, calcareous phyllites, and calcareous

quartzite bearing schists and marbles. The Bündner Schists represent flyschoid metasediments of the Mesozoic age [4]. These were sedimented in the Valais Ocean [5]. During the Alpine orogeny, the described rocks underwent strong multiphase deformation accompanied by a metamorphism, reaching the green schist facies [6]. Structurally, the Bündner Schists are geologically related to the Glockner Nappe [7]. At the rock face in Vals, the following main sets of discontinuities/faults are present (Figure 4): (1) the persistent but folded rock schistosity (s) dipping 30° to the northwest; (2) NNW–SSE to N–S striking; and (3) ENE–WSW to E–W striking, inclined joints, and faults. The discontinuity sets (2) and (3) are related to ductile–brittle deformation in the context with Brenner normal fault [8].



Figure 3. Location of Vals in Tyrol/Austria (red circle).

From a geological point of view, the existing rock mass is inhomogeneous and anisotropic. Soft rocks such as phyllites are intercalated with the hard rocks of calcareous to quartzite-bearing schists and marbles. The uniaxial strength varies from less than 25 MPa to more than 100 MPa. The existing discontinuities are persistent.



Figure 4. Outcrop, representative for strongly folded and jointed rock mass of Bündner Schists in the valley of Vals: S—schistosity (magenta), 2—conjugate discontinuities (yellow), 3—fault plane (red).

On the larger scale, these anisotropic conditions remain the same with depth, so that one can speak of ‘homogeneous anisotropy’. To overcome this anisotropy, a continuum mechanical approach was assigned. The rock mass outcrops are described by applying the GSI chart for flysch sediments [9]. According to that, the rock mass in Vals represents a GSI in the range between 32 and 36.

3. Applied Known Methods and Facts

3.1. The Strength Reduction Method (SRM) in FLAC3D

The finite-difference continuum-mechanic program FLAC3D (Itasca Consulting Group) provides the possibility to calculate safety factors by applying the SRM [2]. In this process, the shear strength of the material is gradually reduced to bring the slope to a state

of limit equilibrium. The SRM was used in the context of MC materials and the simultaneous reduction in cohesion c and friction angle φ . The factor F is defined according to the following equations:

$$c^{red} = \frac{c}{F} \tag{1}$$

$$\varphi^{red} = \arctan\left(\frac{\tan \varphi}{F}\right) \tag{2}$$

By applying the SRM to the HB criterion, FLAC3D locally approximates the HB parameters by a MC criterion:

$$\tau = \sigma' \cdot \tan \varphi_c + c_c \tag{3}$$

Calculating the apparent cohesion c_c and apparent friction angle φ_c in each zone (depending on σ_3):

$$\varphi_c = 2 \cdot \tan^{-1} \cdot \sqrt{N_{\varphi_c}} - 90 \tag{4}$$

$$c_c = \frac{\sigma_c^{UCS}}{2 \cdot \sqrt{N_{\varphi_c}}} \tag{5}$$

where, for compressive stresses ($\sigma_3 > 0$):

$$N_{\varphi_c} = 1 + a \cdot m_b \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1} \tag{6}$$

$$\sigma_c^{UCS} = \sigma_3 \cdot (1 - N_{\varphi_c}) + \sigma_{ci} \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^a \tag{7}$$

Additionally, for tensile stresses ($\sigma_3 < 0$):

$$N_{\varphi_c} = 1 + a \cdot m_b \cdot (s)^{a-1} \tag{8}$$

$$\sigma_c^{UCS} = \sigma_{ci} \cdot (s)^a \tag{9}$$

where σ_{ci} is the unconfined compression strength and m_b , s , and a are HB parameters (see Equations (11)–(13) below) [10].

3.2. Apparent and Equivalent Mohr–Coulomb Parameters

Limit equilibrium can be determined using the MC or the HB criterion. Using the MC criterion, (equivalent) strength parameters c and φ are assumed to be constant for the considered homogenous region. Using the HB criterion, apparent MC parameters (c_c and φ_c) are determined for each individual numerical zone (with variable σ_3). They are thus to be regarded as ‘local’ parameters, i.e., different in every zone, depending on σ_3 (Figure 5).

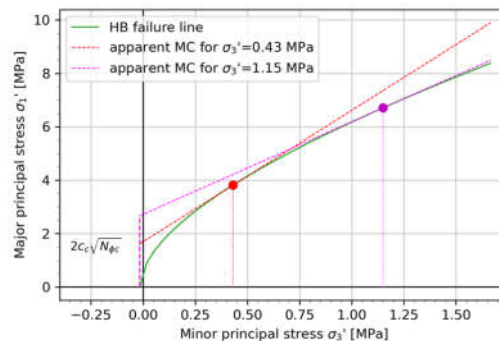


Figure 5. HB failure criterion and its MC approximation with apparent (‘local’) MC parameters (at two locations).

The advantage of ‘local’ over equivalent MC parameters has already been demonstrated in the literature. Ref. [11] studied the accuracy of using equivalent MC parameters to estimate the factor of safety. Especially for steep slopes, it leads to poor estimates of safety factors and poor predictions of detachment volumes. The problem lies in the required estimation of a suitable range for smaller principal stresses (σ_3) over the ‘slope height’ (or ‘tunnel depth’).

Figure 6 shows the HB criterion and equivalent MC envelope. The equivalent MC envelope is one straight line for the entire homogenous region. It cannot fit the HB curve equally well at all depths (i.e., for all σ_3). For lower and higher σ_3 ranges (regions 1 and 3), equivalent MC parameters may overestimate the ultimate shear strength compared to the HB curve.

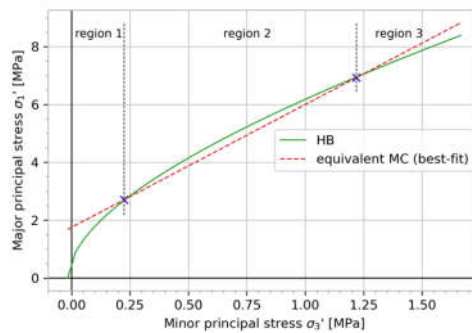


Figure 6. Hoek–Brown criterion and equivalent Mohr–Coulomb envelope.

3.3. Construction of the HB Limit Equilibrium Envelope for a Given Reduction Factor F

The generalized HB criterion [12] for rock masses is nonlinear. This defines the material strength in terms of major and minor principal stresses through the equation

$$\sigma_1 = \sigma_3 + \sigma_{ci} \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s \right)^a, \tag{10}$$

where σ_{ci} is the uniaxial compressive strength of the intact rock material, while

$$m_b = m_i \cdot \exp\left(\frac{GSI - 100}{28 - 14 \cdot D}\right), \tag{11}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3 \cdot D}\right) \text{ and} \tag{12}$$

$$a = \frac{1}{2} + \frac{1}{6} \cdot \left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}} \right), \tag{13}$$

where m_i is an intact rock material property, GSI is the geological strength index, and D is the disturbance factor.

Ref. [13] developed relationships between the shear-normal stress and the principal stress envelopes. The normal and shear stress pair corresponding to a point on a principal stress envelope is given by the following equations:

$$\sigma_n = \frac{1}{2} \cdot (\sigma_1 + \sigma_3) - \frac{1}{2} \cdot (\sigma_1 - \sigma_3) \cdot \frac{\frac{d\sigma_1}{d\sigma_3} - 1}{\frac{d\sigma_1}{d\sigma_3} + 1}, \tag{14}$$

$$\tau = (\sigma_1 - \sigma_3) \cdot \frac{\sqrt{\frac{d\sigma_1}{d\sigma_3}}}{\frac{d\sigma_1}{d\sigma_3} + 1}. \tag{15}$$

For the generalized HB criterion, the following equations relate σ_n and τ to σ_1 and σ_3 [14]:

$$\sigma_n = \frac{1}{2} \cdot (\sigma_1 + \sigma_3) - \frac{1}{2} \cdot (\sigma_1 - \sigma_3) \cdot \frac{a \cdot m_b \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}}{2 + a \cdot m_b \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}}, \quad (16)$$

$$\tau = (\sigma_1 - \sigma_3) \cdot \frac{\sqrt{1 + a \cdot m_b \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}}}{2 + a \cdot m_b \cdot \left(m_b \cdot \frac{\sigma_3}{\sigma_{ci}} + s\right)^{a-1}}. \quad (17)$$

First, the in situ state of the model is calculated. Then, the limit equilibrium routine in FLAC3D ('model factor-of-safety') gradually reduces the apparent ('local') MC parameters φ and c . Limit equilibrium is reached at the first unstable system state ($F_{unstable}$). F_{stable} is the safety factor of the last stable system state.

The failure line for a given reduction factor F can be calculated using the following formula:

$$\tau^{red} = \frac{\tau}{F}. \quad (18)$$

The geometric interpretation of lowering an HB shear envelope by a factor F is shown in Figure 7.

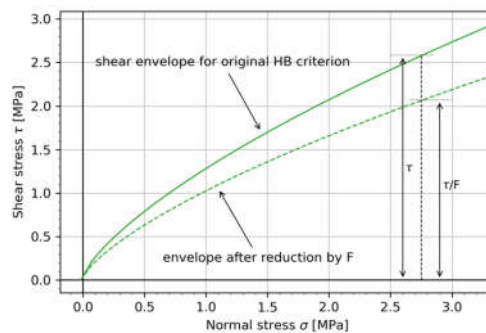


Figure 7. A generalized HB envelope drawn in normal-shear stress space (green), and the resulting envelope after reduction by a factor F (green-dot).

4. The Challenge

The SRM was applied to an MC material model, with MC parameters equivalent to the HB parameters (i.e., equivalent MC parameters). We investigated how different slope heights for calculating the equivalent MC parameters affect the calculation results. What [11] found (see Section 3.2) was confirmed by our calculations. The models show excessively large unstable areas, which cannot be observed in nature (see Section 6.2).

Thus, the HB criterion was used to determine the limit equilibrium using the SRM (see Section 6.3). However, the SRM with HB material requires a local approximation using the MC criterion (apparent MC parameters, see Section 3.1). The reduction factor F calculated in this way cannot be directly used to calculate the HB parameters for the limit equilibrium state.

The SRM leads to a model with chaotically distributed MC parameters in numerical zones (Figure 8). It does not lead to HB parameters for the entire homogeneous area of the model. However, they are required for prediction calculations. The challenge is to find those HB parameters which lead to the chaotic distributions of apparent MC parameters, as shown in Figure 8. In other words, the challenge is to find the HB parameters of the HB envelope for the limit equilibrium state derived via SRM.

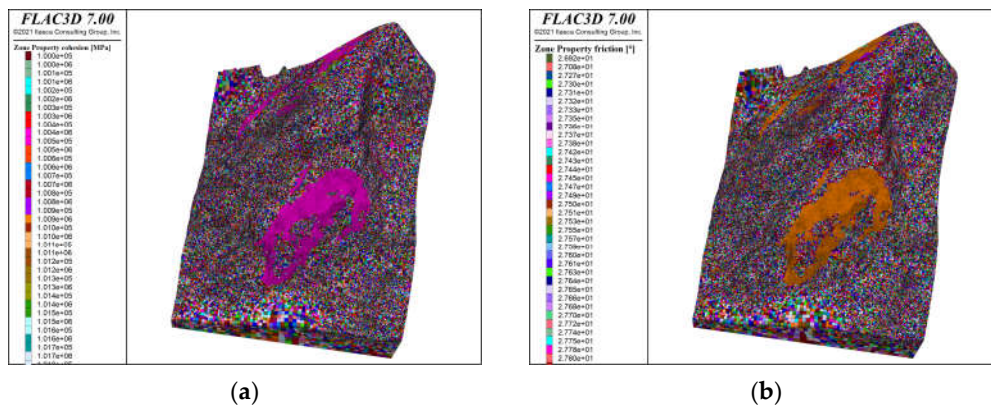


Figure 8. Volume model of chaotically distributed (a) cohesion and (b) friction parameters.

5. Method to Overcome the Challenge: Deriving the HB Parameters Close to the Limit Equilibrium Envelope by Varying GSI and/or D

The reduction factor F cannot be applied to the original HB parameters (as discussed in Section 4).

We need to determine an HB material equivalent to the HB envelope at limit equilibrium state (i.e., reduced by F). This is achieved by adjusting the geological strength index GSI and/or the disturbance Factor D. It is plausible to adjust those parameters due to the following considerations: (1) Based on the statement in Hoek’s GSI chart [9], it is more realistic to consider a range of the GSI, rather than a single value; and (2) Since it can be assumed that the rock mass has been disturbed by previous slope movements, the disturbance factor may be increased.

The laboratory parameters m_i , σ_{di} , and E_i are kept constant.

Figure 9 shows the HB envelopes for the in situ state (green), for the limit equilibrium state (green-dot), and for the HB envelope adjusted to the limit equilibrium state by GSI and/or D (yellow).

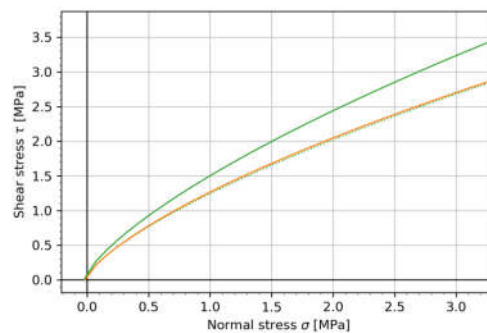


Figure 9. HB failure envelopes in normal-shear stress space; green: in situ state, green-dot: limit equilibrium state, yellow: adjusted to the limit equilibrium state by reducing GSI and/or increasing D.

Since this is an approximation, the derived values of GSI and D should be optimized. We used a concordance check according to [15] to calibrate our model. This is further described in Section 6.4.

6. Stability Analysis of the Valley Flank Vals

We first used the Discrete Element Method DEM (3DEC) for a discontinuum mechanical limit equilibrium study. This failed because of the model size, which required an excessively long computation time. The model size must be sufficiently large to eliminate

the possible effects of boundary conditions. Additionally, the results should be meaningful. The dimensions of the model for the Vals valley flank were set at 560 m wide, 400 m deep, and 510 m high. With such a model size, only large simplifications of the joint system would lead to an acceptable computation time (e.g., increasing the joint spacing).

The continuum mechanical finite difference method (FDM) (FLAC3D) achieves calculations of this model size (654,263 zones and 368,552 grid points) within an acceptable computation time. All results presented in this article were derived via FLAC3D.

6.1. Model Procedure

The distance from the area of interest to the edge of the model was chosen so that the boundary conditions had no influence on the behavior of the model. The side faces were fixed in normal direction and the base surface in x , y , and z directions. The in situ stresses were calculated in two steps. In the first step, a purely elastic material behavior was assumed. Plastic deformations were prevented by setting high rock strengths. In the second step, the actual strength values of the material (Table 1) were assigned and the stress state calculated.

Laboratory testing results [1] and mapping data from the TAC delivered the parameters listed in Table 1. The equivalent MC parameters were determined using the RocData program from [16]. For this purpose, the stress state in the failure surface of the slope movement (shear zone) must be estimated by specifying a ‘slope height’. The slope height related to a possible slope movement was estimated with 80 m. A study of different slope heights was conducted (refer to Section 6.4.1).

Table 1. HB, laboratory, and equivalent MC parameters used for the different limit equilibrium calculations.

State	Initial (Before and After)	Limit Equilibrium		
		Before	After	After Best Fit
Model 1: Equivalent MC Parameters				
φ	42	32.4	31.5	
c (MPa)	0.5	0.353	0.340	
σ_t (MPa)	0.026	0.026	0.026	
$F_{unstable}$		1.420	1.473	
Model 2: Laboratory and HB Parameters				
σ_{ci} (MPa)	45.8	45.8	45.8	45.8
E_i (MPa)	30,000	30,000	30,000	30,000
γ (MN/m ³)	0.02728	0.02728	0.02728	0.02728
GSI	34	34	34	34
D	0	0.37	0.37	0.35
m_i	12	12	12	12
m_b	1.14	0.6654491	0.6654491	0.6891914
s	6.53×10^{-4}	2.329×10^{-4}	2.329×10^{-4}	2.480×10^{-4}
a	0.52	0.5170641	0.5170641	0.5170641
$F_{unstable}$		1.207	1.211	

The model parameters were applied to the entire model. Thus, each respective model was considered a homogeneous region regarding its rock properties. The limit equilibrium (safety factor) was determined for four different models using the SRM (as described in Section 3.1). The four different models are:

- Model 1a: Mohr–Coulomb before the rock fall event 2017;
- Model 1b: Mohr–Coulomb after the rock fall event 2017;
- Model 2a: Hoek–Brown before the rock fall event 2017; and
- Model 2b: Hoek–Brown after the rock fall event 2017.

6.2. Model 1—Mohr–Coulomb

For model 1, the parameters are listed in Table 1, in the column ‘Initial (Before and After)’. The analysis of the valley flank was performed using the MC material model and MC parameters equivalent to the HB parameters.

For the terrain before the rock fall event 2017 (model geometry ‘Before’), the reduction factor F_{stable} of the last stable state was 1.418. The reduction factor $F_{unstable}$ of the first unstable state was 1.420. The safety factor can thus be given as 1.42. Applying Equations (1) and (2) gives the MC parameters $c = 0.353$ MPa and $\phi = 32.4^\circ$ at a limit equilibrium.

For the terrain after the rock fall event 2017 (model geometry ‘After’), the reduction factor F_{stable} of the last stable state is 1.470. The reduction factor $F_{unstable}$ of the first unstable state is 1.473. The safety factor can thus be given as 1.47. Applying Equations (1) and (2) gives the MC parameters $c = 0.340$ MPa and $\phi = 31.5^\circ$ at the limit equilibrium state.

Comparison of ‘Before’ and ‘After’ MC Model

Both model calculations show much larger extents of the moving area than observed. This applies especially to the west and above the observed tension crack. The failure mechanism close to the limit equilibrium state is that of slope creep [17]. This means the continuous decrease in displacements with depth (Figure 10c). A continuous shear band (Figure 10b) separates the moving from the non-moving area (subtype block slope creep, [18]). Due to the extent and depth of the separating shear band (Figure 10a), the movement can be classified as that of a deep-seated landslide.

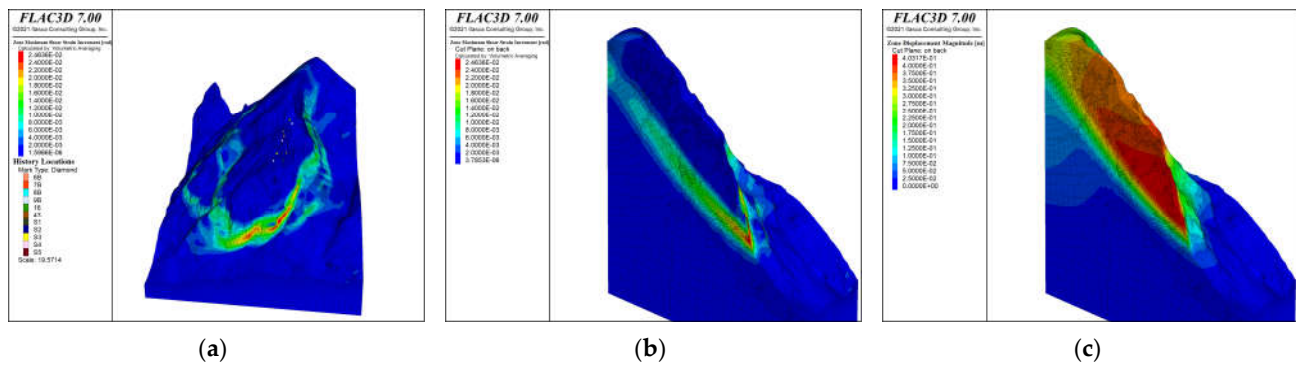


Figure 10. Model 1b ‘after’, $F = 1.47$: (a) contour plot of the shear strain increment; (b) section of the shear zone; and (c) section of the moving area (displacement magnitude).

Table 2 shows a comparison of the calculated reduction factors F_{stable} and $F_{unstable}$ of the two MC models ‘Before’ and ‘After’.

Table 2. Calculated reduction factors for Model 1 MC.

Model 1 MC	F_{stable}	$F_{unstable}$
1a ‘Before’	1.418	1.420
1b ‘After’	1.470	1.473

With a safety factor of approximately 1.47, the ‘After’ model shows a higher overall stability than the ‘Before’ model with a safety factor of approximately 1.42. It can be concluded that the missing rock fall volume of approximately 117,000 m³ (from the 2017 event) increases the overall stability of the ‘After’ model.

6.3. Model 2—Hoek–Brown

For model 2, the parameters are listed in Table 1 (column ‘Initial Before and After’). The analysis of the valley flank is performed using the HB material model implemented in FLAC3D, with the listed HB parameters.

6.3.1. Model 2a—‘Before’

For the terrain before the rock fall event 2017 (model geometry ‘Before’), the reduction factor F_{stable} of the last stable state is 1.200. The reduction factor $F_{unstable}$ of the first unstable state is 1.207. The safety factor can thus be given as 1.20.

The reduction factor F cannot be applied to the original HB parameters (as discussed in Section 4).

We need to determine an HB material equivalent to the HB envelope at limit equilibrium state (i.e., reduced by F). This is achieved by adjusting the disturbance factor D (loosening factor). In this case, the GSI is not adjusted. The necessary disturbance factor D to approximate the limit equilibrium state is 0.37 (with a constant GSI of 34). The slope stability was analyzed with the resulting HB parameters listed in Table 1, in the column ‘Limit Equilibrium Before’.

Figure 11 shows the envelopes for the in situ state (green), for the reduction with $F_{unstable}$ close to the limit equilibrium state (green-dot), and the envelope fitted by the disturbance factor D (yellow).

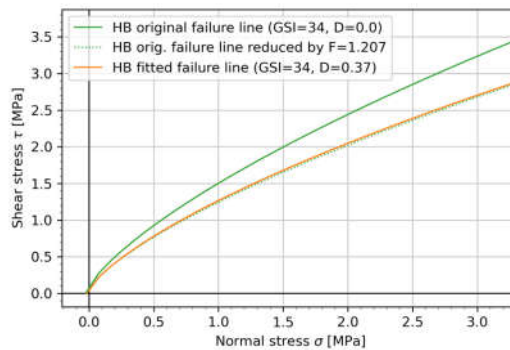


Figure 11. HB envelopes in the sigma-tau diagram for model ‘Before’; green: failure envelope of the in situ state, green-dot: failure envelope of the limit equilibrium state (reduced by $F = 1.207$), and yellow: envelope fitted to the limit equilibrium state by $D = 0.37$.

Figure 12 shows the (a) contour plot of the shear strain increment, (b) the section of the shear zone, and (c) the section of the moving area (displacement magnitude). The volume of the calculated detachment (zones with velocities $\geq 1 \times 10^{-5}$ m/s) is 543,200 m³ [19].

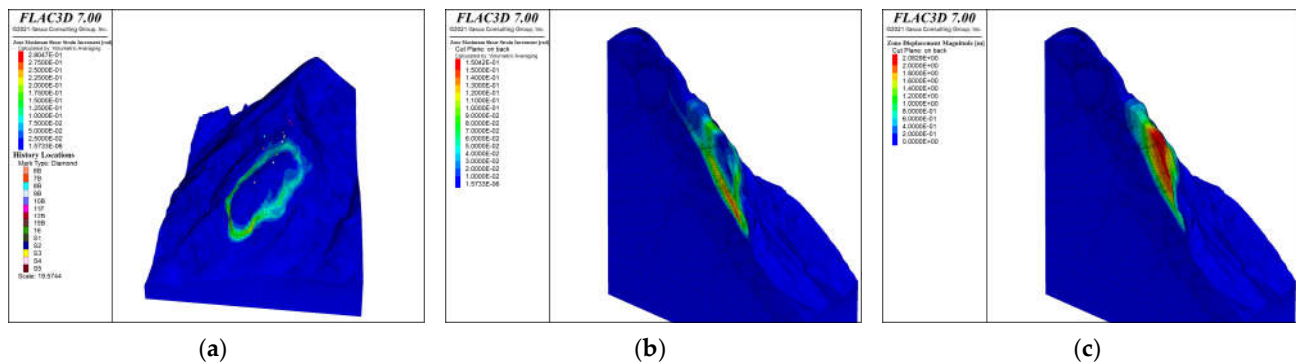


Figure 12. Model 2a ‘Before’, $D = 0.37$: (a) contour plot of the shear strain increment; (b) section of the shear zone; and (c) section of the moving area (displacement magnitude).

6.3.2. Model 2b—‘After’

For the terrain after the rock fall event 2017 (model geometry ‘After’), the reduction factor F_{stable} of the last stable state is 1.207. The reduction factor $F_{unstable}$ of the first unstable state is 1.211. The safety factor can thus be given as 1.21.

We need to determine an HB material equivalent to the HB envelope at limit equilibrium state (i.e., reduced by F). This is achieved by adjusting the disturbance factor D (loosening factor). In this case, the GSI is not adjusted. The necessary disturbance factor D to approximate the limit equilibrium state is 0.37 (with a constant GSI of 34). The slope stability is calculated with the resulting HB parameters listed in Table 1, in the column ‘Limit Equilibrium After’.

Figure 13 shows the envelopes for the in situ state (green), for the reduction with $F_{unstable}$ close to the limit equilibrium state (green-dot), and the envelope adjusted by the disturbance factor D (yellow).

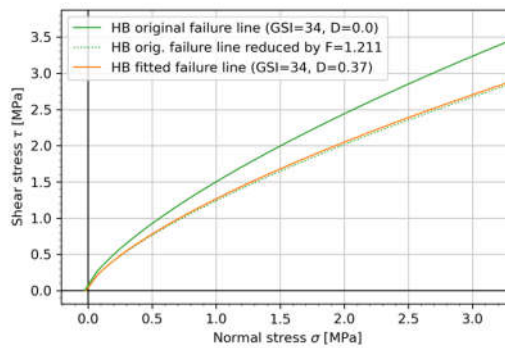


Figure 13. HB envelopes in sigma-tau diagram for model ‘After’; green: failure envelope of the in situ state, green-dot: failure envelope of the limit equilibrium state (reduced by $F = 1.211$), and yellow: envelope fitted to the limit equilibrium state by $D = 0.37$.

Figure 14 shows the (a) contour plot of the shear strain increment, (b) the section of the shear zone, and (c) the section of the moving area (displacement magnitude). The volume of the calculated detachment³ is 495,143 m³ [19].

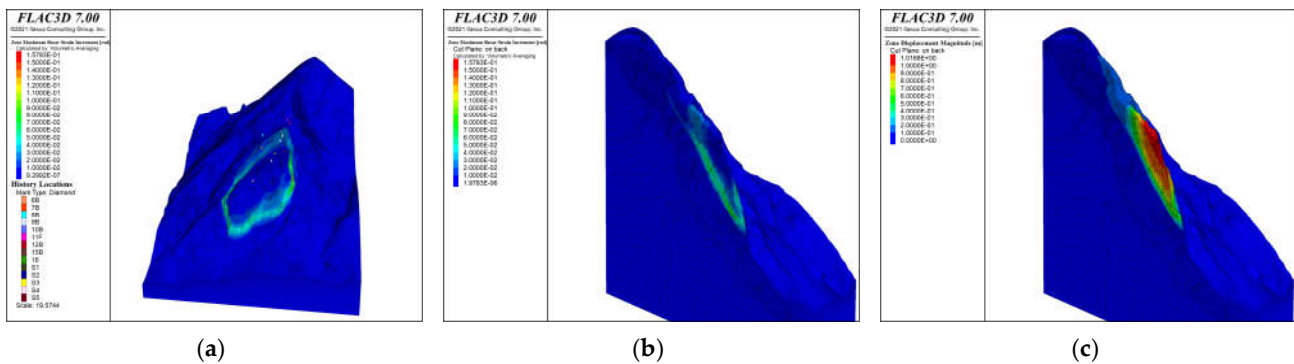


Figure 14. Model 2b ‘After’, $D = 0.37$: (a) contour plot of the shear strain increment; (b) section of the shear zone; and (c) section of the moving area (displacement magnitude).

6.3.3. Comparison of 'Before' and 'After' HB Model

A comparison of the calculated reduction factors F_{stable} and F_{unstable} of the two HB models 'Before' and 'After' is shown in Table 3.

Table 3. Calculated reduction factors for model 2 Hoek–Brown.

Model 2 HB	F_{stable}	F_{unstable}
2a 'Before'	1.200	1.207
2b 'After'	1.207	1.211

The reduction factor F_{unstable} of the 'Before' model is equal to the reduction factor F_{stable} of the 'After' model. This means that we can simulate that the safety of the slope is higher after the wedge of the 2017 event has broken off than before. However, the model cannot tell whether the predicted detachment volume will break off as a whole or in parts.

In both models, the displacements decrease with increasing depth (Figures 12c and 14c).

The volumes of the calculated detachments in the 'Before' and 'After' models are 543,200 m³ and 495,143 m³, respectively. This is a difference of 48,057 m³. In the 'Before' model, the volume of the observed wedge failure of the 2017 event (117,000 m³) is partly included in the calculated detachment volume. Both models show high concordance in terms of location and volume with the moving area identified by the TAC. To further adjust the model parameters, the model 'After' was calibrated.

6.4. Model Calibration

For model calibration, the moving area in the model was compared with the observation. A concordance check according to [15] was performed. The modeled and observed moving areas were projected onto the ground plane, superimposed, and compared. A displacement velocity of 1×10^{-5} m/s was set as the limit for the modeled unstable area [19].

The subarea of the superimposed modeled and observed moving areas (Figure 15) that overlap with one another is true positive (TP). The subarea outside of both the modeled and observed moving areas is true negative (TN). The modeled moving subarea that is outside the observed moving area is false positive (FP). The observed moving subarea that is outside of the modeled moving area is false negative (FN).

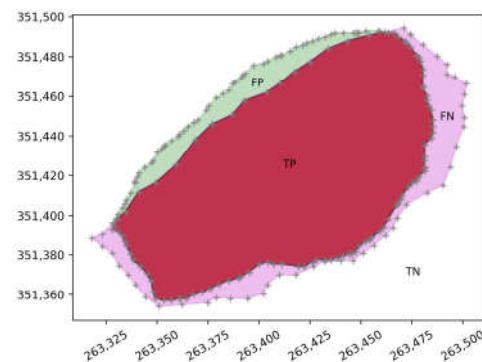


Figure 15. Concordance check by superimposing two areas projected onto the ground plane: the observed moving area (red and magenta) and modeled moving area (red and green; with HB parameters $GSI = 34$, $m_i = 12$, and $D = 0.35$); red: TP (modeled moving area matches observed moving area), green: FP (modeled moving area beyond observed moving area), and magenta: FN (observed moving area not represented in model).

From the thus determined subareas, the following values can be calculated:

- Critical success index (CSI);
- Heidke skill score (HSS);
- Distance to perfect classification (D2PC);
- Accuracy index (ACC).

The *CSI* indicates the percentage of positively correctly modeled areas of an observed region:

$$CSI = \frac{TP}{TP + FP + FN} \quad (19)$$

The *HSS* is used in the evaluation of meteorological forecasts and is an index for categorical forecasts [20]. The *HSS* can take values between $-\infty$ and 1, where 1 denotes the optimum:

$$HSS = \frac{2 \cdot (TP \cdot TN - FP \cdot FN)}{(TP + FP) \cdot (FP + TN) + (TP + FN) \cdot (FN + TN)} \quad (20)$$

The index *D2PC* is determined according to [21] and [15], where *TPR* is the true positive rate and *FPR* is the false positive rate. The *D2PC* takes values between 0 and 1. The optimum of this index is 0:

$$D2PC = \sqrt{(1 - TPR)^2 + FPR^2} = \sqrt{\left(1 - \frac{TP}{OP}\right)^2 + \left(\frac{FP}{ON}\right)^2}, \quad (21)$$

$$TPR = \frac{TP}{TP + FP} = \frac{TP}{OP} \quad (22)$$

$$FPR = \frac{FP}{FP + TN} = \frac{FP}{ON} \quad (23)$$

The *ACC* can assume values between 0 and 1, where 1 indicates the optimum of achieved accuracy. In contrast to *CSI*, the *ACC* takes the value of the true negative (*TN*) into account:

$$ACC = \frac{TP + TN}{TP + TN + FP + FN} \quad (24)$$

6.4.1. Calibration of the MC Model 'After'

We investigated the effect of different slope heights (using the SRM) in a study. The slope height was varied between 65 and 80 m. Another study investigated the effect of a variation of MC parameters (at limit equilibrium state). Cohesion and friction angles were varied between 0.24 MPa and 0.44 MPa, and between 31.1° and 32.6°.

Neither a variation of the slope height nor a variation of the MC parameters could improve the concordance of that modeled with the observed moving area.

6.4.2. Calibration of the HB Model 'After'

The results of the concordance check of the HB 'After' model are listed in Figure 16. A disturbance factor *D* of 0.35 and a *GSI* of 34 gives the highest concordance (i.e., 'best-fit') between the modeled and the observed moving areas (Figure 17).

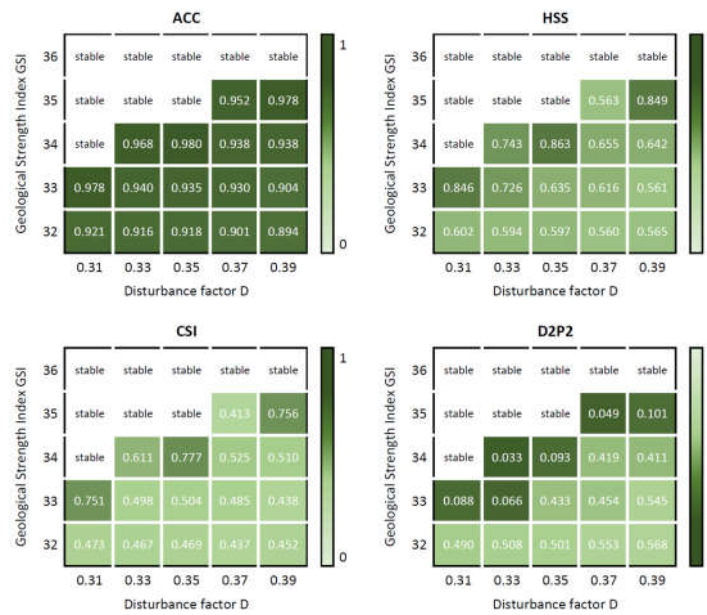


Figure 16. Results of the calibration model HB ‘After’; dark green means best fit.

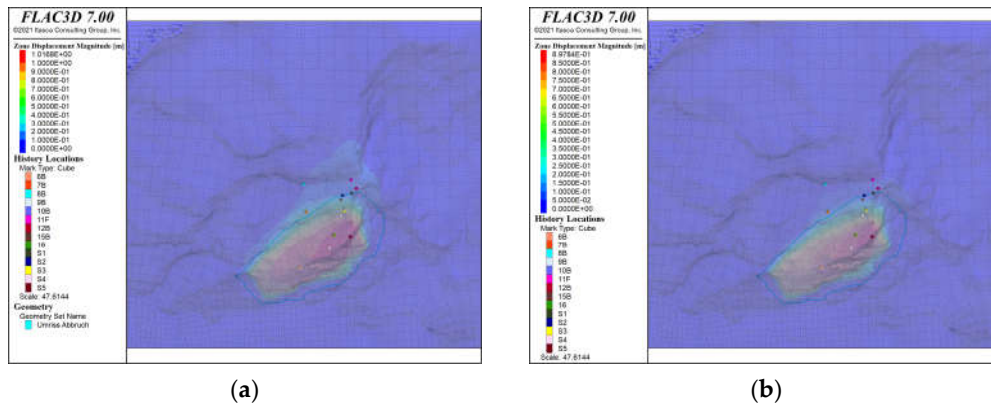


Figure 17. HB model ‘After’: comparison of the moving areas (displacement magnitudes) with (a) D = 0.37 vs. (b) D = 0.35; cyan: outline of the observed moving area.

6.5. Failure Prediction

With the result of the model calibration ‘After’, a predictive calculation of the moving area was performed. The HB parameters are listed in Table 1, in the column ‘Limit Equilibrium After Best-Fit’.

After the 2017 event (in July 2018), five extensometers (S1–S5) and 16 prisms were installed at the Vals valley flank and monitored. At the time of writing, the prisms show negligible displacements. Extensometers S1–S5 show more or less constant displacements between 1 mm (S3) and 10 mm (S2) per year. It can thus be concluded that the situation is stable. Currently, the safety factor is just above 1 or at 1.

The calibrated strength parameters were used to predict the displacements if the safety factor is just below 1. Figure 18a shows the contour plot of the shear strain increment. The unstable slope area (possible detachment) is limited by the shear band shown. It extends over the entire upper rock face from its base area to the observed tension crack (measuring points S1 and S2 in Figure 19a) at the top of the shear zone.

The displacements decrease with increasing depth (Figure 20b). The volume of the calculated detachment (zones with velocities $\geq 1 \times 10^{-5}$ m/s) is 411.277 m³ [19].

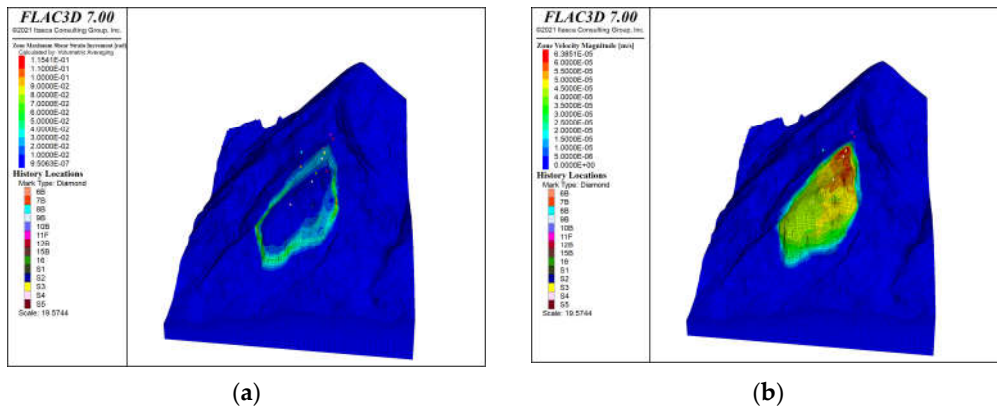


Figure 18. Model ‘Best-Fit’, $D = 0.35$: (a) shear strain increment; and (b) displacement velocities.

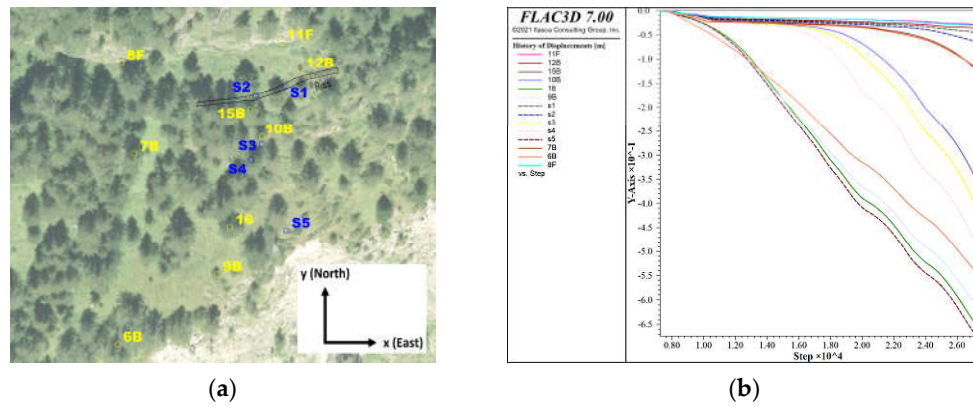


Figure 19. Monitoring points: (a) overview of the actual monitoring points; (b) model ‘Best-Fit’, $D = 0.35$: course of the displacements [m] in negative y direction (i.e., towards South) over the calculation steps.

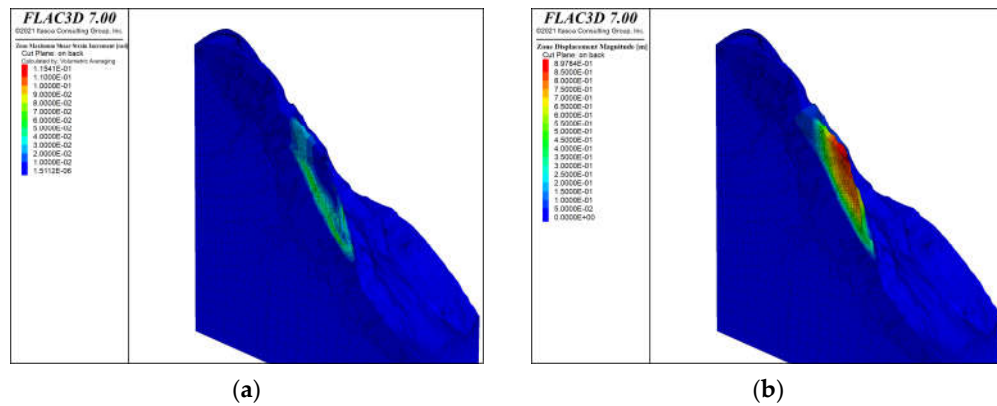


Figure 20. Model ‘Best-Fit’, $D = 0.35$: (a) section of shear zone; and (b) section of moving region (displacement magnitude).

Figure 19 shows the displacements of the monitoring points in the HB model ‘Best-Fit’. Such calculations can only be used for qualitative predictions. The calculated records of displacements show that the monitoring points in the lower moving area (6B, 9B, 16, S5) start moving first. Later, displacements in the middle moving area (10B, S3, S4) increase. They accelerate relatively fast. Lastly, the monitoring points at the head of the moving area (especially around the observed tension crack (S1 and S2), 7B and 15B) show

displacements. The displacement velocities in Figure 19b show that the monitoring points in the middle moving area (10B, S3, S4) accelerate the fastest at the end of the simulation. Note the steepest curves of these monitoring points. These findings can be used in the interpretation of the monitoring and may help in setting alert levels.

7. Discussion

The structural failure of the Vals valley flank in 2017 was a wedge failure. In such a case, the question arises whether a stability assessment using continuum mechanical (smeared) methods is expedient. From a geological point of view, the Vals rock mass is inhomogeneous and anisotropic. For this reason, a discontinuum mechanical calculation (DEM with 3DEC) was set up first. However, in the case in the Vals valley flank, a DEM calculation is not feasible or only feasible with strong simplifications. The extent and anisotropy of the rock face require an increase in joint spacings or a combination of joint orientations. On the larger scale, these heterogeneous and anisotropic conditions remain the same with depth. Thus, we can speak of ‘homogeneous conditions’ in slope scale. In continuum mechanical models, it is state of the art to describe heterogeneous rock masses such as flysch using the HB criterion [9]. We applied a GSI for the flyschoid sequence of the Vals rock mass in the range between 32 and 36.

According to [22] (p. 101ff), the effects of heavy blast damage as well as stress relief due to the removal of the overburden result in disturbance of the rock mass. The disturbance factor D was introduced to describe such stress relief effects in artificial slopes (and tunnels). It is considered that the disturbed rock mass properties using $D = 1$ are more appropriate for these ‘artificial’ rock masses. No blasting has been carried out on the Vals valley flank. We can assume that the former glacier melting and rock mass falls (including the event in 2017) led to stress relief and hence to some disturbance in the rock mass. Since the Vals valley flank is not an artificial slope, we consider initial rock mass properties using $D = 0$ as an appropriate baseline for the SRM. Hence, the calculated safety factors in Table 3 are with respect to the initial rock mass parameters with $D = 0$. We can assume that the Vals valley flank is near limit equilibrium.

The manipulation of both the GSI and D is plausible. It is more realistic to consider a range of the GSI rather than a single value. Additionally, we can assume that the rock mass was disturbed by prior slope movements and stress relief because of former glacier melting. GSI and D are also excellent for calibrating models, as demonstrated by the concordance check described in Section 6.4.

Displacement records cannot be directly implemented, because FLAC uses an attenuation where accelerations are damped (local damping). Thus, it is only possible to make qualitative displacement predictions.

In our discontinuum mechanical and continuum mechanical calculations, the models each consist of one homogeneous region (e.g., same joint spacing/material properties over the whole model). It was not possible to back-calculate the 2017 event with the homogeneous models. Thus, we can assume that it was caused by a local weak zone or a water-bearing layer within the rock mass. However, such conditions could not be mapped in the scarp area.

Our investigations show that a linear material model such as that of MC is poorly suited to represent both the brittle fracture behavior of rock at lower and ductile behavior at higher lateral stresses σ_3 . The same problem exists when converting the HB parameters into equivalent MC parameters. Thus, a linear failure envelope is a poor approximation to the natural conditions. The HB criterion, with its curved failure envelope, is more appropriate. On the other hand, the HB criterion requires the determination of the GSI by field mapping. Additionally, the HB criterion is poorly suited for limit equilibrium studies because the SRM cannot be directly applied. In numerical models, this weakness is solved by introducing the apparent MC parameters. The challenge is to convert the apparent MC parameters back to HB parameters. The presented method shows a feasible way of doing this.

8. Conclusions

The presented method eliminates the difficulty of estimating a suitable range for smaller principal stresses (σ_3) via the ‘slope height’ (or ‘tunnel depth’).

For the present stability analysis with FLAC3D, it was not possible to back-calculate the structural wedge failure from 2017 using the HB criterion. However, it was possible to verify the currently most failure-prone area based on field mapping. However, this was not possible using equivalent MC parameters but only using apparent MC parameters. Using equivalent MC parameters results in a failure mechanism, which cannot be observed in nature, and in a higher safety factor!

Using equivalent MC parameters, it was also not possible to improve the concordance of the moving area between the model and observation by model calibration. This was tried with MC parameters equivalent to the HB parameters for different ‘slope heights’ and a variation of MC parameters. The present investigations clearly show that an application of equivalent MC parameters can lead to a wrong estimation of the overall stability. This was confirmed by [11].

The calibrated model may be used for qualitative displacement prediction. In this case, possible failure is not announced by displacements at the upper end of the moving area, near the observed tension crack. This is rather announced by displacements at the lower end of the moving area (6B, 9B, 16, S5). This information will enable targeted monitoring in the future.

Numerical methods provide reduction factors using the strength reduction method, which to date can only be applied to MC parameters but not to HB parameters. The presented method for calculating HB parameters from the chaotic distribution of apparent MC parameters by manipulating the GSI and D is a promising approach. This enables true limit equilibrium studies and prognostic calculations using the HB criterion with continuum mechanical numerical methods.

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Determination of Meaningful Block Sizes for Rock Fall Modelling

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Abstract

The determination of the so-called design block is one of the central elements of the Austrian Guideline for rock fall protection ONR 24810. It is specified as a certain percentile (P95-P98, depending on event frequency) of a recorded block size distribution (BSD). BSDs may be determined from the detachment area (IBSD) and/or from the deposition area (RBSD). Deposition areas, if available, are generally accessible and measurable without technical aids. However, most measuring methods are subjective, uncertain, not verifiable, or inaccurate. Also, rock fall blocks are often fragmented due to the preceding fall process. The IBSD is (also) required for meaningful rock fall modelling. The statistical method seems the most efficient and cost-effective method to determine IBSDs with many blocks within the full range of block sizes. In current literature, joint properties are often described by the lognormal and exponential distribution functions. Today, we can model Synthetic Rock Masses (SRM) based on Discrete Fracture Networks (DFN). DFNs statistically describe the geometric properties of the joint sets. This way, we can carry out exact rock mass block surveys and determine IBSDs. We asked whether IBSDs derived from SRM models can be described by probability distribution functions and if yes, how well. We fitted various distribution functions to three determined IBSDs of different lithologies. We compared their correlations using the KS test and the MSE method. We show that the generalized exponential distribution function best describes IBSDs across various lithologies, when compared to 78 other distribution functions. This could allow for more certain, accurate, verifiable, holistic, and objective results. Further investigations are required.

Keywords

Rock fall, block size distribution, design block, joint spacing, persistence, joint size, discrete joint network, synthetic rock mass model, probability distribution

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33 1 Introduction

34 The Austrian Guideline ONR 24810 (On, 2021) regulates the design of rockfall protection nets, dams, and
35 galleries. The design is based on the kinetic energy and bounce height of a so-called design block. The design
36 block is specified as a percentile (P95-P98) of a recorded block size distribution (BSD), depending on the event
37 frequency. BSDs may be determined from the detachment area (in the following referred to as in-situ block size
38 distribution IBSD) and/or from the deposition area (in the following referred to as rockfall block size distribution
39 RBSD). The ONR 24810 proposes various methods to determine BSDs. In the detachment area, these include
40 estimation by visual assessment, statistical IBSD, and discrete explicit block measurement. In the deposition area,
41 these include estimation, random axis measurements, the line counting method, the area method, sieve analysis or
42 photosieving by software (Gaich and Pötsch, 2022).

43 Deposition areas, if available, are generally accessible and measurable without technical aids. However, most
44 measuring methods are subjective, uncertain, not verifiable, or inaccurate. Blocks smaller than a fist, are usually
45 not measured. Often, (very) small block volumes are unnoticed or considered minor events and not included in
46 rockfall inventories (Laimer, 2019; De Biagi et al., 2017). This affects the percentiles of the BSD. Neglecting
47 many small blocks in a BSD, results in bigger P95-P98 blocks. In hazard analyses, smaller blocks may play an
48 important role, depending on the protection target. For most realistic distributions of kinetic energy, bounce height,
49 and runout, the entire BSD should be considered in rockfall models (Illeditsch and Preh, 2020). Also, rock fall
50 blocks are often fragmented due to the preceding fall process. RBSDs determined from a few dozen blocks (or
51 defined design blocks thereof) do not seem to be sufficient for meaningful rock fall modelling. If possible, IBSDs
52 should (also) be considered. Visual assessment of rock faces seems subjective and not verifiable. Discrete explicit
53 block measurements with rope access require intensive resources and time and are often unfeasible. The statistical
54 method seems the most efficient and cost-effective method to determine IBSDs with many blocks within the full
55 range of block sizes.

56 The distribution of IBSDs is based on the knowledge that bedding, foliation and joints are commonly closely
57 spaced and have low persistence. Wide spacing and high persistence joints are less common. Consequently, for
58 rock blocks formed by joints, small blocks are more common than large blocks.

59 In a case study, Wyllie (2014), Chapter 8.3.1, estimates the means and standard deviations of lognormal
60 distributions for the rock fall diameter [m] and thickness [m] of discoid-shaped blocks. Based on these
61 distributions, he determines an IBSD [m³].

62 Priest and Hudson (1981) show a histogram of measured joint sizes [m] (i.e., persistence) in a Cambrian sandstone.
63 They fit both, exponential and lognormal curves to this data, for which the correlation coefficients r are 0.69 and
64 0.89, respectively. While the lognormal curve has a higher correlation coefficient, the exponential curve has a
65 better fit at longer joint sizes.

66 Hudson and Harrison (1997), Chapter 7.2.1, show that, when a sufficient large sample of individual joint spacings
67 [m] (more than 200) is plotted in histogram form, a negative exponential distribution is often evident.

68 Palmström (2000) measures the orientation and spacing of three joint sets on a horizontal and vertical surface. He
69 cuts a cube of 10 m edge length by these joint sets and plots the distribution curve of the resulting blocks [m³] in
70 logarithmic scale. The s-shaped curve reminds of a sieve curve. He suggests characterizing the curve with
71 representative volumes, e.g., with the minimum, 25th, 50th and 75th and maximum percentile.

72 Moos et al. (2021) sample eight RBSDs [m³] of different rock fall sites and fit them to a power law distribution of
 73 the form $f(x,a,b) = ax^{-b}$. Generally, the calculated block volumes of certain return periods are significantly larger
 74 than the expert-based maximum block volumes. The fitted power law only corresponds well to the empirical data
 75 for block volumes ≥ 0.05 m³. There is a strong influence of parameter b on the modelled frequency.

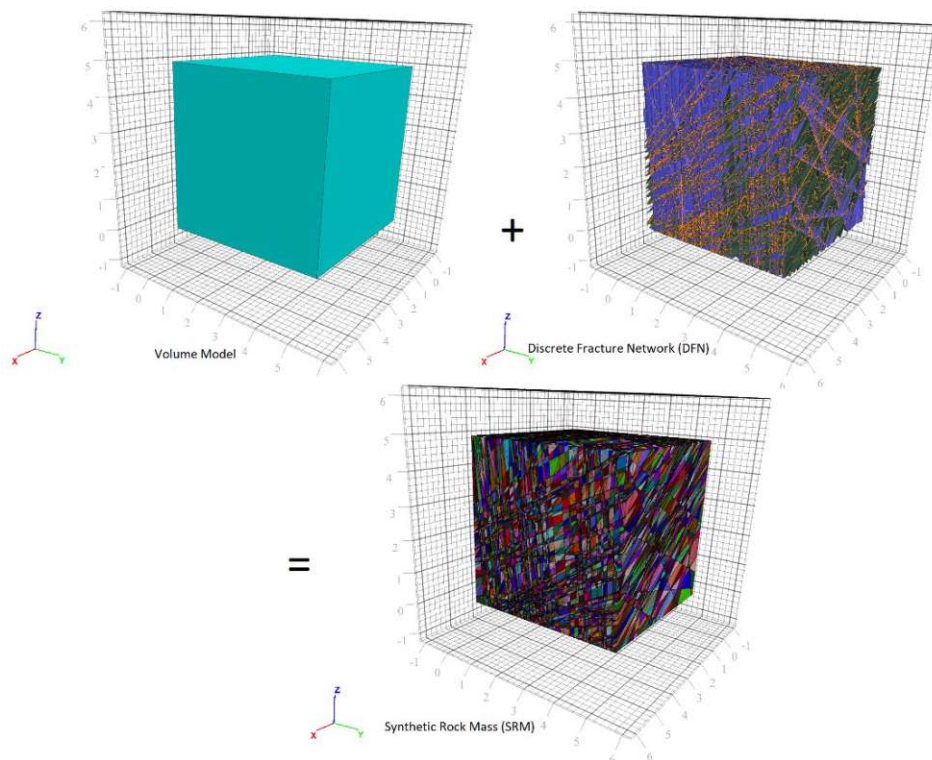
76 Today, Palmström’s method of creating an IBSD by cutting a cube by joint sets, can be applied in a more advanced
 77 way using Synthetic Rock Mass (SRM) models. We asked whether IBSDs derived from SRM models can be
 78 described by probability distribution functions and if yes, how well. Because IBSDs are depending on the joint
 79 properties and they are often described by the lognormal and exponential distribution functions, we asked how
 80 well those functions describe IBSDs.

81 Describing IBSDs by probability distribution functions could allow for more certain, accurate, verifiable, holistic,
 82 and objective results.

83 For this purpose, we calculate different Synthetic Rock Mass (SRM) models based on photogrammetric surveys
 84 by UAV (Unmanned Aerial Vehicle). We carry out exact rock mass block surveys with the help of these models
 85 and determine IBSDs. We fit the lognormal, exponential, and various other distribution functions to the determined
 86 IBSDs. Finally, we compare the correlations between the IBSDs and the distribution functions using the KS test
 87 and the MSE method.

88 2 Method

89 We determine block size distributions from detachment areas (IBSDs) using Synthetic Rock Mass (SRM) models,
 90 fit them to various distribution functions and check their correlations.



91
 92 *Figure 1: Intersection of a volume model with a Discrete Joint Network (DFN) creates a Synthetic Rock Mass (SRM) model.*

93 In SRM models, a Discrete Joint Network (DFN) is intersected with a volume model to simulate the rock mass.
94 The central element here is the DFN. It statistically describes the geometric properties of the joint sets. For this
95 purpose, we consider the distribution of the joint orientation, the joint density (e.g., number of joints per m;
96 reciprocal of joint spacing) and the joint size distribution (persistence) for each individual joint set. A DFN results
97 in a collection of disk-shaped joints whose geometric properties, such as location, orientation, density/intensity,
98 and joint size distribution are subject to a probability distribution. Thus, the joints created in this way do not
99 represent the actual joints in the rock mass. Nevertheless, in this way it is possible to model the joint system very
100 realistically.

101 In detail, the procedure starts with a survey of the rock face by Unmanned Aerial Vehicle (UAV). We took several
102 high-resolution photos of the rock face from different angles and distances with respect to the rock face. Thanks
103 to integrated GPS within the UAV, we know the positions of where the photos are taken. To create a 3D model by
104 photogrammetry, it is necessary to clearly assign each point of the rock face from at least three different
105 perspectives. From the overlapping photos, the software Agisoft Metashape (Agisoft, 2021) creates a point cloud
106 and triangulates it into a mesh. This results in a 3D Digital Elevation Model (DEM) of the rock face. We analysed
107 the point cloud regarding the orientation and distance of the joint sets using the CloudCompare software
108 (Cloudcompare, 2020). *Joint orientations* can be measured at the visible outcrops of the rock face. To measure
109 orientations, CloudCompare creates a flat surface for a certain area if the correspondence of its containing points
110 is large enough. The orientation of the created surface is specified with dip angle and dip direction. We can group
111 the measured joints into joint sets. It is not possible to measure the *joint spacings* in CloudCompare directly. These
112 are defined as normal distances between two joints of the same joint set, which are generally neither parallel nor
113 at right angles to the rock face. However, by back calculating to normal distances and averaging the measurements,
114 we can derive joint spacings. *Joint size* (persistence) is another very important rock mass parameter. It can have a
115 significant influence on the strength and stability of the rock slope. Pahl (1981) has developed a method and
116 equations to calculate the approximate average joint size of a joint set. His method requires a cut-off of small joint
117 sizes and provides an exact solution, if the joint size distribution can be assumed to be exponential. There has been
118 considerable discussion in the literature as to whether the distribution of joint sizes is a negative exponential or a
119 lognormal distribution (Hudson and Harrison, 1997). However, in this case we are interested in block sizes of
120 future rock fall events. They require full detachment from the rock face. Thus, we assume that the blocks are
121 completely cut free by the intersecting joint sets. In other words, we assume 100% persistence for all joints.

122 Based on the above survey information, we can develop a Discrete Joint Network (DFN). A DFN artificially
123 reproduces the existing joint structure as realistically as possible. The *joint orientations* are created by the
124 bootstrapping method. This method assumes that the available random sample is 'representative' of the population
125 from which it is drawn. The bootstrap replaces the theoretical distribution function of a random variable by the
126 empirical distribution function of the sample. So, it is obvious that bootstrapping only works well if the empirical
127 distribution function can approximate the actual distribution function sufficiently well. This requires a certain size
128 of the original sample. Bootstrapping can be understood as a Monte Carlo method, since it repeatedly draws
129 random samples of a distribution. 3DEC offers the possibility to generate the *joint spacings* indirectly via the joint
130 density or joint intensity. The joint density is defined as the area of joints per unit volume [m^2/m^3] (P32), or the
131 length of joints per unit area [m/m^2] (P21), or the number of joints per unit length [m^{-1}] (P10). One square meter
132 of joint in one cubic meter of the rock mass would correspond to a joint density of 1 [m^2/m^3]. The joint density

133 depends only on the joint area per volume. So, several small joint areas can have the same density as a few large
 134 joint areas within the same volume. 3DEC models *joint sizes* as disks. The joint length, or generally the joint size,
 135 refers to the diameter of this disk. We can set the limits of the smallest and largest disk (l_{\min} and l_{\max}) in the software
 136 (Itasca, 2020). In our case, we model disks that go through the entire model domain (100% persistence). We
 137 simplified the determination of the joint density by counting the number of joints per length normal to the joints
 138 (P10), for each joint set. Assuming a persistence of 100%, the number of joints per unit length corresponds to the
 139 area of joints per unit volume, i.e., $P10 = P21 = P32$.

140 By intersecting the DEM with the DFN, we calculate the SRM model. We are not setting any strength parameters
 141 for the rock mass. We are only interested in the volume of the blocks. So, we can rather speak of a synthetic rock
 142 block model than a synthetic rock mass model. With the help of this model, we perform an exact rock mass block
 143 survey and determine a holistic IBSD. The derived IBSDs represent block volumes [m^3]. We want to check their
 144 correlations to various fitted continuous distributions by the Kolmogorov-Smirnov (KS) test and the mean squared
 145 error (MSE) method. We are not interested in properties like density or porosity, which would be affected by cubic
 146 dimensions. In our case, it seems more appropriate to work with size distributions in linear dimensions [m] rather
 147 than in cubic dimensions [m^3]. Furthermore, fitting distribution functions to linear dimensions can provide a more
 148 intuitive visual representation of the size distribution (e.g., grain sizes in a sieve curve). For these reasons, we are
 149 transferring the derived IBSDs from cubic meters to meters by taking the cube root.

150 We use Python to check our derived IBSDs against 79¹ distribution functions (Christopher, 2017). Among them
 151 are also the lognormal and exponential distribution functions.

152 The probability density function (pdf) for lognorm is:

$$f(x, s) = \frac{1}{sx\sqrt{2\pi}} \exp\left(-\frac{\log^2(x)}{2s^2}\right) \quad (1)$$

153 for $x > 0$, $s > 0$, where x is the random variable and s is the standard deviation. Lognorm takes s as a shape
 154 parameter. The probability density above is defined in the ‘standardized’ form. To shift and/or scale the distribution
 155 the loc and scale parameters are used. Specifically, $\text{lognorm.pdf}(x, s, \text{loc}, \text{scale})$ is identically equivalent to
 156 $\frac{\text{lognorm.pdf}(y, s)}{\text{scale}}$ with $y = \frac{x-\text{loc}}{\text{scale}}$. In case of a normally distributed random variable x with mean m and standard
 157 deviation s , $y = \exp(x)$ is normally distributed with standard deviation s and $\text{scale} = \exp(m)$.

158 The probability density function (pdf) for expon is:

$$f(x) = \exp(-x) \quad (2)$$

159 for $x \geq 0$. The probability density above is defined in the ‘standardized’ form. To shift and/or scale the distribution
 160 the loc and scale parameters are used. Specifically, $\text{expon.pdf}(x, \text{loc}, \text{scale})$ is identically equivalent to
 161 $\frac{\text{expon.pdf}(y)}{\text{scale}}$ with $y = \frac{x-\text{loc}}{\text{scale}}$. A common parametrization for expon is in terms of the rate parameter λ , such that
 162 $\text{pdf} = \lambda \cdot \exp(-\lambda \cdot x)$. This parametrization corresponds to using $\text{scale} = \frac{1}{\lambda}$.

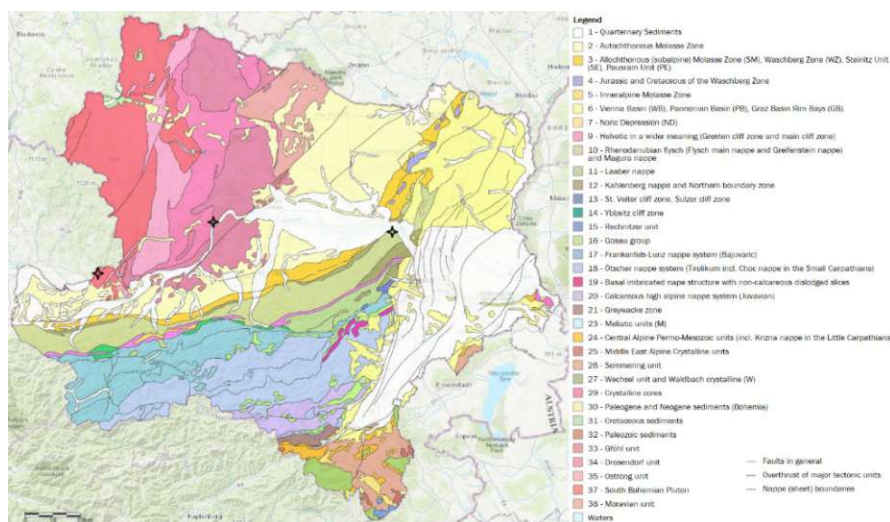
¹ norm, alpha, anglit, arcsine, beta, betaprime, bradford, burr, cauchy, chi, chi2, cosine, dgamma, dweibull, erlang, expon, exponweib, exponpow, fatiguelife, foldcauchy, f, fisk, foldnorm, gamma, gausshyper, genexpon, genextreme, gengamma, genlogistic, genpareto, genhalflogistic, gilbrat, gompertz, gumbel_l, gumbel_r, halfcauchy, halflogistic, halfnorm, hypsecant, invgamma, invweibull, johnsonsb, johnsonsu, laplace, logistic, loggamma, loglaplace, lognorm, lomax, maxwell, mielke, nakagami, ncx2, ncf, nct, norm, pareto, powerlaw, powerlognorm, powernorm, rdist, reciprocal, rayleigh, rice, recipinvgauss, semicircular, t, triang, truncexpon, truncnorm, tukeylambda, uniform, vonmises, wald, weibull_min, weibull_max, wrapcauchy, ksone, kstwobign.

163 The Kolmogorov-Smirnov test, also known as the KS test, is a non-parametric statistical test. It assesses whether
 164 a sample comes from a specific distribution. In other words, the test compares the observed data with the predicted
 165 data (also known as the one-sample KS test) under the null hypothesis that the two distributions are identical. It is
 166 sensitive to both location and shape differences between the sample and reference distributions. The KS test results
 167 in the test statistic (D) and the p-value. These two components are essential in interpreting the outcome of the KS
 168 test. The test statistic (D) is a numerical value that compares the sum of vertical distances of all data points
 169 (supremum) of the cumulative distribution function (cdf) of the sample to the sum of vertical distances of all data
 170 points of the cdf of the reference distribution. The D value ranges between 0 and 1. A value of 0 means a perfect
 171 match between the sample and reference distribution. The p-value is a probability that measures the strength of
 172 evidence against the null hypothesis. It determines whether the observed difference (D) is meaningful and reliable
 173 or can be attributed to random chance. If the p-value is smaller than the chosen significance level (e.g., $p \leq 0.05$),
 174 we can consider the result statistically significant. We can reject the null hypothesis in favour of concluding that
 175 the sample does not follow the reference distribution. If the p-value is greater than the significance level, the result
 176 is not statistically significant. We cannot reject the null hypothesis. There is no sufficient evidence to claim a
 177 difference between the sample and the reference distribution. We choose a confidence level of 95%; that is, we
 178 reject the null hypothesis in favour of the alternative if the p-value is less than 0.05 (i.e., 5%).

179 The Mean Squared Error (MSE) is a statistical metric, also known as the Mean Squared Deviation. It measures the
 180 average squared differences between the observed values and the predicted values in a dataset. It quantifies the
 181 accuracy of a prediction by assessing how well it fits the observed data points. For each data point, the squared
 182 difference between the observed value and the predicted value is calculated. Squaring the differences ensures that
 183 negative differences do not cancel out positive differences. All squared differences are summed up and divided by
 184 the total number of data points. This results in the average squared difference, which is the MSE. Lower MSE
 185 values indicate a better fit of the model to the data (Hedderich and Sachs, 2020).

186 3 Results

187 We applied the described method to three different rock faces in three different areas within Lower Austria:
 188 Tiefenbach, Spitz, and Greifenstein, as shown in Figure 2.



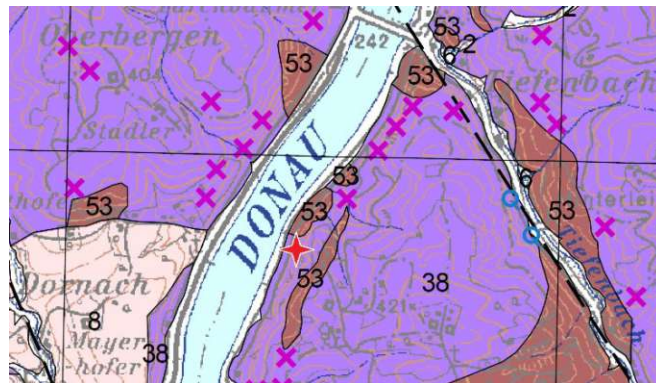
189
 190 *Figure 2: Project areas (from West to East) Tiefenbach, Spitz and Greifenstein within Lower Austria (black)*

191 3.1 Tiefenbach

192 The rock face in Tiefenbach (Figure 3) is in the West of Lower Austria, along the state road Greiner Straße (B 119)
193 next to and along the Danube, which represents the border to Upper Austria there. The rock face is about 25 m
194 high. The geological unit (Figure 4) is the so-called Weinsberger Granite (38): coarse-grained biotite granite with
195 porphyritic large potassium feldspar (Mississippium) next to metablastic to diatectic paragneiss (53), relics of
196 biotite-rich paragneiss ('pearl gneiss') (Moser and Linner, 2019).



197
198 *Figure 3: Photograph of the Tiefenbach rock face*

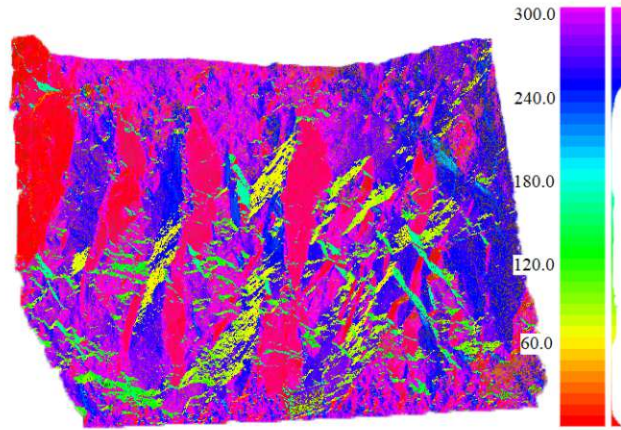


199
200 *Figure 4: Geological map (1:50 000) of the project area Tiefenbach. The red star marks the location of the rock face. The*
201 *geological unit is the so-called Weinsberger Granite (38): coarse-grained biotite granite with porphyritic large potassium*
202 *feldspar (Mississippium) (Moser and Linner, 2019).*



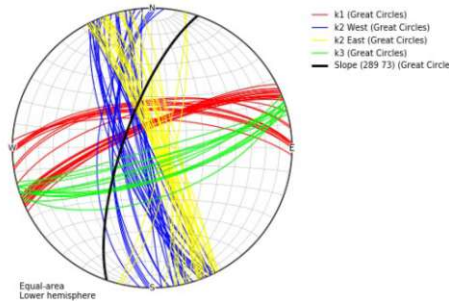
203
204 *Figure 5: 3D photogrammetry model (Agisoft, 2021) of the Tiefenbach rock face. The structural geological analysis was*
205 *limited to the marked area (Red).*

206 The 3D photogrammetry model of the Tiefenbach rock face is shown in Figure 5. We limited the structural
 207 geological analysis of the Tiefenbach rock face to the marked area in Red, because of the low vegetation outcrop.
 208 The investigated rock face has a length of about 38 m, a slope height of 20-25 m and an average inclination of 73°.
 209 Figure 6 shows the 3D point cloud model of the selected Tiefenbach rock face coloured according to dip direction.



210
 211 *Figure 6: 3D point cloud model (Cloudcompare, 2020) of the selected Tiefenbach rock face, coloured according to dip*
 212 *direction (degrees): k1 (Red), k2 (Blue/Yellow), k3 (Green).*

213 The analysis via CloudCompare resulted in 102 joint orientations (Helm, 2023). We grouped them into three joint
 214 sets: k1 (red), k2 (blue and yellow) and k3 (green) with 30, 58 and 14 measured orientations, respectively (Figure
 215 7).



216
 217 *Figure 7: Equal-area lower hemisphere plot of the 102 measured joints (great circles) and the slope (Black) in Tiefenbach,*
 218 *using OpenStereo (Grohmann and Companhia, 2017); for the density plot refer to Figure 9 (left).*

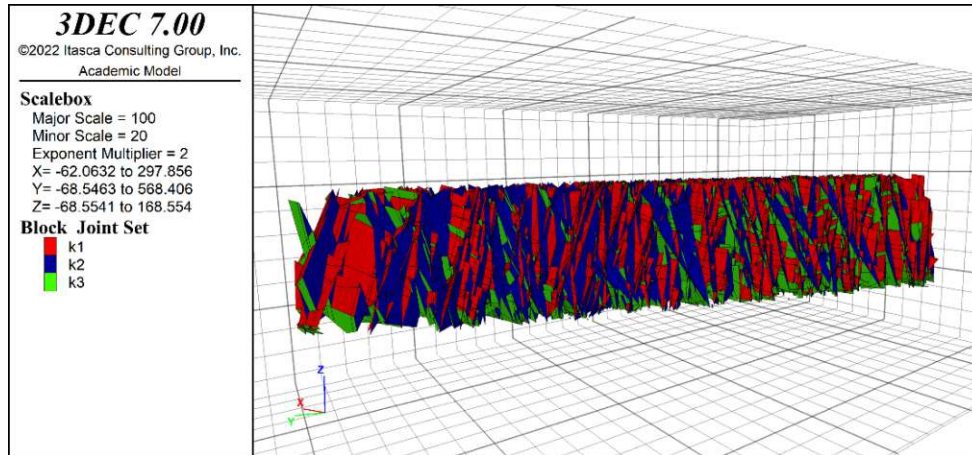
219 We used all measured joint orientation to generate the DFN. They provide a sufficiently large sample for
 220 bootstrapping. Table 1 lists the mean dip direction and dip, and the determined joint densities for each joint set.

221 *Table 1: Tiefenbach DFN data: joint directions and joint density*

Joint set	Dip dir. (mean)	Dip (mean)	Joint density
k1	337	76	0.20
k2	069	87	0.42
k3	162	77	0.30

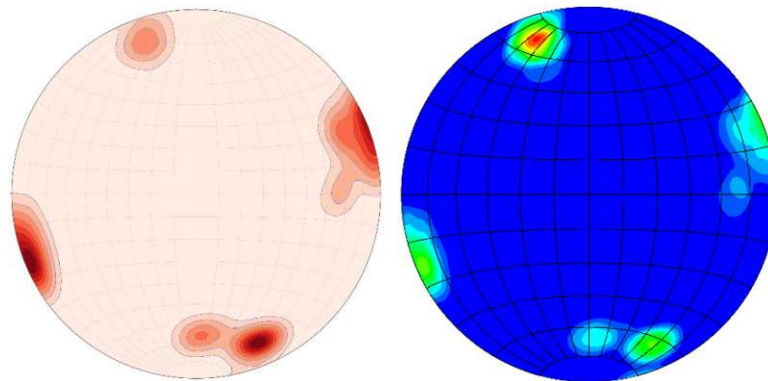
222
 223 The dimensions (x y z) of the model domain are approx. 230 x 500 x 100 m. We cut the model domain by two
 224 planes parallel to the slope (289/73) with a 50 m distance. The resulting 3D volume model of the slope is approx.

225 500 m long, 50 m deep (into the rock mass) and 100 m high. Figure 8 shows the joint sets of the rock mass slope
 226 in Tiefenbach, where the y-axis points North.



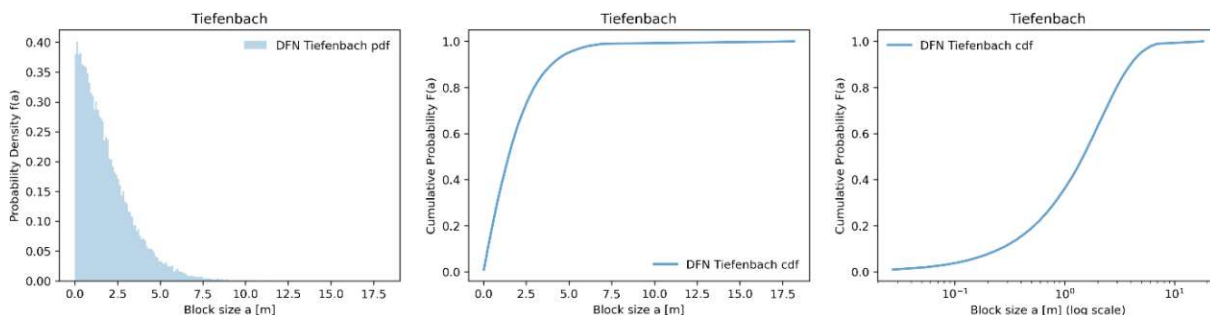
227
 228 *Figure 8: 3DEC SRM model of Tiefenbach showing the block joint sets (y-axis = North).*

229 Figure 9 compares the equal-area lower hemisphere density plots of the 102 measured joint poles (left; using
 230 OpenStereo (Grohmann and Companhia, 2017)) and the 94 created joint poles in the SRM model (right) for
 231 Tiefenbach. They correspond very well.



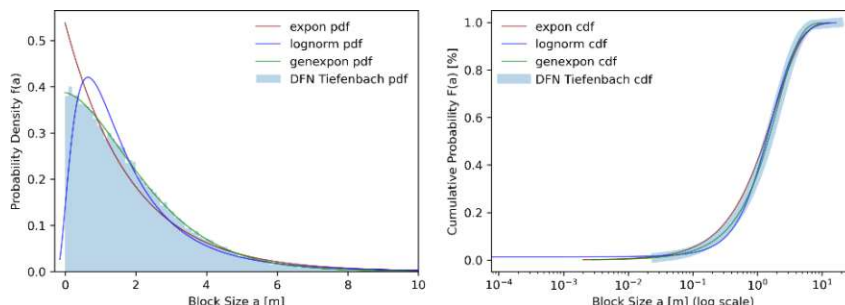
232
 233 *Figure 9: Equal-area lower hemisphere density plots for Tiefenbach; Left: of the 102 measured joint poles (using*
 234 *OpenStereo; max ~337/76 and ~069/87); Right: of the 407 created joint poles in the SRM model (using 3DEC; max.*
 235 *~162/77) for Tiefenbach.*

236 We derived 60,503 block volumes from the Tiefenbach SRM model. Assuming the block shape cuboid, we
 237 calculated their edge lengths by taking the cube root. The IBSD [m] of the Tiefenbach slope is plotted in Figure
 238 10.



239
 240 *Figure 10: IBSD of the Tiefenbach slope with 60,503 blocks (edge length a in [m]).*

241 Our check against 79 different distribution functions with Python found that neither the exponential nor the
 242 lognormal distribution fits well. Rather, the generalized exponential distribution function fits best. We show this
 243 both graphically in Figure 11 and numerically using the KS test and the mean squared error (MSE) method (as
 244 described above).



245
246
247 *Figure 11: IBSD of the Tiefenbach slope [m] (blue bars) with the fitted exponential (maroon), lognormal (blue) and generalized exponential (green) probability density functions (left: pdfs, right: cdfs).*

248 The fitted parameters of the three tested continuous distribution functions are listed in Table 2 and the KS test and
 249 MSE results are listed in Table 3.

250 *Table 2: Fitting parameters of the fitted distribution functions; SRM Tiefenbach*

Fitting parameters	expon	lognorm	genexpon
loc	1.51e-04	-3.19e-01	1.51e-04
scale	1.86	1.68	2.48
shape par. 1	-	s = 0.74	a = 0.96
shape par. 2	-	-	b = 1.14
shape par. 3	-	-	c = 0.73

251 *Table 3: Computational test results of the KS tests and MSE method; SRM Tiefenbach*

test results	expon	lognorm	genexpon
KS: p	7.14e-159	1.39e-72	0.58
KS: D	5.49e-02	3.70e-02	3.16e-03
MSE	26.36	51.30	11.92

252 *Table 4: Comparison of Quantiles for the SRM Tiefenbach*

Quantile	DFN [m ³]	DFN [m]	expon [m]	lognorm [m]	genexpon [m]
0	3.46e-12	1.51e-04	1.51e-04	-3.19e-01	1.51e-04
25	2.95e-01	0.67	0.53	0.70	0.67
50	3.19	1.47	1.29	1.36	1.46
75	18.13	2.63	2.58	2.46	2.63
95	119.53	4.93	5.57	5.40	4.94
96	144.90	5.25	5.98	5.87	5.24
97	179.02	5.64	6.52	6.50	5.62
98	228.30	6.11	7.27	7.44	6.15
99	340.97	6.99	8.56	9.18	7.04
100	5,973.04	18.14	inf	inf	inf

254 With a p-value near 0.00 we reject the null hypothesis in favour of concluding that the sample does not follow the
255 reference distributions. This is the case for the exponential (expon) and the lognormal (lognorm) distribution. With
256 a p-value of 0.58 (> 0.05) we cannot reject the null hypothesis. This indicates that there is no sufficient evidence
257 to claim a difference between the sample and the generalized exponential (genexpon) reference distribution.

258 Comparing the MSE values for the expon, lognorm and genexpon distribution functions, the genexpon distribution
259 function has the lowest MSE value, indicating its best fit. A comparison of the quantiles is listed in Table 4.

260 3.2 Spitz

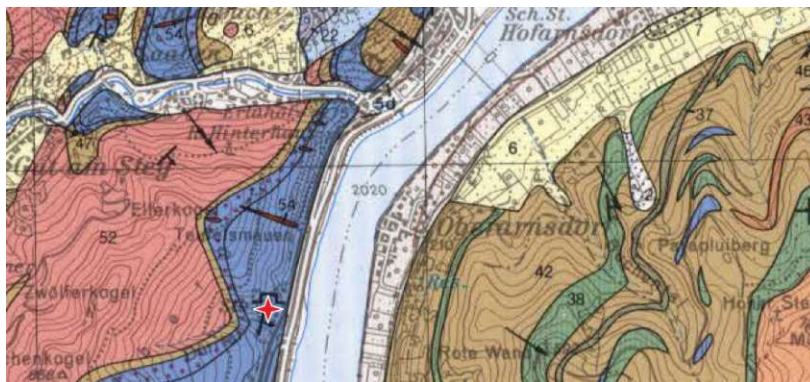
261 The rock face in Spitz (Figure 12) is a former marble quarry in the middle of Lower Austria, West of the state road
262 Donau Straße (B 3) next to and along the Danube. The geological unit (Figure 13) is marble, ribbon marble and
263 silicate marble (46) (Fuchs et al., 1983).



264
265 *Figure 12: Photograph of the Spitz rock face*

266 The 3D photogrammetry model of the Spitz rock face is shown in Figure 14. The investigated rock face has a
267 length of about 70 m, a slope height of up to 30 m and an average inclination of 84° .

268 Figure 15 shows the 3D point cloud model of the selected Spitz rock face coloured according to dip direction.



269
270 *Figure 13: Geological map (1:50 000) of the project area Spitz. The red star marks the location of the rock face. The*
271 *geological unit is marble, ribbon marble and silicate marble (46) (Fuchs et al., 1983).*



Figure 14: 3D photogrammetry model (Agisoft, 2021) of the Spitz rock face.

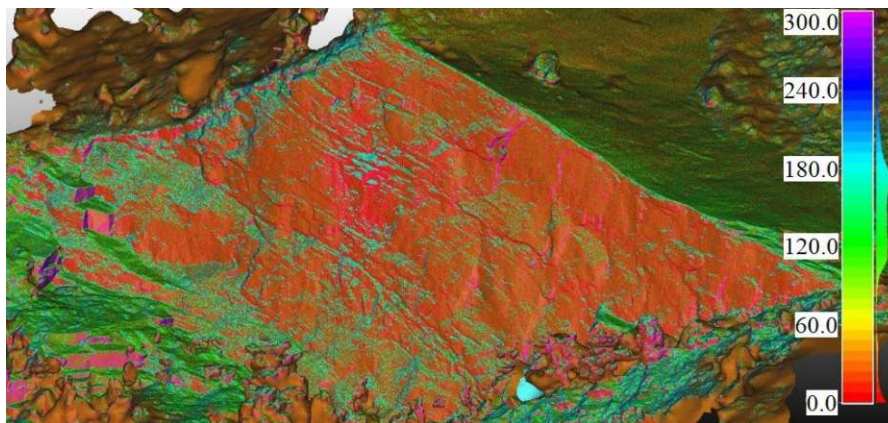


Figure 15: 3D point cloud model of the selected Spitz rock face, coloured according to dip direction (degrees): SE (Green), N/S (Red/Cyan), NW (Purple).

The analysis via CloudCompare resulted in three main joint sets forming the rock face: SE, N and NW with 132, 55 and 21 (total 208) measured orientations, respectively (Figure 16).

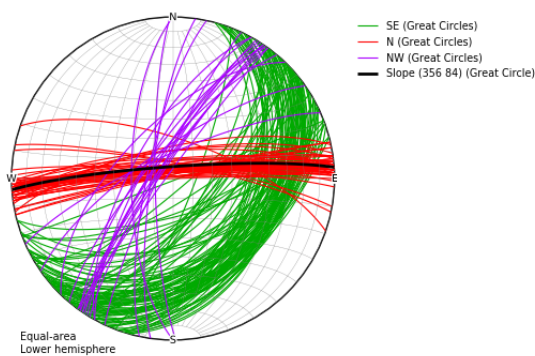


Figure 16: Equal-area lower hemisphere plot of the 208 measured joints (great circles) and the slope (Black) in Spitz, using OpenStereo; for the density plot refer to Figure 18 (left).

To generate the DFN, all measured joint orientations are used to provide a sufficient large sample for bootstrapping. Table 5 lists the mean dip direction and dip, as well as the determined mass densities and joint size for each joint set.

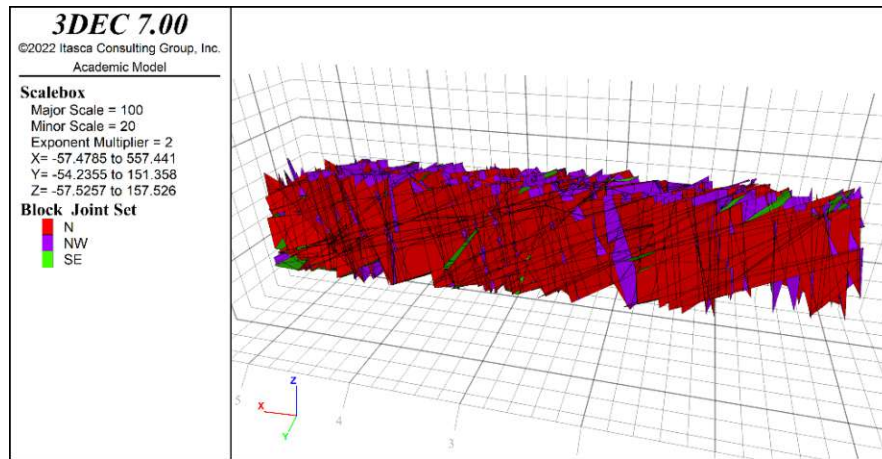
285

Table 5: Spitz DFN data: joint directions and joint density

Joint set	Dip dir. (mean)	Dip (mean)	Joint density
SE	135	40	0.23
N	356	85	0.18
NW	303	80	0.28

286

287 The dimensions (x y z) of the model domain are approx. 500 x 100 x 100 m. We cut the model domain by two
 288 planes parallel to the slope (356/84) with a 50 m distance. The resulting 3D volume model of the slope is approx.
 289 500 m long, 50 m deep (into the rock mass) and 100 m high. Figure 17 shows the joint sets of the rock mass slope
 290 in Spitz, where the y-axis points North.

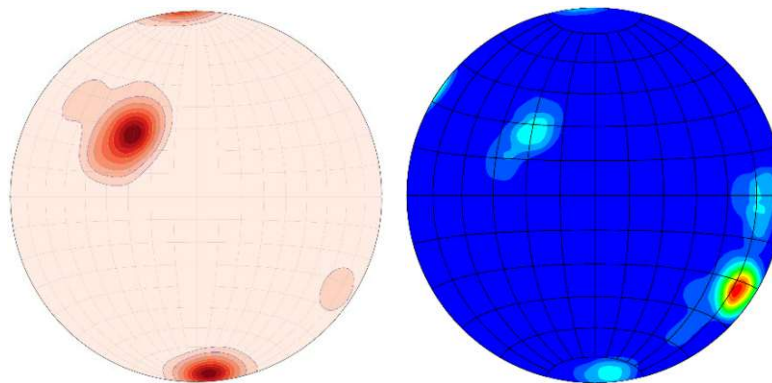


291

Figure 17: 3DEC SRM model of Spitz showing the block joint sets (y-axis = North).

292

293 Figure 18 compares the equal-area lower hemisphere density plots of the 208 measured joint poles (left) and the
 294 174 created joint poles in the SRM model (right) for Spitz. They correspond very well.



295

296 Figure 18: Equal-area lower hemisphere density plots for Spitz; Left: of the 208 measured joint poles (using OpenStereo;
 297 max ~356/85 and ~135/40); Right: of the 185 created joint poles in the SRM model (using 3DEC; max. ~303/80).

298

299 We derived 26,253 block volumes from the Spitz SRM model. Assuming the block shape cuboid, we calculated
 their edge lengths by taking the cube root. The IBSD [m] of the Spitz slope is plotted in Figure 19.

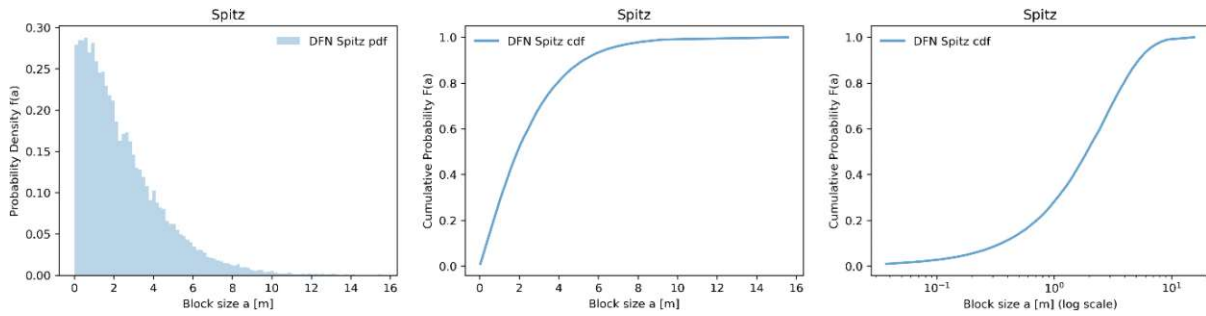


Figure 19: IBSD of the Spitz slope with 26,253 blocks (edge length a in [m]).

Our check against 79 different distribution functions with Python found that neither the exponential nor the lognormal distribution fits well. Rather, the generalized exponential distribution function fits best. We show this both graphically in Figure 20 and numerically using the KS test and the mean squared error (MSE) method (as described above).

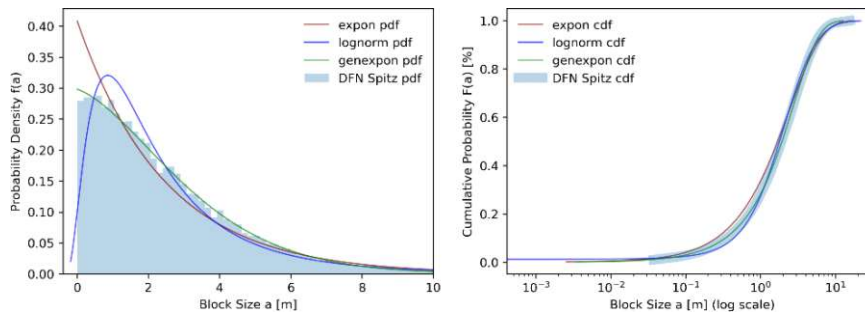


Figure 20: IBSD of the Spitz slope [m] (blue bars) with the fitted exponential (maroon), lognormal (blue) and generalized exponential (green) probability density functions (left: pdfs, right: cdfs).

The fitted parameters of the three tested continuous distribution functions are listed in Table 6 and the KS test and MSE results are listed in Table 7.

Table 6: Fitting parameters of the fitted distribution functions; SRM Spitz

Fitting parameters	expon	lognorm	genexpon
loc	9.45e-05	-4.07e-01	9.45e-05
scale	2.45	2.20	2.20
shape par. 1	-	s = 0.75	a = 0.66
shape par. 2	-	-	b = 1.01
shape par. 3	-	-	c = 0.32

Table 7: Computational test results of the KS tests and MSE method; SRM Spitz

KS test results	expon	lognorm	genexpon
p	1.33e-69	5.64e-33	0.16
D	5.51e-02	3.78e-02	6.95e-03
MSE	45.80	89.08	18.60

With a p-value near 0.00 we reject the null hypothesis in favour of concluding that the sample does not follow the reference distributions. This is the case for the exponential (expon) and the lognormal (lognorm) distribution. With

316 a p-value of 0.16 (> 0.05) we cannot reject the null hypothesis. This indicates that there is no sufficient evidence
 317 to claim a difference between the sample and the generalized exponential (genexpon) reference distribution.

318 *Table 8: Comparison of quantiles for the SRM Spitz*

Quantile	DFN [m ³]	DFN [m]	expon [m]	lognorm [m]	genexpon [m]
0	8.45e-13	9.45e-05	9.45e-05	-4.07e-01	9.45e-05
25	0.70	0.89	0.70	0.93	0.88
50	6.94	1.91	1.70	1.80	1.94
75	41.21	3.45	3.40	3.24	3.48
95	275.69	6.51	7.34	7.11	6.49
96	332.99	6.93	7.88	7.73	6.87
97	411.60	7.44	8.59	8.55	7.35
98	544.03	8.16	9.58	9.79	8.02
99	792.58	9.25	11.28	12.09	9.13
100	3,776.36	15.57	inf	inf	inf

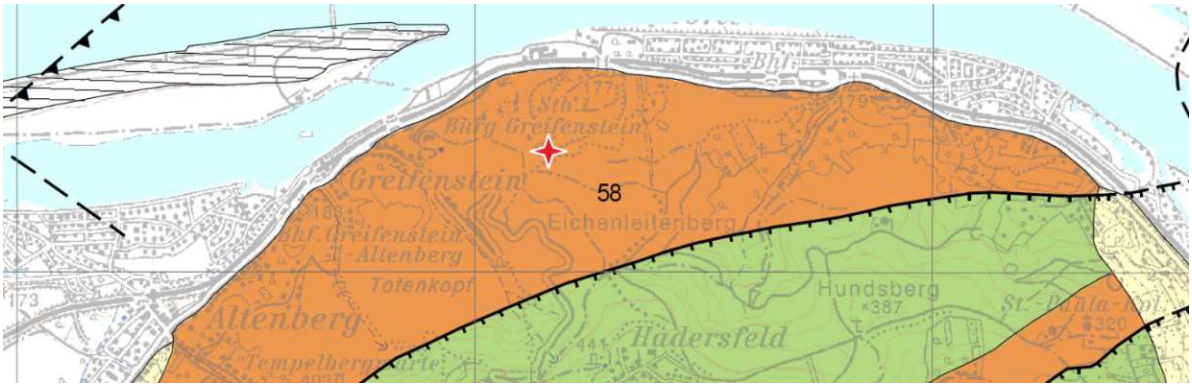
319
 320 Comparing the MSE values for the expon, lognorm and genexpon distribution functions, the genexpon distribution
 321 function has the lowest MSE value, indicating its best fit. A comparison of the quantiles is listed in Table 8.

322 **3.3 Greifenstein**

323 The rock face in Greifenstein is within a former quarry (until 1993) and landfill North of Vienna, also close to the
 324 Danube, located within the Rhenodanubian flysch zone (Figure 21 and Figure 22). This zone extends from Vienna
 325 to Vorarlberg north of the northern Limestone Alps. Basically, the flysch zone was formed by sedimentary
 326 depositional processes of rivers in the sea of that time. Due to the different flow velocities, sedimentary layers of
 327 different thickness were formed. About 42 million years ago the ‘Eurasian Plate’ and the ‘Adriatic Plate’ collided
 328 and the flysch zone came to the surface (Egger and Coric, 2017). The easternmost part of this Rhenodanubian
 329 flysch zone is called the Greifenstein Formation (58): fine- to coarse-grained, medium- to thick-bedded,
 330 siliciclastic sandstone and clay shale (Ypresium). (Kreuss, 2020)



331 *Figure 21: Photograph of the Greifenstein rock face*



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Figure 22: Geological map (1:50 000) of the project area Greifenstein. The red star marks the location of the rock face. The geological unit is the so-called Greifenstein Formation (58): fine- to coarse-grained, medium- to thick-bedded, siliciclastic sandstone and clay shale (Ypresium). (Kreuss, 2020)



337

338

Figure 23: 3D photogrammetry model (Agisoft, 2021) of the Greifenstein rock face.

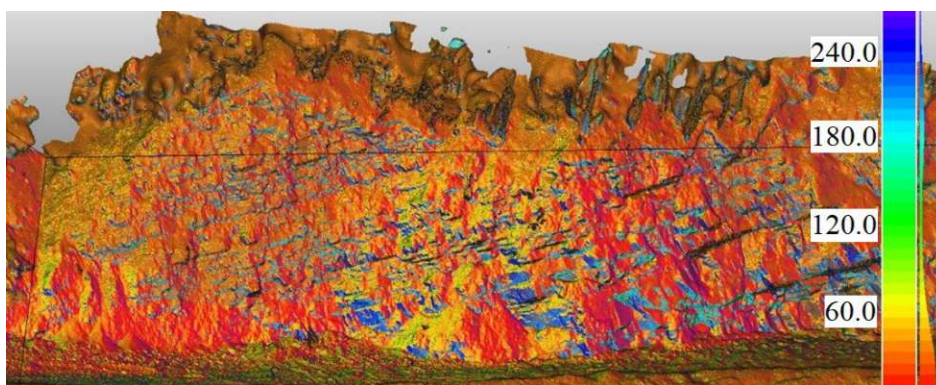
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341

The 3D photogrammetry model of the Greifenstein rock face is shown in Figure 23. The investigated rock face has a length of about 80 m, a slope height of 20-27 m and an average inclination of 70°. (Wiesinger, 2023)

Figure 24 shows the 3D point cloud model of the selected Spitz rock face coloured according to dip direction.



342

343

344

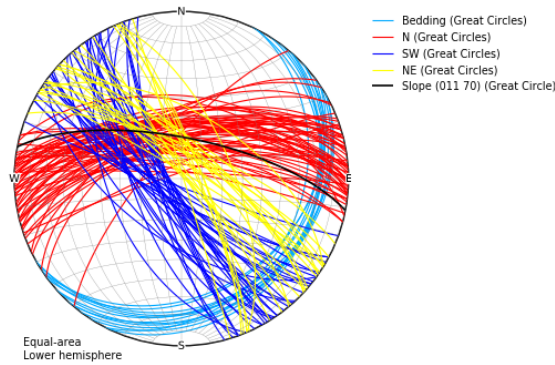
Figure 24: 3D point cloud model of the selected Greifenstein rock face, coloured according to dip direction (degrees): Bedding (Light-Blue), N (Red), SW/NE (Blue/Yellow).

345

346

347

The structural geological analysis of the Greifenstein 3D rock face model resulted in 143 joint orientations, which could be grouped into three joint sets: the bedding, the Northeast-Southwest joint set and the North joint set (Figure 25). (Wiesinger, 2023)



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349

350

Figure 25: Equal-area lower hemisphere plot of the measured joints (great circles) and the slope (Black) in Greifenstein, using OpenStereo; for the density plot refer to Figure 27 (left).

351

352

353

To generate the DFN, all measured joint orientations are used to provide a sufficient large sample for bootstrapping. Table 9 lists the mean dip direction and dip, as well as the determined mass densities and joint size for each joint set.

354

Table 9: Greifenstein DFN data: joint directions and joint density

Joint set	Dip dir. (mean)	Dip (mean)	Joint density
Bedding	135	23	2.38
N	357	65	0.95
SW-NE	236	82	0.95

355

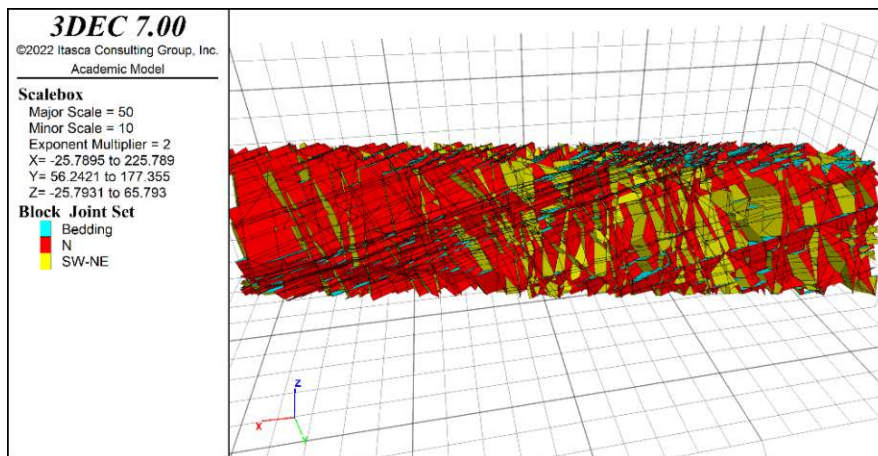
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The dimensions (x y z) of the model domain are approx. 200 x 160 x 40 m. We cut the model domain by two planes parallel to the slope (011/70) with a 20 m distance. The resulting 3D volume model of the slope is approx. 200 m long, 20 m deep (into the rock mass) and 40 m high. Figure 26 shows the joint sets of the rock mass slope in Greifenstein, where the y-axis points North.



360

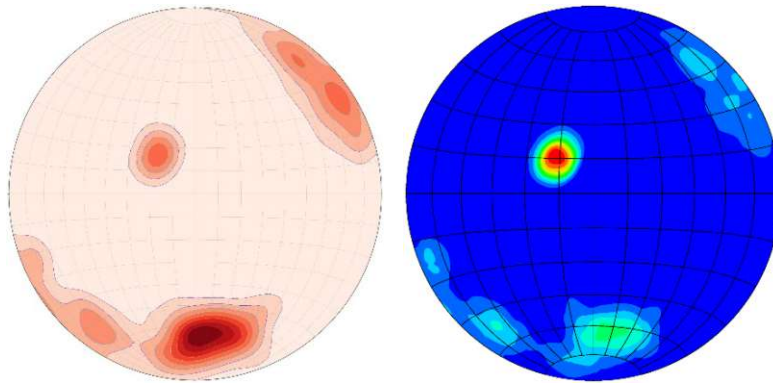
361

Figure 26: 3DEC SRM model of Greifenstein showing the block joint sets (y-axis = North).

362

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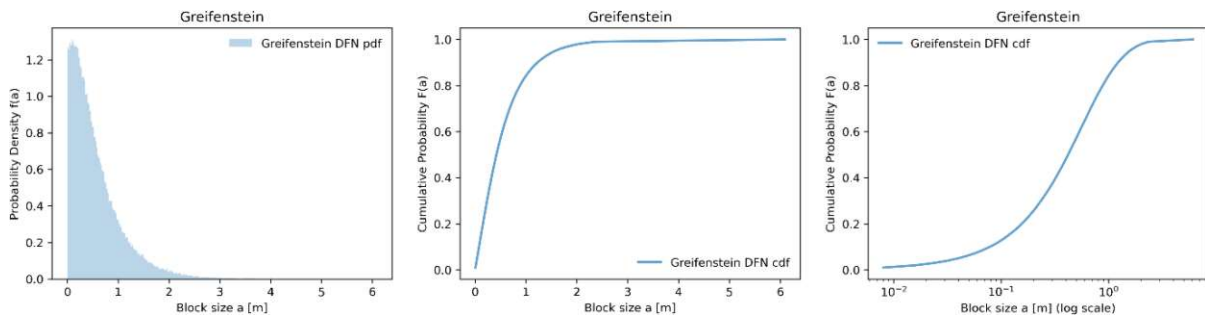
Figure 27 compares the equal-area lower hemisphere density plots of the 143 measured joint poles (left) and the 381 created joint poles in the SRM model (right) for Greifenstein. They correspond very well.



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Figure 27: Equal-area lower hemisphere density plots for Greifenstein; Left: of the 143 measured joint poles (using OpenStereo; max. $\sim 357/65$); Right: of the 381 created joint poles in the SRM model (using 3DEC; max. $\sim 135/23$).

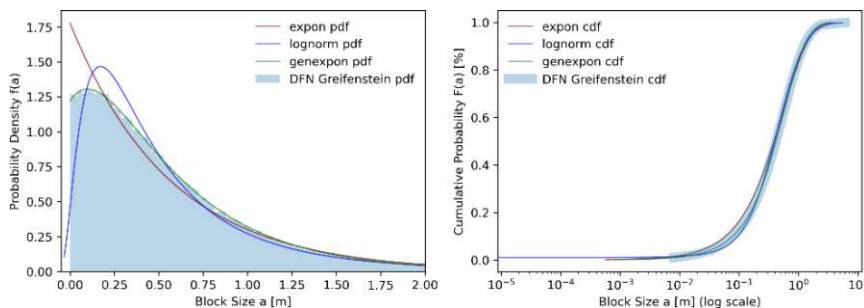
367 We derived 143,029 block volumes from the Greifenstein SRM model. Assuming the block shape cuboid, we
 368 calculated their edge lengths by taking the cube root. The IBSD [m] of the Greifenstein slope is plotted in Figure
 369 28.



370
371

Figure 28: IBSD of the Greifenstein slope with 143,029 blocks (edge length a in [m]).

372 Our check against 79 different distribution functions with Python found that neither the exponential nor the
 373 lognormal distribution fits well. Rather, the generalized exponential distribution function fits best. We show this
 374 both graphically in Figure 29 and numerically using the KS test and the mean squared error (MSE) method (as
 375 described above).



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Figure 29: IBSD of the Greifenstein slope [m] (blue bars) with the fitted exponential (maroon), lognormal (blue) and generalized exponential (green) probability density functions (left: pdfs, right: cdfs).

379 The fitted parameters of the three tested continuous distribution functions are listed in Table 10 and the KS test
 380 and MSE results are listed in

381 With a p-value near 0.00 we reject the null hypothesis in favour of concluding that the sample does not follow the
 382 reference distributions. This is the case for the exponential (expon) and the lognormal (lognorm) distribution. With

383 a p-value of 0.58 (> 0.71) we cannot reject the null hypothesis. This indicates that there is no sufficient evidence
 384 to claim a difference between the sample and the generalized exponential (genexpon) reference distribution.

385 *Table 10: Fitting parameters of the fitted distribution functions; SRM Greifenstein*

Fitting parameters	expon	lognorm	genexpon
loc	1.34e-05	-7.06e-02	1.34e-05
scale	0.56	0.46	0.71
shape par. 1	-	s = 0.81	a = 0.87
shape par. 2	-	-	b = 0.58
shape par. 3	-	-	c = 2.97

386
 387 Comparing the MSE values for the expon, lognorm and genexpon distribution functions, the genexpon distribution
 388 function has the lowest MSE value, indicating its best fit. A comparison of the quantiles is listed in Table 12.

389 *Table 11: Computational test results of the KS tests and MSE method; SRM Greifenstein*

KS test results	expon	lognorm	genexpon
p	1.66e-239	6.13e-99	0.68
D	4.39e-02	2.82e-02	1.90e-03
MSE	2.41	6.38	1.78

390 *Table 12: Comparison of quantiles for the SRM Greifenstein*

Quantile	DFN [m ³]	DFN [m]	expon [m]	lognorm [m]	genexpon [m]
0	2.42e-15	1.34e-05	1.34e-05	-0.07	1.34e-05
25	7.45e-03	0.20	0.16	0.20	0.19
50	7.45e-02	0.42	0.39	0.40	0.42
75	0.46	0.77	0.78	0.73	0.77
95	3.87	1.57	1.68	1.69	1.57
96	4.77	1.68	1.81	1.85	1.68
97	6.14	1.83	1.97	2.06	1.82
98	8.36	2.03	2.20	2.38	2.02
99	13.11	2.36	2.59	2.99	2.36
100	225.20	6.08	inf	inf	inf

391
 392 Additionally, we have examined whether there is a difference between the block size distribution derived from a
 393 rock slope and a block size distribution derived from the rock mass (i.e., a cube) in the example of Greifenstein.

394 The dimensions (x y z) of the model domain are approx. 60 x 60 x 60 m. Figure 30 shows the joint sets of the rock
 395 mass cube in Greifenstein, where the y-axis points North.

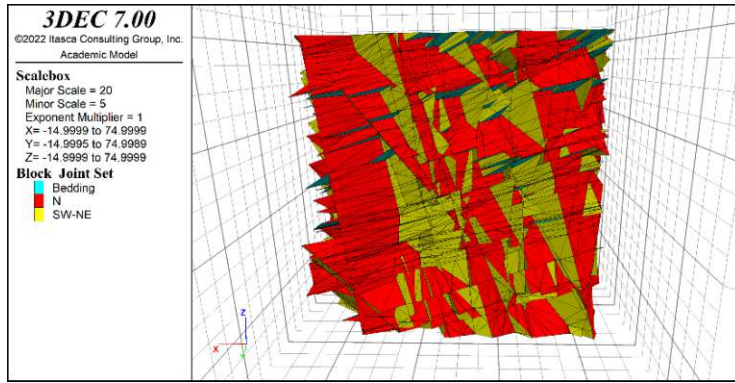


Figure 30: 3DEC SRM model of the Greifenstein 60 m cube showing the block joint sets (y-axis = North).

134,711 block volumes were derived from the SRM model and assuming the block shape cuboid, their edge lengths were calculated taking the cube root. The distributions of the block sizes are plotted in Figure 31.

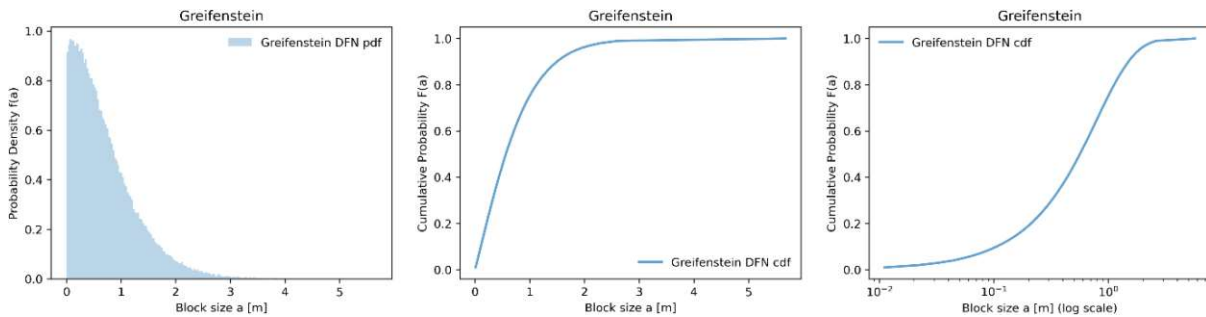


Figure 31: IBSD of Greifenstein 60 m cube with 134,711 blocks (edge length a in [m]).

Again, neither the exponential nor lognormal distributions fit well. Rather, the generalized exponential distribution function fits best. We show this both graphically in Figure 32 and numerically using the KS test and the mean squared error (MSE) method (as described above).

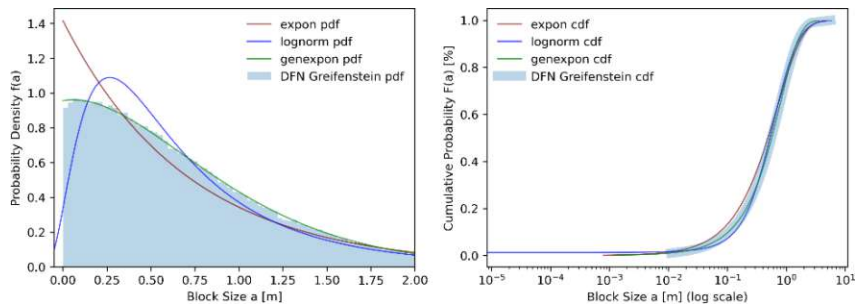


Figure 32: IBSD of the Greifenstein 60 m cube [m] (blue bars) with the fitted exponential (maroon), lognormal (blue) and generalized exponential (green) probability density functions (left: pdfs, right: cdfs).

The fitted parameters of the three tested continuous distribution functions are listed in Table 13 and the KS test and MSE results are listed in Table 14.

With a p-value near 0.00 we reject the null hypothesis in favour of concluding that the sample does not follow the reference distributions. This is the case for the exponential (expon) and the lognormal (lognorm) distribution. With a p-value of 0.09 (> 0.05) we cannot reject the null hypothesis. This indicates that there is no sufficient evidence to claim a difference between the sample and the generalized exponential (genexpon) reference distribution.

414

Table 13: Fitting parameters of the fitted distribution functions; SRM Greifenstein 60 m cube

Fitting parameters	expon	lognorm	genexpon
loc	8.55e-05	-1.34e-01	8.55e-05
scale	0.71	0.66	0.96
shape par. 1	-	s = 0.71	a = 0.92
shape par. 2	-	-	b = 1.48
shape par. 3	-	-	c = 0.66

415

Table 14: Computational test results of the KS tests and MSE method; SRM Greifenstein 60 m cube

test results	expon	lognorm	genexpon
KS: p	0.00	4.03e-133	0.09
KS: D	6.42e-02	3.37e-02	3.38e-03
MSE	3.82	6.15	1.48

416

Table 15: Comparison of quantiles for the SRM Greifenstein cube 60 m

Quantile	DFN [m ³]	DFN [m]	expon [m]	lognorm [m]	genexpon [m]
0	6.25e-13	8.55e-05	8.55e-05	-1.34e-01	8.55e-05
25	0.02	0.26	0.20	0.28	0.26
50	0.18	0.56	0.49	0.53	0.57
75	0.99	1.00	0.98	0.94	1.00
95	6.32	1.85	2.12	2.00	1.84
96	7.47	1.96	2.27	2.17	1.95
97	9.28	2.10	2.48	2.39	2.08
98	12.05	2.29	2.76	2.72	2.27
99	17.84	2.61	3.25	3.33	2.58
100	182.72	5.67	inf	inf	inf

417

4 Discussion

418 The equal-area density plots proof that it is possible to represent joint systems using a DFN. The joint orientations
 419 of the left and right density plots correspond very well (see Figure 9, Figure 18, Figure 27). The pole densities of
 420 the measured joint planes (left plots) represent joints exposed on the rock face. So, wall-building joints may be
 421 measured more frequently than other joints. This can lead to a situation where joint surfaces of lower joint density
 422 (with greater joint distances) are measured more frequently on the rock face, and vice versa. Thus, measurements
 423 from the rock face may be distorted. The SRM density plots (on the right) reflect the ‘true’ joint densities of the
 424 joint sets (relative to each other). For example, in Greifenstein the joint set N (357/65, Figure 24, red) is dominating
 425 on the rock face and measured more frequently than the bedding (135/23, Figure 24, light blue). The surface areas
 426 (outcrops) of the bedding are very small (see Figure 27, left). Yet, the bedding has a much higher joint density
 427 than the other two joint sets. This is reflected in the density plot in Figure 27 on the right. We can conclude that
 428 the density plot of the SRM is more realistic and more meaningful than the density plot of the measured joints on
 429 the rock face.

430
 431 We chose the slope and cube dimensions of the SRM models to create many blocks for our investigations. In
 432 Tiefenbach, a slope size of approximately 500 x 50 x 100 m generated 60,503 block volumes. In Spitz, a similar

433 slope size generated 26,253 block volumes. In Greifenstein, with a bedding of relatively high joint density, a slope
 434 size of approximately 200 x 20 x 40 m generated 143,029 block volumes, and the Greifenstein 60 m cube generated
 435 134,711 block volumes. The size of the model should correspond to the homogeneous area investigated.

436 We show that neither the lognormal nor the exponential distribution functions describe IBSDs [m] well. To fit
 437 best, the lognormal is shifted with a negative loc parameter. This results in negative block volumes. Both, the
 438 lognorm and expon have relatively long tails. This results in much larger blocks compared to the IBSDs. We also
 439 tried to fit Python's power law distribution in the form $f(x,a) = ax^{-1}$ to our derived IBSDs (both in [m³] and in
 440 [m]). No correlations could be found. The generalized exponential distribution function best describes block size
 441 distributions [m] across three various lithologies when compared to 78 other distribution functions via the one-
 442 sample KS test and the MSE method.

443 The probability density function (pdf) for genexpon is:

$$f(x, a, b, c) = \left(a + b(1 - \exp(-cx)) \right) \exp \left(-ax - bx + \frac{b}{c}(1 - \exp(-cx)) \right) \quad (3)$$

444 for $x \geq 0, a, b, c > 0$, where x is the random variable and a, b , and c are shape parameters. The probability density
 445 above is defined in the 'standardized' form. To shift and/or scale the distribution the loc and scale parameters are
 446 used. Specifically, $genexpon.pdf(x, a, b, c, loc, scale)$ is identically equivalent to $\frac{expon.pdf(y,a,b,c)}{scale}$ with $y =$
 447 $\frac{x-loc}{scale}$.

448 Above generalized exponential distribution is an extension of Marshall and Olkin's bivariate exponential
 449 distribution (Ryu, 1993). The three shape parameters provide quite a bit of flexibility for analysing any skewed
 450 dataset. Table 16 lists the genexpon fitting parameters for the different investigated slope and cube sizes. The
 451 location parameters are essentially zero.

452 *Table 16: Genexpon fitting parameters for the investigated locations Tiefenbach (slope size 500 x 50 x 100 m),*
 453 *Spitz (slope size 500 x 50 x 100 m) and Greifenstein (slope size 200 x 20 x 40 m and 60 m cube).*

genexpon fitting parameters	SRM Tiefenbach	SRM Spitz	SRM Greifenstein	
	500x50x100	500x50x100	200x20x40	cube 60 m
loc	1.51e-04	9.45e-05	1.34e-05	8.55e-05
scale	2.48	2.20	0.71	0.96
shape par. 1	a = 0.96	a = 0.66	a = 0.87	a = 0.92
shape par. 2	b = 1.14	b = 1.01	b = 0.58	b = 1.48
shape par. 3	c = 0.73	c = 0.32	c = 2.97	c = 0.66

454
 455 For the Greifenstein slope and the Greifenstein cube calculations we used the same DFN data (Table 9). The
 456 models differ in their dimensions and orientation. We cut the slope model parallel to the mean rock face orientation.
 457 The cube model is North-South oriented, independent of the observed rock face orientation. Comparing the results
 458 (Table 16), the genexpon fitting parameters, especially b and c , do vary. It is unclear, how these differences affect
 459 rock fall modelling results. Modelling a cube, independent of the slope direction, would be much more practicable
 460 when investigating a quarry, for example. Further investigations on the sensitivity of the scale and shape
 461 parameters are required.

462 Comparing the quantiles of the Greifenstein slope and the Greifenstein cube models (Table 17), the blocks of the
 463 cube model are generally slightly bigger. Due to the bigger dimensions of the cube in the y-direction (20 m vs 60
 464 m), bigger blocks are built. The maximum block volumes depend on the size of the homogeneous area, as also
 465 recognized by Laimer (2019) and Moos et al. (2021).

466 *Table 17: Comparison of quantiles [m] for different SRM models*

Quantile	Tiefenbach slope		Spitz slope		Greifenstein slope		Greifenstein cube	
	DFN	genexpon	DFN	genexpon	DFN	genexpon	DFN	genexpon
0	1.51e-04	1.51e-04	9.45e-05	9.45e-05	1.34e-05	1.34e-05	8.55e-05	8.55e-05
25	0.67	0.67	0.89	0.88	0.20	0.19	0.26	0.26
50	1.47	1.46	1.91	1.94	0.42	0.42	0.56	0.57
75	2.63	2.63	3.45	3.48	0.77	0.77	1.00	1.00
95	4.93	4.94	6.51	6.49	1.57	1.57	1.85	1.84
96	5.25	5.24	6.93	6.87	1.68	1.68	1.96	1.95
97	5.64	5.62	7.44	7.35	1.83	1.82	2.10	2.08
98	6.11	6.15	8.16	8.02	2.03	2.02	2.29	2.27
99	6.99	7.04	9.25	9.13	2.36	2.36	2.61	2.58
100	18.14	inf	15.57	inf	6.08	inf	5.67	inf

467
 468 The DFN- and genexpon-quantiles correspond very well (see Table 17). Comparing the P98 of the genexpon cdfs,
 469 the block sizes seem plausible, considering the different lithologies and joint systems. In Spitz, where we could
 470 observe huge marble blocks in the deposition area (Figure 33, left), the P98 block edge length in the genexpon cdf
 471 is 8 m (512 m³). In Greifenstein, where we could observe a sandstone bedding of relatively high joint density
 472 (Figure 33, right), the P98 block edge length in the genexpon cdf is approximately 2 m (8 m³). The P98 block edge
 473 length in the genexpon cdf of the Tiefenbach granite lies in between, with approximately 6 m (216 m³).



474
 475 *Figure 33: Photographs of the deposition areas; Left: Spitz with huge marble blocks vs Right: Greifenstein with smaller*
 476 *sandstone blocks due to rather dense bedding.*

477 As already criticized by Laimer (2019), the use of P95-P98 appears too high for rock formations, which form very
 478 large rock fall blocks (> 10 m³). He had sufficient data from the Dachstein Formation (limestones and dolomites)
 479 to show that, for limestone, the return periods of P95 to P96 blocks (0.15-2.25 m³) range from 23.5 to 56.5 years.
 480 This corresponds to the service life of a conventional rock fall protection barrier. P97 and P98 limestone blocks
 481 have return periods of more than 100 years.

482 With our holistic rock mass block surveys of relatively large rock masses (homogeneous areas), the problem of
483 very large (P95-P98) blocks is even more evident. Our SRM approach does not consider whether blocks are
484 kinematically able to fail. IBSDs may include blocks of very high return periods. One approach to deal with this
485 problem could be a kinematic analysis of the blocks using the Block Theory (Goodman and Shi, 1985). However,
486 this approach does not consider return periods. For the design of rock fall protection measures and hazard analyses,
487 rock fall frequencies (magnitude to frequency relations M/F (Corominas et al., 2018)) and return periods play an
488 important role. This requires knowledge of the events on the one hand and defining a worst-case scenario
489 (depending on the protection target) on the other. Our IBSD delivers knowledge of all possible events within a
490 homogeneous area including their frequencies (i.e., magnitude to frequency relations). Based on a defined worst-
491 case scenario, events/block sizes of higher return periods may be neglected (cut off). This could be achieved based
492 on (few) information available, like silent witnesses of estimated age, experience reports from residents or records
493 of past events.

494 **5 Conclusion and Outlook**

495 We could find a probability distribution function, which describes in-situ block size distributions (IBSDs) very
496 well. It may be generally usable for block size distributions (BSDs). Further investigations are required on the
497 sensitivity of the scale and shape parameters.

498 We show that both, the lognormal and the exponential distribution function do not describe IBSDs well enough.

499 Describing IBSDs by probability distribution functions could allow for more certain, accurate, verifiable, holistic,
500 and objective results.

501 With the presented method, it is possible to determine IBSDs based on photogrammetry and SRM models. The
502 investigation of many more sites of different lithologies using this method could result in a catalogue
503 recommending a range of scale and shape parameters for certain lithologies in the future. This requires locations
504 of sufficiently large outcrops with low vegetation.

505 The effect on rockfall modelling should be further investigated, e.g., comparing the runout, kinetic energies, and
506 bounce heights of modelling whole IBSDs vs genexpon pdfs vs design blocks.

507 Utilizing a distribution function together with a catalogue of suitable fitting parameters may offer the benefit of
508 enabling the determination of an IBSD and, consequently, a meaningful assessment of a design block, even with
509 only a limited number of block size measurements (silent witnesses).

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554

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559 **Competing Interests**

560 The authors have no relevant financial or non-financial interests to disclose.

561 **Author Contributions**

562 Conceptualization: Mariella Illeditsch and Alexander Preh; Methodology: Mariella Illeditsch and Alexander Preh;
563 Formal analysis and investigation: Mariella Illeditsch; Writing - original draft preparation: Mariella Illeditsch;
564 Writing - review and editing: Mariella Illeditsch and Alexander Preh; Supervision: Alexander Preh.