Chapter 2
State of the Art

2.1 Characterisation of Rockfalls

According to the detachment process landslides can be divided in 5 the process-groups: slide, flow, drift, topple, and fall. Each of these process types can be combined with the type of material: rock, debris, soil or mud. This PhD thesis focuses the process of rockfalls, which implies the detachment of a rock at a steep slope surface (Cruden and Varnes 1996). The detached rock material descends the slope in the possible modes of motion namely falling, bouncing, rolling or sliding.

During the free fall two different modes of falling can occur: either the block rotates around its centre of mass or a translation of the centre of the block takes place (Azzoni et al. 1995: 712). If the slopes inclination angle falls below 76 degrees, falling transforms to bouncing. During the bouncing process the tangential and normal coefficients of restitution and the angle between the block-trajectory and the slope would affect the rebound behaviour most (Hungr and Evans 1988 in Dorren 2003). Below an inclination angle of 45° the block would continue rolling (Ritchie 1963 in Dorren 2003: 72). The velocity of the process "fall" can be classified as very rapid to extremely rapid, meaning 0.3 m/min to >3 m/s (Cruden and Varnes 1996).

Due to rock volume we can further classify the process of rockfall to debris fall (<10 m^3), boulder fall (10–100 m^3), block fall (>100 m^3), cliff fall (10^4–10^6 m^3) and Bergsturz (>10^6 m^3) increasing the volume (Whalley 1984: 218).

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2.2 Towards Characterizing Rock Mass: Influencing Parameters and Classifications

The formation of cracks and later rock joints depends on the material properties of the rock, especially on the materials strength and on the induced stresses as well as their direction. In case of failure, the induced stresses exceed the materials strength.

Discontinuities represent interstices in the rock mass along which no tensile stress can be transferred (Priest 1993: 5). Different kinds of discontinuities can be divided referring to their origin or geological history (Priest 1993: 10 ff.). Faults develop through induced tectonically shear stress which exceeds the shear strength of the rock material along a particular plane (Kersten 1990 in Priest 1993). Faults in general are often grouped in sets, so called fault zones. A characteristic feature would be the core of a fault zone consisting of powdered rock (fault gouge or fault breccia), which is surrounded by the disintegrated rock mass. Joints represent fractures in a rock mass along which little or no displacement has occurred (Price 1966 in Priest 1993: 12). The joint frequency and occurrence is mostly controlled by lithology and bed thickness. This type of discontinuity can be divided in two groups: systematic joints, which mostly run in planar and at least sub-parallel joint sets and non-systematic joints, which cannot be grouped together in a sense of joint sets. Bedding planes are characterized by physical or chemical changes in the material during the deposition, for example grain size or dolomite content. These changes can, but do not have to lead to parallel or sub-parallel fractures drawing through the rock mass. The cleavage can be separated into two types: the fracture cleavage and the flow cleavage. The fracture cleavage is not connected to any kind of parallel aligned minerals and describes cemented parallel fractures in a rock mass. The flow cleavage is dependent on recrystallization of minerals and a parallel alignment of metamorphic minerals in a typical sense, like mica, leading to the classical type of foliation.

The discontinuities generally contribute to an increasing destabilisation of the rock mass meaning that the strength of a rock mass disintegrated through joints would always be less than the materials strength.

The Characterisation of Joints

The discontinuity pattern in a rock mass can be described in terms of the following criteria: the orientation, the persistence, the spacing and the surface conditions (Fig. 2.1). The mentioned objectives can be quantitatively recorded in the field due to scanline analysis using a fact sheet (Priest 1993: 33, Norrish and Wyllie 1996: 391). It is recommendable to additionally record the aperture width and the discontinuity filling, especially for geotechnical purposes (ISRM 1978: 351 ff.). The different parameters of discontinuities determine the degree of disintegration, the joint properties and subsequently the potential of failure. The objectives of crack development and the shear abilities of rock joints are described in Sect. 2.2.2 in more detail.

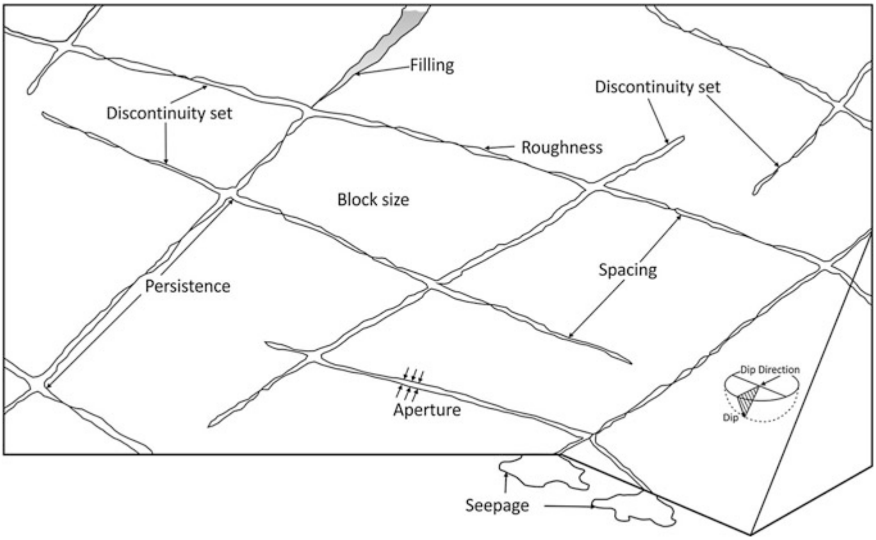

Fig. 2.1 Illustration of the joint parameters influencing the rock mass (in Hudson and Harrison 1997, p. 116)

The orientation of a discontinuity is usually measured as dip and dip-direction and can be visualized in a stereographic projection, for example in a Schmidt Net. By use of a Schmidt Net the spatial distribution of the discontinuities can be visualized in relation to each other. The discontinuity persistence describes the visible trace length of a discontinuity on a rock face above and below the location of recording, named as upper and lowers semi trace length (Hudson and Priest 1979; Priest 1993). The persistence can also be connected to a five step classification from very low persistence (<1 m) to very high persistence (>20 m) (ISRM 1978: 335). The challenge with persistence is the assessment of discontinuity progression in the internal of the rock mass. The spacing represents the distance between the discontinuities of one set (Fig. 2.2). The spacing can be determined along a scanline and afterwards be transformed to a true spacing, which represents the spacing measured perpendicular to the joint surfaces. The spacing and persistence of different joint sets have a main effect on the cubature of critical blocks.

The surface conditions include information about the fracture roughness, which could be determined by the joint roughness coefficient (JRC, 1 = very smooth to 20 = extremely rough; Fig. 2.3) (Barton 1973: 314 ff., Barton and Choubey 1977: 19) and the curvature or waviness of a discontinuity characterised in 5 classes, 1 = planar and 5 = very curved (Priest 1993: 44). The qualitative description of discontinuity surface roughness is characterized according to 3 main types (stepped, undulating or planar), each divided into the three subtypes: rough, smooth and slickensided (ISRM 1978: 343). The visual comparison of the given roughness profiles with the rock surfaces should usually be carried out in dip direction. For the

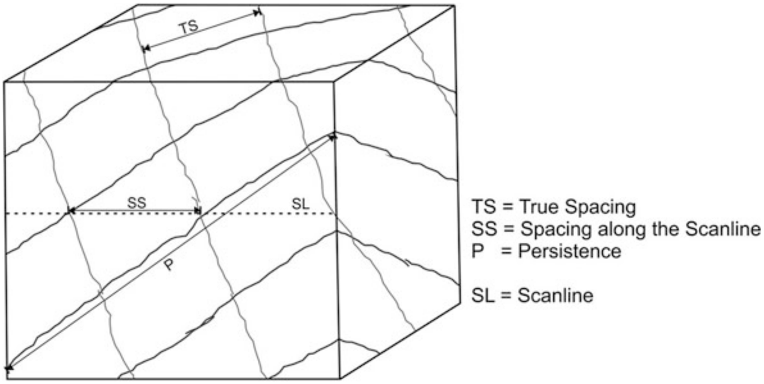

Fig. 2.2 The terms of persistence and spacing visualized in a block diagram

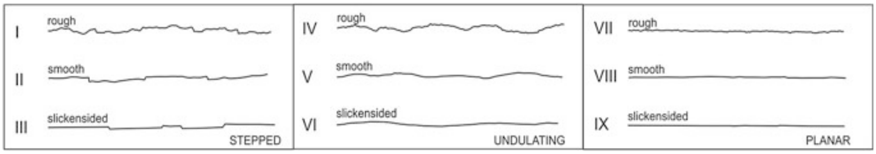

Fig. 2.3 Quantitative roughness classification categorized according to three main types: stepped, undulating and planar; each divided in three subtypes: rough, smooth and slickensided (after ISRM 1978: 343)

use of the subclasses termed “slickensided” a certain evidence for shear displacement has to be provided.

The aperture width is measured perpendicular to the rock walls of an opened joint. The aperture can be classified referring to three categories: closed features, gaped features and open features, which are subdivided in nine categories, from very tight (<0.1 mm) to cavernous (>1 m) (ISRM 1978: 352) (Table 2.1).

Table 2.1 Description of joint apertures referring to ISRM (1978: 352)

Aperture	Description	
<0.1 mm	Very tight	“Closed” features
0.1–0.25 mm	Tight	
0.25–0.5 mm	Partly open	
0.5–2.5 mm	Open	“Gapped” features
2.5–10 mm	Moderately wide	
>10 mm	Wide	
1–10 cm	Very wide	“Open” features
10–100 cm	Extremely wide	
>1 m	Cavernous	

It should be noted that a difference between the terms aperture and width exists due to the filling of joints. A joint filled with air or water shows an aperture whereas a joint filled with clay or other sedimentary material shows a width of opening (Fig. 2.1).

The infilling of a discontinuity represents the material separating two joint walls from each other. Factors like the filling material, the materials particle size, the water content, the joint wall roughness, the fracturing of the surrounding rock mass or the degree of weathering play an important role for the geotechnical behaviour of the filled discontinuities (ISRM 1978: 355 ff.).

Rock Mass Classification Systems

For classifying a rock mass originally most input was given in the field of tunneling (Terzaghi 1946 in Hoek 2000), where the rock mass was classified according to the rock properties and the degree of transection. The rock mass rating system (RMR) suggests the characterization of the rock mass referring to a score system, in which the geotechnical parameters are weighted according to their degree of influence on the rock mass destabilization (Bieniawski 1976 in Hoek 2000: 47). The Rock Quality Designation (RQD) gave input to the topic of rock mass classification using drilling cores and was actually linked to a rock mass classification system (Deere et al. 1969). The critical parameters of rock faces endangering certain highway sections can be assessed using the rockfall hazard rating system which was set up as a score system to characterize the effect on a certain highway section (Pierson 1991). The Slope Stability Probability Classification (SSPC) provides a three steps approach for characterizing a certain rock mass (Hack et al. 2002). The system includes the characterization of the rock mass (exposure rock mass, ERM) the characterization of the bedrock material (reference rock mass, RRM) and the stage of the slope after construction taking future weathering into account (slope rock mass, SRM). Nevertheless the slope and joint geometry affect the volume and shape of rockfall material. Since the joint persistence is most difficult to determine in situ, the effect of joint geometry and persistence on rock slope stability can be assessed via probabilistic approaches like SLOPESIM (Einstein et al. 1983).

2.2.1 The Mechanics of Failure Preparation and Causes

The Total Friction Concept

The geotechnical hazard analysis at rockfall source areas along the B 305 is performed using the total friction concept (Barton and Choubey 1977). The total friction concept takes the roughness of rock joints into account, when considering the Mohr-Coulomb failure criterion (Patton 1966 in Barton 1973, Goldstein et al. 1966 in Barton 1973).

Thus, the shear stress at failure τ is approached by

$$\tau = \sigma * \tan(\varphi_b + i) \tag{2.1}$$

where σ is the normal stress, φ_b is the material-immanent basic friction angle and i is the dilation angle that expresses effective roughness. The friction term is composed of two parameters, the basic friction angle φ_b and the dilation angle i of the joint surface. The basic friction angle is defined as the friction angle determined by shearing two planar rough "sawn or sand blasted" rock surfaces (Barton 1973, 1976; Barton and Choubey 1977).

The dilation angle i represents the asperity angle from the horizontal. As i is difficult to parameterise, Barton (1973) modified this equation for the shear stress at failure,

$$\tau = \sigma * \tan\left[JRC * \log\frac{JCS}{\sigma} + \varphi_b\right] \tag{2.2}$$

where JCS is the joint wall compressive strength and JRC is the joint roughness coefficient.

For rough and undulating surfaces the envelope of the peak shear strength shows a curved character (Barton 1973; Barton and Choubey 1977).

The roughness of rock joints can be categorized in first and second order asperities, depending on the considered scale (Selby 1982). The joint roughness coefficient can be assumed as a parameter for second order asperities since undulation is not taken into account. The **JRC** can be quantified using two different approaches. First the roughness of natural rock joints can visually be compared to 10 standardized profiles with JRC values from 0 (very smooth) to 20 (very rough) (Barton and Choubey 1977; Fig. 2.4).

More reproducible, the direct quantitative (in situ) method is assessing the JRC by using a profile gauge (Barton and Choubey 1977). The JRC-value is computed via the first derivation of roughness amplitudes (the gradient Z_2), which are determined in defined intervals of length (Tse and Cruden 1979; McCarroll 1997). Using linear regression analysis the JRC can be determined via Z_2 (Tse and Cruden 1979). The work of Yang et al. (2001) is based on the findings of Tse and Cruden (1979) and provides a better correlation between Z_2 and the JRC for the regression analysis. Theoretically, the JRC can be determined in a back-analysis of direct shear tests, assuming that shear strength, normal stress as well as JCS and basic friction angle are measured (Barton and Choubey 1977).

The JCS can be approached by both, uniaxial compressive strength (UCS) testing in the laboratory and Schmidt-Hammer in situ testing. UCS is definitely the more accurate test of material performance. However, Schmidt Hammer testing is better capable of defining spatial heterogeneity along rock joints with different degrees of weathering and thus material performance of rock bridges (Barton 1973: 328). For unweathered samples or rock joints the JCS is equivalent to the uniaxial compressive strength of the material (Barton 1973; Barton and

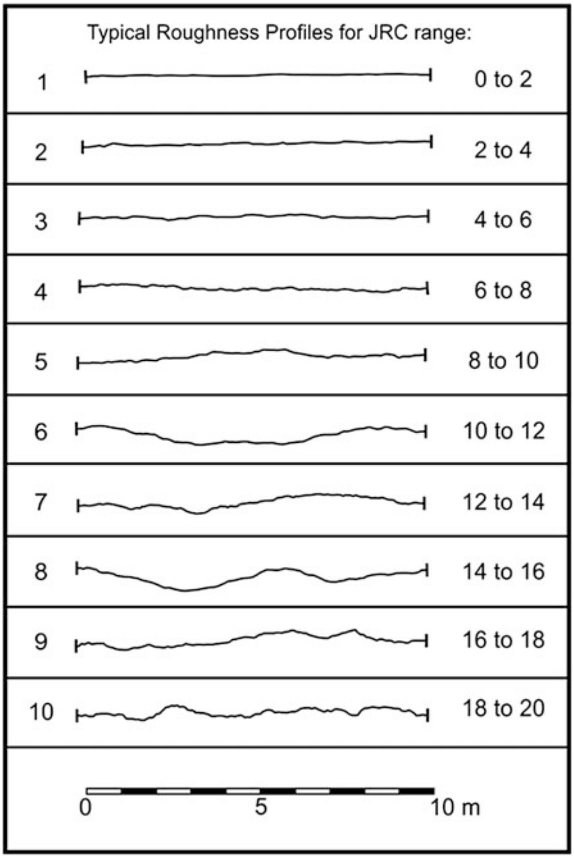

Fig. 2.4 Typical JRC profiles for comparing fracture roughness of rock faces with the graphical classification

Choubey 1977). For weathered material, the JCS is reduced in comparison to the UCS and shows a wider scatter (Barton 1973). In addition, surface roughness and moisture content contribute to a reduction of the JCS (Aydin and Basu 2005; Barton and Choubey 1977; Goudie 2006).

Causes

The causes are landslide promoting factors which do not instantaneously lead to failure. The causes can be divided into internal and external causes (Howe 1909: 44–49; Terzaghi 1960: 88 in Erismann and Abele 2001: 107). This subdivision is based on the shearing properties of material stating that internal causes are those which lead to a sliding mechanism without external influence and without a change in surface conditions. The external causes lead to an increase in shearing stresses. The example of gravitational forces, which can be grouped neither to internal nor external causes, might indicate the issue of applying this classification in practices. However it is a first approach of suggesting a classification scheme. It seems more suitable to divide the promoters of landslides in causes; meaning factors which induce landslides over a certain period of time and one trigger, which instantaneously leads to failure (Varnes 1978 in Cruden and Varnes 1996: 76).

A classification of causes is given by four basic parameters and their interaction (Erismann and Abele 2001: 109):

- The slope angle β of the underlying ground surface
- The coefficient of friction μ between the potential mass of failure and the ground
- The cohesion c between the critical mass and the surrounding rock mass
- The external forces (for example the hydraulic pressure in rock joints)

Further classification suggests the categorisation into the following four classes (Cruden and Varnes 1996: 70): geological causes, morphological causes, physical causes and human causes. For the process of rockfalls a few examples for each group are mentioned as:

- Geological causes: adversely oriented discontinuities, weathered materials
- Morphological causes: toe-erosion; steep valley flanks
- Physical causes: thawing, intense rainfall, freeze-and-thaw weathering,
- Human causes: excavation of the slope toe

Both classification systems interact with each other since for example the geological causes like weathered materials would affect the coefficient of friction as well as the parameter of cohesion.

2.2.2 Critical Fracture Propagation (Progressive Failure) and Triggers

Continuous Crack Propagation

For the failure of a potential rock mass a growth of cracks must occur so that a coherent failure plane can be formed. In rocks this implies the transition from rock bridges to roughness contacts, which are separated by cracks. A rock bridge is defined as a segment of intact rock separating co-planar or non-coplanar discontinuities from each other (Kemeny 2005: 36).

If a critical mass fails due to crack propagation over a certain period of time two terms can be classified referring to the location of stress attack (Erismann and Abele 2001: 121 ff.): if one single location or rock bond is affected by several load cycles the process would be named as "fatigue"; if many locations at a failure surface are affected by changing stress concentration this would be called as "progressive failure". So the failure of a single rock bridge could be considered as fatigue in the general context of progressive failure. The term "fatigue" was introduced giving the following detailed definition (Visser 1980 in Erismann and Abele 2001: 120): "… fatigue is the property of material to fail after many repetitions of a deforming stress, which by itself is not high enough to cause failure." This definition implies that a failure occurs not until several load cycles have occurred. The following formula was set as a general idea for fatigue crack propagation (Paris 1962 in Erismann and Abele 2001: 120):

$$\frac{dc}{dN} = (C\Delta K)^n \tag{2.3}$$

where dc represents the crack growth per load cycle (with N as the cycle number), ΔK stands for the variation of stress concentration and C and n are constants, determined experimentally. The relation shows that the crack growth increases due to increasing stress concentrations. Or the other way round, if the cracks grow further, the stress concentration will increase during the load cycles. Since the driving force of a critical mass can be assumed as more or less constant due to a direct relation to gravity, the crack growth is increasing in an accelerated pace.

Assuming a failure surface of several square meters, repeating load cycles lead to failure of one or more rock bridges. The consequence would be a redistribution of stresses among the remaining rock bridges and asperity contacts. With every time one or more intact rock contacts fail, the induced stresses affect the remaining intact contacts over a certain period of time until the critical mass fails as a whole. The redistribution of stresses affects interlocking asperity contacts in the same way as intact rock bridges. Interlocked roughness contacts could also be sheared off due to increasing shear stresses. A sketch of the transformation from rockbridges to roughness contacts is given in Fig. 2.5.

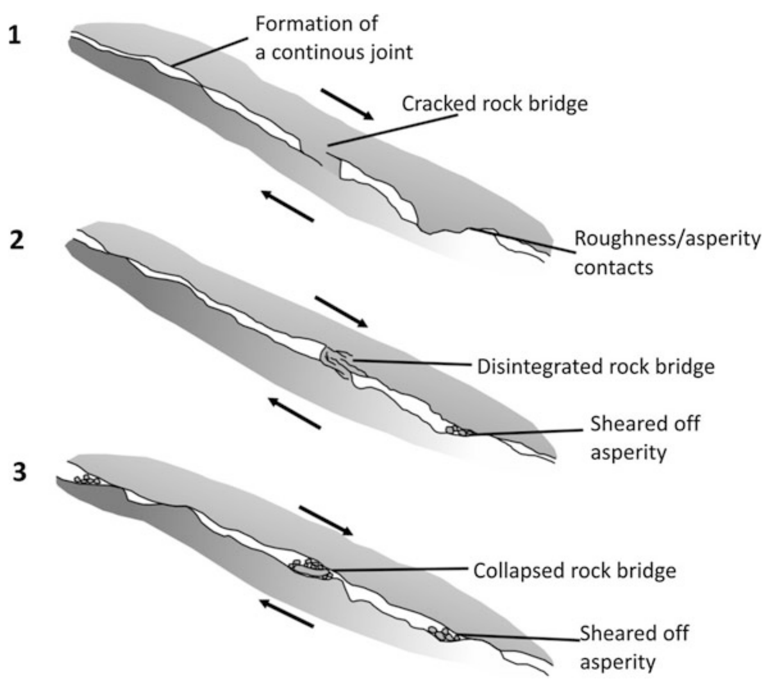

Fig. 2.5 Stages in transformation of rock bridges to asperity rock-rock contacts at a failure surface

The formation of cracks and at larger scale discontinuities has to be embedded in the context of stress regime affecting the different part of a critical rock mass. There are three types of discrete stress distributions which are classified due to the orientation of the stress vector in relation to the surface of a potential joint (Erismann and Abele 2001: 116 ff.):

- Mode I: the tensile stress vector is oriented perpendicular to the joint surface. This mode is named according to the mouth of an alligator.
- Mode II: The shearing stress vectors are oriented parallel to the shearing plane but perpendicular to the margin surface of a potential block. This mode is named referring to the mouth attitude of a bulldog.
- Mode III: The shearing stress vectors are oriented parallel to the shearing plane and also parallel to the edge of the block. This mode is named according to the mouth attitude of a cow.

The three modes of stress distribution can occur considering a case of planar failure on an inclined failure surface (Fig. 2.6). At the detachment plane tensile stress is dominating, which leads to crack opening from top to bottom. At the

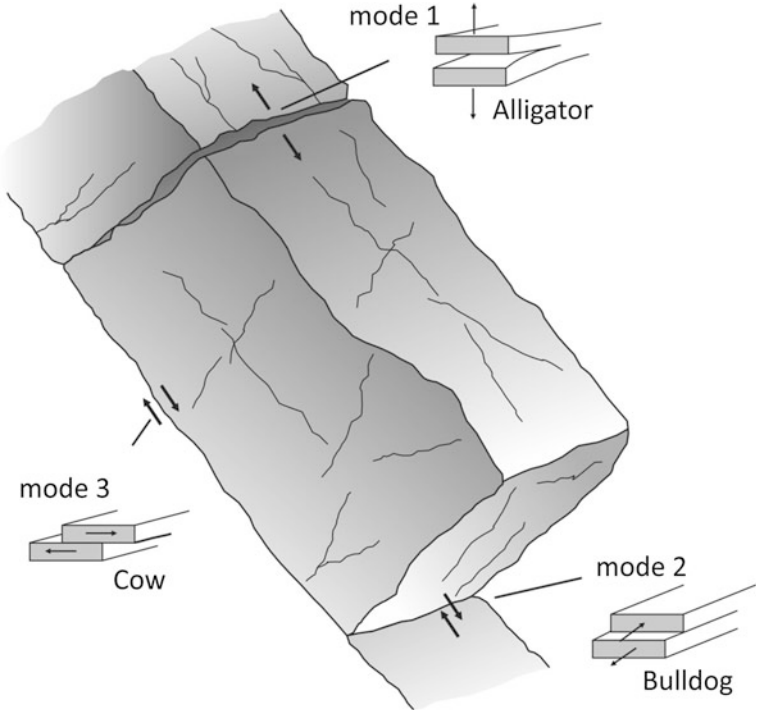

Fig. 2.6 The mechanical modes of failure named according to animal's mouth attitudes (after Erismann and Abele 2001: 116) illustrated using the example of planar rock mass failure

bottom of the block mode II can be assumed, whereas the shearing vectors at the lateral edges of the block can be grouped to mode III.

Trigger Mechanisms

"The trigger of an event is the last cause which puts an end to the balance between driving and retaining forces and thus sets the mass moving."(Erismann and Abele 2001: 108)

In many cases triggers can be seen as external drivers, for example: intense rainfalls, rapid snowmelt, water-level changes, earthquake shaking and volcanic eruption (Wieczorek 1996: 76 ff.). For rockfalls especially the thawing of ice and ice segregation can play a certain role as triggers (Krautblatter et al. 2013). A trigger leads to a strong decrease in the retaining forces so that failure occurs as the consequence.

An issue with the assignment of triggers is that in several cases an exact trigger cannot be determined. If the in Sect. 2.2.1 mentioned causes affect a potential mass over a critical period of time, this could lead to material fatigue of the rock mass, which consequently leads to failure. The consequence is a progressive failure of a critical mass.

Types of Detachment: Topple, Slide (Block and Wedge Sliding), Rock Slumping

The detachment process of rock masses depends on several factors: Joint systems and their orientation, spacing of the joints and inclination of the slope/rock face. The mentioned factors influence the released volume, the block shape and especially the detachment process (Fig. 2.7). The general detachment mechanisms for rock slopes could be divided into the following groups: planar sliding, wedge

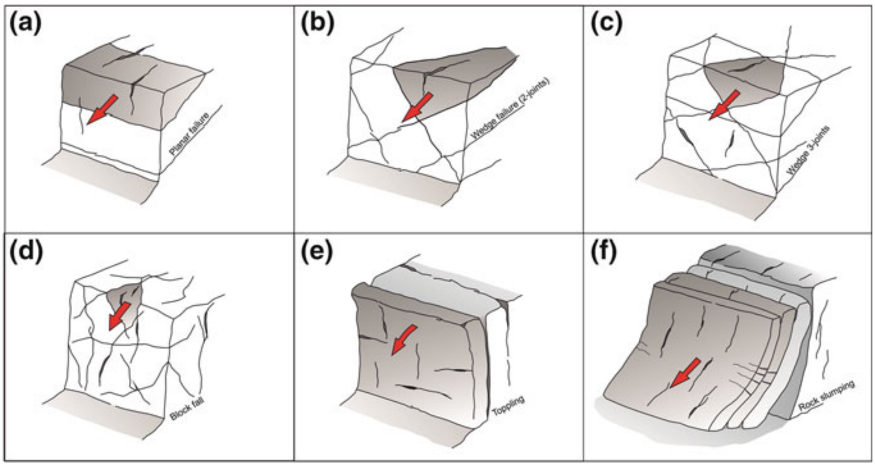

Fig. 2.7 Examples of rock mass failures referring to their mode of release: Planar sliding (**a**), wedge sliding (**b** and **c**), free fall (**d**), toppling (**e**) and rock slumping (**f**)

sliding, toppling, rock slumping, buckling and free fall (Whalley 1984; Goodman and Kieffer 2000).

The planar failure occurs on one single plane of failure due to undercutting of a rock slope by one joint system (Fig. 2.7a). If two joints are intersecting in an adverse way and the line of intersection is inclined parallel to the slope this could result in wedge sliding. The wedge could be defined by 2 or more joints (Fig. 2.7b, c). If a joint system draws through a rock mass totally undefined, blocks of an undefined shape could be released and descend the slope (Fig. 2.7d). In case of a very steep and parallel to the slope inclined joint system toppling can occur. In case of toppling a rock slab fails in a forward rotational movement, since the centre of gravity is exposed over the edge of the rock slab. The tension cracks between the rock slabs show a V-shaped form (Fig. 2.7e). Due to a missing toe support rock slabs are released in a backward rotation, which leads to rock slumping. The rock slabs are separated by A-shaped tension cracks (Fig. 2.7f).

The spacing of a joint system affects the volume of the released mass due to the variation in length and width.

2.2.3 Rockfall Run-Out Modelling Approaches

A Short History of Rockfall Modelling

In this paragraph a short overview of the history of rockfall modelling should be given, whereas the detailed description of the different modelling types is presented in Sect. 2.2.3.

The need for run-out assessment of rockfalls goes back to the 1930s (Heim 1932). Heim was the first author who described the behaviour of blocks during free fall and the subsequent modes of motion through the travelling process. The complexity of the impact of a falling block/boulder on the slope surface including a potential degree of fragmentation and the dispersion of trajectories is illustrated in detail (Heim 1932, 64 ff.). The first empirical approach of run-out modelling, namely the method of the “Fahrböschung” (Fig. 2.8), suggests the measuring of the angle between a horizontal line and the connecting line between the release area and the farthest travelled block at the run-out area (Evans and Hungr 1993; Heim 1932). A similar approach is given by the “minimum shadow angle”-principle including the angle between a horizontal line and the connecting line between the toe point of the source area and the farthest travelled block of a rockfall mass (Evans and Hungr 1993). One of the enhanced empirical models is the run-out-ratio model, describing the ratio between the horizontal length of the run-out distance and a combined horizontal length of the talus slope and the rock face (McClung and Lied 1987 in Dorren 2003). This model is based on a run-out approach for avalanche modelling.

The next step in the development of rockfall models are the process based models reducing the rockfall process to a 2D section, where the lateral dispersion is not considered. The modes of motion during the falling process were combined of flying and collision stages, where the flying stages were modelled as parabolic

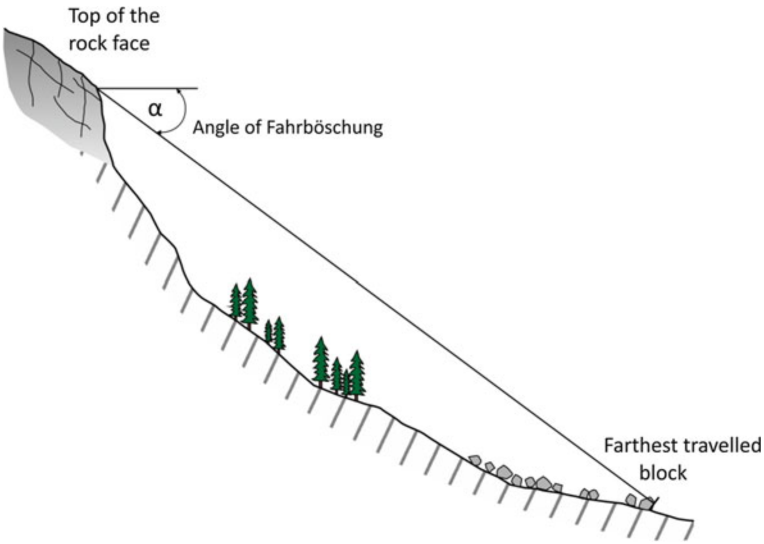

Fig. 2.8 Illustration of the Fahrböschung as an empirical approach for run-out assessment

throws with velocities in x and y direction, where the acceleration of the block is based on gravity (Dorren 2003). There are plenty of process based models describing the rockfall process in a 2D limited space as mentioned above (Azzoni and Freitas 1995; Hungr and Evans 1988 in Dorren 2003; Statham 1976).

The 3D rockfall models are based on a GIS approach or a combined GIS process based approach. The GIS based models are composed of three modules: the determination of the source area, the calculation of falling tracks and the calculation of the run-out (Hegg and Kienholz 1995 in Dorren 2003). The evolution of the GIS based models reaches from models which simulate the block sliding over the slopes surface to models simulating an initial free fall and the subsequent bouncing and rolling. The GIS-process based models, namely Rockyfor 3D can be used for rockfall calculations at regional scale (Dorren 2003).

The Scope of Rockfall Modelling

The applied purpose of rockfall modelling is the improvement of hazard assessment, including the run-out prediction of possible events as well as the design of mitigation measures. Apart from any certain code the most important outcomes are:

- The run-out distance: to determine whether critical blocks will reach endangered infrastructure or not.
- The velocity and kinetic energy: as linked dimensions to design the load capacity of mitigation measures.
- The jumping heights: to adapt the height of fences or walls to the critical jumping height of descending blocks.

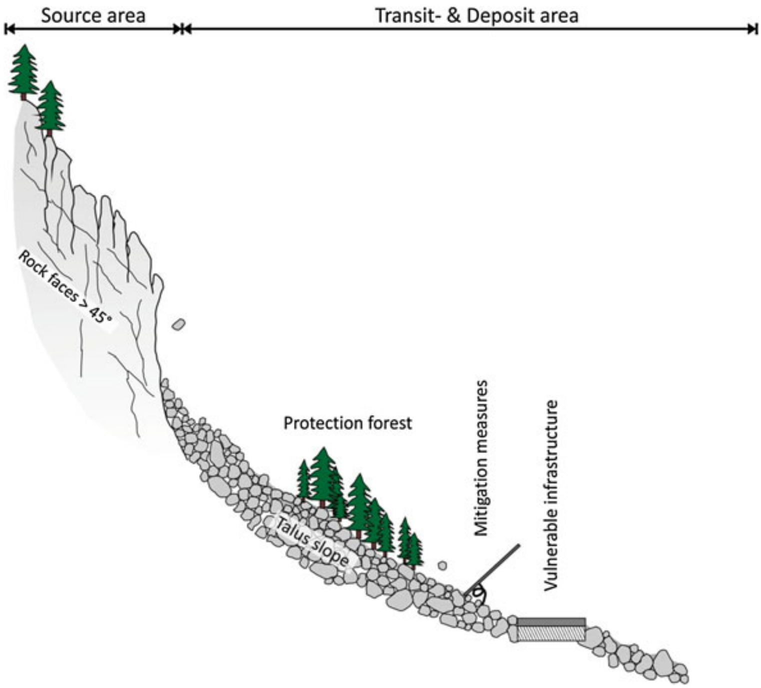

Fig. 2.9 General sketch of the rockfall process areas: the source area, the transit- and deposit area. Protection forest as well as fences or walls are retaining measures at endangered slopes. The critical inclination angle is assumed as 40° for the rockfall source areas (after Dorren 2003)

The crucial input parameters can be categorized according to the process areas of a rockfall slope (Fig. 2.9). For the source area the block parameters like the dimensions, the shape and the material density have to be defined. The transit and deposit area need to be defined by roughness and damping parameters of the ground to assess the decelerating forces for the block (Dorren 2012; Krummenacher et al. 2005; Guzetti et al. 2002). Since natural obstacles like protection forest have a decelerating effect on descending blocks, the forest stand is an important parameter which is implemented into the state of the art 3D rockfall codes (Dorren 2012; Bartelt et al. 2013 Krummenacher et al. 2005).

Modes of Interaction During the Rockfall Process and Modelling Approaches
The rockfall process is characterized by the following stages of motion: the detachment-, the transition- and the stopping or deposit phase. The detachment often leads over to a free fall stage, where the duration of the free fall depends on the inclination angle of the terrain. The transition stage is composed of flying and collision stages, which is called bouncing (Dorren 2003). As the total kinetic energy is decreasing, the mode of motion could change from bouncing to rolling until the block finally stops. This point is reached, when the kinetic energy loss due to

collisions with the underground, forest stems or obstacles is so high that the block is forced to stop.

Since the current thesis deals with 3D rockfall modelling, the state of the art in terms of modelling refers to the state of the art 3D-codes.

Interaction Block—Underground

The interaction of a descending block with the underground material is an important influencing factor of the blocks energy loss. The state of the art 3D rockfall codes nevertheless differ in the approaches of describing the contact between boulder and underground.

One of the most applied approaches is to characterize the process of energy loss due to slope contacts by two main parameter sets (Dorren 2012; Guzetti et al. 2002; Krummenacher et al. 2005):

- The damping: represented by the normal and tangential coefficient of restitution (R_N and R_T). The coefficient of restitution is defined as:

$$R = \frac{V_{ac}}{V_{bc}} \tag{2.4}$$

  where v_{ac} is the speed after collision and v_{bc} is the speed before collision.
- The roughness: represented by undulation of the underground material properties

The coefficients of restitution are important for the calculation of the penetration depth of the block into the ground material (R_N) and the calculation of the velocity after the rebound (R_T). Their integration into the algorithm depends on the code (Bourrier et al. 2009; Dorren 2012; Guzetti et al. 2002).

The slope roughness can be described by the diameter of blocks building a talus slope (Krummenacher et al. 2005). An enhanced way of determining the "true" effect of the slope roughness to descending blocks would be the assessment of the mean obstacle height (MOH). The MOH mirrors the height of an obstacle considered in the descending line of the falling block (Dorren 2012; Fig. 2.10) Apart from any approach of describing the roughness, the slope roughness has got an evident effect on blocks with a small diameter compared to the amplitudes of roughness. The effect of slope roughness decreases if the block diameter increases in comparison to the amplitudes of slope roughness.

In most codes where the rebound approach is based on the coefficients of restitution, a complex or natural block shape is not considered. The Code STONE 3D considers the block as a lumped mass, implying that the mass is concentrated in one central point (Guzetti et al. 2002: 1082). The code Rockyfor3D provides the possibility to enter a block shape. Nevertheless during the contact block—slope, the boulder is considered to be a sphere with a diameter composed of the mean of the two longest block axes (Fig. 2.11; Dorren 2012: 13 ff.).

One of the most actual rebound approaches provides the possibility to take the shape of a descending rock fully into account (Fig. 2.11). The physical modelling

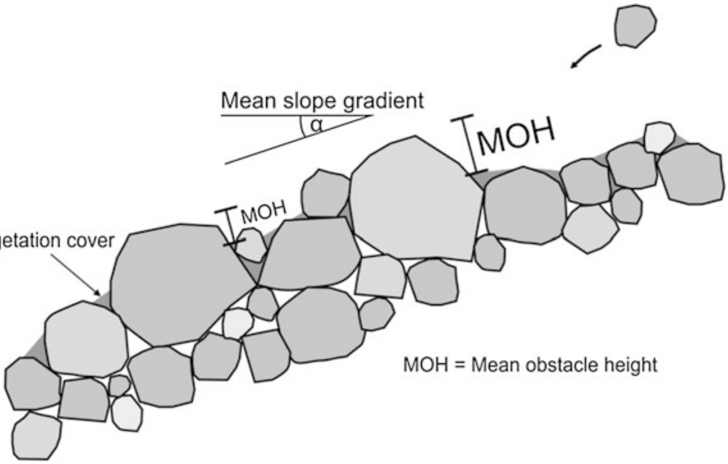

Fig. 2.10 Illustration of the Mean obstacle height (MOH) (after Dorren 2012), which implies the height of an obstacle in falling direction of a descending rock

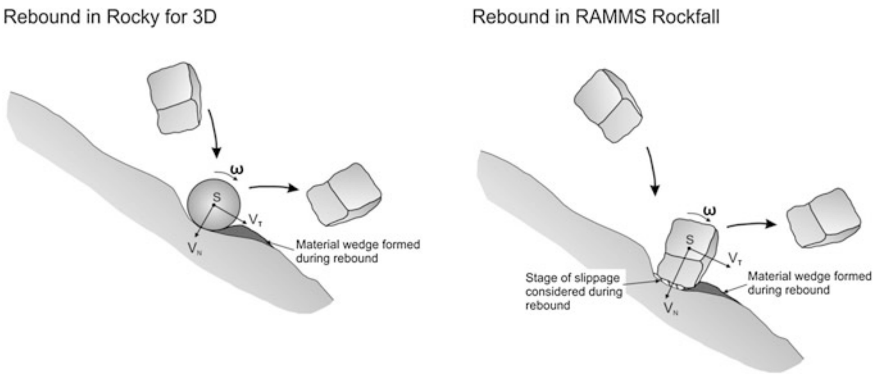

Fig. 2.11 *Left* Rebound analysis in Rockyfor 3D: The rock is considered as a sphere during rebound, with its centre of mass at the centre point of the sphere. *Right* Rebound analysis in RAMMS::Rockfall: the rock shape is taken into account during rebound, the block stick-slides over the terrain before take-off

approach is based on the non-smooth contact dynamics method and was implemented into the software RAMMS::Rockfall. The descending block is modelled as a rigid complex shaped body, which is characterized by its centre of mass and inertia vector (Leine et al. 2013; Bartelt et al. 2013).

The following important issues in terms of rock—slope interaction are considered in this model:

- The three dimensional motion of a 3D shaped rigid body in space (with three degrees of freedom for translational motion and three degrees of freedom in terms of rotation)

- The consideration of a 3D shaped block during the slope contact (the models described above consider the block as a sphere during the block/terrain interaction)
- The stage of slippage during the rebound process

In the code RAMMS::Rockfall the terrain geometry is described by a high resolved digital terrain model (DTM). The terrain materials properties are defined referring to the Coulomb-Friction law, where the terrain material is determined by the friction coefficient and the coefficients of restitution (Bartelt et al. 2013).

The Interaction Block—Forest Stand

The protective effect of mountain forests was in the focus of research over the last years (Berger et al. 2002; Jahn 1988; Dorren et al. 2007; Stoffel et al. 2006). The presence of forest stand on alpine hill slopes leads to a decrease in velocities, rebound height and kinetic energy. Consequently the run-out distance will be reduced due to forest stand (Dorren et al. 2005). Studies on the required species of a protective forest show, that broadleaved trees would have the more robust properties against rockfall than coniferous trees (Stokes 2006: 84; Dorren and Berger 2005: 69). Nevertheless a mix of both tree species would be most effective since coniferous trees are more effective in preventing avalanche damage (Stokes 2006). Apart from the tree species it seems to be more effective to provide a forest stand with a large number of trees instead of a decreasing number of trees with a large diameter at breast height.

To directly compare the effect of protection forest on rockfall hazard, real size experiments were carried out on a slope providing an equivalent geological set up, but with and without forest stand (Dorren et al. 2006).

The forest stand can quantitatively be described by the following parameters:

- Tree species
- The diameter at breast height (DBH)
- The stock density, which can be assessed via:
  - The number of tree stems per hectare
  - The mean tree free distance (MTFD) (Krummenacher et al. 2005)

The drag effect of the forest stand can be implemented in rockfall codes using different approaches. A detailed approach of describing forest stand is to define homogenous areas for the forest stand by mapping. For each area the number of tree stems per hectare and the percent of coniferous trees have to be determined. To assess the resistance against stem breakage, the diameter at breast height is specified as a mean value with the associated standard deviation. The described parameters are required for the software package Rocky for 3D. In this code the forest stand is defined pixel based, meaning that the number of trees is randomly defined for each pixel with the associated DBH (Dorren 2012).

Another approach is that the forest stand is simply defined as an additional drag force depending on the tree height. Every time the block descends the mean forest height the drag force is applied. This approach is applied in the code RAMMS:: Rockfall (Bartelt et al. 2013).

2.3 Key Questions and Research Gap

The frame of this research work is set by the topic of rockfall hazard assessment. The described state of the art demonstrates the stress field between rockfall run-out modelling and the mechanics of failure preparation in advance of release.

The submitted PhD thesis aims to link the field of detachment and failure mechanics to the field of rockfall run-out analysis by addressing the following key-questions:

1. Detachment mechanics at the rockfall source area:
   (a) How can we achieve reconnaissance about the failure mechanics of mid-magnitude blocks?
   (b) How can we analyse the mechanical behaviour of a critical block by analysing the limit equilibrium stage?
   (c) How can we provide accurate information on the degree of potential fragmentation as input parameter for rockfall modelling?
2. Quantitative magnitude assessment and detachment processes:
   (a) How can we validate assumed detachment processes by scanline and kinematic analyses?
   (b) How can we provide a structured way of evaluating rock volumes in a quantitative way?
   (c) How can we transfer our knowledge from block recording to reasonable information in terms of rockfall modelling?
3. Run-out modelling
   (a) How does an increased slope roughness affect the rockfall run-out?
   (b) How do areas of windfall affect the rockfall run-out in terms of increased slope roughness?
   (c) How can we consider potential degrees of fragmentation in detailed 3D modelling studies and which assumptions do we have to take?

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