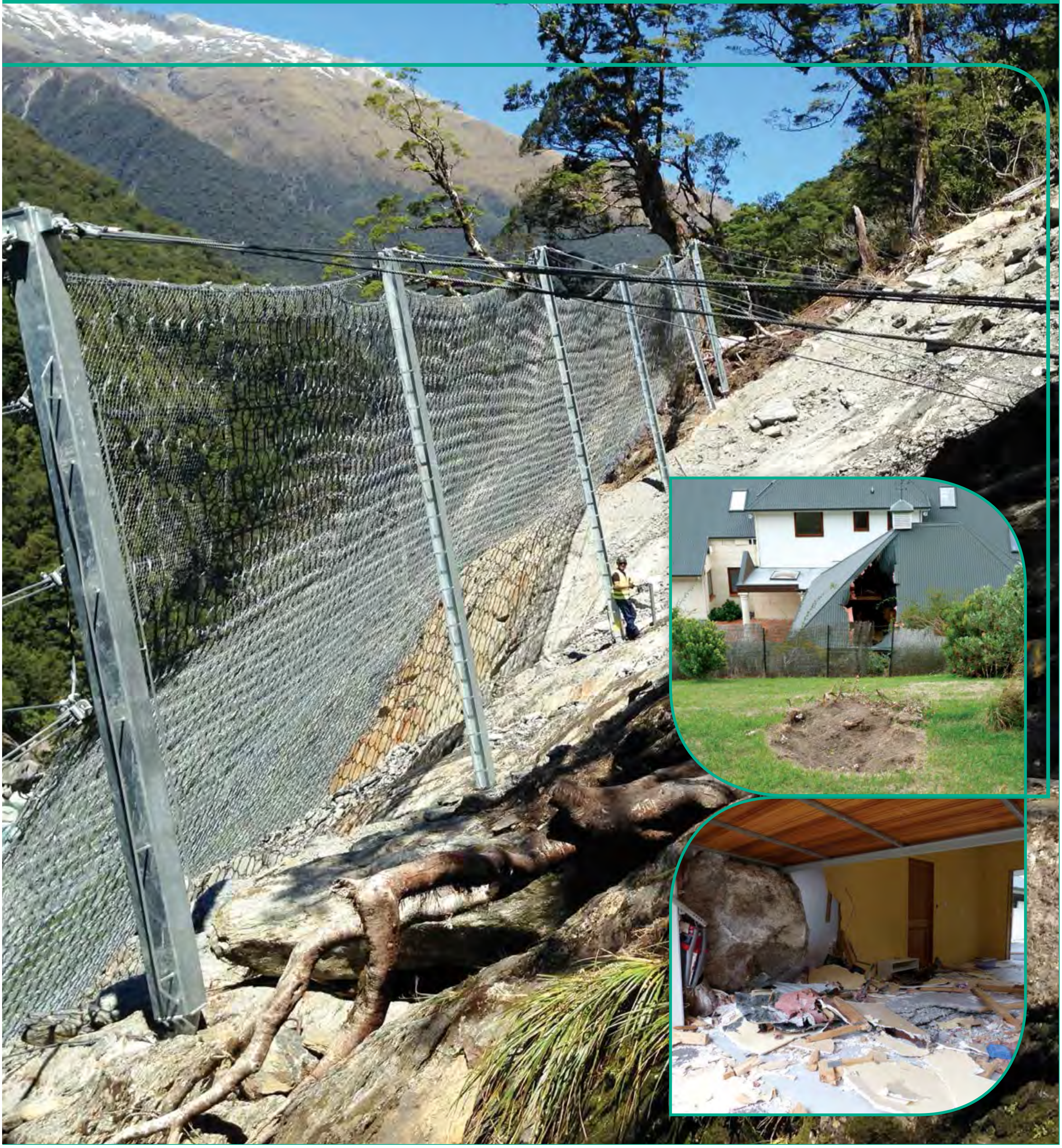
ROCKFALL:

Design considerations for passive protection structures



MINISTRY OF BUSINESS, INNOVATION & EMPLOYMENT
HĪKINA WHAKATUTUKI

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Ministry of Business, Innovation & Employment (MBIE)
Building System Performance Branch
PO Box 1473
Wellington 6140

New Zealand Geotechnical Society
PO Box 12 241
Wellington 6013

Transport Agency
Private Box 6995
Wellington 6140

KiwiRail
Wellington Railway Station
P O Box 593
Wellington 6140

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Important notice

Submissions by the geotechnical community to the Ministry are encouraged. The document will be updated following the six month period for comments after the issue date.

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FOREWORD

The problem of rockfall and protecting people and infrastructure from its impacts is not a new issue for New Zealand. However, with the occurrence of the 2010/2011 Canterbury Earthquake sequence, issues associated with rockfall received significant attention when more than 100 houses in the Port Hills of Christchurch were hit and/or penetrated by falling rocks. This event tested existing rockfall protection structures within the Port Hills and the performance of these structures was variable. This variable performance was in part due to lack of guidance on the engineering design of these types of structures.

New rockfall protection structures were considered in many areas around the Port Hills as a means to mitigate the rockfall risk to dwellings and infrastructure. Recognising the need for more consistent and reliable engineering design, the Christchurch City Council prepared a Technical Guide on Rockfall Protection Structures in March 2013. This document focusses on issues specific to Christchurch and the Port Hills.

It has since been recognised that it would be a valuable exercise to expand on this guidance to make it more applicable nationally. The Ministry of Business, Innovation & Employment (MBIE) agreed to undertake this project by producing this guidance for the design of passive rockfall protection structures.

GLOSSARY OF KEY TERMS

The following definitions are used in this document:

At-source Mitigation Measures	Works undertaken within the rockfall source area that are intended to reduce the potential for rockfall to occur.
Block	*Also boulder, rock block.* An individual piece of rock. No particular size is implied by the term.
Building Consent Authority	The regional or local authority responsible for issuing a building consent for the construction of the rockfall protection structure.
Bund	An earthen embankment (sometimes reinforced) that is used as a passive rockfall protection structure.
Design Block	The size (in m^3) of the rock block that is selected for the design of the rockfall protection structure.
Design Capacity	The level of energy (usually in kJ) that a passive RPS is intended to withstand based on design block travelling at the design velocity.
Design Velocity	The speed of the design block, usually estimated using rockfall modelling software.
Designer	The geotechnical engineer or other qualified geo-professional who is responsible for undertaking the design of a rockfall protection structure.
ETAG 027	European testing standard for rockfall net fences.
Energy Capacity	The energy rating of a rockfall net fence system, as measured via standardised testing (most commonly ETAG 027).
MEL	Maximum Energy Level. The energy level of the Design Block travelling at 25m/s. This is a term from ETAG 027 and used only in conjunction with flexible rockfall barriers.
Passive Mitigation Measures	Works undertaken downward of the rockfall source area that are intended to reduce the effects of falling rock.
RPS	An acronym for Rockfall Protection Structure(s).
SEL	Service Energy Level. The SEL is equal to 1/3 of the MEL. This is a term from ETAG 027 and used only in conjunction with flexible rockfall barriers.

1 INTRODUCTION

1.1 Purpose and Scope

This document provides technical guidance for the design of passive rockfall protection structures (RPS) that act to reduce the effects of falling rock on people and/or infrastructure. The document sets out a general methodology for undertaking design of passive RPS within the context of the New Zealand Building Code. It also serves as a guide to inform others about the design process and the nature of the work involved in designing passive rockfall protection structures.

The document is a review of rockfall literature and current practice with emphasis on experience in Europe and North America where much research on passive RPS is underway. As a result of the evolving nature of passive RPS design, the information presented herein is intended to be informative.

Passive RPS are one of the possible means of mitigating risk posed to people and infrastructure by falling rock. This document includes an overview of the rockfall risk mitigation process as context to where passive RPS fit into the risk mitigation framework. Its purpose and focus, however, is on the technical aspects related to the design of passive RPS.

Information about relevant topics has been summarised, and references are listed to direct the reader to more in-depth information.

1.2 Rockfall Definition

For the purposes of this guidance, the following definition of rockfall has been adopted (Turner & Schuster, 2012):

- A very rapid slope movement in which bedrock material is detached from a steep slope and descends by falling, bouncing, rolling or sliding
- It can involve gravel-size particles up to large rock masses
- It relates to the fall of individual or several rock blocks, where there is little interaction between the individual blocks.

Rockfall events can be defined over a continuum from the fall of a single block to the fall of many thousands of blocks such as occurs in a rockfall avalanche-type event. This document focusses on the fall of individual or relatively small numbers of blocks. This document should not be used to design passive RPS where the hazard comprises the fall of many blocks (debris avalanches such as cliff collapses) or other types of landslide debris possibly moving as flows (eg debris flows), where the debris generally moves as a mass[1]. However during earthquakes, a multitude of rocks may be triggered from a given source area. Special attention should be given to the clearance of the RPS after each earthquake event.

1.3 Passive Rockfall Protection Structures

Passive rockfall protection structures are engineered structures constructed at a location distant from the rockfall source that intercept or divert falling or rolling rocks. They are not intended to prevent rockfall from occurring, but rather to mitigate its effects. The types of passive RPS addressed in detail in this document are:

- Flexible barriers
- Deformable rigid barriers
- Attenuators
- Catch areas.

Other types of passive RPS exist, however these are considered to be the most likely to be constructed within New Zealand.

1.4 Audience

The audience for this document is:

- Experienced geotechnical professionals seeking guidance on current approaches used for the design of passive RPS; and
- Territorial and building consent authorities which may be required to assess building consent and resource consent application documents related to passive RPS.

1.5 Exclusions

This document specifically does not address the following:

- Rockfall hazard assessment and guidance on decisions around whether or not action needs to be taken to mitigate the risk associated with the assessed rockfall hazard
- Rockfall mitigation works undertaken at the rockfall source that act to prevent rockfall from occurring (eg scaling works, rock bolts, pinned mesh, etc)
- Rockfall associated with cliff collapse, debris flows or other types of landslide (eg slumps, slides) (Hungr et al., 2014).

1.6 Scope

This document provides guidance on:

- rockfall risk mitigation
- technical design considerations
- compliance with Building Code.

1 Large volumes of falling rock (or rock and soil) behave differently to individual blocks. The behaviour (velocity and kinetic energy) of large volume masses is more difficult to understand and quantify; research is ongoing internationally to better understand this behaviour.

2 ROCKFALL RISK MITIGATION PROCESS

Rockfall hazard is most commonly addressed within the a framework of risk management. This involves characterising the rockfall hazard and evaluating the risk posed by the hazard. Hazard and risk are defined as follows (AGS, 2007):

A simplified overview of the risk management process is shown in Figure 1. A summary of the steps in this process is given to provide the contextual framework for the selection and design of passive rockfall protection structures that moderate the consequences of a rockfall event.

- **Hazard:** a condition with the potential for causing an undesirable consequence; this includes describing (defining) the location and the probability of occurrence within a given time period.[2]
- **Risk:** a measure of the probability and severity of an adverse effect to health, property or the environment; often calculated by multiplying the probability of occurrence by the consequences (quantitative).

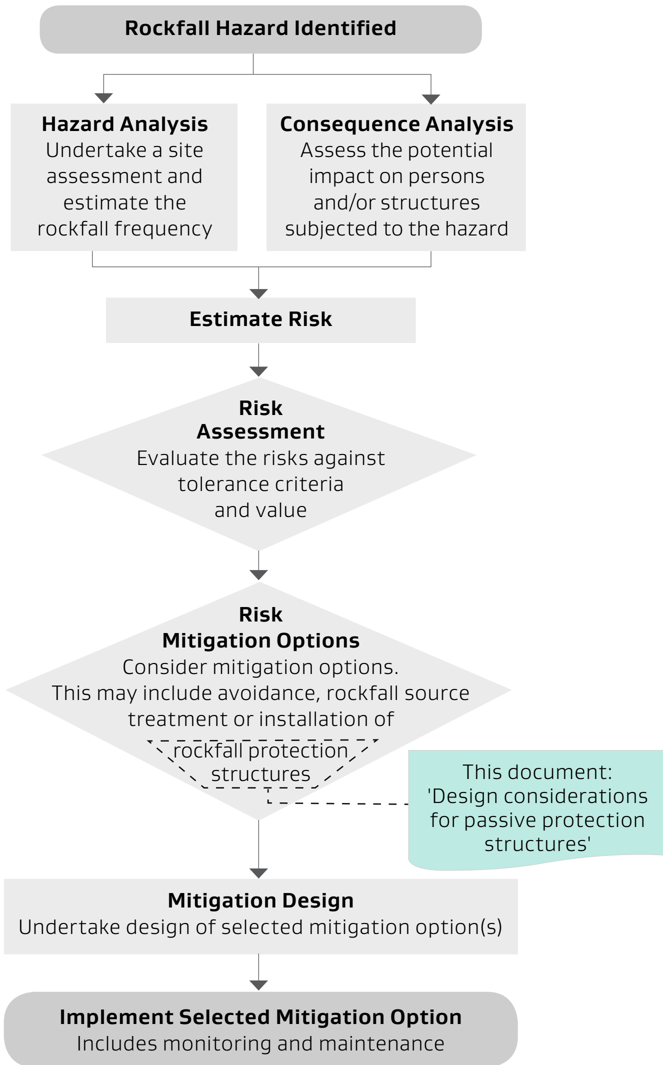
Once the risk is estimated, it is assessed using defined criteria so as to judge what constitutes an acceptable or tolerable level of risk. If the risk is judged to be too high, then a number of actions may be undertaken to mitigate the risk.

For the particular case of rockfall, mitigation options may include (AGS, 2007):

- **Avoid the risk:** move the people/infrastructure away from the hazard.
- **Reduce the frequency of an event:** undertake stabilisation or removal works at the rockfall source.
- **Reduce the consequences of an event:** install defensive measures downward of the rockfall source to protect people and/or infrastructure (*focus of this guidance document*).
- **Manage the risk:** install monitoring, warning systems, signage.
- **Accept the risk:** take no action.
- **Transfer the risk:** require another authority to accept the risk, or compensate, as by insurance.
- **Postpone the decision:** where there are significant uncertainties, undertake additional studies to reduce the uncertainties.

It may be the case that more than one mitigation option is used, such as removing a portion of the source rock, constructing a rockfall barrier and installing warning signs.

Figure 1: Overview of the Rockfall Risk Management Process (based on AGS, 2007)



2 AS/NZ31000:2009 defines risk as a combination of likelihood and consequence. Hazard, as defined by AGS (2007), largely aligns with the AS/NZ 31000:2009 definition but includes time-based a probability component. The Switzerland approach of rockfall risk managment process is presented in Appendix C.

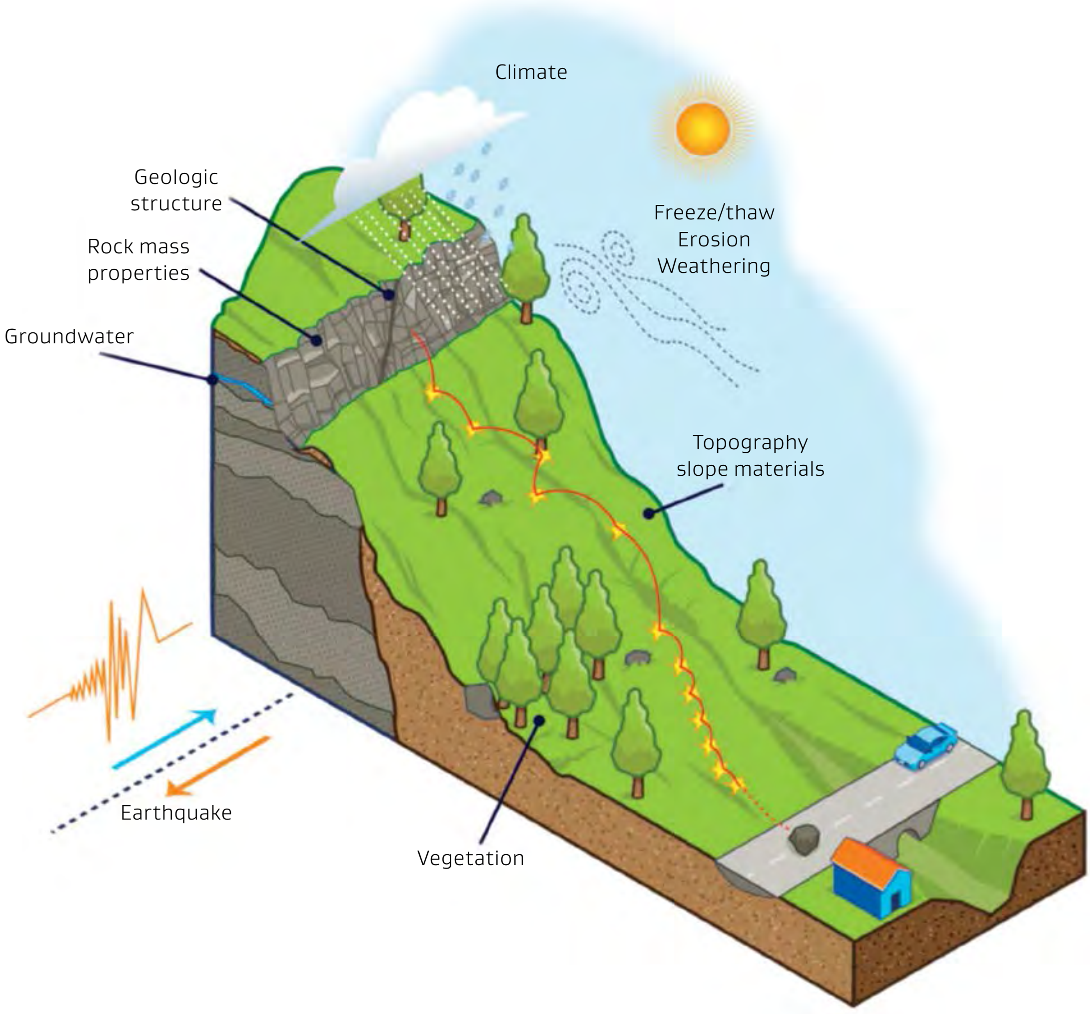
2.1 Rockfall Processes and Site Assessment

Rockfall is a complex process that is influenced by a number of variable factors. These factors that affect the incidence and behaviour of individual falling rocks can be divided into internal parameters and external influences (Volkwein et al., 2011; Turner and Schuster, 2012). Internal parameters are those that are intrinsic to the source rock mass and slope. External influences are conditions that can change, sometimes very rapidly, and alter the forces acting on the rock source such that rockfalls are triggered.

- **Internal parameters:** rock mass properties (rock type, discontinuities, strength, block size/shape), slope morphology (height, angle, profile shape), slope materials, vegetation, groundwater.
- **External influences:** weathering, erosion, climate (rain, snow, freeze/thaw), seismic activity, human activity (excavation, blasting, water table changes, surface water control, deforestation).

Together these factors (Figure 2) influence the frequency of rockfall and the path, speed, mode of travel (eg bouncing, rolling, etc) and travel distance of rocks as well as their resistance to breaking apart during travel. It is important to understand and document these factors as they play a role in the selection and design of rockfall mitigation measures.

Figure 2: Factors that influence rockfall



2.2 Site Assessment

The site assessment is a critical component of the work involved in the design of rockfall protection structures. The quality of data collection directly affects the accuracy of the site model, which in turn affects the analyses used for the design and selection of appropriate protection measures (Turner and Schuster, 2012).

The site assessment should answer the following questions (Caltrans, 2014):

1. Is there a rockfall problem, under what conditions is it a problem, and where is it located?
2. What is the nature and frequency of past rockfall events at the site?
3. Where the source area, travel paths and run-out zones are and what are the properties of each?
4. What is the likely motion of the rock travelling down the slope (rolling, bouncing or sliding)?
5. What size rocks typically reach the base of the slope (or point of interest) and how does this compare to their size in the source area?
6. How far do the rocks roll past the base of the slope (or point of interest)?

The site assessment is usually performed as a combination of desktop studies and field investigations. Desktop studies are used to collect known information about the site, including the history of past rockfall events. They are most useful when undertaken in advance of the field assessment to inform and focus the field investigations. The field investigations are aimed at characterising the geomorphology of the site and the nature of the geological processes operating, in particular the site-specific nature of the rockfall source areas, run-out zone and past rockfall events.

The site assessment should include an in-depth geological and rock mechanical study including nature and distribution of rock types, joint patterns, hetorage veities, etc. Based on this information a detailed and comprehensive geological model should be established.

The information to be collected during the site assessment is set out in Table 1, together with references that describe methodologies and techniques for collecting the information. Not all of this information may be relevant for a particular site; information that plays a critical role in the design of passive RPS is listed in red.

It is important to recognise that the site does not stop at artificial boundaries, such as roads or property boundaries. As a minimum, the site assessment area should be large enough to encompass the entire source area and run-out zone. Useful information may be gained by examining the area outside of these limits, especially if the setting and rockfall processes are similar. The scale of the site assessment, both in aerial extent and in budget, should be commensurate with the scale of the rockfall hazard.

This is often a critical point in the workflow either because of low budget or lack of time. It may be better to take precautionary measures in order to gain time for further analysis.

Table 1: Information to be typically collected for site assessment

ITEM	PURPOSE OR REASON	INFORMATION TO BE COLLECTED OR PREPARED	SOURCE OR REFERENCE
DESKTOP STUDIES			
Geological Maps	Useful for providing general understanding of regional and local geologic setting, lithology and geologic structure.	Maps showing regional geology, local geology; geological structures.	GNS; regional/local council; university; technical publications; consultant reports.
Site aerial and terrestrial photography	Useful for site mapping; can possibly identify changes to site if historic photos are available.	Aerial photographs (orthorectified, oblique, stereopairs); historical site photos.	LINZ; regional/local council; commercial aerial photography/survey company; museums and Council archives; Google Earth imagery, satellite imagery.
Topographic Survey	Critical for rockfall trajectory modelling.	Published topographic maps; other available topographic information.	LINZ; regional/local council; commercial aerial photography/survey company.
Reports	Useful for providing information on previous rockfall occurrence at/near site and to understand local hazards.	Regional hazard assessment; site reports; accounts of previous rockfall; highway incident/maintenance reports.	GNS; regional/local council; technical publications; consultant reports; EQC; NZTA; media reports.
Rockfall Trigger and Frequency Assessment	Critical for assessing hazard and risk; this guides selection and design of rockfall mitigation options.	Rainfall records; temperature records; earthquake records; hazard studies, newspaper articles and insurance data.	NIWA; GNS; regional/local council; Turner and Schuster (2012) [section 5.4.2]; Moon et al. (2005); AGS (2007); historical archives.
FIELD INVESTIGATIONS – SOURCE CHARACTERISATION			
Source Location and Distribution	Critical for understanding the hazard; used for rockfall trajectory modelling.	Map of rockfall source areas.	
Rock Mass Characteristics	Important for understanding the nature of the source rock; this affects block size, shape, failure modes and fragmentation.	Lithology, rock strength, weathering, joint characteristics (spacing, persistence, condition, etc).	IAEG (1981); NZGS (2005); Hoek and Bray (1981); GSL (1977)
Block Size	Critical for estimating impact load for passive RPS design.	Measurements of block sizes in-situ (outcrop); debris or talus pile; individual fallen blocks.	IRSM (1978); ONR (2013); Dorren et al. (2007)
Block Shape	Can affect run-out distance and block rotational energies generated.	eg tabular, rounded; should be documented for in-situ and fallen blocks.	Turner and Schuster (2012)
Failure modes	Important for understanding how blocks fall from the source area.	Topple, wedge, planar sliding; individual blocks vs mass failures.	Hoek and Bray (1981)

ITEM	PURPOSE OR REASON	INFORMATION TO BE COLLECTED OR PREPARED	SOURCE OR REFERENCE
FIELD INVESTIGATIONS – SLOPE CHARACTERISATION			
Slope Topography	Critical for assessing rockfall trajectory paths.	Site-specific survey; LiDAR survey high resolution LIDAR-DEM allows showing surface without the vegetation, UAV (unmanned aerial vehicle) photography with photogrammetry.	Commercial aerial photography/survey company.
Slope Morphology	Important for assessing rockfall trajectory paths; this may locally affect rockfall hazard and risk where topographic focussing and shedding features are present.	Map identifying main morphological features (eg slope breaks, drainage), potential focussing and shedding/shielding features; and potential launch features; UAV photography.	Townsend and Rosser (2012)
Slope Materials	Important for rockfall trajectory modelling.	Description and map of location of rock and soil; potential seasonal variations in slope conditions (wet/dry) should be noted.	NZGS (2005); Saunders and Glassey (2006)
Groundwater and surface water	Important as these may play a role in triggering rockfall.	Location of surface drainage and seeps.	Site observations.
Vegetation	Necessary if vegetation is to be considered in rockfall trajectory modelling.	Description and map of location and distribution of vegetation types, including tree spacing and diameter of trees.	Dorren et al. (2007); Jonsson (2007)
FIELD INVESTIGATIONS – PREVIOUS ROCKFALL			
Recent Rockfall Events	Provides important information for understanding rockfall behaviour at the site.	Location (run-out distance); block size & shape; number of blocks; evidence of rockfall trajectory (trails, impact marks); evidence of bounce height, silent witnesses.	Site observations; air photos; reports; media; anecdotal information from observers.
Historic and geomorphic evidence of rockfall and other landslide processes	Provides important information for understanding geologic processes that may affect site and the frequency of rockfall events.	Location (run-out distance); block size & shape; relative age based on final resting position (eg on top of ground or partially buried).	Site observations; dating and cosmogenics (eg Mackey and Quigley 2014).
Empirical rockfall run-out models	Useful for comparison with observed rockfall run-out distance in order to identify the extent of the hazard zone.	Slope angle measurements between source area and furthest observed fallen rock; measurements to be based on selected approach(es) (eg alpha minus beta, run-out ratio, Fahrboeschung and shadow angle methods).	Keylock and Domaas (1999); Dorren (2003); Lied (1977); Evans and Hungr (1993); Massey et al. (2012) and (2014); Heim (1932); Hungr et al. (2005); Turner and Schuster (2012)

Assessing historic rockfall where evidence may have been disturbed

It may be the case that the site assessment is being undertaken in an area where the historic evidence of rockfall has been removed (such as for land development), where it has been disturbed by human works (such as road construction) or where it has been affected by natural processes (such as landslides or rivers, which may transport rock to or from the site). When this occurs it can be difficult to ascertain some aspects of rockfall process, including block size and run-out distance. The geologist or engineer undertaking the site assessment should consider whether there is any potential for the site to have been affected by any of these types of site modification activities or processes.

Where the evidence has been removed or otherwise disturbed, alternate means should be considered for the site assessment. This may include obtaining information via any of the following:

- Historic photographs
- Anecdotal evidence (previous reports, newspaper, local residents)
- Trenching (to expose deposits) and site investigation
- Field observations in an area with similar source and slope characteristics.

In particular, where rocks may have been removed from the run-out zone, the run-out distances should be compared with estimates obtained using empirical methods, such as Fahrböschung (reach angle) or shadow angle (Evans and Hungr, 1993). This is good practice even when the evidence has not been removed, as the observed rockfall run-out distance may not necessarily be the maximum possible run-out distance.

Rockfall Risk

Quantitative risk-based approaches are increasingly being used in practice as a result of recent refinements in hazard zoning, improvements in digital technologies, and increased knowledge and records of rockfall behaviours and their triggering mechanisms.

Use of fully quantitative methods can be difficult in practice because of the costs involved, especially for agencies (such as transportation) that may be faced with a large number of problematic sites. For this reason, many transport agencies use semi-quantitative approaches that aid in setting priorities around risk mitigation actions. In situations where life or economic risk is a significant issue, such as in the Port Hills of Christchurch following the 2010/2011 Canterbury earthquakes, rigorous risk assessment methods have been used, however these are by no means the norm for all rockfall risk assessment situations.

There is always some degree of uncertainty in risk estimates, whether due to the methodology used, assumptions made and/or the time and cost investment expended. The limitations of any risk based study should be acknowledged. Understanding the approximate risk is better than not having any estimate of the risk, as this will provide important information for the decision-making process.

References: Fell et al. (2005); Massey et al. (2012); Turner and Schuster (2012)

3 ROCKFALL MITIGATION

This chapter provides an overview of a range of rockfall mitigation measures that could be considered for a particular site. More detailed information is provided for passive rockfall protection structures, as these are the focus of this document.

3.1 Mitigation Measures

Rockfall mitigation measures can broadly be categorised as engineered measures and non-engineered measures (Turner and Schuster, 2012). Engineered measures are interventions that either reduce the occurrence of rockfalls or diminish their effects; these include source stabilisation works, protection works and avoidance measures. Non-engineered measures are interventions which do not directly affect the rockfall process; these include warning signs and monitoring programmes.

Figure 3 illustrates the range of mitigation measures and where they may be employed along the slope and Section 3 provides more commentary on mitigation measures. Table 2 provides a summary of some of the more widely-used mitigation measures, together with brief commentary. A more comprehensive discussion on the range of mitigation measures can be found in Turner and Schuster (2012), as well as references in listed in Table 2.

Figure 3: Application of mitigation measures along slope profile

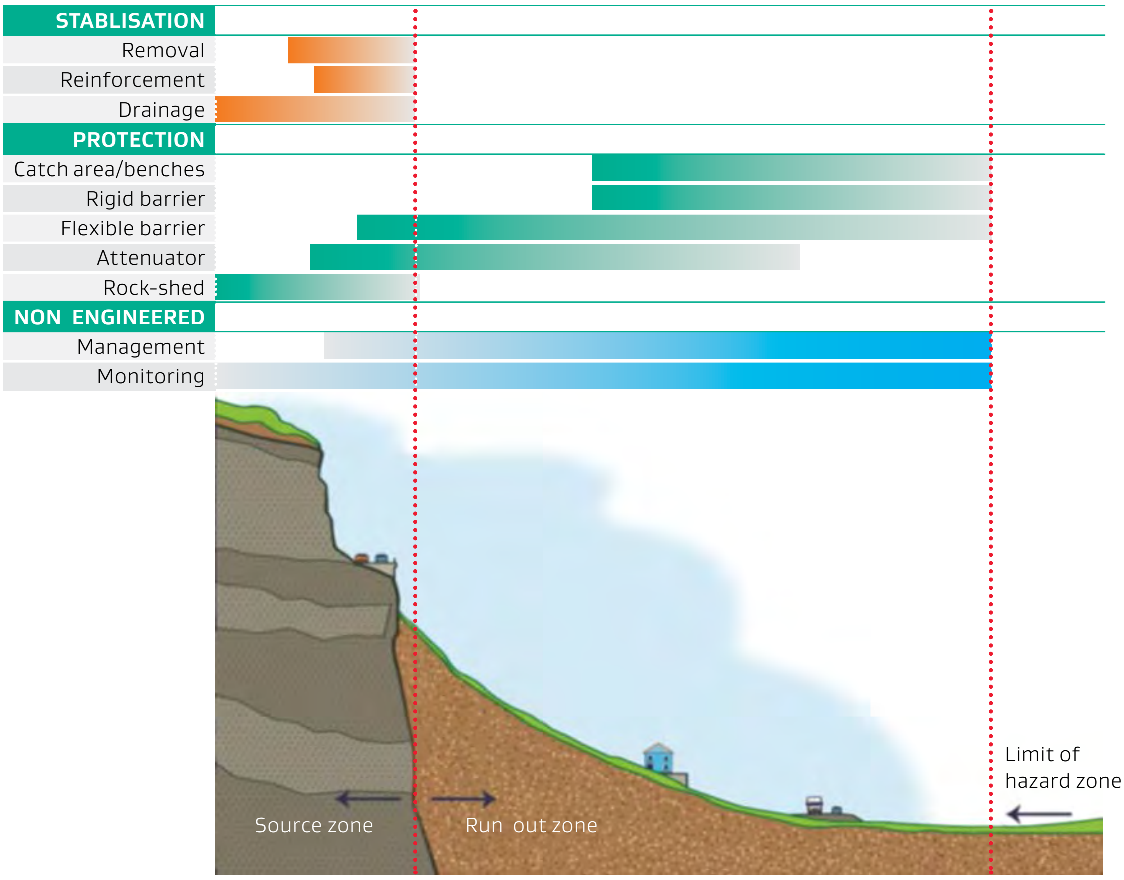


Table 2: Summary of rockfall mitigation options.

MITIGATION MEASURE	DESCRIPTION / PURPOSE	TYPES	ADVANTAGES / WHERE USED	LIMITATIONS	REFERENCES
AVOIDANCE MEASURES					
Avoidance	Situating a new facility or moving an existing facility, road or persons away from the rockfall source.	Re-position/ re-align.	Can provide permanent mitigation of the hazard. Usually only considered for areas where there is a significant exposure or for development of a new (greenfield) site.	Existing infrastructure may need to remain to maintain access/services; space limitations; relocation may not be viable or may be expensive.	Turner & Schuster (2012) [Ch. 12]
		Tunnel.		Hazards related to confined space; expensive.	
		Elevated Structure (viaduct).		Structure must completely span active rockfall area; expensive.	
STABILISATION MEASURES					
Removal	Partial or complete removal of source rock to reduce the occurrence of rockfall. May include modification of the slope profile to remove features that act as launch points for falling rock.	Scaling (hand tools, air bags, light blasting, water blasting, excavator).	Useful in areas with limited source area. Relatively less expensive.	Effective for short-term, but less effective as long-term solution. May need regular scaling programme or additional protection measures to maintain desired level of protection. May expose further susceptible rock face.	Turner & Schuster (2012) [Ch. 13]; FHWA (1989)
		Blasting (large-scale), Slope re-shaping.	Can result in a more stable rock face with reduced maintenance costs.	Potential for damage from flying rock and/or debris. Possible right-of-way and environmental issues. Debris consisting of smaller rock blocks may pose new risk, particularly for short term.	
Reinforce-ment	Secure source rock in place to reduce the occurrence of rockfall.	Dowels, Shear Pins (untensioned), Rock Bolts (tensioned).	Can be used for individual blocks (spot bolting) or for rock masses (pattern bolting).	Slope access difficulties. Effectiveness affected by block size.	Turner & Schuster (2012) [Ch. 13]; Hoek and Bray (1989); Muhunthan et.al. (2005)
		Shotcrete.	Useful for small block size, erosion protection.	Reduces slope drainage. May be affected by freeze-thaw conditions or by earthquake shaking. Visual impacts. Quality and durability affected by skill in application.	
		Buttress.	Typically for support of key block.	Height limitations. Slope access difficulties. Visual impacts.	
		Cable Lashing, Whalers, Lagging.	Useful for individual blocks.	Typically movement must occur to develop full resistance. Visual impacts.	
		Anchored mesh, cable nets.	Useful over large areas for a range of block sizes.	May accumulate rockfall debris. May change loading conditions if significant debris accumulates.	

MITIGATION MEASURE	DESCRIPTION / PURPOSE	TYPES	ADVANTAGES / WHERE USED	LIMITATIONS	REFERENCES
Drainage	Removal or reduction of surface water and/ or groundwater to reduce the occurrence of rockfall. Commonly used with other mitigation techniques.	Surface drains.	Used where surface flows affect rock face stability.	Slope access and layout difficulties. Maintenance important. Environmental issues.	Turner & Schuster (2012) [Ch. 13]
		Weep drains (boreholes).	Used in areas where groundwater affects rock face stability.	Difficult to quantify need and to verify effectiveness. Require regular maintenance.	
PROTECTION MEASURES					
Catch Areas	A shaped catch area, usually constructed at the base of a slope, that is used to contain rockfall.	Ditch/berm.	Often used along transportation corridors. Can retain large volumes. May be combined with other types of structures.	Right-of-way limitations. Large area may be required for high slopes. Requires regular clean-out and maintenance to preserve effectiveness. Material can roll through, especially in over-design events.	Pierson, Gullixson & Charrie (2001); Turner & Schuster (2012) [Ch. 14]
Barriers	Wall-type structure used to intercept and contain falling rock.	Rigid Barrier – stiff materials (concrete, timber).	Used for relatively lower energy impacts; can have small footprint area.	Stiff materials are more prone to damage by higher-energy events; typically not useful for high energy impacts.	Turner & Schuster (2012) [Ch. 14]
		Rigid Barrier – deformable materials (earthen embankment, mechanically stabilised earth wall, gabion wall).	Capable of sustaining multiple high energy impacts, depending on construction. Design life less affected in aggressive (corrosive) environment. Facing can be adapted for aesthetic requirements.	Construction limited to relatively flatter slopes (< about 20°). Can require relatively large footprint area. Requires regular inspections and clean-out. May need to consider slope stability and surface drainage issues, depending on location.	Lambert & Bourrier (2013); Grimod & Giacchetti (2013); Peila (2011); Ronco, Oggeri & Peila (2009); Brunet, et al. (2009); Wyllie (2015) [Ch. 10]
		Flexible barrier (rockfall fence).	Can be installed in difficult-to-access locations; has relatively low mass. Can be installed quickly.	Require space for downward deflection. Requires regular inspections and maintenance, including clean-out. Clean out can be difficult if installed in difficult-to-access location. Possible issue if multiple impacts per event are anticipated. Design life relatively more affected in aggressive (corrosive) environmental conditions.	Grimod & Giacchetti (2014); UNI (2012); ONR (2013); Turner & Schuster (2012) [Ch. 15];

MITIGATION MEASURE	DESCRIPTION / PURPOSE	TYPES	ADVANTAGES / WHERE USED	LIMITATIONS	REFERENCES
Suspended Mesh/ Cable Nets	Flexible wire or cable mesh structure suspended across a chute or over a rock face. Used to intercept rocks, attenuate their energy and direct them into a catchment area.	Hybrid drapery, Attenuator.	Useful for high rockfall frequency and where debris can be guided into a collection area. Required maintenance relatively smaller than for flexible barriers. Can be installed in difficult-to-access locations.	Usually requires a debris collection area. Must consider debris and snow loads on anchors. Generally limited to rock sizes less than about 1.2m, depending on mesh type. Design life relatively more affected in aggressive (corrosive) environmental conditions.	Arndt, Ortiz & Turner (2009); Turner & Schuster (2012) [Ch. 16]; Glover (2012); Wendeler C., Denk M. (2011)
Rock Sheds	Covered, usually concrete, structure used to intercept and divert rockfall. Typically used only for transportation routes.	Rockfall Shed.	Used in steep-sided valleys with high rockfall frequencies. Can deviate water flow and small debris flows. Relatively low maintenance. Typically used only for transportation routes.	Must consider downward issues. Expensive.	Wyllie (2015) [Ch. 11]
NON-ENGINEERED MEASURES					
Manage-ment	Alert users to potential for rockfall and fallen rock debris.	Warning Signs.	Inexpensive.	Users become accustomed to signs and ignore warnings. Does little to mitigate the risk unless it discourages people from traversing the area.	Turner & Schuster (2012) [Ch. 17]
Monitoring	Installation of instruments to detect incipient rockfall.	Monitoring.	Useful in remote locations (eg railroads); can be used to generate notifications or automated closures.	Can be limited lead time for events, especially in the case of heavy rainfall or earthquakes.	

3.2 Rockfall Energy

One of the key concepts in the design of passive RPS is that of the kinetic energy of the rock block as it moves downward. This energy changes along the block trajectory, increasing in free-fall and reducing after impacts with the ground. There are three types of energy that are considered:

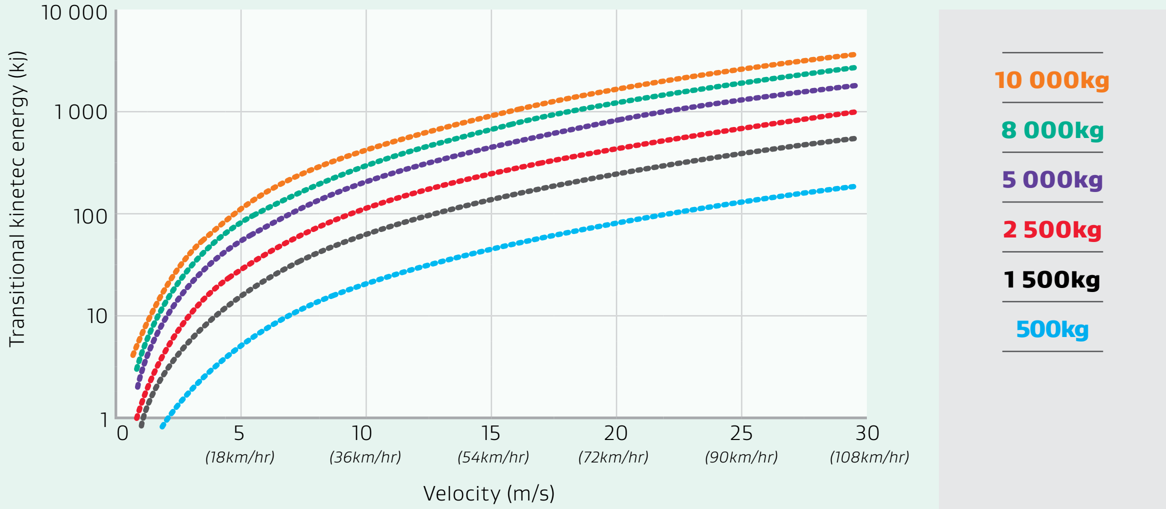
- **Translational Kinetic Energy:** energy of a non-rotating body due to its motion; this is defined in mechanics as ½ mv^2, where m is the mass of the body (in this case, the rock block), and v is the velocity, or speed at which the rock block travels.
- **Rotational Kinetic Energy:** energy due to rotation of a block; this is defined in mechanics as ½ Iw^2, where I is the moment of inertia around its axis of rotation, and w is the angular velocity (or its speed of rotation).
- **Total Kinetic Energy:** the sum of the Translational Kinetic Energy and Rotational Kinetic Energy.

For a rotating block travelling downward, the majority of its total energy comprises of translational energy; rotational energies are generally in the order of 10–15% of the total energy; however they can be as high as about 40% depending on the slope and block geometry (Turner & Schuster, 2012). The design and modelling of passive RPS typically considers only the translational energy, however it is important to keep in mind that a block with a high rotational energy can have a detrimental effect on passive RPS. For example, depending on its shape, a high rotational velocity can result in rupture of net fences, or it could cause the block to travel up and over the face of an embankment structure. For ease of quantification Figure 4 provides a graphical representation of the translational energy levels generated by travelling rock blocks of various sizes and speeds.

Rockfall energy: understanding the magnitude

Rockfall energy is an important component of passive RPS design. The graph below has been prepared to help conceptualise and give a feel for the magnitude of the energy in terms of the block size, mass and velocity. The graph shows the energy plotted against the block velocity for a range of block masses; an inset table provides an indication of the physical size of the block for the range of block masses. Rotational energy has not been considered in this graph.

Figure 4: Approximate translational kinetic energy vs Velocity of rock and boulder size



3.3 Passive Rockfall Protection Structures

Passive rockfall protection structures act to either capture a rock or to control its trajectory once it has fallen. These measures are mostly of the following types:

- Catch areas
- Rigid barriers
- Flexible barriers
- Attenuator systems
- Rock sheds.

Figure 5 shows the range of impact energies for which the different types of passive structure could be considered.

3.3.1 Catch Areas

Catch areas, sometimes referred to as "rockfall catchment areas", are engineered ditches that are designed to stop and capture falling rocks before they impact the structure at risk. These solutions are often employed along transportation routes where slope geometry and space permit. Catch areas may be combined with barrier systems, especially where there are constraints on the space available for the catch area.

Catch areas have been in use for some time and much work on the performance and design of catch areas has been undertaken in the US, particularly by the Washington and Oregon state departments of transportation (Ritchie, 1963; Pierson et.al., 2001).

3.3.2 Rigid Barriers

Rigid barriers are structures that act to either contain or deflect rockfall, with the structure being sufficiently stiff to withstand the kinetic energy imparted by the falling rock (Turner and Schuster, 2012). Rigid structures undergo relatively little to no downward deformation when impacted and they can therefore be constructed close to the assets they are protecting. There are a variety of barrier types in use today with a wide range of energy capacities depending on their materials and geometry; some examples are shown in Figure 7. The basic types of barriers are:

- **Earthen embankments (berms or bunds):** These can be constructed in a range of shapes and sizes to suit the site, and with varied internal reinforcing elements and facing materials (soil, rip-rap). They act through a combination of deformation and internal compaction to absorb the energy imparted by falling rock blocks. Depending on their construction, these structures can withstand multiple impacts with very high energy.
- **Structural walls:** Structural walls are steep-faced rigid structures that may be constructed of concrete, timber, steel or gabion baskets. They generally have a smaller footprint and cross-sectional area than an earthen embankment. Barriers constructed with stiffer materials (concrete, timber, steel) are generally suitable for lower energy impacts.

Figure 5: Range of energy capacities for a variety of passive RPS

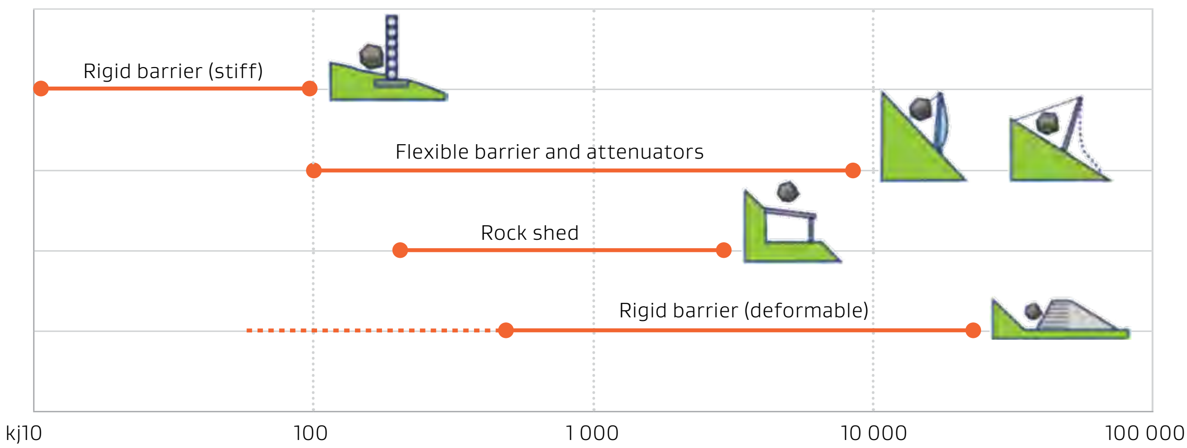
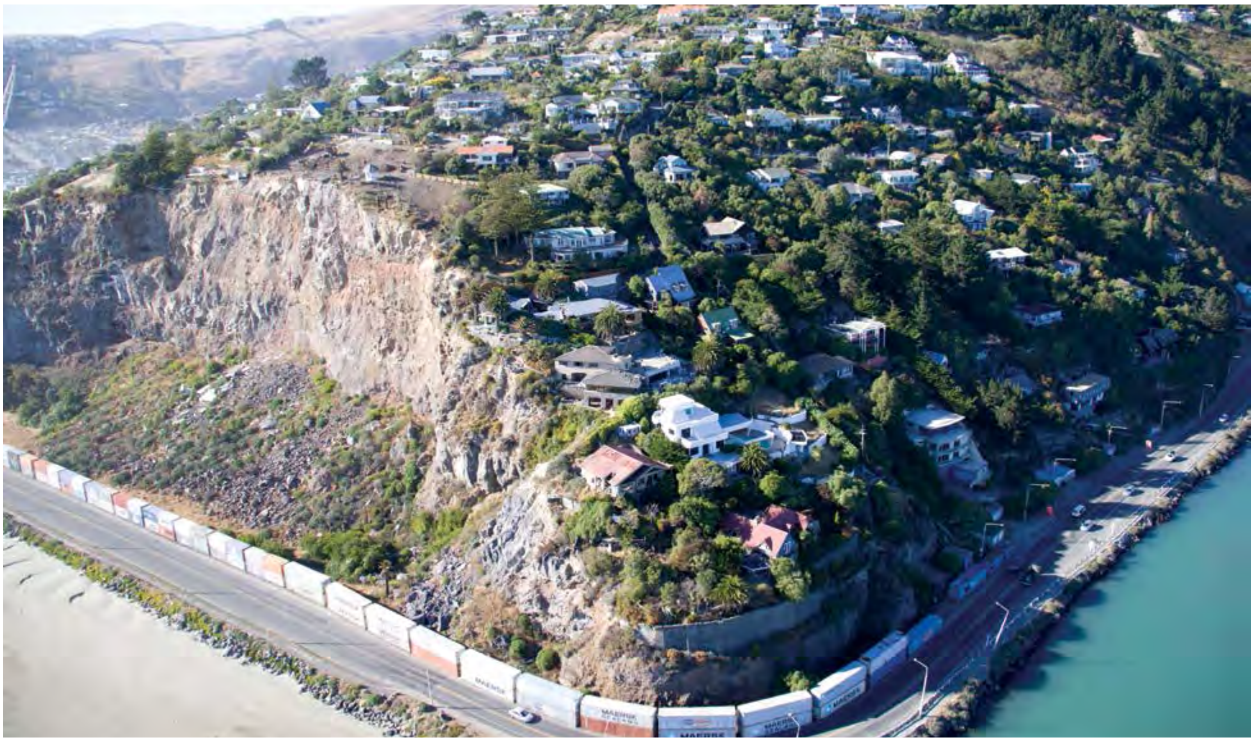


Figure 6: Catchment area with container barrier



All rigid barriers must be checked for stability, for the likelihood of the rocks to roll up and over the structure, for dynamic performance in terms of ultimate resistance (piercing or collapse due to unraveling or element failure) and for serviceability energy (maximum acceptable deformation for easy maintenance). These types of barriers are often combined with an upslope catch area that is used to contain the rock blocks. Further information on the design of rigid barriers is provided in Section 4.4.3.

3.3.3 Flexible Barriers

Flexible barriers, or rockfall net fences (fences), are lightweight structures that act to contain rockfall by deforming downward and dissipating the energy of the falling rock. A fence consists of net panel that is suspended from a series of posts and cables that are anchored into the ground; several energy-absorbing components, such as breaking elements, are incorporated into the system to help dissipate the energy and transfer it into the ground. A schematic (Figure 9) and photograph (Figure 10) of a flexible rockfall barrier are shown opposite.

It is common practice today to use fence systems that have undergone standardised physical testing to verify their capacity to stop rocks travelling with a specified energy. Most testing today is carried out using the ETAG 027 guideline, although at least one other testing guideline (BAFU) is also in use (see the additional commentary box for discussion). The testing allows the manufacturer to specify the energy level the fence is capable of withstanding, as well as give an indication of the residual height. Because of the considerable expertise, time and cost involved in developing fence systems, they are proprietary systems that are sold as kits by a relatively small group of manufacturers. A kit is a construction product consisting of several components, which are placed on the market together with one common quality certification. Currently these kit systems are available with energy ratings between 100kJ and 8500kJ, where 100kJ is the lowest energy level that can be certified using the ETAG 027 procedure.

Where lower rockfall energies (<100kJ) are encountered at a site where passive RPS is needed, the designer may choose to use a 100kJ ETAG 027-rated system, or may choose to use an alternate system that has either testing and validation to demonstrate its energy capacity; and/or design using first principles.

Figure 7a: Stiff rockfall barrier earth embankment

Figure 7b: Mechanically stabilised earthbund rockfall barrier

Figure 8: Rigid rockfall barrier (example)

Figure 9: Schematic of rockfall barrier (after EOTA, 2013)

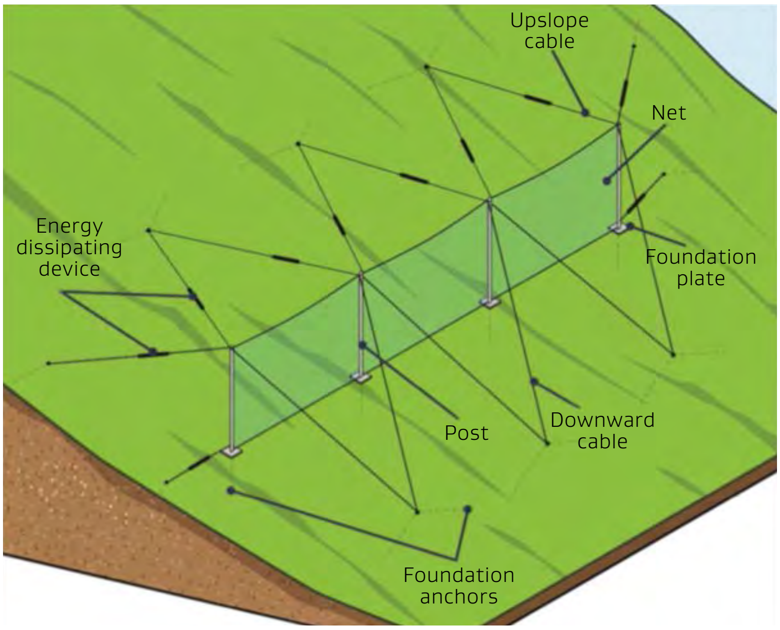


Figure 10: Flexible rockfall barrier (example)

Figure 11: Flexible rockfall barrier (driven railway irons plus cable connection)

Standardised Testing of Rockfall Fences

In order to address issues related to the variable performance of flexible rockfall fences, work was undertaken in Europe to develop guidelines for physical testing. Guidelines were originally developed by the Swiss (BAFU) in 2001 (Gerber, 2001 amended 2006) and more recently in 2008 by the European Organisation for Technical Approvals (EOTA, 2008). The guideline developed by EOTA is called ETAG 027 – Guideline for European Technical Approval of Falling Rock Protection Kits (EOTA, amended April 2013) and it is the more widely used today.

The ETAG 027 guideline covers the manufacture and testing of the individual fence components (posts, breaking element, etc) as well as the physical testing of the assembled fence kit. For a fence kit to receive a European Technical Approval according to ETAG 027, it is tested in one of eight energy classes ranging from 100kJ to >4,500kJ. The fence kit must successfully pass two tests (MEL[1] and SEL[2]) in which a boulder impacts the centre of the middle fence panel of a three-panel fence. In order to achieve reliable and repeatable results, the test is frequently carried out by means of a boulder dropped vertically, however tests carried out in inclined test facilities are also permitted.

A number of measurements are recorded during the test, which includes loads in cables and on anchors, fence deformations (deflections), residual net height and the distance between the lateral posts and the net (lateral gaps). Loads on anchors measured during the test are used by the manufacturer to specify minimum anchor capacity. Measurement of the residual net height is used to determine a classification of Category A, B and C based on the height of net following the MEL test[3].

While ETAG 027 is useful for comparing fence systems developed by different manufacturers, the procedure has some limitations related to cost and technology. That the designer must consider including:

- The system is not tested for impacts other than in the centre of the fence panel; in practice the falling rock may impact anywhere on the fence, including posts and anchor cables.
- Rotational effects are not considered, as the system is tested with a non-rotating block.
- The test does not account for the “bullet effect” in which a smaller block travelling at a higher velocity (same energy rating) could potentially punch through the net.

A list of proprietary rockfall fences that have currently achieved ETAG 027 approval is available on the ETAG website at http://valideta.eota.eu/pages/valideta/ and http://issuedeta.eota.eu/pages/issuedeta/ (note that both websites should be checked due to a change in the approval process at EOTA).

Note: that systems with retention capacity above 2,000kJ are not lightweight anymore. Up to 2,000kJ installation in inaccessible zones is possible wiht small helicopters. This is often crucial for cost effective fence solutions.

1 MEL = Maximum Energy Level. The fence must catch and stop a single MEL boulder; the residual height is measured and the fence is classified as Category A, B or C based on its residual height.

2 SEL = Service Energy Level. The fence must catch and stop two successive drops of the SEL boulder; no repairs are allowed after the first drop. (SEL ≈ 1/3 * MEL).

3 Category A is for systems where the residual height is ≥50% of the nominal (initial) height; for Category B, the residual height is between 30 to 50% of the nominal height, and for Category C, the residual height is ≤30% of the nominal height (or where a longitudinal support rope has broken). (ETAG 027 should be consulted for further explanation).

3.3.4 Attenuator Systems

An attenuator is a type of flexible fence system that is intended to slow falling rocks rather than to capture them. Attenuators reduce the energy of falling rocks by controlling their trajectory over a part of their travel path. The reduction in energy allows the falling rock to be more easily captured by other passive RPS situated downward. Attenuators are usually constructed using rated flexible barrier systems that are modified to incorporate a draped net "tail". When rocks impact the structure, they travel beneath the tail, which forces the block to impact the ground, losing energy with each impact.

An attenuator is shown in a schematic (Figure 12) and photographs (Figure 13 and 14) opposite.

Attenuator systems have been in use for about 20 years and are becoming more widely used as the understanding of their behaviour and performance improves. Their design to date has largely been based on empirical methods and judgement. Current work, including physical testing, numerical modelling and review of the performance of existing systems, is being undertaken in an effort to develop a design approach for these systems. Full-scale testing of systems has been conducted both in Europe and North America (Arndt et al., 2009; Glover et al., 2012; Wyllie & Shevlin., 2015).

3.3.5 Rock Sheds

Rock sheds, or rockfall protection galleries, are reinforced concrete roof slabs that are either covered with an energy-absorbing layer of material, or are shaped such that they deflect rockfall over the structure at risk (Figure 15). They are one of the most costly types of passive RPS and are commonly constructed to protect roads and railway lines situated below steep-sided valley walls where there are frequent rockfalls. They also provide protection against avalanches.

Much research into the design of rock sheds has been undertaken particularly in Switzerland and Japan, both of which have published design guidelines for these structures. Research is ongoing, particularly in regard to energy-absorbing materials that are used to cover the roof slab. Rock sheds are not addressed further in this document; design references are listed in Table 2.

A recently-developed alternative to rigid sheds is a flexible gallery structure (Figure 16), several of which have been constructed in Europe (Wendeler & Denk, 2011).

Depending on the circumstances several attenuators may be used to progressively slow a travelling rock block.

Figure 12: Schematic of attenuator system

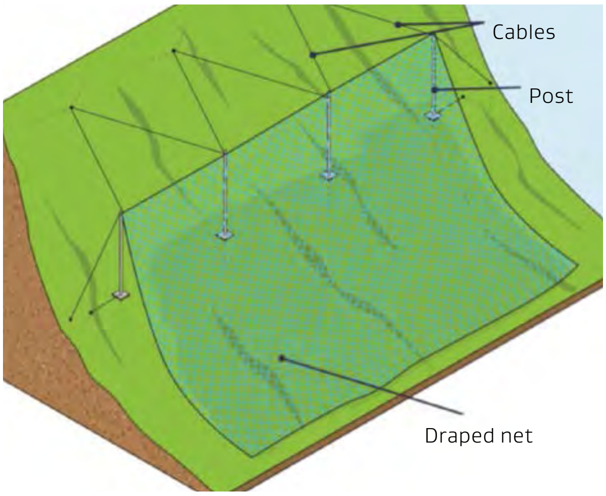


Figure 13: Rockfall attenuator system at Christchurch Gondola (photo Geofabrics)

Figure 14: Rockfall attenuator system at Diana Falls (photo Geovert)

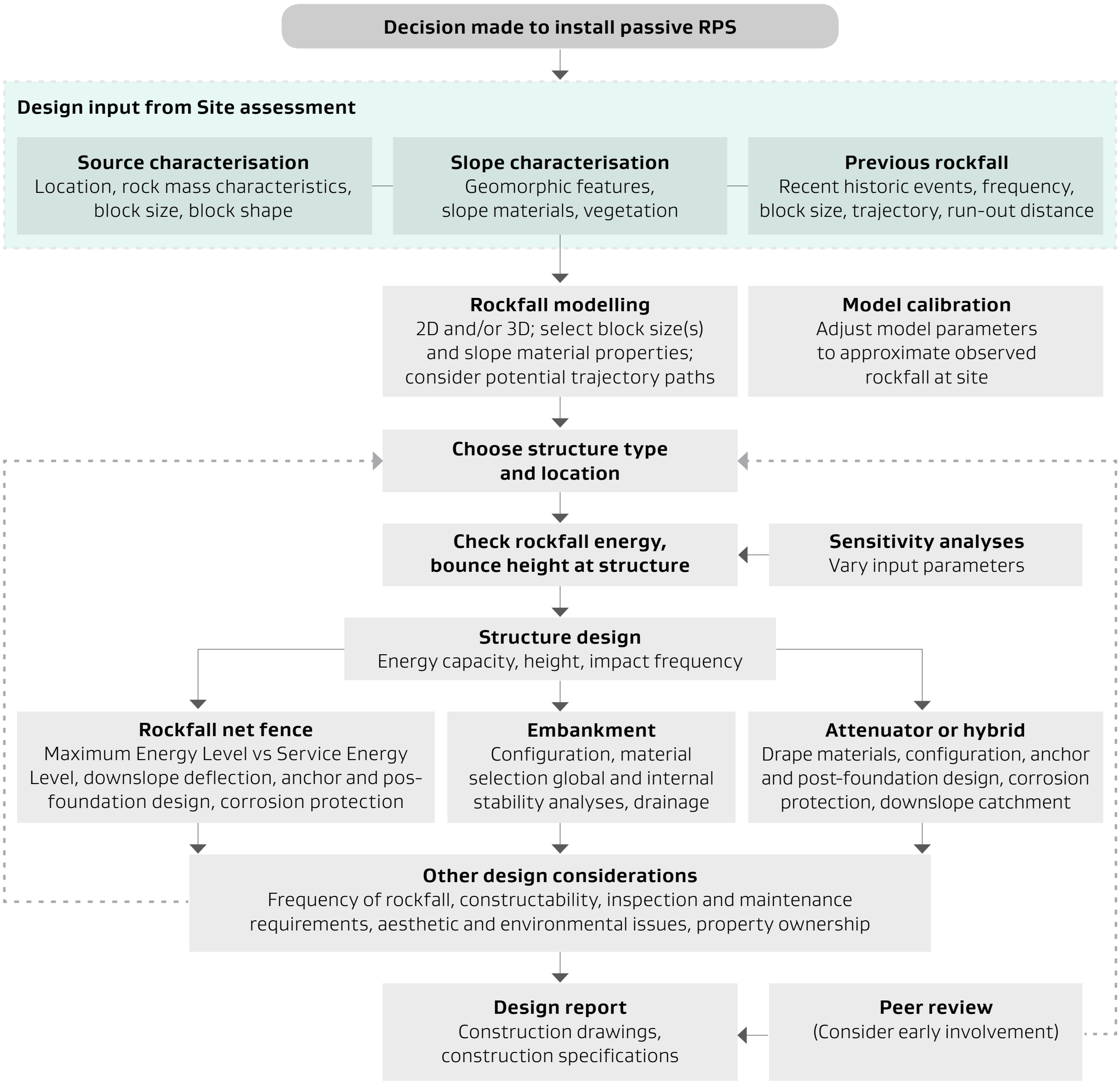
Figure 15: Rock shed at Arthur's Pass

Figure 16: Flexible gallery structure, Switzerland (photo Geobrugg)

4 DESIGN OF PASSIVE ROCKFALL PROTECTION STRUCTURES

This chapter focuses on the engineering aspects of passive RPS design. A general framework for the design process is illustrated in the flowchart in Figure 17.

Figure 17: Overview of passive RPS Design Process



4.1 Design Block

An important input for passive RPS design that is selected by the designer is the rock block size[3]. An oversized design block could result in a costly, over-engineered structure. If the design block is too small, the structure could be overwhelmed and not achieve its design objective if impacted by a larger rock block. The number of blocks that could potentially impact the structure per rockfall event – ie single or multiple[4] – also needs to be understood where feasible.

Selection of design blocks for the Passive Rockfall Protection System design is crucial. Usually three scenarios with 30, 100, 300 year recurrence are selected, each with a different design block size.

The design block size should be based on information collected from the site assessment, ideally from measurements of block size made both at the rockfall source and from fallen blocks located in the run-out zone. From a practical standpoint, it is often difficult to make accurate measurements of block size and there will naturally be some uncertainty. As a result, there will be some degree of "engineering judgement" that is used in the selection of the design block size.

In practice, consideration to assessments on rope, or with UAV or drones should be given.

The designer should consider the following when selecting the design block size:

- Block size in the identified source area/s
- The size and location of all blocks that have travelled past the potential structure location (giving consideration to whether or not the evidence of previous rockfall may have been removed or disturbed [Section 2.2])
- The size of blocks at the source in comparison with the size of fallen blocks in the run-out zone (this allows for evaluation of the tendency of the blocks to break apart as they fall – which is relatively common).

The design block should generally be among the largest of the blocks observed at the site in the wider vicinity of the feature being protected (where geomorphology is similar), acknowledging that it may be possible for a larger block to fall. When a statistically relevant number of block size measurements exists, the design block selected is commonly in the order of the 95th percentile boulder (eg ONR, 2012).

3 Velocity is also an important input for passive RPS design as it also has a significant effect on the energy, but velocity is predominately governed by the nature of the slope in the run-out zone (ie slope angle, slope materials, etc). The velocity is usually estimated via rockfall modelling, acknowledging that the designer may choose to vary the velocity estimate depending on the site conditions and their variability.

4 For the purpose of this guideline, "multiple" blocks means several individual blocks that fall during a single event.

4.2 Rockfall Modelling

Rockfall modelling is used to simulate falling rock trajectories and is commonly used to estimate the velocity (and hence energy) and bounce height of the rock block along the slope profile. This information is then used, together with factors of safety, in the process of selecting the type, location, size and energy capacity of the rockfall protection structure.

Many researchers and practitioners note the importance of using an experienced modeller for this work. Ideally this should be a geoprofessional who is experienced in rockfall issues, and who is knowledgeable about the model parameters and how they can be realistically varied in order to calibrate the model against observed rockfall behaviour (Turner and Schuster, 2012; Volkwein, 2011).

4.2.1 Limitations of Modelling

It is very important that the designer understands the limitations of attempting to model rockfall trajectories. While the approach and mathematics/mechanics for a particular software programme may vary (eg lumped mass, ability to vary block shape, definition of coefficients of restitution, etc), all of the programmes are modelling a complex stochastic process. Input data is difficult to quantify and may vary over time. Some of the complexities that give rise to uncertainties in the model include:

- **Site conditions and characterisation:**
  - measuring boulder size distribution
  - estimating size of in-situ blocks/ boulders within source area
  - identifying potentially unstable blocks
  - boulder shape
  - defining varying slope materials
  - resolution of topographic survey (eg published 20m contours vs 1m contours obtained from LiDAR survey
  - variation of modelled slope profile from the actual rockfall trajectory path.
- **Behaviour of falling rocks:**
  - unpredictable rockfall paths complicated by geomorphology (eg focussing gullies or shedding ridges)
  - boulders may break apart
  - boulders may collide
  - rotational energy (eg disk-shaped blocks)
  - boulder' shape (eg with one very short axis).

Despite these complexities, rockfall models are currently the most useful means of understanding the behaviour of falling rocks and estimating their energies and bounce heights. The designer must be aware that there are inherent uncertainties in the model output. The model output must be assessed for its reasonableness (refer to Section 4.2.5).

4.2.2 Software

Although deterministic and probabilistic approaches softwares have been developed, the most common approach to simulating rockfall trajectories to date has been to use softwares based on deterministic approach. They perform computations on selected 2D slope profiles along potential rockfall trajectories. Recent advances in computer technology and the ability to obtain detailed topographic information (eg LiDAR) has led to an increase in the use of 3D modelling. Either 2D or 3D modelling or a combination of the two may be used for design.

A list of modelling approaches and some currently available software packages for rockfall modelling is provided in Table 3, together with commentary. The table is not intended to be an exhaustive discussion of the different approaches, but instead to give an overview of the types of approaches that are typically used to assess rockfall. References to studies containing more detail on the various methods discussed are provided. Rockfall software development is ongoing and it is anticipated that additional software packages and development of applications will be available in the future. A more comprehensive list of available software packages and approaches is given in Turner and Schuster (2012).

It is the responsibility of the designer to choose the most appropriate and cost-effective analysis method based on project scope, site characteristics, and the quantity and quality of information available from the site assessment, it is also important to keep in mind that little will be gained by using a sophisticated computer model with limited site information. In some cases, this may lead to the use of a combination of both 2D and 3D modelling[5].

Table 3: Overview of rockfall modelling approaches and software

SOFTWARE PACKAGES	DESCRIPTION	COMMENT	REFERENCES
	2D APPROACHES		
RocFall, CRSP, Rockfall	Rockfall simulation software can be generally split into two types based on the "kinematic principle" adopted: i) Lumped mass; and ii) Rigid body. Lumped mass refers to motion analysis that ignores the shape and size of the rock block, while rigid body takes these factors into account. Both compute the translational and rotational velocity and energy.	RocFall (distributed by RocScience) is probably the most widely used two-dimensional rockfall assessment and analysis software. Initial versions were lumped mass, but recent versions are now hybrid (both lumped and rigid body). As this software is widely used there is a large body of literature and data from the "back analysis" of fallen rocks. Earlier versions of the software were also independently checked by the Geotechnical Engineering Office (GEO) in Hong Kong. It is still prudent to check the results from such models to those derived from the more "well established" empirical approaches such as the shadow-angle approach.	Turner and Schuster (2012)
	3D APPROACHES		
HY-STONE, RAMMS, Rockyfor, CRSP-3D	Three-dimensional lumped mass, rigid mass and hybrid rockfall modelling software is now becoming more common place in practice, especially in areas of complex terrain. The main benefit of such approaches is that the true shape of the topography, both in the rockfall source area and along the potential run-out paths, can be taken into account in the assessment process.	Modelling software will continue to develop and become more complex, and so too will the need to determine the parameters that such models require, making such approaches more applicable for detailed site-specific studies. However, it is still prudent to check the results from such models with those derived from the more "well-established" empirical approaches such as the shadow angle and 2D approaches.	Volkwein et al. (2011); TRB (2012); Vick (2015)
ArcMap, GRASS, ILWIS	GIS software is typically used for regional-scale rockfall hazard assessments, and increasingly to assess the results from two and three dimensional rockfall simulation software packages. There is a plethora of commercial and open source GIS software that covers all sectors of geospatial data handling.	GIS software has become an industry standard way of calculating, analysing and visualising rockfall information to produce hazard and risk maps. Such software can also be used to display and interrogate the results from two- and three-dimensional rockfall simulations.	Turner and Schuster (2012); Volkwein et al. (2011)

Note: for deterministic models, a significantly smaller amount of runs is usually needed in order to cover the possible spread of results.

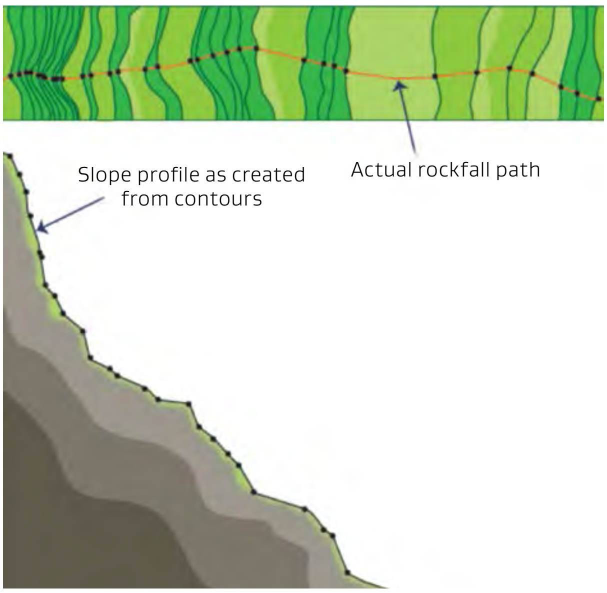
5 Discussions with practitioners indicate that when 3D modelling is used, it is often only used for complex terrain to help identify critical rockfall trajectories; from this information, one or more trajectories are commonly selected and a 2D model is then used to evaluate energy and bounce height for passive RPS design.

4.2.3 Rockfall Paths

Energy and bounce height parameters of passive RPS design are usually based on a simulation of rockfall trajectory along one or more critical rockfall paths. These parameters can be evaluated by using either 2D or 3D modelling software. The rockfall paths should incorporate the source area, the proposed passive RPS and the structure at risk. The selection of a rockfall path may be guided by distribution and frequency of rockfall paths generated by 3D rockfall (or GIS) models. However site observations and engineering judgement must be used to decide whether there may be other possible critical trajectories, especially if the site contains gullies, ridges, cliffs or launch features.

A slope profile must be generated along the selected path(s), keeping in mind that the number of points selected along the profile may impact model results. An example of a rockfall paths and slope profile selected for analysis is shown in Figure 18. At least 1000 trajectories[6] through proposed passive RPS location should be simulated along the selected rockfall path(s). If the proposed passive RPS location is at the end of the run-out path where the number of boulders intercepting the structure location is very small, then the number of trajectories may need to be increased so that the designer is satisfied that the energies and bounce heights are realistic for the site.

Figure 18: Slope profile along rockfall path (based on Dorren, 2003)



4.2.4 Slope Material Properties

Properties are assigned to various slope materials in a model usually by means of the *coefficient of restitution* (COR), however there are other approaches in use. Discussions of slope material properties and the various approaches used in their estimation are presented in Chapter 8 of Turner and Schuster (2012) and in Volkwein (2011).

The COR is a means of estimating the loss of kinetic energy that occurs when a block collides with the ground. There are usually two types of COR that are assigned in a model: normal and tangential. The normal COR represents the kinetic energy dissipation that occurs normal to the slope (as by collisions) while the tangential COR represents the kinetic energy dissipation that occurs parallel to the slope (as by rolling or sliding). There are multiple definitions for COR in use by various software packages that are computed based on either velocity or energy ratios before and after impact.

A number of references provide published values for COR for various slope materials. These values are often obtained from back-analysis at particular sites, and values may be influenced by block size and impact velocity. The published values should be used as a guide only, as they often refer to specific case histories. Differences in block size and impact velocity can affect the value of the COR, and these values are often not listed in the references.

The designer should use any published values cautiously and should be aware of how the values were developed, what definition of COR has been used in their development, and whether or not the COR definition is the same as in the selected software package used for design. The designer should always calibrate the model against observed rockfall behaviour in order to estimate COR values appropriate for the site. Model calibration is discussed in Section 4.2.5.

6 This value is recommended based on guidance from Italian standards (UNI 11211). Higher numbers of paths have not been found to significantly change the results and they could require a significant calculation effort.

Slope Material Properties: Lessons Learned in Christchurch

The predominant material covering the slopes in the Port Hills around Christchurch is loess. Seasonal variations (wet winters and dry summers) strongly affect the moisture content of the loess and its material properties. Differences in boulder run-out distances were observed for rockfall that occurred in summer and winter months – in summer the loess is dry and hard, resulting in longer run-out distances; in winter, the ground is wet and soft due to increased rainfall, resulting in relatively shorter run-out distances.

Figure 19: Example of boulder run-out affected by wet ground conditions in loess, Port Hills, Christchurch

4.2.5 Model Calibration

Nearly all of the literature that discusses rockfall modelling stresses the importance of calibrating the rockfall model against actual observed or inferred[7] rockfall that has occurred at the site (Turner and Schuster, 2012; Volkwien, 2011). Calibration involves altering the slope model parameters (coefficients of restitution, slope roughness, etc) in order to match or approximate observed boulder run-out distances and bounce heights. It may be necessary to run two different calibrations – one for run-out distance and one for bounce heights.

The actual rockfall behaviour on the slope can be evaluated using a variety of means, including the following:

- Observations of evidence from recent rockfall (fresh scars at source, loose rocks, talus, impact marks, vegetation disturbance, fresh tree scars, run-out distance)
- Observations of evidence from historic events (tree impact marks, run-out distance to fallen rocks or talus deposits)
- Observations made during rock-rolling experiments carried out at the site (noting that this may not be feasible because of safety issues or the presence of downward infrastructure and assets).

The actual and modelled rockfall run-out distance should also be checked against empirical models and published data, such as Fahrboeschung and shadow angles (refer Turner and Schuster, (2012).

An example of rockfall model calibration that was carried out in the Port Hills (Christchurch) using observed boulder trails for earthquake-generated rockfalls is presented in Massey et al. (2012).

4.2.6 Sensitivity Analyses

Once the rockfall model is calibrated, sensitivity analyses should be performed by varying model parameters in order to investigate the effects of changes on the boulder energies and bounce heights. Model parameters will depend on the software being used, however the following should be considered for sensitivity analyses:

- block size
- slope discretisation

7 Where evidence of previous rockfall may have been disturbed or removed.

- slope material types/locations (including vegetation, if considered)
- block shape, and
- initial conditions (including block velocity/ acceleration – *note that modelling experience suggests that effects of initiation velocities tend to become non-critical once boulders have impacted a few times on the slope*).

4.2.7 **Vegetation**

Vegetation may be present on the slope in a range of types, sizes and densities. It may act to retard rockfall energies and bounce heights by reducing the travel velocities, however it is very difficult to account for (quantify) the effects of vegetation with confidence (Turner and Schuster, 2012).

The designer should consider very carefully whether or not to include the effects of vegetation in the model either in part or full, as the characteristics of the vegetation may change over time and it could be completely or partially removed by either natural (die-back, fire) or intentional means (harvesting).

If vegetation is to be considered in the analysis, the model should be calibrated against known rockfall trajectories through vegetated areas. At a minimum, sensitivity analyses should be carried out for slopes with and without vegetation to understand the impacts of the vegetation and its potential removal.

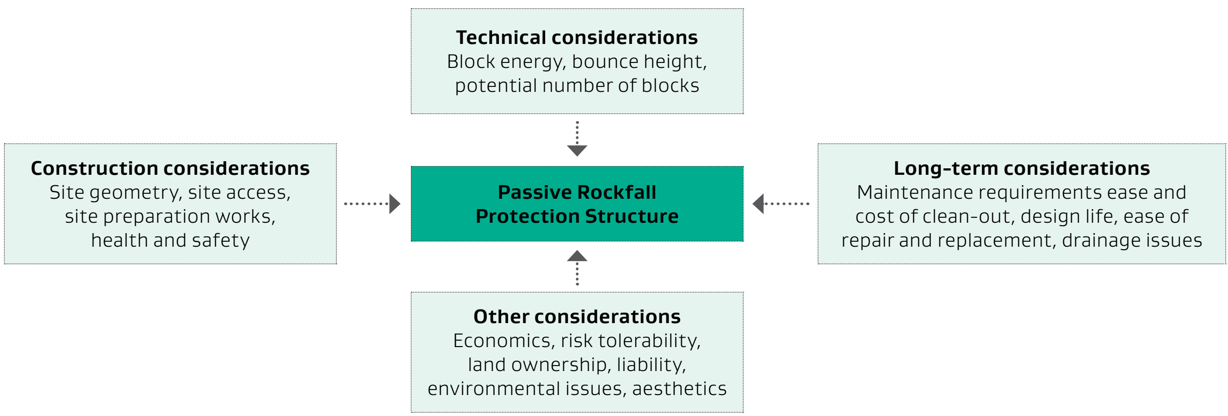
Much work has been undertaken in Europe in an effort to quantify the effects of forests on rockfall trajectories; this work is described in Dorren et al. (2007) and Jonsson (2007).

4.3 Structure Location, Type and Length

The selection of passive RPS location and type should be considered in parallel, as the choice of location may influence the type of structure best suited to that location. The selection process will depend on a number of factors that may include those illustrated in Figure 20. This is not considered an exhaustive list. Where there are multiple options for location and/or passive RPS type, the decision may be best evaluated using options or optimisation studies that consider the factors listed below.

Generally the first step in this process is to choose potential structure locations. This should be done considering the results from the calibrated rockfall model run using the design block. Ideally the structure will be situated where the block energy and bounce heights are relatively lower in comparison with energy and bounce profiles along the slope, as this will minimise the size of the structure. This location could be close to the source (before the block gains significant energy) or it could be toward the lower limit of the run-out zone where the slope is relatively flatter (after the block has lost much of its energy). Additionally it could take advantage of some topographic feature (such as a natural or man-made bench) that tends to cause a reduction in block energy and/or bounce height.

Figure 20: Factors affecting selection of passive RPS type and location



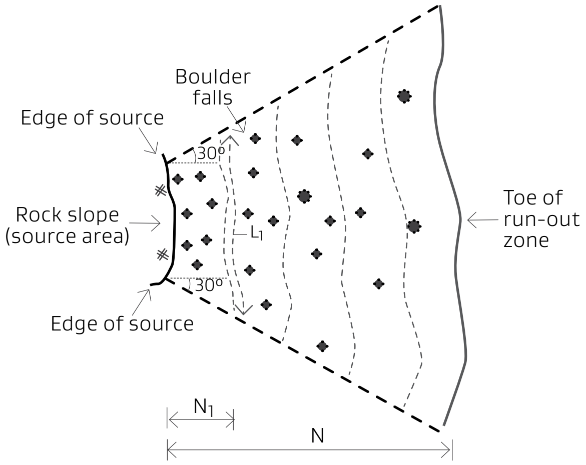
Outside of the technical considerations of block energy and bounce height, the following considerations may also affect the choice of passive RPS location:

- **Required length:** the length of the structure will depend on where it is situated with respect to the rockfall source and the area being protected and the possible spread laterally across the slope of the boulder run-out
- **Site access:** both temporary access for construction and permanent access for clean-out, maintenance and replacement
- **Drainage** and overland flow paths
- **Legal issues:** property boundaries and land ownership; passive RPS should also not deflect rockfall across property boundaries where it has the potential to cause damage
- **Constructibility:** construction company and manufacturer of the system should be involved at earliest stage of design.

Once one or more locations are selected, the type of structure best suited to the location(s) can then be considered. Some of the more important factors to be considered in selecting structure type are the site geometry (particularly slope angle), the energy and bounce height, and the anticipated frequency of rockfall impacts (for structure maintenance and clean-out considerations).

A discussion of the various passive RPS types, and their advantages and limitations, is summarised in Table 2, with more detailed discussions included in the associated references. The design considerations for the various structure types are discussed in more detail in Section 4.4. Figure 20 shows the spread angle of boulder falls, after GNS report CR-2011-301.

Figure 20: Spread angle of fallen boulders observed, after GNS report CR-2011-311, C. Massey, 2011



Once the structure location and type are selected, the rockfall model(s) should be re-run in order to confirm energies and bounce heights at the structure location. It is likely to physically re-visit the site prior to finalising the passive RPS location in order to fully consider the suitability of the site in relation to its effectiveness for potential rockfall paths, as well as for consideration of installation, maintenance, access and safety issues.

4.4 Design Philosophy

The design of rockfall protection structures is a field that is evolving. Design approaches are either relatively new (in the case of rockfall fences and embankments) or are under development (in the case of attenuator systems). Part of the reason for this is the complicated response of the structure when subjected to the dynamic impact load from a falling rock.

The design guidance provided herein is based on currently available approaches developed mostly in Europe by practitioners and researchers who manufacture, construct, test and use the systems. The approaches described have been developed using a combination of physical testing and numerical modelling of controlled tests or real-world impacts on existing structures. The design approaches are being updated as the understanding of the behaviour and performance of these systems improves.

Many of the passive RPS addressed in this document are proprietary systems that are designed and tested by individual manufacturers at approved testing sites using standard testing methodologies. It is common for the manufacturer or supplier to offer support to the designer for their particular system. The local designer is ultimately responsible for the design of the passive RPS. The designer needs to use their knowledge of the site and NZ practices to lead the design, supplementing the process with relevant information and experience from the manufacturer.

The designer must choose an approach that is suited to their particular site, assessed risk and structure; it may be useful to use multiple approaches and compare the results from each. No matter what method is used, it is important that the design basis and the rationale behind the decisions made are clearly explained in the Design Features Report, refer Section 6.

4.5 Flexible Barriers

The design approach for flexible barriers, or rockfall net fences, is perhaps the most well-developed of the three types of passive RPS addressed in this document, with design guidance recently being published by the Austrian Standards Institute (ONR, 2012)[8] and Italian Standards Institute (UNI, 2012). These documents are likely to be further developed as more information about the performance of fences designed using these approaches, becomes available.

For rockfall net fences, the main design aspects that need to be considered include the following:

- The design approach eg Maximum Energy Level (MEL), Service Energy Level (SEL) or an alternative bespoke design approach
- Sizing of the fence (both for energy capacity and height)
- Downward deflection of fence
- Design of anchors
- Design of fence post foundations
- Corrosion protection
- Capability of local construction contractors to implement a design.

4.5.1 MEL or SEL Design Basis

In the first instance, the designer must decide whether to design on the basis of Maximum Energy Level (MEL) or Service Energy Level (SEL) loading conditions. Appendix A clarifies how MEL and SEL design approach aligns with NZS1170.5. These are terms introduced in the ETAG 027 testing guideline and their use in design is further described below (Peila and Ronco, 2009). Only one approach should be followed:

- **MEL Design.** The barrier is designed to withstand one impact with the design energy; it is expected that the barrier will need to be repaired (or in the worst case replaced) following impact. This approach may be applicable to situations where anticipated frequency of rockfall events is low. MEL corresponds approximately to the Ultimate Limit State (ULS) in terms of NZS1170.5.
- **SEL Design.** The barrier is designed to withstand multiple impacts with the design energy; it is expected the barrier will require limited to no repair following an impact. This may be applicable for areas where the anticipated frequency of rockfall events is high, or where the fence is installed in a location that is difficult to access for maintenance work.

The energy ratings listed by manufacturers are their MEL design load. For a barrier with an energy rating of 3000kJ, the design block for an MEL design would have an allowable energy of 3000kJ, while the design block for a SEL design would have an allowable energy of 1000kJ.

4.5.2 Flexible Barrier Size Selection

The sizing of the barrier for both energy capacity and bounce height should consider the results of rockfall modelling together with observations made during the site assessment. The general approach proposed by ONR (2013) and UNI (2012) is to apply partial Factors of Safety (FoS) on various input and output parameters from the rockfall modelling. These partial factors account for uncertainties in the input assumptions and also the fact the ETAG 027 certification of a structure was performed in ideal conditions and assumes (Grimod and Giacchetti, 2014):

- the rock block impacts centrally in the net system in all cases
- only one block impact at MEL and two at SEL
- symmetric construction profile over 30m in length.

A summary of the design process as outlined in the above references is provided in Table 4. The designer should consult the references[9] for a more detailed discussion, explanation and current recommendations for partial Factors of Safety.

Most of the literature refers to the difficulty in estimating bounce height and the limitations of rockfall models for estimating bounce heights. It is important that the bounce heights be investigated in the modelling

8 These are national standards recently developed in Austria and Italy for use in those respective countries.

9 At the time of publication, these documents are only available in Italian (UNI 2012) and German (ONR 2013). The Italian approach is referenced in Grimod and Giacchetti (2014). It is understood that an English translation of the ONR document will be made available when the updated version of the document is released, which is anticipated in late 2015. Both documents are currently under revision.

through model calibration and sensitivity analyses. Evidence of bounce heights from the site assessment must also be considered if this data is available from recent rockfall events.

For proprietary systems with ETAG 027 approvals, the barrier kits are offered with a standard height that has been used during the testing. There is some variance allowed for slight modifications to the barrier height up to about 1m, depending on the barrier class. If the designer wishes to modify the barrier height, the fence manufacturer should be consulted to confirm the allowable height change is consistent with the assumed capacity.

Table 4: Comparative summary of design approaches for flexible barriers

PARAMETER	UNI 2012 (Italian)	ONR 2013 (Austrian)
Design Block Size	Design block is selected on the basis of "engineering judgement" from the site assessment and considering the nature of the risk.	Use a 95th to 98th% block size depending on the rockfall frequency; larger block sizes are used for higher frequency events.
Factor of Safety – Rockfall modelling	Partial FoS are applied to various parameters (boulder mass, velocity and energy) based on the level of detail of site assessment and confidence in the input parameters used for the rockfall simulation.	Partial FoS are applied to the boulder energy and manufacturer fence energy rating based on the Damage Consequence Class*. **Qualitative measure of human, economic, social and environmental impacts; generally it is based on the type of infrastructure at risk (eg a hospital or a rural road).*
Rockfall Model Output – Energy	Boulder energy is computed using the 95%ile translational velocity; *Energy* = ½m v^2 (m = mass; v = translational velocity) Partial FoS applied to energy based on type and use of infrastructure at risk.	Uses 99%ile translational kinetic energy.
Rockfall Model Output – Bounce Height	Uses 95%ile bounce height from the rockfall simulation plus a minimum clearance height. Partial FoS are applied based on the level of detail of site assessment and confidence in the input parameters used for the rockfall simulation.	Uses 95%ile bounce height from the rockfall simulation. Partial FoS are applied based on the Damage Consequence Class.
Barrier Energy Rating	Partial FoS applied as a reduction factor to the specified barrier energy rating.	Partial FoS applied as a reduction factor to the specified barrier energy rating.

4.5.3 Barrier Deflection

Flexible rockfall barriers can undergo considerable deflection when impacted by rockfall. The potential deflection of the fence in the downward direction needs to be checked against the location of the particular asset being protected. The Italian standard (UNI, 2012) recommends applying a FoS between 1.0 and 1.5 to the maximum elongation of the fence (as measured during the ETAG 027 testing) depending on the design basis (MEL or SEL) and the number of functional fence modules (panels).

4.5.4 Anchor and Post Foundation Design

The anchors connecting the barrier posts and cables into the ground are the main mechanism for load transfer to the ground; the anchors are normally installed through a concrete plinth and base plate. Anchor loads are specified by the manufacturer based on the loads measured directly during the ETAG 027 test for a particular system. The designer, rather than the manufacturer, is responsible for providing a design and construction specification for the ground anchors. Guidance may be provided by the supplier, but it should be verified by site specific design.

In general, a Factor of Safety should be applied to the anchor loads provided by the manufacturer, and proof tests should be performed to confirm the performance of the anchors. Subsurface investigations may be required to confirm rock type and depth to bedrock, and a pull-out test on a trial anchor may be needed to confirm design grout – ground bond strengths. If the anchors or foundations are installed in rock, then discontinuity orientations should be checked to evaluate the potential for the formation of potentially unstable blocks that might move under anchor loads. Group effects may need to be considered depending on the anchor spacing.

The Factor of Safety selected for design and the number of proof tests conducted will depend on the potential variability of site conditions (rock type, quality, depth to bedrock, etc) and the previous experience of the designer with the particular site conditions (eg if they have previously installed many anchors in similar ground and are confident about the anchor performance, then a lower Factor of Safety and lower numbers of proof tests may be selected).

There is no New Zealand standard for anchor design, however reference is made in the New Zealand Bridge Manual to the following international standards/guidance for design and installation of ground anchors:

- BS 8081:2015, Code of practice for ground anchors[10]
- BS EN 1537:2013, Execution of special geotechnical work – ground anchors
- FHWA-IF-99-015, Ground anchors and anchored systems.

Fence Post Foundation Systems

It has been observed recently that some fence post foundation systems have been designed with very large, reinforced concrete plinths and deep micro piles. This is particularly the case where the foundations have been designed according to methods used for standard building foundations that are intended to resist loads while undergoing minimal displacement. These large foundations can be difficult and costly to construct and repair or replace.

Recent observations and experience with post foundations indicates that they can sustain some vertical and horizontal displacement while still remaining functional; in this case, the foundation deforms with the barrier, aiding the energy dissipation of the barrier system. This is currently a topic of discussion amongst international practitioners.

It is anticipated that in the future designs will incorporate fence post foundations that are constructed without a concrete plinth. This is already recommended by the Austrian agency for forest and avalanches (WLV). This will result in foundation systems that are easier and less costly to install without negatively impacting on the overall capacity of the passive RPS system.

Reference: Giacchetti et al., 2014.

Additionally Eurocode 7 (BS EN 1997-1:2004+A1:2013) provides guidance on design of anchors.

It is recommended that the anchor design be carried out using the procedures recommended in one of these documents. It is noted, however, that research and testing is currently underway in Europe on the performance of anchors under different loads, including typical shock loading received from barrier impacts. This research is not yet published, but it may affect the anchor design methodology for flexible barriers in the future.

4.5.5 Corrosion Protection

Outside of rock impacts and foundation design, the design life of the structure will be governed by degradation due to corrosion of the fence components. Appropriate corrosion protection for the installation location must be applied to the fence and components (AS/NZS 2312 and AS/NZS 4534-2006). The fence manufacturer can provide information on corrosion protection coatings, their design life and maintenance requirements.

4.6 Deformable Rigid Barriers

Reinforced earth embankments are the most-widely used type of deformable rigid barriers in use in current practice (Lambert, 2013), noting that there are a wide variety of materials and configurations that have been used in their construction (Peila, 2011). As such, the focus of this section is on these types of structures.

Lambert (2013) describes a number of approaches that have been used for embankment design that have been used to date. A number of these approaches are based on determining the penetration depth of the block into the embankment, and this is the approach that is described herein.

Embankments dissipate the kinetic energy of falling rocks (Grimod and Giacchetti, 2013) via:

- **Plastic deformation:** approximately 80–85% of the energy is dissipated by formation of an impact crater in the upslope face of the embankment,
- **Friction loss:** approximately 15–20% of the energy is dissipated via sliding along layers involved in the impact; and

10 This reference from the Bridge Manual has been updated to reflect the 2015 update of BS 8081.

- **Elastic deformation:** approximately 1% of the energy is dissipated through settlement of the soil grains at the impacted surface.

Design guidance has recently being published by the Austrian Standards Institute (ONR, 2012) and Italian Standards Institute (UNI, 2012). It is understood that the Austrian guidance relating to embankments is currently under revision, with anticipated release in late 2015.

The design of embankments involves determining the embankment geometry, which is based on the design block diameter, energy and bounce height. These values are used to estimate the penetration depth using a series of charts that have been developed from numerical modelling and physical tests.

The calculation for impact capacity of a given structure is carried out using static loads to substitute for the impact loads. Those static load are calculated based on the conservation of momentum principle and taking into account a coefficient of absorption relative to stiffness of the bend.

Designs are based on either the ultimate limit state (ULS) or serviceability limit state (SLS), which are defined as follows (Grimod and Giacchetti, 2013):

- **Ultimate Limit State:** The maximum resistance of the embankment, without collapse; collapse is taken as when the centre of gravity of the deformed section of embankment lies outside of the embankment structure (deformation >50% of embankment thickness at impact height). The embankment will require substantial repair or replacement following an impact by the design block.
- **Serviceability Limit State:** The maximum deformation permitted to allow easy repair of the structure; this is taken to be no more than 20% of the embankment thickness at impact height (and usually no more than 50–70cm) on the upslope site; and usually no more than 30–40cm of deformation on the downward side.

The main design aspects to be considered for embankments are summarised in Table 5 and illustrated in Figure 21.

Figure 21: Schematic of rockfall protection embankment (after Wiley, 2015; Grimod and Giacchetti, 2013)

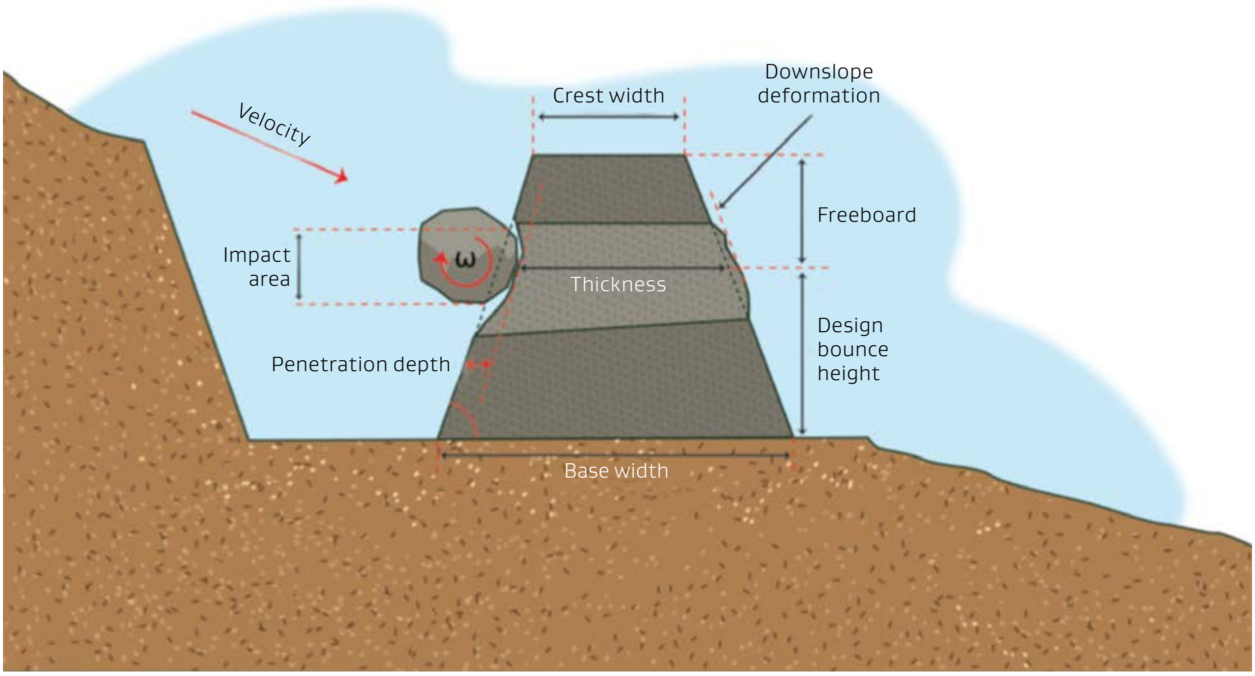


Table 5: Embankment design considerations

DESIGN FEATURE	CONSIDERATION
Design block energy and bounce height	Same approach used for net fences, embankments and attenuators (Section 4.2).
Foundation bearing capacity	Foundation bearing capacity based on vertical load from the bund over the width of the footprint. The bearing capacity should be checked against soil rupture as well as settlement.
Embankment height	A freeboard above the design bounce height must be included. This height provides a normal force that acts above the impact area and generates shear resistance to the punching load (Wylie, 2015). Where space is limited, a net fence may be used to extend the height of the embankment so that it can catch the infrequent higher bounce rock or larger volume without substantially increasing the footprint of the structure.
Embankment width	Embankment width is governed by a minimum top crest width of >1.0m and a minimum width at impact of >2 times penetration depth.
Upslope face	The upslope face angle should be as steep as possible to minimise the potential for blocks to roll up and over the structure. The potential for a block to shatter should be considered in the selection of the upslope facing material.
Materials	– Facing materials (type, ease of repair) – Backfill materials (gradation; min/max particle size; permeability; pH) – Placement requirements (compaction).
Stability Analyses	Need to undertake the following stability analyses: – Internal stability of embankment (static & seismic loading) – Global stability of slope and embankment (static, seismic and impact loading) – Dynamic resistance to penetration on the upslope face – Dynamic resistance to sliding of the downward face.
Durability and Reparability	The facing element of the protection embankment must be durable to resist environmental exposure. It should be easily patched under the SLS condition; or re-constructed in the affected section under the ULS condition.
Upslope ditch	This may be incorporated into the design and may include a layer of energy-absorbing material. Rockfall simulations should be re-run to account for any changes to the slope profile upslope of the structure.
Service road	This should be incorporated to allow for regular maintenance, including removal of fallen rocks from behind the embankment. The geometry should allow access for equipment suitable for rock removal.
Drainage	Drainage through and around the structure must be considered to minimise the potential for ponding, erosion and instability issues.

4.7 Attenuators

There are currently no published design approaches for attenuators. Designs to date have been undertaken using an empirical approach that is based on limited field testing and observation (Arndt et al., 2009; Wyllie D., Shevlin T., 2015; Glover et al., 2012). Table 6 summarises design considerations.

Table 6: Design considerations for attenuator systems

DESIGN FEATURE	CONSIDERATIONS
Design block energy and bounce height	Same approach is used for net fences, embankments and attenuators (Section 4.2) .
Materials	Usually constructed using an ETAG 027 (or other) rated fence system with alterations made to the netting configuration to create a tail that can be draped over the ground downward of the structure.
Energy capacity	Follows a MEL-based design for rockfall net fences; the dynamic loads in the structure are reduced since the rock block is not retained by the structure.
Height	Same approach as for rockfall net fences.
Drapery design	Need to consider material type (weight, durability) and length. This should take into account the slope over which the drape will rest (slope angle, roughness) and potential slicing forces that could result from the rotation and bouncing of the block. The intent of the drape is to force additional collisions between the boulder and the ground, while allowing the boulder to exit the drape. If the drape is not properly designed, material could be retained beneath the drapery (affecting its future performance) or it could exit with little reduction in the boulder energy.
Anchors and Fence Posts	As for rockfall net fences, noting that the anchor loads will usually be less than for a rockfall net fence. The load imposed by the weight of the drapery may need to be considered for design of the fence post spacing.
Corrosion Protection	As per net fence design. The design life will be affected by the frequency of rockfalls intercepted by the structure and climate conditions (eg proximity to sea).
Downward containment structure	May be a ditch or other type of passive protection structure.

While an ETAG 027 (or other) rated system should be used for attenuator systems, the system in this configuration does not have an ETAG 027 (or other) rating as the system is not intended (nor designed) as a barrier to stop falling rocks.

4.7.1 Catch Areas

Catch areas have been widely used for some time, particularly along roadway corridors, and design guidelines are available (Pierson et al., 2001). They are often used in combination with other types of passive RPS. The main design features for catch areas are summarised in Table 7.

Table 7: Design considerations for catch areas

DESIGN FEATURE	CONSIDERATIONS
Location	Usually located at the base of a slope, however may be located in upslope or mid-slope areas depending on the nature of the source area, the slope geometry and where the at-risk facility is located on the slope. The location should consider access for debris removal.
Width	Should allow for the initial impact and subsequent roll-out of the rock. The height and geometry of the slope above the ditch affect the width.
Shape	Commonly has a gentle back-slope and a depth to minimise potential for rocks to roll out of a ditch.
Substrate	A soft, loose material such as uncompacted sand or pea gravel may be used to help attenuate the energy of falling rock, noting that choice of substrate may be influenced by access for maintenance.

4.8 Inspection and Maintenance

Regular inspections and maintenance are integral to maintaining optimal performance of the passive RPS for its intended design life. The frequency of inspection will depend on the structure type, anticipated frequency of rockfall events, and general site conditions. As part of the passive RPS design, the designer should prepare an inspection and maintenance programme that provides for the following:

Table 8: Inspection and Maintenance Considerations

TYPE OF INSPECTION	DESCRIPTION	SUGGESTED FREQUENCY
Regularly Scheduled	This should include removal of any accumulated debris, vegetation, inspection of mechanical components (eg posts, cables, anchors, netting, brakes, etc) and corrosion protection, and inspection and cleaning of any drainage works.	1–5 years, depending on anticipated rockfall frequency and corrosivity of environment.
Post-Event	This should include inspection of all components for damage, clean-out of any debris, and repair or replacement of any components damaged by the rockfall. The passive RPS should be returned to its intended design capacity following the maintenance.	As required by rockfall or trigger event.
Source Area	This should include visual inspection of potential rockfall sources to assess whether there have been any changes in condition, such as loosening or movement of blocks.	5–10 years, or as warranted following rockfall or trigger event.

If a proprietary system is installed, then a recommended inspection and maintenance programme may be provided by the manufacturer.

The designer should provide a maintenance manual to the asset owner. The manual should detail the recommended inspection and maintenance programme and include proforma for undertaking and documenting regularly scheduled and post-event inspections and maintenance. It should also list any reporting that may be required as part of the consent conditions.

Maintenance costs

The cost of maintenance over the lifetime of the structure is an important consideration when selecting a mitigation option. The cost will depend on the type and size of structure, its design life, the ease of access for repair and debris removal, and the rate of debris accumulation (rockfall frequency). These costs can be considerable, particularly if regular debris removal is required at a site with difficult access.

As an example of how lifetime maintenance costs have been considered in design elsewhere, CalTrans (California Department of Transportation) has found that for their installations, maintenance costs for rockfall net fences increase significantly when impact energy levels reach or exceed 1000kJ. As a result, they rarely install rockfall net fences with capacities greater than 1000kJ. Instead, they consider alternative mitigation measures to address rockfall issues at sites where energies in excess of 1000kJ are anticipated (Turner and Schuster, 2012; CalTrans 2014).

Note: this example is not intended to imply that this particular practice adopted by CalTrans should be used elsewhere. The decisions around use and selection of passive RPS depend on a variety of factors that need to be considered on a case-by-case basis.

5 REPORTING

The work undertaken to investigate and assess rockfall issues, and to select and design passive RPS should be documented in one or more reports. The information contained within the reports may vary depending on the particular site issues, the scope of the project, and the stages in which the work is undertaken.

Information that should be prepared and presented for both technical review and consenting is summarised in Table 9. This list is not comprehensive and is intended as a guide only. Not all of the studies may need to be undertaken if the information is not relevant to the site.

Table 9: Summary of information to be included in reports

FACTUAL INFORMATION	
Desktop Studies	– Topographic survey information (list type and source) – Aerial photographs – Published geological maps – Historical information about the site, including any previously observed rockfall information – Previous site reports (list and briefly describe relevant content) – Trigger assessment (seismic, climate, other) and an estimate of rockfall frequency – if feasible.
Site Assessment	– Site description (geological setting, slope characteristics) – Engineering geological map (delineate source areas, slope materials, vegetation, run-out areas, location and size of fallen rock, location of structures at risk) – Source rock characterisation (rock type, strength, jointing, block sizes) – Slope materials characterisation (soil type and properties, rock type and properties plus any climatic dependency) – Results of any field or laboratory testing undertaken – Water conditions (surface and subsurface) – Vegetation (type and size).
RISK INFORMATION	
Risk Assessment and Analysis (not covered in this document)	– Description of approach selected and risk criteria – Estimate probability of rockfall impacting persons/structure at risk – Identify and evaluate consequences of rockfall impact – Decision on whether mitigation is required.
DESIGN INFORMATION	
Evaluation of Mitigation Options	– Identify range of possible mitigation options for site – Cost-benefit considerations – Constructability issues – Selection of most feasible mitigation alternative – Performance criteria.

DESIGN INFORMATION	
Design	– Selection of design block size, including basis for selection – Rockfall analyses: software used, Input criteria (block size(s), slope profile, slope materials); results (energy, bounce height) – Selection of optimal passive RPS size and location – Stability analyses (slope, foundation, earthworks) – Anchorage design – Corrosion protection – Design life – Plans and sections illustrating: location of structure at risk, rockfall protection structure, analysis sections, other relevant information.
CONSTRUCTION INFORMATION	
Construction Drawings	– Site plan (including details of slope morphology and rock mass or rock source as appropriate) – Structure plan and cross-section(s) – Foundation/Anchorage details – Other construction information (site access, drainage diversion) – Vegetation/landscape plan – Legal boundaries.
Construction Specification	– Nature of the ground where structure is to be constructed – Materials – anchors; corrosion protection, grout, mesh, cable – Anchorage testing requirements – Materials specification (type and testing) – Construction sequence – Installation – Stressing and acceptance testing – Hold points.
As-Built Information	– As-built site plan and drawings – Product certifications – Asset owners' operations and maintenance manual.
OTHER INFORMATION	
Inspection and Maintenance Plan	– Requirements for regular inspections and maintenance – Requirements for post-event inspections.
Safety in Design	– Documentation of Safety in Design assessment, including statement of how the outcomes of the assessment have been incorporated into the design, construction and operational phases.
Additional Information that may be required	– Environmental Impacts (flora and fauna) – Environmental impacts caused by works (at-source works, changes to drainage, sediment control) – Visual impact assessment – Archaeological assessment – Iwi consultation.

5.1 Building Code Compliance Pathway

From a Code compliance perspective, it is the building consent applicant's responsibility to confirm that the consent application establishes the capacity of the passive RPS to absorb the impact energy from falling boulders. This is done via:

a an agreed design and construction methodology such as outlined in Section 4 of this document; and

b a design features report documenting the design philosophy and the accompanying building plans and specifications (refer Section 5 of this document).

The application would then be considered by the building consent authority and follow the normal building control processes for engineered structures.

For passive RPS one means of demonstrating compliance with Clause B1 of the Building Code is:

- Adopting an accepted design standard such as ONR24810 (ONR, 2013) or UNI11211-4 (UNI, 2012) discussed in Section 4.4.2 of this document; and
- specifying a structure that complies with an internationally established quality and load testing system such as the European ETAG 27 quality system.

Note: This requires the designer to determine the design load or energy requirements and then specifying a structure that has a CE certificate confirming it meets the ETAG 27 requirements for the specified level of rockfall impact energy. It is also noted that all elements of the passive RPS will need to be covered by the design including (for flexible passive RPS) anchor points, anchor capacity and foundation details, as these aspects may not be covered by the ETAG 27 certificate.

Table A1 summarises the key points made above.

Table 10: Means for demonstrating Building Code compliance

SYSTEM	ACCEPTED MEANS FOR DEMONSTRATING COMPLIANCE WITH CLAUSE B1
Flexible Barrier	– Designed using an internationally accepted design standard such as ONR24810 or UNI11211-4; and – Specifying a proprietary system with for example ETAG 027 approval (or equivalent) and installed as per the manufacturers recommendations – Other system evidence derived from physical tests.
Attenuator/ Hybrid	– Refer Section 4.4.4 of this document – The attenuator system design uses ETAG 027 certified (or equivalent) components – Other system evidence derived from physical tests.
Bund/ Embankment	– Designed using an accepted design standard such as ONR24810 or UNI11211-4; and/or based on technical information published in a recognised technical journal which has been peer reviewed. Note: Will need to demonstrate other issues such as slope stability and surface water are adequately addressed. – Other systems: Such as numerical modelling of block/structure interaction (eg Finite element) using generally accepted principles.

6 OTHER DESIGN CONSIDERATIONS

Additional considerations need to be taken into account as part of the design and construction of passive RPS. This is not a comprehensive list and site conditions may require that other design considerations be made.

6.1 Health and Safety

Rockfall due to its nature poses inherent health and safety issues that must be considered throughout the site assessment, design, construction and operation stages of the project. In particular, health and safety plans should be developed for site assessment work and these should carefully consider the rockfall hazards posed to staff who are undertaking field work within the rockfall hazard area. If the hazard is significant, it may be the case that remote sensing methods (eg aerial photography, UAV) are used, or that physical works (eg scaling, temporary protection measures) are undertaken to reduce the hazard to workers. The same considerations apply for construction activities undertaken in the rockfall hazard area, noting that the exposure time of workers to the hazard is likely to be increased during passive RPS construction.

Under the Health and Safety at Work Act 2015, everyone (including owners, designers and contractors) will have an obligation to manage risk, consult and coordinate to improve safety. Including a "safety in design" approach to the passive RPS lifecycle process will assist in meeting the obligations of this Act. The purpose of safety in design is to integrate risk identification and assessment techniques early in the design process to eliminate or, if this is not reasonably practicable, minimise the risks to health and safety throughout the life of the structure being designed; the structure life includes construction, operation, maintenance and disposal. More discussion is provided on this subject in Appendix A.

6.2 Legal and Property Ownership Issues

It may be the case, particularly if the passive RPS is being installed for protection of a residential structure, that property boundaries could impact the optimal selection, design and location of a passive RPS. For example, the best site from a technical perspective may be located on land that is not owned by the party seeking to install the passive RPS. In this case it may be necessary to engage in discussions with other property owners (who may include private individuals or government entities) or to re-locate the structure to a different (and perhaps less-than-optimal) location. If the passive RPS is located near a property boundary, care needs to be taken so that rocks are not deflected by the passive RPS onto neighbouring properties. Issues related to construction access, legal easements, maintenance access, and liability may also need to be considered.

The potential complexities that could arise in regard to legal and property ownership issues should not be underestimated and these should be identified at the beginning of the project.

7 REFERENCES

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APPENDIX A. **REGULATORY CONSIDERATIONS**

The following discussion is an overview of the building regulatory system that is relevant to the design of passive rockfall protection structures (passive RPS) and is targeted at building consent officials but may be of interest to passive RPS designers.

A1 Overview of the Regulatory System

The regulation and performance of buildings in New Zealand sits under the following three-part framework (MBIE, 2014):

1 The Building Act, which contains the provisions for regulating building work.

2 The various building regulations, which include amongst other items the New Zealand Building Code (the Code), prescribed forms, list of specified systems, definitions of 'change the use' and 'moderate earthquake'.

3 The Building Code, contained in Schedule 1 of the Building Regulations 1992, which sets performance standards all new building work must meet, and covers aspects such as stability, protection from fire, access, moisture, safety of users, services and facilities, and energy efficiency. Verification methods and/or acceptable solutions are provided for all Code clauses and are one way, but not the only way, of complying with the Code.

A1.1 **Building Act**

Passive RPS are buildings for the purpose of the Building Act (BA). BA principles outlined in section 4 (s4) of the Act and other sections particularly relevant to the design and construction of rockfall protection structures include:

- All building work needs to comply with the Building Code, whether or not a building consent is required (s 17)
- Buildings need to be durable (s 4(2)(c))
- Under the Building Act a rockfall protection structure is regarded as a building (s 8)
- The whole-of-life costs of a building (including maintenance) need to be considered (s 4(2)(e))
- The importance of standards of building design and construction in achieving compliance with the Building Code (s 4(2)(f))
- Other property needs to be protected from physical damage resulting from the construction, use, and demolition of a building (s 4(2)(j))
- Owners, designers, builders and building consent authorities each need to be accountable for their role in obtaining consents and approvals, ensuring plans and specifications for building work will meet the Building Code (s 4(2)(q))
- The building consent authority must have "reasonable grounds" to grant a building consent (s 49)
- Buildings with specified intended design lives (s 113).

A1.2 **Building Code**

The New Zealand Building Code sets out the performance criteria to be met for all new building work. The Code does not prescribe how work should be done but states how completed building work and its parts must perform. Aspects covered include stability, durability, protection from fire, access, moisture, safety of users, services and facilities, and energy efficiency.

The Building Act provides a number of pathways that designers may follow to achieve compliance with the Building Code:

1 Acceptable solutions provide a prescriptive means of meeting the Code. If followed by the designer, the designer must be granted a building consent as they are deemed to comply with the Code. This is the simplest path but is not available for rockfall protection structures.

2 Verification methods provide a prescriptive design method, which if followed by the designer will produce a design that is also deemed to comply with the Code. This path requires more scrutiny than designs that follow an acceptable solution to check that correct assumptions are used and that any calculations used in the design are correct. Again, there is no specific verification method for rockfall protection structures and therefore this path is not available for the consenting of rockfall protection structures.

3 Product certification (CodeMark) obtained by product suppliers to demonstrate their product meets the performance requirements of the Code. This is a possible path for manufacturers of rockfall protection structures to pursue. To date there are no Code Marks issued for rockfall protection structures in New Zealand and therefore this path is not available for the consenting of rockfall protection structures.

4 Alternative solutions whereby designers demonstrate to the satisfaction of the building consent authority (BCA) that a design solution, not covered directly with an acceptable solution or verification method, does achieve the performance requirements of the Code. Demonstration may include fundamental engineering design and expert review, history of use, or testing of the design or product. If it can be demonstrated to the BCA that the performance criteria are achieved, the BCA must grant a building consent. Presently this is the only available pathway to demonstrate Code compliance for passive RPS.

The following extracts from the Building Code are particularly relevant to the design and consenting of rockfall protection structures.

Clause B1 (Structure) and clause B2 (Durability) of the Code describe the required building deformation performance in service when subject to frequent load events.
In particular the following clauses are relevant:

- **B1.3.1** – *Buildings, building elements* and *site* work shall have a low probability of rupturing, becoming unstable, losing equilibrium, or collapsing during *construction* or *alteration* and throughout their lives.
- **B1.3.3** – Account shall be taken of all physical conditions likely to affect the stability of *buildings, building elements and site work*.
- **B2.3.1** – Building elements must, with only normal maintenance, continue to satisfy the performance requirements of this code for the lesser of the specified intended life of the building, if stated, or the life of the *building*, being not less than 50 years.

Clause A1

8.0 Ancillary

- **8.0.1** – Applies to a building or use not for human habitation and which may be exempted from some amenity provisions, but which are required to comply with structural and safety-related aspects of the building code. Examples: a bridge, derrick, fence, free-standing outdoor fireplace, jetty, mast, path, platform, pylon, retaining wall, tank, tunnel or dam.

A passive rockfall protection structure is covered by the ancillary building definition above and therefore needs to comply with the structural and safety-related aspects of the Code. A key difference to the above list of examples of ancillary buildings is that the passive RPS is generally installed to protect an asset below the passive RPS from a travelling rock block (boulder) impact load, with boulders travelling in some cases at high velocity and therefore containing a significant amount of energy. In addition, the intended life of the passive RPS may need to be specified due to the protective role of the structure and the uncertainty of durability (refer Section A1.4 below).

A1.3 Linkage between Building Code and the Passive RPS design

The design of a conventional building uses AS/NZS1170 and NZS1170.5 to determine SLS and ULS loadings. In particular; NZS1170.5 is used to determine the seismic load.

The design of an passive RPS differs from the normal aspects of a building design in that the loads it is designed to withstand are largely independent of forces applied by seismic (that generates the initial rock block movement), wind and snow/ice loads and therefore AS/NZ1170 Parts 1 to 4 and NZS1170.5 design loadings do not apply directly to the design of passive RPS. The exception to this is the case of an earth bund where seismic loads must be considered in the slope stability analysis of the bund.

As described in Section 4.4.2.1, the MEL and SEL design approaches correspond approximately to ULS and SLS design load cases in terms of AS/NZ1170 Parts 1 to 4 and NZS1170.5.

Except for typically low energy rigid passive RPS structures, passive RPS are also different in that they are designed to yield plastically both at rockfall impact loads lower than the design ultimate load and at the ultimate limit state load. The plastic deformation of the passive RPS under impact loads greatly improves the ability for the resisting system to sustain significant loads by reducing the forces applied to a passive RPS. The trade off in this instance is that passive RPS may require maintenance, repairs or replacement subject to the size and type of impact.

A1.4 Specified Intended Life

In terms of the Code durability requirements, the required life of the building of at least 50 years may not be appropriate for a passive RPS, because of the nature of its construction and the vulnerability of its structural system to corrosion.

Hence, the building consent applicant and the territorial authority should consider a specified intended life under clause 113 of the Building Act by defining the intended life by some alternative means. For example, the specified intended life of a passive RPS could be specified as the minimum of:

- X years (*for corrosion reasons for example*), or
- a single significant rockfall impact or several less significant rockfall impacts as determined by specified maintenance inspection regime (*Note: due to the reduced ability to take further rockfall impacts after some plastic deformation has occurred following a rockfall impact event*).

In addition, the consent applicant should consider specifying an inspection regime so as to monitor compliance with the building consent and address when repairs/maintenance is required or the passive RPS should be replaced altogether. The Code specifies that durability is based on "normal" maintenance being carried out, so what is proposed must be reasonable for any current and future owner to understand and accept. It is recommended that, as a minimum, inspections are undertaken annually and/or after a significant rockfall impact event. Further discussion on this issue is provided in Section 4.5 of this guidance[11].

A2 Other Regulatory Considerations

A2.1 Hazard Notices (sections 71–74)

A 'hazard notice' issued under the Building Act is only required when a building is on land that is subject, or likely to be subject, to a natural hazard. The natural hazards are listed in section 71 of the Building Act, and include falling debris. The BCA will have to assess whether the natural hazard is 'likely' to occur. The Court's guidance on this section is that this requires a common sense approach involving considerations of fact and degree (*Logan v Auckland City Council (2000)* 4 NZ ConvC 193, 184(CA) at [33]).

The informal test of a 1/100 year trigger event resulting in a natural hazard having to have more than temporary and minimal effect on the property is a useful rule of thumb[12,13].

A 1/100 year return period event equates to approximately a 40% probability that it will occur over the 50 year design life of a building. The challenge is that the return period for rockfall is very difficult to estimate as it can be caused by a number of different triggers including rainfall, freeze/thaw effects, human activities, natural weathering processes and (like Canterbury) significant seismic events. If there is a documented history of rockfall at a particular location and rockfall debris that can be mapped and dated, then this can help quantify the return period of occasional events (TRB, 2012 and Mackey et al. (2014)). Smaller seismic events with or without rockfall may also provide a guide as to minimum rockfall return periods. What is clear is the >6000 falling boulders generated in the Port Hills by the Canterbury earthquake sequence, particularly in the February 2011 aftershock event, was probably in the order of a one in 7000 year event (Mackey et al. (2014) and Vick (2015))[14]. The Canterbury experience would also suggest larger

11 Appendix F of AS/NZS 4534 – 2006 (Zinc and Zinc Aluminium alloy coatings on a Steel Wire) provides guidance on how to determine design life in different atmospheric conditions.

12 DBH 2012 – Department of Building and Housing (2102) Guidance to CERA and Canterbury Building Consent Authorities to repair and rebuild houses on land potentially subject to inundation, particularly with respect to the application of sections 71 to 74 of the Building Act 2004.

13 DBH (Department of Building and Housing) Determination 2008/82 Building consent for a storage shed on land subject to inundation at 58 Brookvale Lane, Taupaki.

14 According to Vick (2015), ". . ...prior to the CES, historical rockfall events in the Port Hills have been infrequent, and minor in their impact. According to written (mainly newspaper) records over the past 100 years, there have been several historic rockfall events causing minor damage to public and private property, and only slight inconvenience to livelihoods; most are associated with urbanisation of the hill suburbs and none were earthquake-generated directly. Seismicity was considered a contributing factor in only two events, where recent (at the time) distal earthquake events combined with prolonged storm events were thought to be a trigger. . .. . .".

seismic events can generate numerous simultaneous boulder falls from the more jointed/fractured rockfall source areas in close proximity[15]. Clusters of similar aged historic boulders may serve as a useful guide to this possibility occurring at a particular site.

Where a property is subject to a natural hazard, a hazard notice is not required if adequate provision is made to protect the land.

The factors that a BCA should weigh when considering if the land was subject to a natural hazard would be:

a Whether the land is intimately associated with the building. Generally what needs to be considered is whether the unprotected land above the passive RPS is used in the normal course of events as part of the use of the protected land below the passive RPS.

b The amount of land affected if the rockfall occurred after the passive RPS was in place; and

c The extent to which the land would be affected by the rockfall, and the steps that would be required to remediate the land.

A2.2 Producer Statements

Producer statements are used for design and construction purposes to assist BCAs to establish compliance with the Building Code and the Building Act. Producer statements have no statutory status under the Building Act 2004; however they remain in common use today. The Ministry of Business Innovation and Employment (MBIE) has issued guidance on issuing and accepting certificates of design work, producer statements and design features reports during the building consent process[16].

IPENZ has also provided some guidance on the use of producer statements which can be accessed via the following link.

http://www.ipenz.org.nz/ipenz/forms/pdfs/PN01-Guidelines-on-Producer-Statements.pdf

A2.3 Resource Management Act

Territorial authorities may have planning rules that require the installation of an passive RPS to have resource consent depending on the particular circumstances of the project. Conditions attached to resource consent are another mechanism to manage the maintenance aspect of a passive RPS if it is not covered by the terms of the building consent.

A2.4 Building Consent Exemption

Building consent exemption would be unusual; however, there may be some circumstances where a building consent may not be required by a TA. Schedule 1 of the Building Act provides the TA with the discretion to waive the requirement for a building consent if it is satisfied that the building work is likely to comply with the Building Code (BA Schedule 1 Section 2). An example of when this could occur is when, in the BCA's[17] opinion, sufficient documentation has been submitted as part of a resource consent application to demonstrate compliance with the Building Code.

If an exemption is provided, the structure must still comply with Building Code requirements (BA Section 17).

A2.5 Local Government Official Information and Meetings Act 1987 – Inclusion of information in a LIM

Section 44A of the Local Government Official Information and Meetings Act 1987 requires any special features or characteristic of the land to be included in a land information memorandum (LIM), including information about potential falling debris. This is a much lower test than "likely" in Section 71 of the Building Act. Information about potential hazards that affect a property should be included in the LIM if it relates to that property.

15 The term "boulder flux" was commonly used in Canterbury to describe this effect and was generally associated with the collapse of a rock bluff or bluffs above and into a valley shaped topography immediately below the bluff.

16 Guidance on the use of Certificates of Work, Producer Statements, and Design Features Reports by Chartered Professional Engineers under the new Restricted Building Work regime. www.building.govt.nz

17 Note: Some functions under the Building Act (BA) are the responsibility of territorial authorities (TAs) and others are the responsibility of building consent authorities (BCAs), refer BA Section 12. This difference allows for private organisations, if accredited and registered, to be BCAs but with restrictions on what functions they can undertake. Notwithstanding this difference, currently only TAs are accredited as BCAs. Only a TA can exempt work from requiring a building consent.

A3 Safety in Design

The purpose of safety in design is to integrate risk identification and assessment techniques early in the design process to eliminate or, if this is not reasonably practicable, minimise the risks to health and safety throughout the life of the structure being designed; this includes maintenance and disposal stages.

The core elements of safety in design are:

- **Persons with control:** persons who make decisions affecting the design of products, facilities or processes are able to promote health and safety at the source. This includes designers, owners and builders.
- **Risk management:** identification of risks to the health and safety of persons over the life of the structure, assessment of these risks and implementation of design treatments to eliminate, or where not practicable, to minimise the risks to health and safety of persons through designed control measures.
- **Lifecycle:** consideration of reasonably foreseeable risks that may occur during construction, operation, inspection, maintenance, repair, modification, replacement and removal of the structure.
- **Consultation:** consult, cooperate and coordinate activities with all other persons who have a work health or safety duty in relation to the design. This could include construction contractors, owners, end users and decision makers.
- **Information transfer:** effective communication and documentation of design and risk control information between all persons involved in the phases of the lifecycle.

High level considerations for applying safety in design to passive RPS are summarised in the table below. This is not a comprehensive list and site conditions may require that other design considerations be made. More detailed discussion of safety in design can be found in the Health and Safety Reform Bill (2014), IPENZ (2006) and NZTA (2014).

Table A1: Safety in design considerations

LIFECYCLE STAGE	SAFETY IN DESIGN CONSIDERATIONS AND POTENTIAL HAZARDS
Site hazard assessment and design	– Falling rocks/boulders during inspection mobilised by natural (climactic/seismic) or human activities (works) – Lack of emergency site egress or avoidance space (eg in the event of rockfall during inspections).
Construction[18]	– Upslope hazards (rockfall) mobilised by natural (climactic/seismic) or human activities (works) – Site access (operation on steep slopes, loss/blockage of access) – Remote sites (rope, helicopter access, weight of passive RPS parts) – Site egress or avoidance space (eg in the event of rockfall) – Downward effects (land instability, temporary drainage, rockfall impacting people, structures, infrastructure) – Disturbance to nearby structures – Construction materials (availability, ability to transport to site and constructability).
Inspections and Maintenance	– Upslope and downward hazards (rockfall, erosion) – Ability to maintain permanent site access – Equipment that can readily be used for vegetation and rockfall debris clearing and removal.
Repair / Replacement	– Removal of rockfall debris and other debris – Removal and replacement of damaged passive RPS elements.

A3.1 Additional References

The following may be useful resources when considering Health and Safety considerations in passive RPS design:

Safe Work Australia. (2014, July). Guide for Safe Design of Plant. Australia: Safe Work Australia

Health and Safety at Opencast Mines, Alluvial Mines and Quarries (2015) Worksafe NZ

18 The paper by Grimod et al. (2014) "Certified deformable rockfall barriers: tests design and installation" has a useful discussion on design and construction issues associated with difficult to access and hazardous sites.

APPENDIX B. ROCKFALL PROTECTION STRUCTURE DESIGN WORKED EXAMPLE

The following is intended as a worked example for design of passive rockfall protection structures in accordance with the MBIE guidance document Rockfall: Design Considerations for Passive Structures. The relevant section of the guidance will be referred to for easy cross referencing where appropriate. The document provides the following information as examples:

- An assessment of the engineering geology that could be undertaken to describe the geological setting, topography of the site, rock mass characteristics of the source area and failure mechanisms
- Rockfall Assessment, including the nature of past rockfall, rock size and runout distribution, potential initiating factors,
- Rockfall Modelling, including 2D and 3D assessment, model calibration, distribution of rockfall endpoints, bonce height and energy.

At this point, the worked example progresses two parallel Rockfall Protection Structure (RPS) designs:

1 A geogrid reinforced earth bund at relatively high level on the slope, subject to relatively high bounce heights and impact energies, and

2 A proprietary rockfall fence located at relatively low level on the slope and subject to lower bounce heights and energies.

For each RPS design, the document then provides examples of:

- Design energies (MEL/SEL for flexible RPS or SLS/ULS for earth bunds),
- Sizing (dimensioning) of the barriers;
- Worked examples of the designs for each barrier option, including typical drawings and design calculations.

> **Note**
>
> The worked example uses a hypothetical location, based on an existing slope in Christchurch. Certain elements of the worked example are fictitious. For example, the geology of the rockfall source has been adjusted to provide a demonstration of the information that could be collected during site assessment.
>
> The worked example does not provide an example risk assessment, as the form of this may vary considerably depending on the client, project objectives, location and consenting issues.

B1 Site Description

The subject site lies on gently sloping land between approximately 125m and 300m laterally from a series of rock bluffs (Figure B1). The site is currently undeveloped.

> **Note**
>
> For the purposes of this assessment, let us assume that the property owner plans to subdivide and develop the area, and that the intent of the rockfall protection works design is to present two options for the owner to assess cost vs benefit scenarios (Construction costs are outside of the scope of work of this document but may be a significant consideration for the client).

The slope above the property rises from around 38m above sea level (masl) at the eastern boundary of the property to around 280masl at the level at the rockfall source. Vegetation on the slopes above the property is restricted to grasses. A mature shelter belt is located on the slope below the rock bluffs; however, the belt is mostly to the south of the expected source area for boulders affecting the subject property.

The effect of the Shelter Belt has been ignored for rockfall modelling purposes in the worked example. Depending on the width and type of vegetation, vegetation may have a significant effect on boulder runout distance and energy. Practitioners should consider vegetation effects both for risk assessment and boulder trajectory modelling purposes.

Figure B1: Site Location and Extent (indicated contours at 5 m interval)

B2 Engineering Geological Assessment

B2.1 Geological and Topographical Setting

The site is underlain by rocks of the Lyttelton Volcanic Group, typically comprising dark grey to black Basalt to grey & Green Trachyte. The Lyttelton Volcanic Group rocks are overlain by dark grey Hawaiite to Trachyte interbedded with red-brown pyroclastic deposits of the Mt Pleasant Formation.

Overlying the both volcanic groups in the lower parts of the slope is wedge of Loess Colluvium which typically comprises Silt with occasional volcanic rock blocks. The Colluvium is anticipated to have a thickness of up to around 5 m. On the upper slopes wind-blown Silt (Loess) is apparent) and is typically less than a few metres thickness.

The lower part of the slope in the vicinity of the site is typically inclined at around 15° over a slope distance of 210 m and is typically a relatively even surface. Above this, the ground slope steepens to around 36° over a slope distance of around 135 m. In this upper part of the slope, there is significant variation in the ground surface over a short distance as a result of low height rock outcrops, which tends to result in an undulating ground surface.

The rockfall source comprises two distinct areas as shown on Figure B2 and Photograph B1. The upper source comprises a continuous rock bluff located approximately 175 m laterally from the upslope side of the property and is approximately 14 m high. The lower source is located approximately 125 m laterally from the upslope side of the site. The lower source is typically more distributed compared to the upper, and comprises a zone approximately 40m in height and is typically sloped at 45°.

An approximately 4 m wide access track has been cut into the slope immediately above the property. A significant gully feature is apparent to the north of the site and has a typical width of around 20 m and is up to approximately 5 m deep.

B2.2 Site Walkover Assessment

During site assessment the following observations were made:

- The rock bluffs above the site consist of two distinct volcanic materials as follows:
  - The upper bluff line typically comprises subrounded to rounded reddish brown vesicular basaltic blocks within an ash matrix. The basalt blocks are typically strong[1] to very strong, however the surrounding Ash matrix is typically very weak rock. This rock mass appears to be a pyroclastic flow, derived from an explosive eruption episode. For the purposes of this report, we have referred to the upper bluff lines and pyroclastic material as 'Source Area 1'. Defects within the pyroclastic material are typically very widely spaced with variable orientation, and are persistent up to around 10 m.
  - The lower rock bluffs consists of very strong dark grey porphyritic basalt. The basalt ranges from vesicular to non-vesicular depending on location in the outcrop. Defects within the basaltic rock mass are typically widely to very widely spaced and are moderately steep to steep. Two broadly orthogonal persistent (up to 5 m) defect sets can be recognized within the basalt rock mass, and are likely to represent cooling joints. A third set can also be observed, which typically has a variable orientation and persistence. The basalt exposed in the lower bluff line is interpreted to be derived from a sequence of lava flows which underlies the pyroclastic material. The lower basaltic rock bluffs have been termed 'Source Area 2' in this report.
- A number of fallen boulders are apparent on the slopes above and in the vicinity of the proposed site as shown on Figure 2 (following page). Boulders from both identified source areas can be observed on these slopes, however the following distribution is apparent:
  - As indicated on Figure 2, rocks derived from the upper pyroclastic flow (Source Area 1) are typically restricted to the mid to upper slopes of the site
  - Boulders derived from the basaltic rock outcrop (Source Area 2) are typically more widely distributed compared to those derived from the pyroclastic material. Basaltic boulders can be observed downhill (west) side of the site and have runout across the full width of site
  - A number of basaltic derived boulders have come to rest within the gully, and typically have a higher concentration compared to the less confined slopes which comprise the site itself. The gully to the north of the site has clearly acted as a topographical focus for boulder runout
- Based on our site observations, boulders derived from the upper pyroclastic source are typically equant and rounded, with a typical diameter of around 1.2 m. The largest observed boulder had a diameter of approximately 2.4 m.
- Boulders derived from the lower basaltic rock source were typically angular and tabular to prismatic, with block lengths typically somewhat greater than width, with a typical radius of around 1.2 m. The largest observed boulder from this source had a diameter of approximately 2.8 m (refer Table B1). This is interpreted to be an effect of the cooling joints noted within the rock mass exposed in the source area. The typical joint patterns at the source were between 0.5 m and 2.5 m which suggests the blocks typically do not break up substantially in transit.
- Tree scarring damage from impact of a relatively large boulder can be observed at the edge of the stand of pine trees towards the southeastern corner of the site. Impact damage can be observed to around 3.2 m height above ground level (Photograph B3).

1 Rock mass descriptions are made in general accordance the NZ Geotechnical Society "Field Description of Soil and Rock" Dec 2005.

Photo B1: View of rockfall source areas from site

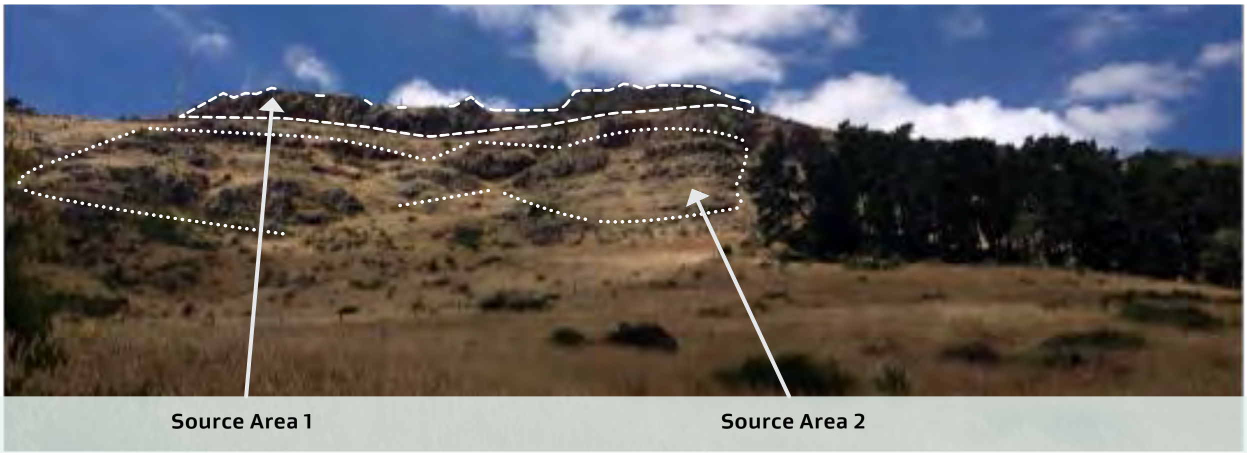


Photo B2: Basaltic block at approximately mean size on lower part of site

Photo B3: Tree impact damage (outlined) from boulder impact (note that the boulder has been mechanically broken up after impact).

Photo B4: Source Area 1 – Typical character of upper source area (Pyroclastics)

Photo B5: Source Area 2 – Typical character of lower source area

B2.3 Interpretation

It is considered that the following failure mechanisms are likely within the two rock masses:

- Boulder release within the upper pyroclastic material is likely to be a result of differential weathering, whereby the weaker ash matrix is preferentially removed, leading to loss of support of the significantly stronger basaltic blocks
- Rock fall within the lower basaltic flows is due to a defect controlled failure, principally rock toppling and/or planar sliding.

The observed distribution in boulder runout distance is principally due to differences in particle shape. The tabular to prismatic nature of some of the basaltic boulders means that there may have been potential for the boulders to be able to "cart-wheel" down the slope, allowing them to achieve a significant runout distance. Whilst boulders derived from the pyroclastic rock source fall from greater height, their typically equant and rounded shape means that they had less ability to cart-wheel, resulting in lower bounce heights and therefore greater frictional resistance due to ground contact.

Typically observed sizes within the boulder runout field are similar to the defect spacing and block size in the outcrops, which suggests that boulders are not subject to significant breakup .

B2.4 Boulder Size Data

Of the proportion of boulders that came to rest on the slopes below the identified source areas the fallen rock database suggests the following size variation.

B3 Rockfall Modelling

B3.1 Existing Data

A high level three-dimensional rockfall modelling exercise has been previously undertaken in the area of the site to aid with land policy decisions. This modelling takes into account three dimensional topographical effects such as gullies and ridge lines which may tend to concentrate or reduce potential boulder runout paths.

> The 3D modelling work was high level and area wide in approach and it was not intended to be used as a design tool without more detailed calibration. There was no specific model calibration with filed data at any one location. Even without specific calibration in some locations the 3D modelling aligned well with actual boulder data in other cases the modelling aligned less well. However, the modelling was very useful in identifying areas where topographical effects were important and was an important input into the policy development decisions.

Table B1: Boulder size

	SOURCE AREA 1 (PYROCLASTICS)	SOURCE AREA 2 (BASALT)
Typical Block Shape	Equant, Spherical	Equant to Tabular, Angular
Typical Dimension (diameter / length)	1.2 m (mean) 1.7 m (95th percentile)	1.0 m to 1.3 m (mean) 1.7 m (95th percentile)
Typical Volume	0.9 m^3 (mean) 2.6 m^3 (95th percentile)	1.5 m^3 (mean) 5.1 m^3 (95th percentile)[1]
Typical Density	2700 kg / m^3	2850kg / m^3

Notes:

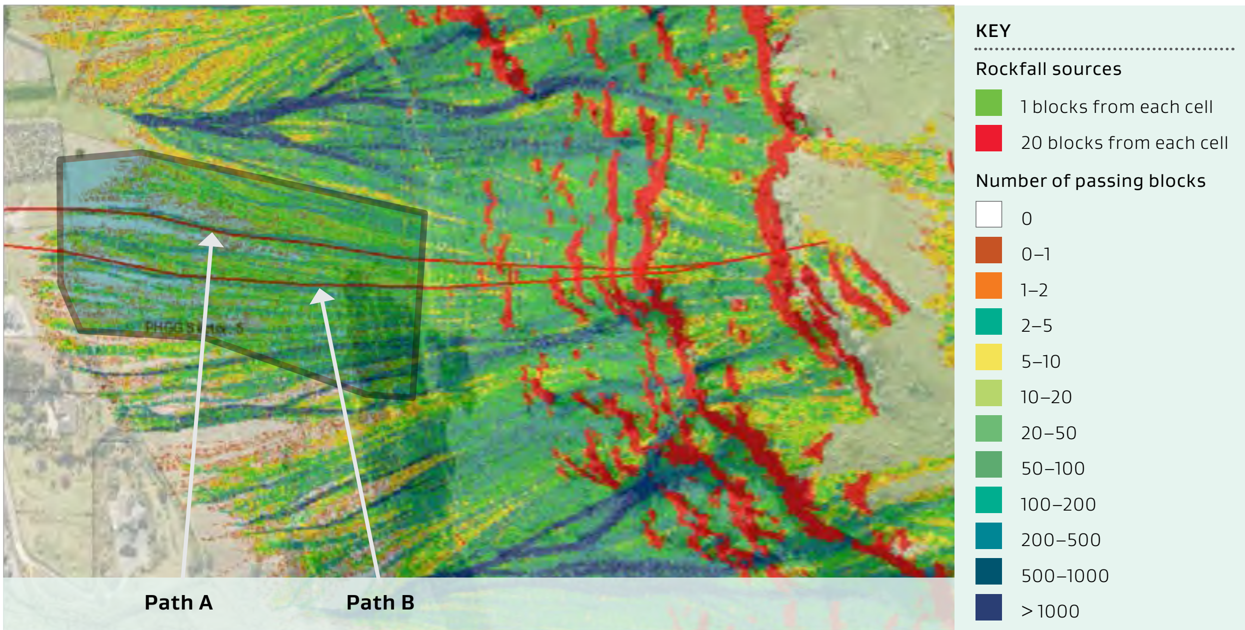
1. A factor of 0.7 may be applied to the volume calculations of boulders measured in the field to account for the irregular (non-cube) shape of the boulders (see for example, Section 5.1.2 of Massey et al 2012).
2. The 95 percentile boulder can be difficult to estimate if there is a limited boulder data set on the site in question (refer Section 4.1 of the guidance) which to base the design. If there is evidence of boulders on the slope either new or historic then selecting the largest of the most common boulders would not be unreasonable.
3. The design percentile will be a function of the level of risk reduction that is required and should be assessed on a case by case basis. The 95th percentile bounce height and energy are based on the boulder size distribution and have been assumed as the design values for the purposes of this worked example and follows the approach outlined in ONR (2012) refer MBIE guidance Section 4.1.

The analysis used spherical boulders at the 95th percentile boulder volume for all boulder falls in the Port Hills region based on data collected from field inspection. It was also assumed for this model that all vegetation was removed (bare slope model). Furthermore, any cell with an inclination greater than 45° was assumed to be a boulder source. The outcomes of the study are therefore likely to be somewhat conservative, but remain extremely useful to denote critical sections for rockfall remedial design purposes.

The 3D modelling considered 1 m square source cells, each containing 20 boulders. The run-out distances and preferred travel directions are then calculated assuming the boulders contained within each cell are released. It is therefore important to note that, particularly for determining preferred boulder fall paths, the numbers of boulders passing any particular location are only a statistical assessment of the relative concentrations of boulder travel paths. The results of the assessment do not necessarily mean that boulders would be dislodged in any particular location in a large earthquake event.

The results of the 3D modelling of the boulder paths for the section of slope above the site are shown on Figure B3. Preferred boulder paths are indicated by higher transit values. The results of modelling suggest that the main rockfall areas which may affect site are located almost due east of the site. Two preferred boulder paths (Path A and B; refer Figure B3) are interpreted based on the results of the 3D modelling; however these are very similar in terms of both orientation and ground surface.

Figure B3: Results of 3D modelling



Notes:

1 It is recognised that existing 3D modelling will not be available in many areas of New Zealand. In this case, practitioners should consider what effects topography may have to assess the most representative 2D section (or sections) for a 2D analysis.

2 It may be argued that a third section, through the southeastern corner of the worked example site should be assessed as there is clearly some concentration of preferred boulder fall paths in this area. This has not been undertaken as part of this worked example for simplicity.

B3.2 Site Specific Rockfall Modelling

Two dimensional rockfall modelling along the preferred path orientations noted in the 3D rockfall modelling (refer Section 4.1 of the guidance document).

Rockfall modelling, using the proprietary software package 'Rocfall' v5.0, was undertaken along both preferred rockfall paths to assess appropriate design parameters (bounce height and energy) for potential rockfall systems to reduce the risk to the site. Both sections produced similar results in terms of bounce height and energy and therefore only the results for Path A are presented.

Rockfall assessment was undertaken assuming 'Rigid Body' analysis assuming two source areas as shown in Figure BA.1 of Attachment A and summarised in Table B2 below.

For all modelled sections, 0m chainage has been taken as an arbitrary point approximately 20 m above (east) of Source Area 1. As shown in A.1 of Attachment A, the site is located between 195 m and 450 m.

B3.3 Back Analysis

Slope parameters (restitution coefficients and surface friction angles) for the assessment have been assessed based on modelling the distribution of rockfall end points observed in the field (refer Attachment A). Guidance on the range of coefficients of restitution for these slope materials are provided within the software manual and in Massey (2012) for this region. The results of back analysis are shown in Figure BA.1 to BA.3 of Attachment A.

In summary:

- The modelling distribution of boulder runout distances matches closely with observed fallen boulder locations;
- Boulders from the upper pyroclastic source (Source Area 1) typically come to rest on the upper to mid-part of the slope, however;
- Rocks derived from the basaltic rock source (Source Area 2) typically show a distribution of end points that are further downslope compared to the Source Area 1.

It is therefore considered that the material parameters reflect the conditions observed at the site and are appropriate for design purposes and therefore a parameter sensitivity analysis was not undertaken in this instance.

Note

Perhaps the most difficult part of rockfall assessment is to produce a model that accurately describes the site conditions, in particular the runout distribution and bounce height. In all case, this will depend on careful engineering geological mapping of the source and runout zones. In areas where recent boulder fall has not occurred, field trials (where appropriate) to assess runout and bounce could be considered. Depending on the modelling program used, practitioners may need to consider assessing two models, one for runout distribution and the second for bounce height (refer Section 4.2.5 of the guidance document).

Table B2: Rockfall Modelling Parameters

	UPPER SOURCE (PYROCLASTICS)	LOWER SOURCE (BASALT)
Modelled Block Shape in Rocfall V5.0 [1]	Octagon	Super Ellipse 5:6
Typical Density	2700 kg / m^3	2850 kg / m^3
Modelled Mass	mean 2,440 kg std.GHYLDWLRQ 2,740 kg 95th percentile 6,945 kg	mean 4,335 kg std.GHYLDWLRQ 6,190 kg 95th percentile 14,500 kg
Boulder Initial Velocity [2]	1.5 m / s	1.5 m / s

Notes:

1 The available shapes in Rocfall 5.0 are somewhat limited, but have significantly improved compared to previous versions. The shape names are set within the software. Other software packages allow greater flexibility when assigning particle shape characteristics.

2 The initial velocity has been widely adopted for rockfall modelling purposes in the Christchurch region; however it is very high in relation to published information (see for example Gerstenberger et al, 2007). Experience suggests that even with very high values, effects of initiation velocities tend to become non-critical once boulders have impacted a few times on the slope. In most scenarios therefore, gravitational effects dominate the rockfall modelling; however, this should be assessed on a case by case basis.

B3.4 Boulder Energy and Bounce Height

The results of our modelling are presented in Sections A.4 and A.5 of Attachment A and are summarised below:

- 95th percentile bounce heights of between 4.5 m to almost 9 m are apparent in the upper part of the site between approximately location 80 m and 250 m on the modelled section. There is considerable variation in bounce height with relatively short distance up and down the slope, which is interpreted to be an effect of the significant topographical variation as well as the modelled surface conditions (coefficients of restitution, friction angle);
- Downslope of around cross section location 250 m, bounce heights reduce considerably and by around 300 m are typically less than 1.5 m (refer Attachment A; Figure A.4);
- There is considerable variation in the 95th percentile energy calculated along the modelled section. The 95th percentile total kinetic energy (rotational + translational) approaches 2400 kJ on the upper slopes (around location 150 m), and typically reduces to less than 400 kJ downslope of location 325 m (refer Attachment A; Figure A.5);
- Boulders from the upper pyroclastic source (Source Area 1) typically come to rest on the upper to mid-part of the slope (refer Attachment A; Figure A.3), however;
- Rocks derived from the basaltic rock source (Source Area 2) typically show a distribution of end points further downslope compared to the Source Area 1.

A summary of the results of rockfall assessment are provided in Table B2.

Observations of boulder roll trajectory on the Port Hills suggest that boulders may deviate by up to 30° from the downslope direction principally as a result of individual boulder shapes, which depending on how the boulder is rolling or bouncing, may impart a directional bias. This is not captured in 2D analysis, but should be considered in regard to assessing the length of any RPS barrier.

Note

Design of the length of an RPS should be considered as a risk-based approach against the possible distribution of boulder paths and should be assessed on a case by case basis by the Practitioner. Practitioners should also take care that the design parameters determined on a collector or barrier location are not subject to significant variation with short changes in slope distance and thus as accurately possible reflect the local site conditions.

Non-Earthquake Rockfall

In the pilot study for assessing life-safety risk from rockfalls report GNS note that there is historical precedent of rockfalls occurring on occasions other than severe earthquake shaking. Whilst this is most likely due to severe storm events, boulder release can also occur under dry weather conditions.

We have modelled the effect of non-earthquake induced rockfall assuming a zero release velocity. The results of the analysis show that the probability of a boulder reaching the site is not significantly different to the probability of a boulder reaching the site under earthquake acceleration.

Table B2: Results of Rockfall Modelling

PARAMETER	SECTION UPPER RPS LOCATION	SECTION LOWER RPS LOCATION
Maximum / 95th percentile bounce height at proposed RPS locations	8.2 m / 3.3 m [1]	1.5 m / 1.1 m
Maximum / 95th percentile kinetic energy	3890 kJ / 900 kJ [1]	518 kJ / 178 kJ

Notes:

1 It can be observed on Figures A.4 and A.5 in Attachment A that the distribution of boulder bounce heights and energies at the proposed upper barrier location vary considerably over a short distance. The results calculated in the software at the barrier itself have therefore been adjusted slightly to reflect the typical bounce height and energy in the local area (refer green and red lines on Figures A.4 and A.5). However, if there is flexibility in RPS location then the final site would be chosen to align the lowest bounce heights and energies.

2 Bounce height is calculated as the difference between boulder interception height on each collector and the elevation of the ground surface at the collector.

B4 Proposed Rockfall Mitigation Works

B4.1 General

Two options for the rockfall protection structure (RPS) are considered in this worked example as follows:

- A proprietary rockfall fence installed adjacent to the upslope property boundary (upper RPS location). The fence has been designed to accommodate the 95th percentile boulder energy and bounce height determined on the basis of site specific rockfall modelling. Design calculations are resented in Attachment C.
- A geogrid reinforced earth bund constructed at approximately the mid-distance on the slope (lower RPS location).

Note

The design philosophy for an earth bund differs from rockfall fence. For a rockfall fence, careful consideration needs to be given of the potential for multiple boulder impact (termed boulder flux) to occur, which will determine whether the fence is designed to SEL or MEL energy levels. This is due to overall design of the system; braking elements and the height of the fence may be affected by a single boulder impact and need to be allowed for in the design process (refer Section 4.5.1 of the guidance document). Multiple boulder impacts on an earth bund are of less concern as it would be very unlikely that two design boulders would impact at exactly the same location on the structure. With this in mind, the following energies have been assumed in this worked example:

- For the lower earthbund, SLS energy will be considered as the 95th percentile energy, whilst ULS will be assumed as the maximum recorded value, or 3 x SLS, whichever is lower.
- For the proposed rockfall fence, the MEL will be considered as 3 x 95th percentile energy in accordance with the guidance document the lower earthbund, SLS energy will be considered as the 95th percentile energy, whilst ULS will be assumed as the maximum recorded value, or 3 x SLS, whichever is lower.

The following sections provide the typical design process for the two RPS options outlined above. Typical Design Drawings are provided in Attachment B for both RPS options.

B4.2 Rockfall Fence (Upper Barrier Location)

The results of rockfall modelling suggest that boulders released from the Basaltic rock mass (Source Area 2) provide higher bounce heights and energies to the upper barrier location (refer A7.6 and 7.7) and have therefore been adopted for design purposes.

Note

The sum of the two source distributions produces a slightly reduced 95th percentile compared to the Source Area 2 by itself. In this case the Source Area 2 95th percentile energy was taken as this was more conservative.

The design requirements are summarised in Table B3.

Table B3: Summary of design requirements

DESIGN PARAMETER	DESIGN VALUE	SOURCE
Design Kinetic Energy	900 kJ (SEL) 2700 kJ (MEL)	Rockfall Modelling (Attachment A7.2); MEL calculated as 3 x SEL
Design boulder Source Area 2 – diameter and volume	1.8 m	Table 1
Design Bounce Height (to boulder centre)	3.4 m	Rockfall Modelling (Attachment A7.1)
Design Impact Location	4.15 m	Section 1, Attachment C
Minimum Fence Height	4.8 m – 5.0 m	Section 1, Attachment C

B4.1.1 RPS Design

A 3000 kJ proprietary fence is considered to have sufficient capacity for the anticipated MEL energy.

In terms of design height, Piela & Ronco (2009) suggest that the design interception height of a rockfall fence should be greater than the modelled interception height plus a clearance (f) that is not less than half the average size of the design boulder. Gimrod & Giacchetti (2014) suggest a similar approach, with a suggested minimum value equal to 0.5 m.

Typical design drawings are presented in Attachment B. As indicated in Attachment C, a minimum overall height of the rockfall fence of 4.8 to 5.0 m above existing ground surface is calculated. This is equivalent to the minimum barrier height supplied by the manufacturer for the energy rating of the fence of 5 m and the proposed fence is therefore suitable.

B4.1.2 Anchor Design

Lateral and Upslope Anchors

The Manufacturer's design calls for the fence to be supported by:

- Two lateral anchors, one at each end of the proposed fence.
- A series of upslope anchors located 4 m upslope from the fence. From each anchor point a wire cable will extend to each adjacent fence post.

> **Note**
>
> Anchor positions and numbers as well as the spacing of supporting posts will vary depending on the proprietary fence proposed and its energy category.

For the purposes of the worked example, let us assume that the depth of soil is limited to less than around 1.0 m and therefore anchors may be drilled into rock to resist the applied loads. The grout to country bond in rock is assessed to be 500 kPa.

To meet the anchor design loads supplied by the Manufacturer, based on this bond capacity, the anchors will require the following minimum bond lengths in rock:

- **Lateral Anchors:** 3.0 m
- **Upslope Anchors:** 2.5 m

Anchor capacity calculations are provided in Attachment C.

B4.1.3 Base Plate Foundations

Supporting posts for the rockfall fence are proposed to be constructed at approximately 10m intervals along the length of the fence with some adjustment to best fit local topographic variations. The base plates will be supported on a 28 MPa concrete plinth. The Manufacturer's base plate design calls for two anchors; one anchor inclined at approximately 60° from horizontal orientated upslope (to accommodate shear loading) and a second anchor on the downslope side (to accommodate compressive load). Anchor bond lengths of 2.0 m and 2.75 m are calculated to resist the shear and compressive loads (Attachment C).

B4.1.4 Inspection and Maintenance Requirements

Requirements for regular inspection will be dictated by the environmental setting in which the rockfall fence is constructed, and need to be assessed based on the maintenance requirements. Maintenance of the rockfall fence would be expected to be undertaken between every one and five years and would include:

- Checking the tension of clamped connections,
- Removal of vegetation within the area local to the fence,
- Removal of any accumulated rock debris; and
- Checking the state of the corrosion protection and patching as necessary

Inspection of the fence would also be required following rockfall impact (or an event likely to cause a rock impact (earthquake or storm).

ETAG 027 provides an assumed working life of 25 years without rock impact under normal environmental conditions and 10 years under aggressive conditions provided that:

- The recommendations in the Manufacturer's maintenance manual are adhered to.
- The fence is not subject to major rock impact. In the event of major impact, it is likely that several elements of the RPS will have suffered significant damage and will be required to be replaced. Specific engineering assessment would be required in this event.

All in-ground anchors are expected to have a design life of not less than 50 years.

B4.3 Earth Bund (Lower Barrier Location)

The results of rockfall modelling suggest that boulders with sufficient travel distance to reach the lower barrier are derived from Source Area 2 only. The design requirements are summarised in Table B2.

Table B2: Summary of design requirements

DESIGN PARAMETER	DESIGN VALUE	SOURCE
Design Kinetic Energy	178 kJ (SEL) 518 kJ (MEL)	Rockfall Modelling (Attachment A8.2); ULS taken as maximum energy
Design Boulder width (Source Area 2) diameter and volume	1.8 m 5.1 m^3	Table 1
Design Bounce Height	1.1 m	Rockfall Modelling (Attachment A8.1)
Design Impact Location	1.95 m (bounce height to boulder centre + radius)	Section 1, Attachment E
Minimum Height (above ground on upslope side of bund)	2.75 m	Section 2, Attachment E
Minimum Width at Crest	1.5 m	Section 3, Attachment E

B4.2.1 RPS Design Geometry

The RPS geometry has been calculated based on Ronco et al (2009) and summarised as follows. The minimum embankment height is calculated as the factored interception height of the boulder plus the thickness of the upper reinforced soil layer. The interception height is taken as the height to the top of the boulder (ie bounce height to boulder centre + radius). As indicated in Attachment C, a minimum overall height of the earth bund of 2.75 m above existing ground surface is calculated.

The MBIE guidance document recommends the following criteria to assess the minimum width of the bund:

- At least 2 times the anticipated penetration of the ULS boulder energy, and;
- At least 5 times the anticipated penetration of the SLS boulder energy.

In this case, the anticipated penetration of a 5.1 m^3 boulder with 178 kJ kinetic energy at SLS is expected to be 0.42 m (Attachment E), which requires the minimum bund width at the point of impact to be least 2.1 m. This means that the minimum width of the bund at crest will be required to be approximately 1.5 m. The side slopes of the bund are at 76° which means that the base width will be approximately 3.5 m wide.

Note

The dimensions indicated above to not allow for ground slope angle, nor any upslope drainage requirements. Depending on bund height, the extent of earthworks may be quite extensive and may need to be carefully considered in terms of horizontal extent, slope stability as well as access and maintenance requirements.

A cross section through the bund is presented in Attachment D. Example calculations are provided in Attachment E.

B4.2.2 Bearing Capacity, Internal and Global Stability

Bearing capacity of the foundation soil has been assessed as outlined in Section 4 of Attachment E. The foundation soils are considered to provide adequate bearing for the proposed earth bund structure.

Internal stability of geogrid reinforced earth bunds is unlikely to be a significant design concern as the bund is not subject to external soil load. Design of the geogrid reinforcement can be undertaken either in consultation with the manufacturer or in slope stability analysis software.

An assessment of global stability has not been undertaken as part of this worked example as it is unlikely to be of significant concern for the relatively gentle slopes and stiff soil materials. However, global stability should be considered as part of the design process and include:

- An assessment of design seismic accelerations: boulder initiation may only occur under seismic events significantly larger than ULS; accordingly;
- An evaluation of the degree of acceptable deformation of the embankment may need to be considered, and;
- The proximity of structures to the earth bund (for example cut slopes and buildings)

B4.2.3 Inspection and Maintenance Requirements

Similarly to rockfall fences, regular inspection requirements will be dictated by the environmental setting in which the rockfall fence is constructed,and need to be assessed based on the maintenance requirements. Maintenance of the rockfall bund would be expected to be undertaken between every five to 10 years and include:

- Removal of vegetation from the bund which could be large enough to damage the reinforcing grids,
- Removal of any accumulated rock debris; and
- Checking the state of the corrosion protection of the reinforcing grid and patching as necessary

Inspection of the bund would also be required following rockfall impact (or an event likely to cause a rock impact (earthquake or storm).

Earthbunds are limited maintenance structures, and would not be expected to require significant maintenance unless a rockfall occurs which penetrates the bund by 20% (ie a SLS event). In this case, some localised repair to the damaged and deformed area of the bund would be required.

B5 Construction Considerations

Regardless of the type of rockfall protection structure being considered, the following construction factors are raised for practitioners to consider:

- Topographic survey: Adequate topographical control at the location of the RPS is necessary to ensure that any proposed structure can be readily built. A minimum contour interval of 0.5m is recommended. The locations of property boundaries need to be well understood to ensure that the length of the RPS is sufficient to provide the required level of protection without encroachment onto neighbouring land or encroaching onto boundary setbacks (if possible)
- In addition to the above, careful siting of rockfall fences in particular should be considered Bends in fences should be minimised to avoid additional ground anchoring. A fences' ETAG certification relies on it being constructed along contour without significant vertical height difference. The certification may become problematic where large height variations are necessary. If large height variation is required, then specific design may need to be undertaken (this would normally be done by the manufacturer)
- The material used for facing of an earthbund should be carefully considered. Geotextiles exposed to weather may breakdown over time unless vegetation is able to establish. Bunds subject to water flow may need at least a partial facing of non-erodible material.
- Earth bunds in particular require relatively large laydown areas to allow material stockpiling. Adequate space is an important consideration, as is accessibility to the worksite for earthmoving equipment.
- In sites with confined space, difficult access or steep slopes, rockfall fences will likely be significantly easier to construct.
- For earth bunds, construction may have significant effect on overland flow paths. In particular, difficulties may arise with concentrating overland flow on sites with potential or actual soil contamination (HAIL or LLUR). Whilst extra cost may be incurred, consideration may need to be given to designing a free-draining base to the earth bund (as shown on Appendix D).

B6 Alternative Options

The intent of this worked example is to provide an outline of the design process for two common rockfall protection structures. Particularly towards the distal end of the distribution of rockfall runout points, other mitigation options may become practical (catch areas, rigid timber barriers, vegetation planting etc). Practitioners may wish to consider these options as they may have significantly lower construction cost.

B7 References

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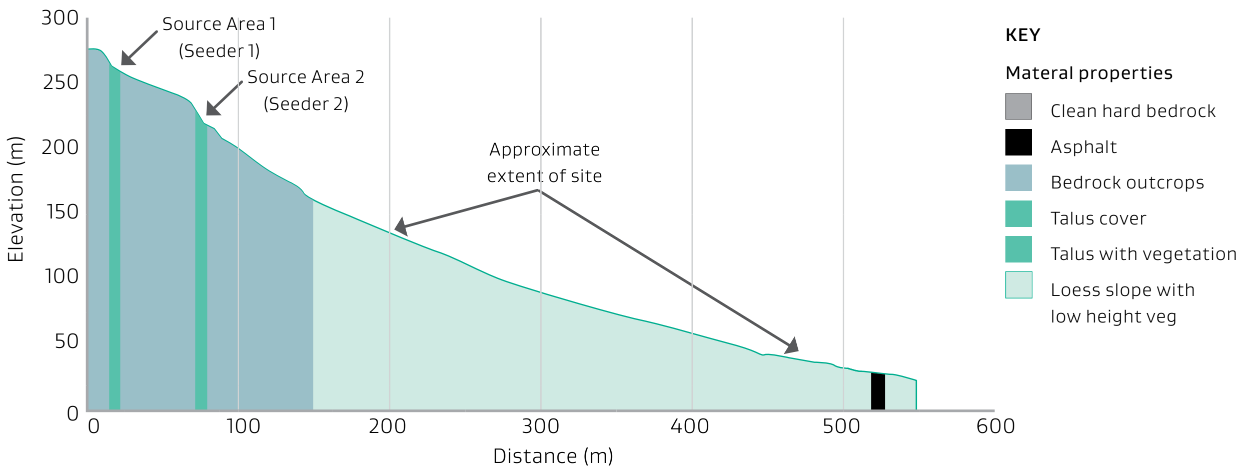
Ronco, C, Oggeri, C. & Piela, D. (2009): *Design of reinforced ground embankments used for rockfall protection*. Nat. Hazards Earth Syst Sci. 9 1189–1199.

Richards, L. (2012, June 18). Port Hills Rockfalls, Rockfall mitigation – design strategy.

APPENDIX B – ATTACHMENT A.
2D ROCKFALL MODELLING

A.1 Cross section topography



Properties

	MEAN	DISTRIBUTION	STANDARD DEVELOPMENT	RELATIVE MINIMUM	RELATIVE MAXIMUM
CLEAN HARD BEDROCK					
Normal restitution	0.53	Normal	0.04	0.12	0.12
Tangential restitution	0.99	Normal	0.04	0.12	0.01
Dynamic friction	0.577	Normal	0.048	0.144	0.144
Rolling resistance	0.289	Normal	0.024	0.072	0.072
ASPHALT					
Normal restitution	0.4	Normal	0.04	0.12	0.12
Tangential restitution	0.9	Normal	0.04	0.12	0.1
Dynamic friction	0.577	Normal	0.048	0.144	0.144
Rolling resistance	0.289	Normal	0.024	0.072	0.072
BEDROCK OUTCROPS					
Normal restitution	0.35	Normal	0.04	0.12	0.12
Tangential restitution	0.85	Normal	0.04	0.12	0.12
Dynamic friction	0.532	Normal	0.045	0.135	0.135
Rolling resistance	0.266	Normal	0.023	0.069	0.069
LOESS SLOPE WITH LOW HEIGHT VEGETATION					
Normal restitution	0.4	Normal	0.03	0.09	0.09
Tangential restitution	0.65	Normal	0.03	0.09	0.09
Dynamic friction	0.324	Normal	0.035	0.105	0.105
Rolling resistance	0.071	Normal	0.018	0.054	0.054

Seeders

Seeder 1

PROPERTIES

Location: (68.125, 252.046)

ROCKS TO THROW

Number of rocks: Set in project settings
Rock types: Basalt

	MEAN	DISTRIBUTION	STANDARD DEVELOPMENT	RELATIVE	
				MINIMUM	MAXIMUM
INITIAL CONDITIONS					
Horizontal velocity (m/s)	1.5	Normal	0.15	0.45	0.45
Vertical velocity (m/s)	1	Normal	0.15	0.45	0.45
Rotational velocity (º/s)	0	None			
Initial rotation (º/s)	0	Uniform		0	360

Seeder 2

PROPERTIES

Location: (12.628, 290.016)

ROCKS TO THROW

Number of rocks: Set in project settings
Rock types: Pyroclastics

	MEAN	DISTRIBUTION	STANDARD DEVELOPMENT	RELATIVE	
				MINIMUM	MAXIMUM
INITIAL CONDITIONS					
Horizontal velocity (m/s)	1.5	Normal	0.15	0.45	0.45
Vertical velocity (m/s)	1	Normal	0.15	0.45	0.45
Rotational velocity (º/s)	0	None			
Initial rotation (º/s)	0	Uniform		0	360

Rock types

Pyroclastics

PROPERTIES

Colour:

Shapes: Octagon

	MEAN	DISTRIBUTION	STANDARD DEVELOPMENT	RELATIVE MINIMUM	RELATIVE MAXIMUM
PYROCLASTICS					
Mass (kg)	2442	Lognormal	2737	2000	8211
Density (kg/m^3)	2700	None			

Basalt

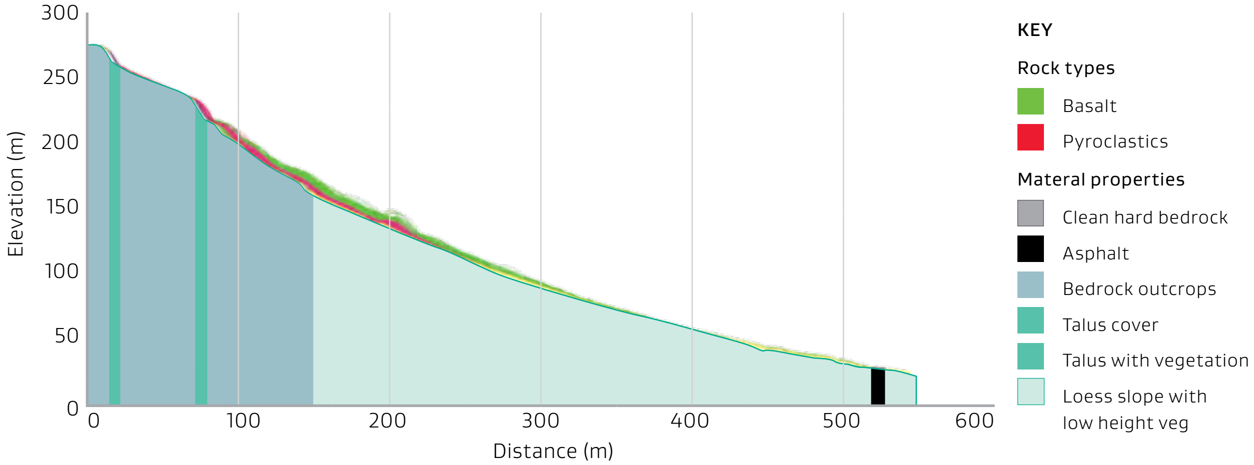
PROPERTIES

Colour:

Shapes: Super ellipse ˆ4 (5:6)

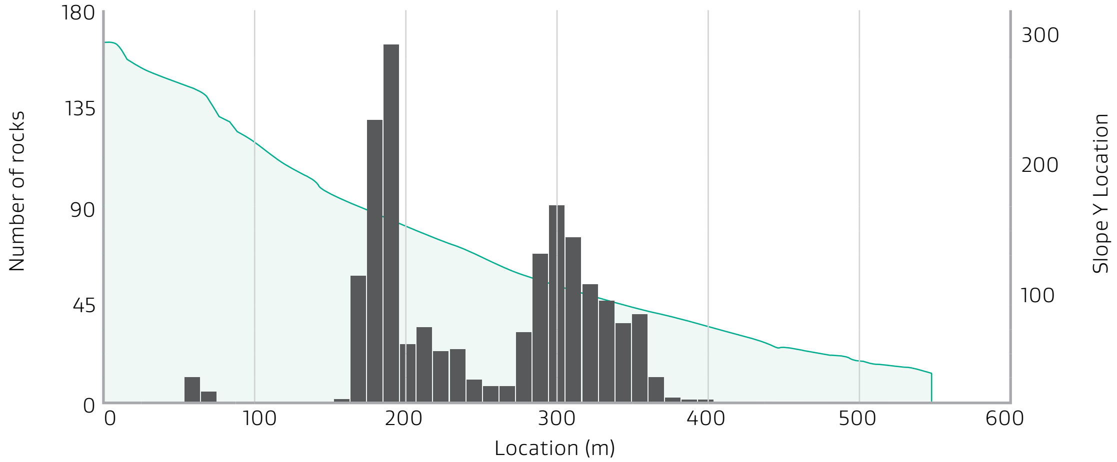
	MEAN	DISTRIBUTION	STANDARD DEVELOPMENT	RELATIVE MINIMUM	RELATIVE MAXIMUM
BASALT					
Mass (kg)	4334	Lognormal	6188	3334	18564
Density (kg/m^3)	2850	None			

A.2 Cross section modelling results

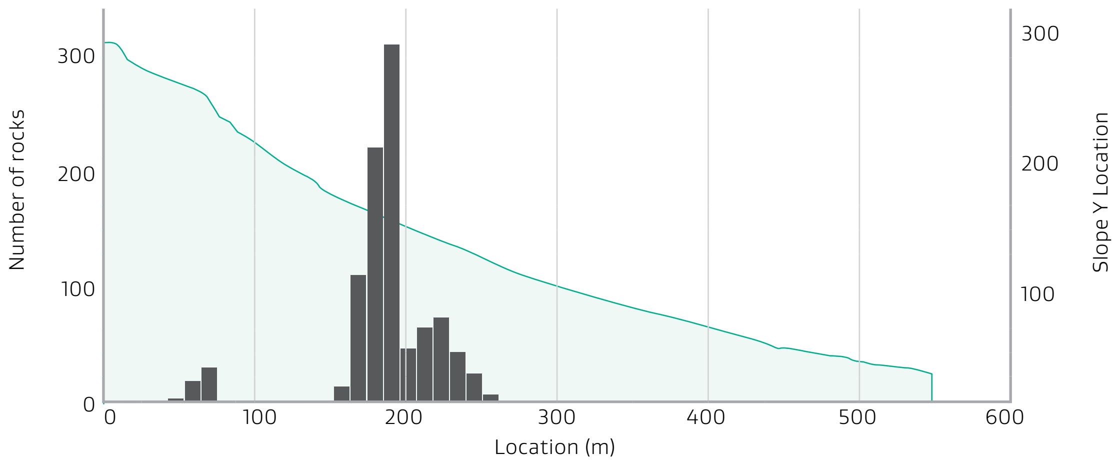


A.3 Boulder run-out distance

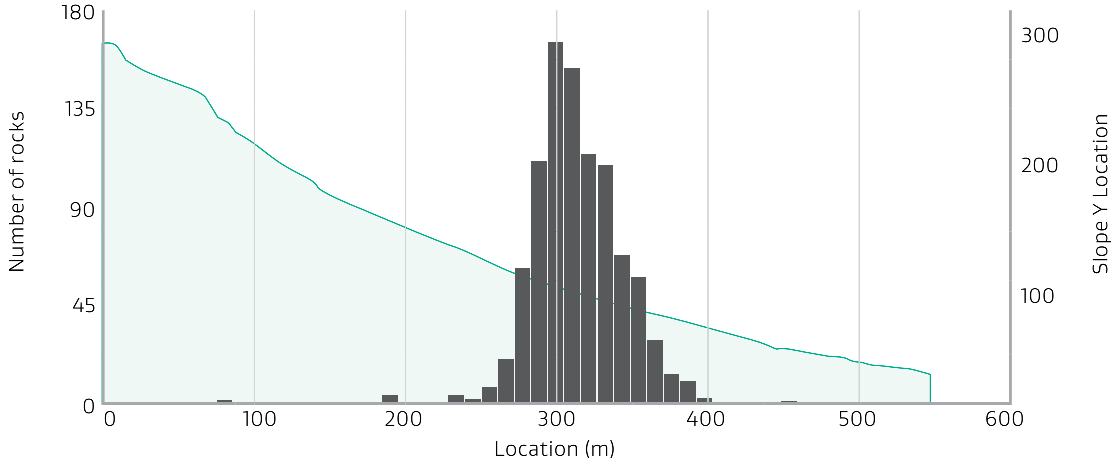
Both sources



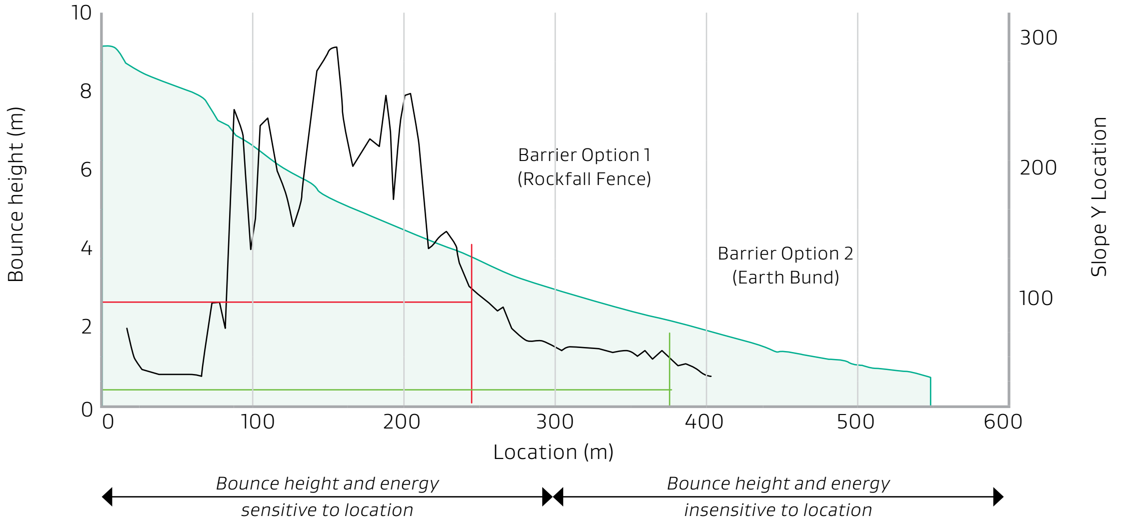
Upper Source (Pyroclastics)



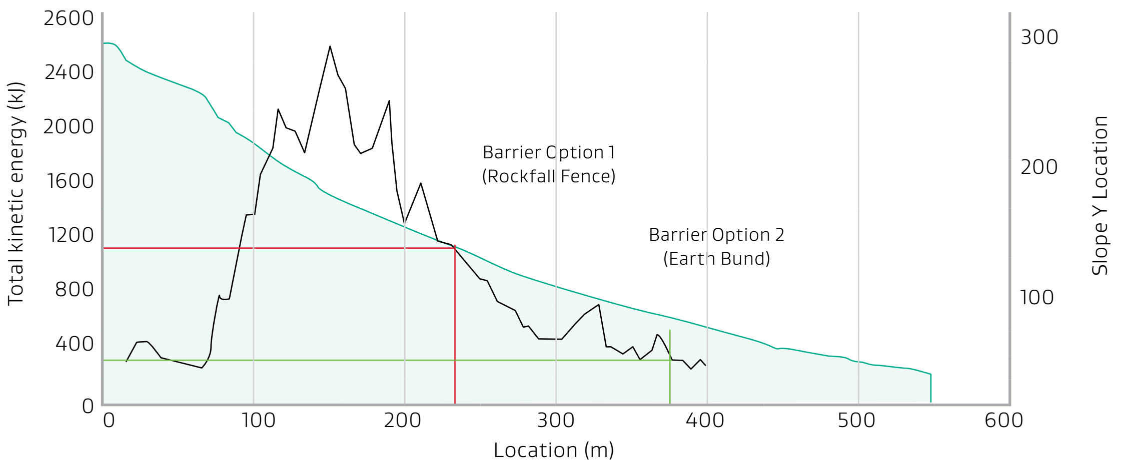
Lower Source (Basalt)



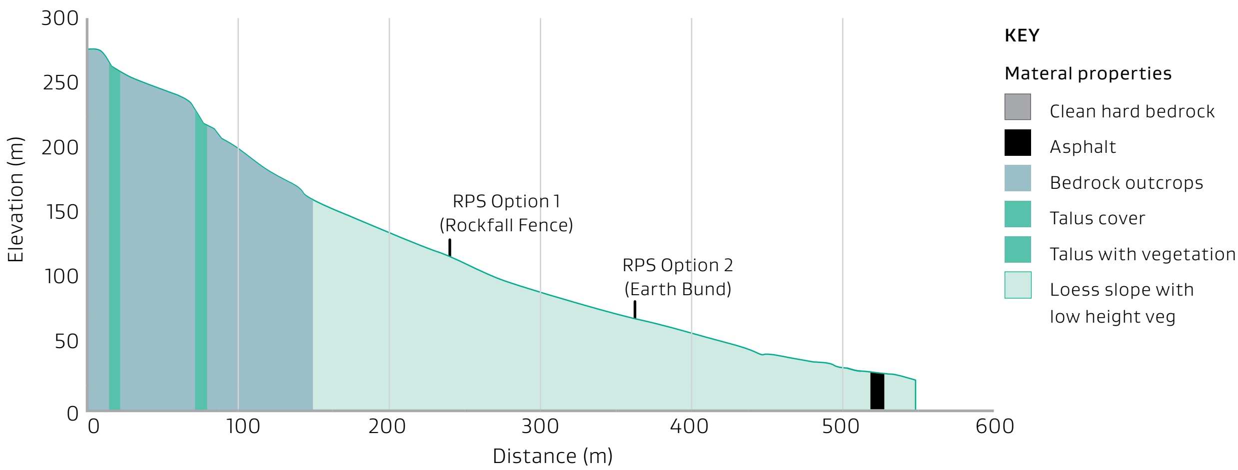
A.4 95th Percentile bounce height



A.5 95th Percentile energy



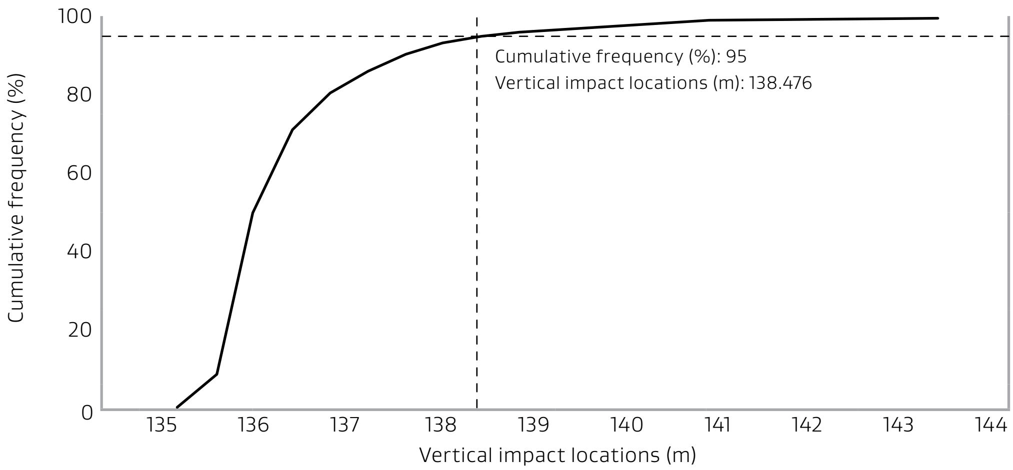
A.6 Cross Section – Proposed RPS Locations



A7. Upper RPS (Rockfall Fence) Results

All Rockfall Sources

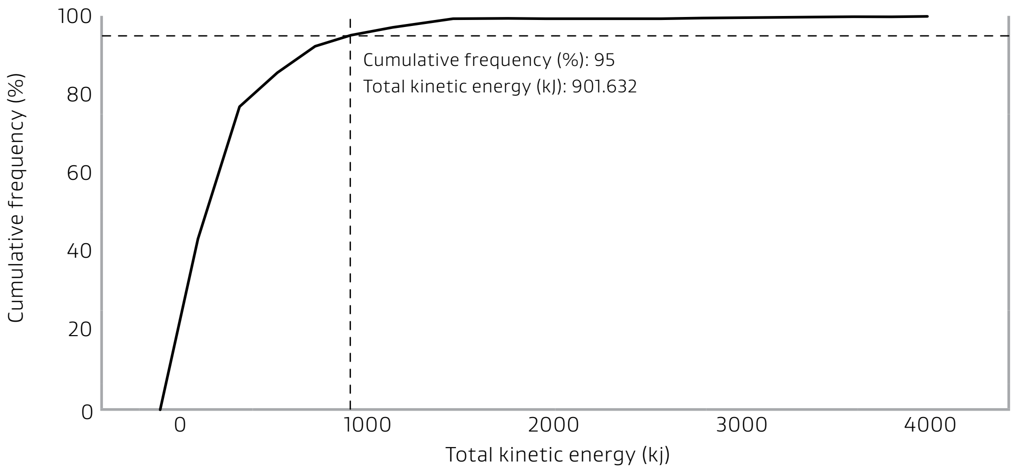
A7.1 95th Percentile bounce height to boulder centre (base of RPS is at 135.1m)



Total number of rocks on collector 1: 1530

Vertical impact locations:
min = 135.388
max = 143.264

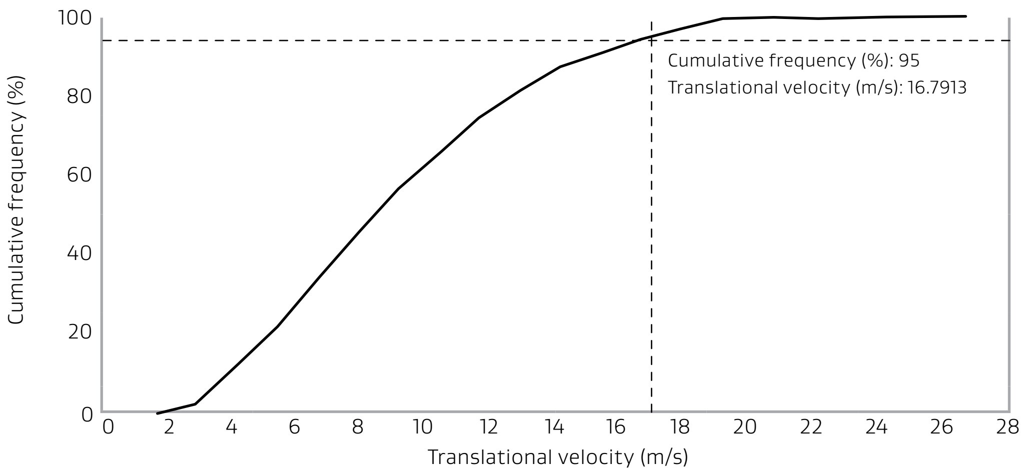
A7.2 95th Percentile Energy on RPS



Total number of rocks on collector 1: 1530

Total kinetic energy:
min = 2.10544
max = 3890.54

A7.3 95th Percentile Translational Velocity on RPS

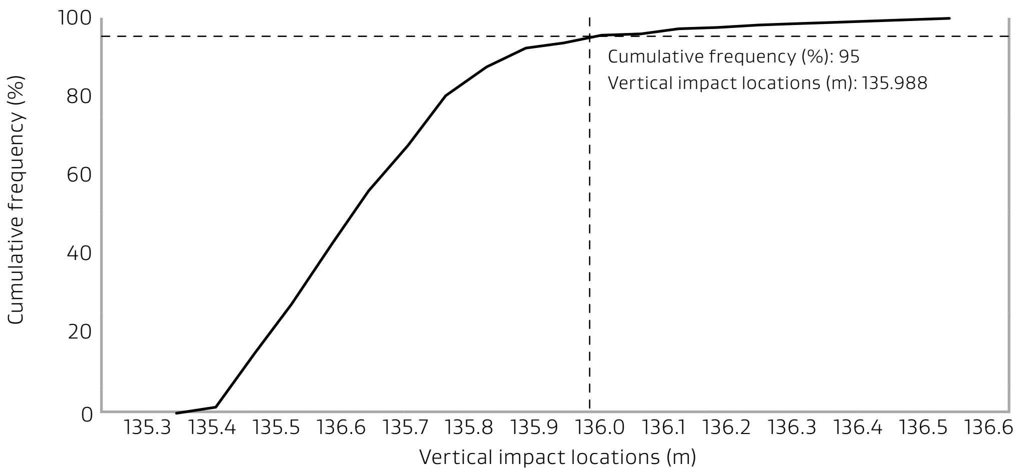


Total number of rocks on collector 1: 1530

Translational velocity:
min = 2.1438
max = 25.8436

Upper Rockfall Source (Pyroclastics)

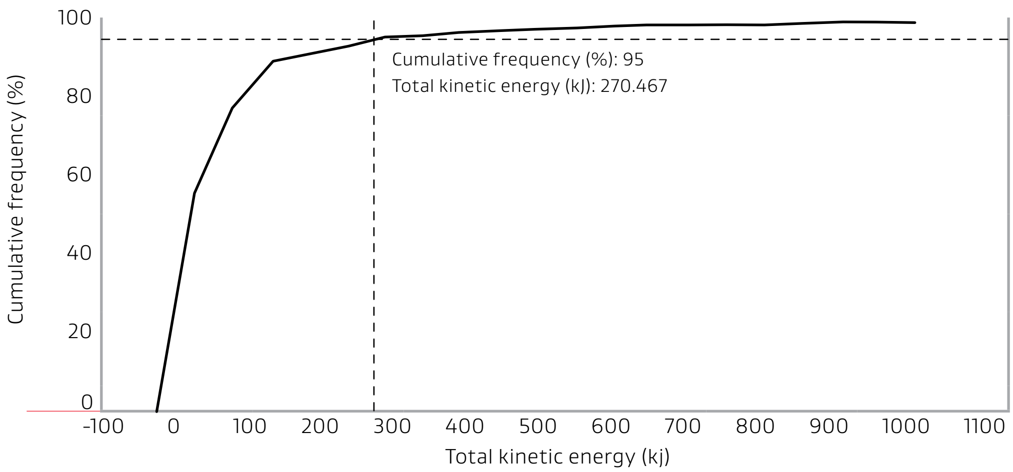
A7.4 95th Percentile Bounce Height (base of RPS is at 135.1m)



Total number of rocks on collector 1:
164

Vertical impact locations:
min = 135.388
max = 136.52

A7.5 95th Percentile Energy

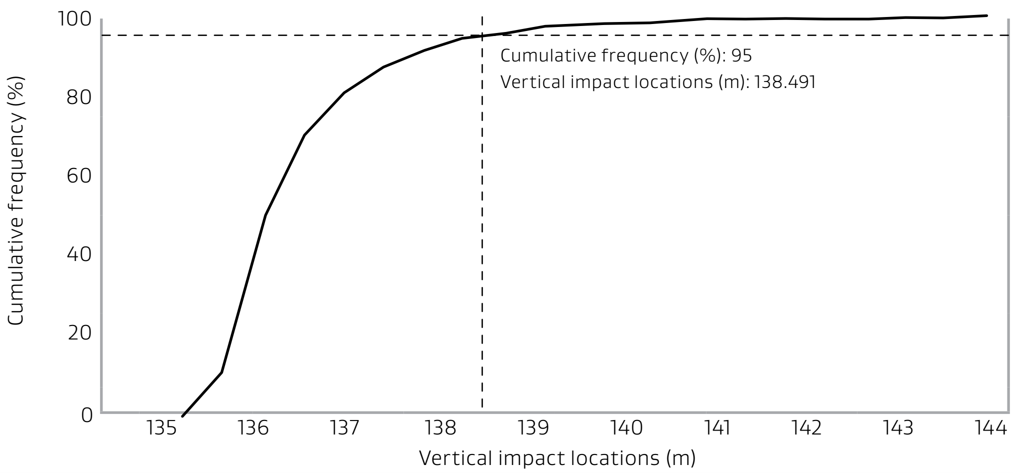


Total number of rocks on collector 1:
164

Total kinetic energy:
min = 1.92618
max = 983.134

Lower Rockfall Source (Basalt)

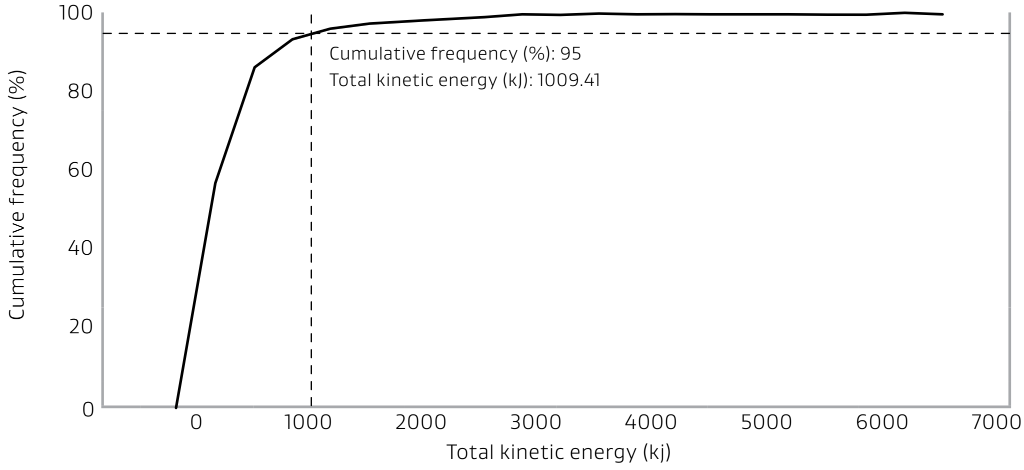
A7.6 95th Percentile Bounce Height (base of RPS is at 135.1m)



Total number of rocks on collector 1:
2894

Vertical impact locations:
min = 135.388
max = 143.814

A7.7 95th Percentile Energy on RPS

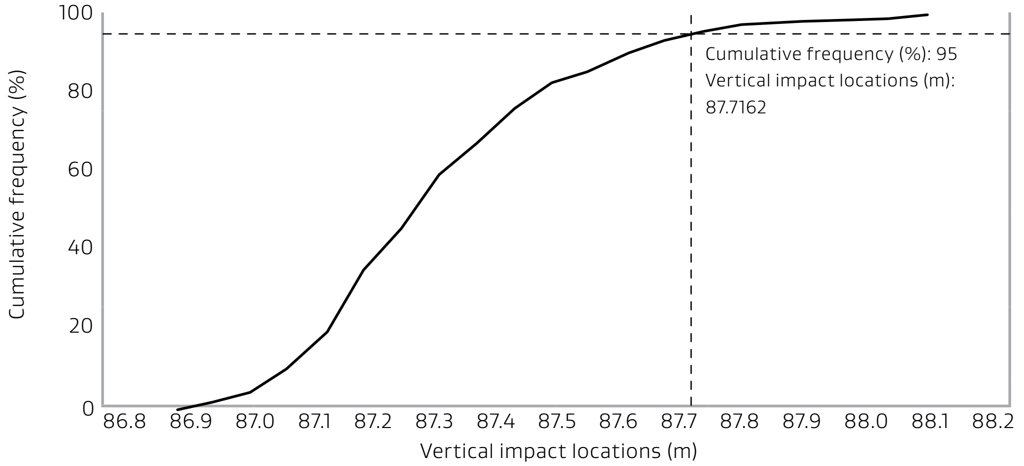


Total number of rocks on collector 1:
2894

Total kinetic energy:
min = 3.41405
max = 6367.65

A.8 Lower RPS (Rockfall Fence) Results

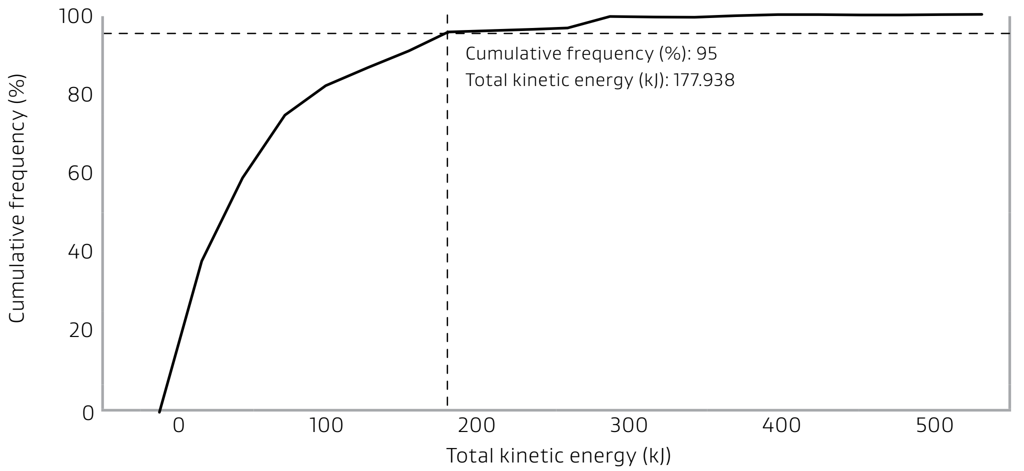
A8.1 95th Percentile Bounce Height to boulder centre (base of RPS is at 86.6m)



Total number of rocks on collector 2:
144

Vertical impact locations:
min = 86.9124
max = 98.0694

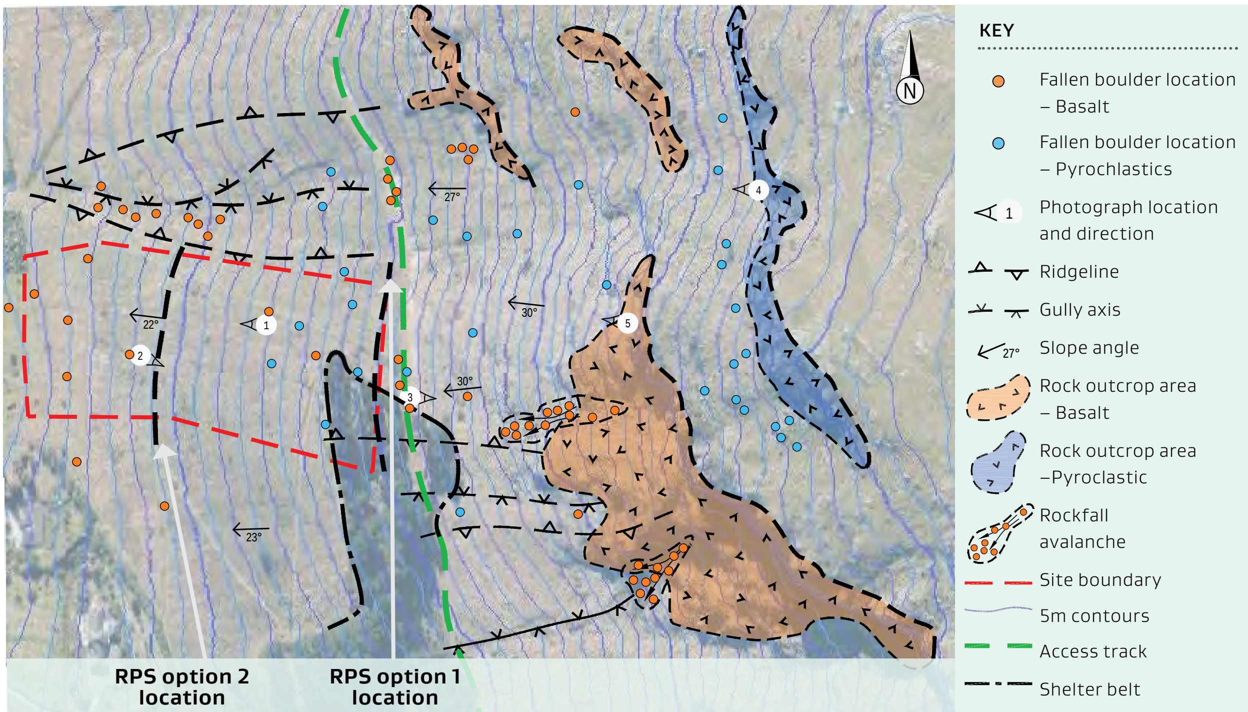
A8.2 95th Percentile Energy on RPS



Total number of rocks on collector 2:
144

Total kinetic energy:
min = 2.30787
max = 517.955

APPENDIX B – ATTACHMENT B.
TYPICAL ROCKFALL PROTECTION FENCE DRAWINGS



Components and materials

Post: HEA160 profile (UNI 5397), S275JR steel (EN 10025)

Base plate: dimensionss 400 x 550 mm, thickness 15 mm, S235JR steel (EN 10025)

Steel cables: ø 20 mm (6 x 36 + WSC) (EN 12385–4), wire tensile strength 1770 MPa

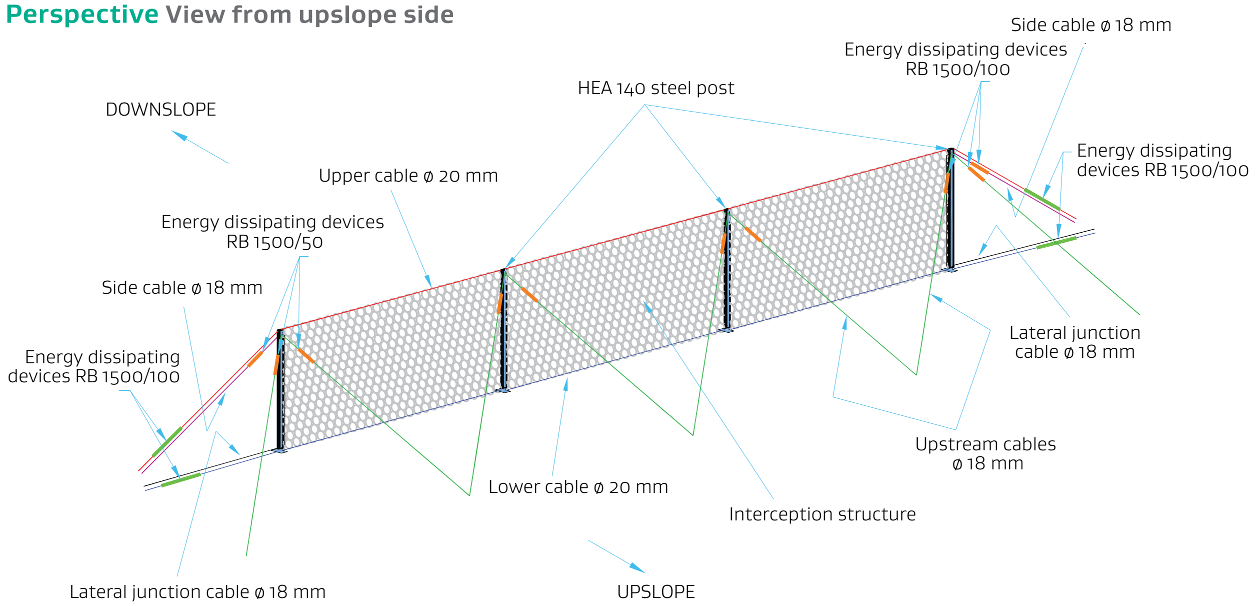
Steel ring panels: ring net panel type ASM 3–4–350/300, heavily galvanized wire ø 3.00 mm (EN 10244–2, Class A), wire tensile strength ≥ 1380 MPa

Additional net layer: hexagonal double twisted wire mesh, mesh type 8 x 10, wire ø 2.20 mm (EN 10223–3)

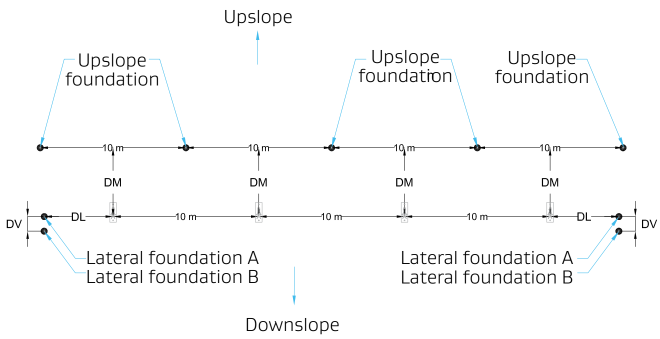
Shackles: Ushape M20, S275JR steel galvanized (EN 10025)

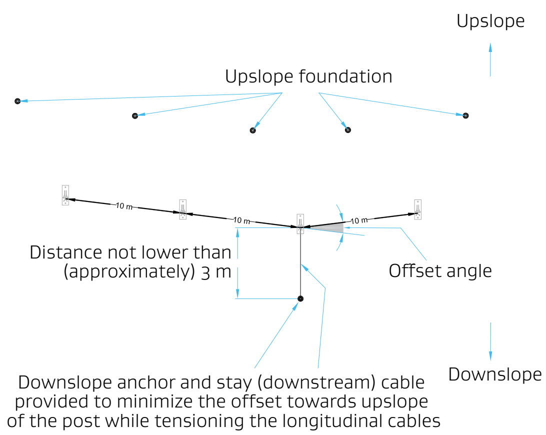
Clamps: for steel cables ø 20 mm (EN 13411–5)

Perspective View from upslope side



Foundation Plan view

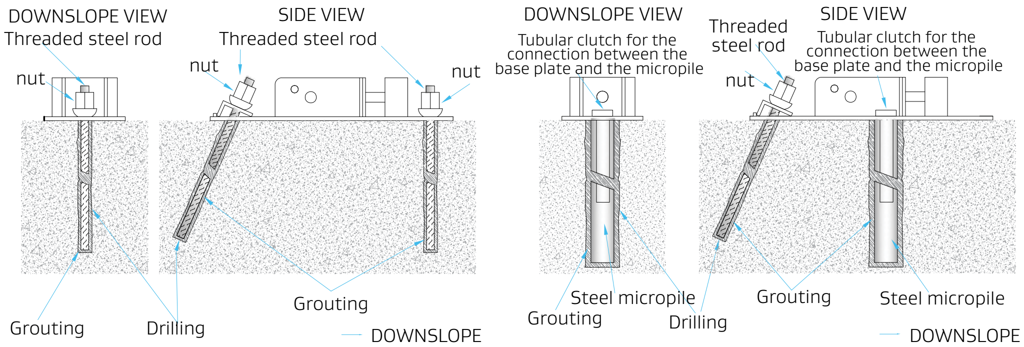




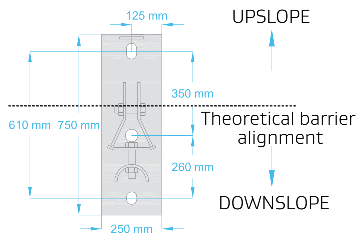
Note: DM, DV e DL values vary on the post height H, as shown below.
For further details refer to the 'Installation manual'.

H (m)	DM (m)	DV (m)	DL (m)
4.0	4.0	1.5	4.5
4.5	4.5	1.5	4.5
5.0	5.0	1.5	5.0

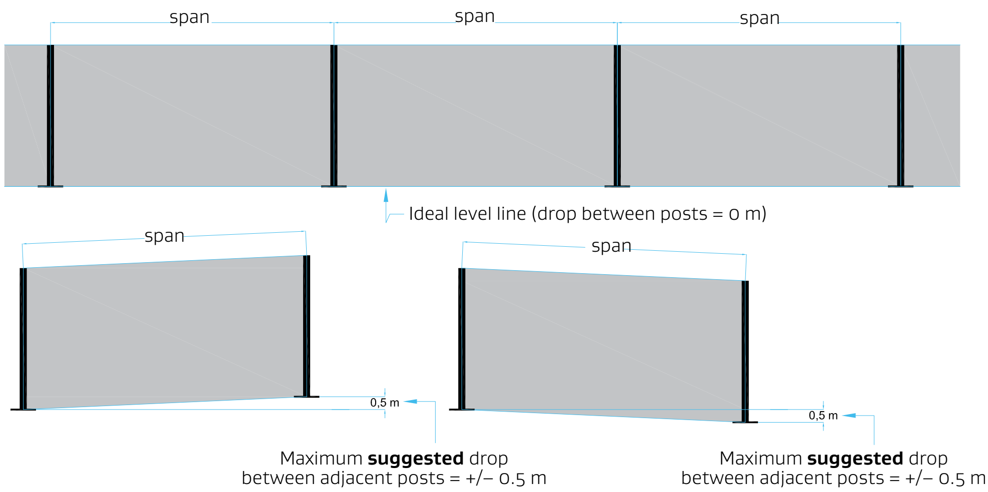
Foundations (with and without micropile) for Base Plate Cross sections



Base Plate Plan view

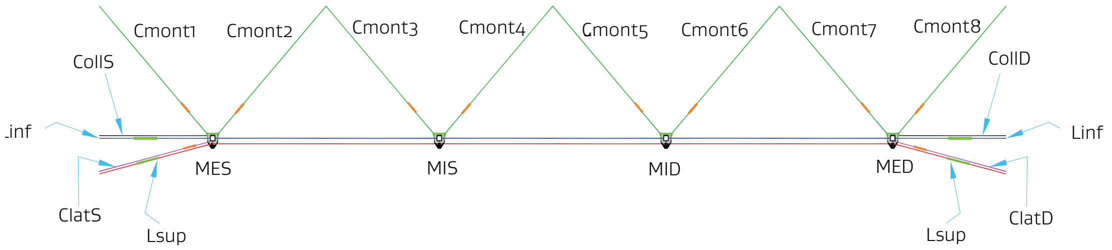


Scheme of maximum post drop Front view



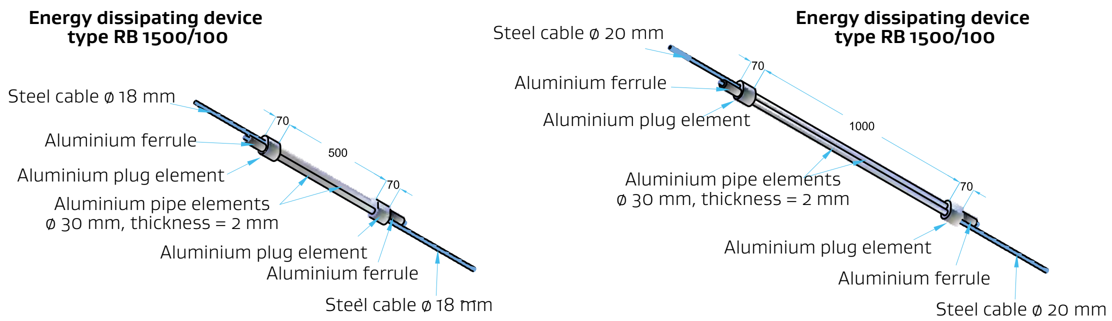
Note: the reported values refers to the standard barrier installation, and they are mainly intended to obtain an easy and quick assembly. Anyway, higher drops can be allowed without reducing the barrier functionality.

Schematic barrier with three spans Plan view

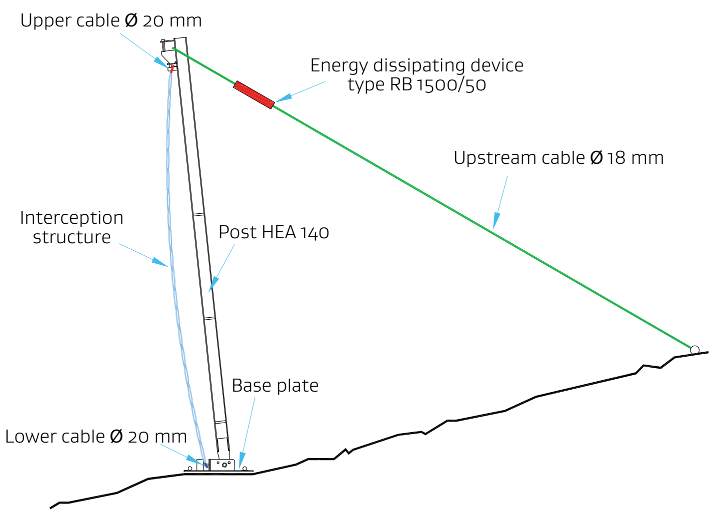


IES	left lateral post	Lsup	upper longitudinal cable
IIS	left internal post	Linf	lower longitudinal cable
IID	right internal post	ClatS	left side cable
IED	right lateral post	ClatD	right side cable
		CollS	left lateral junction cable
		CollD	right lateral junction cable
		Cmont1+8	upstream cables

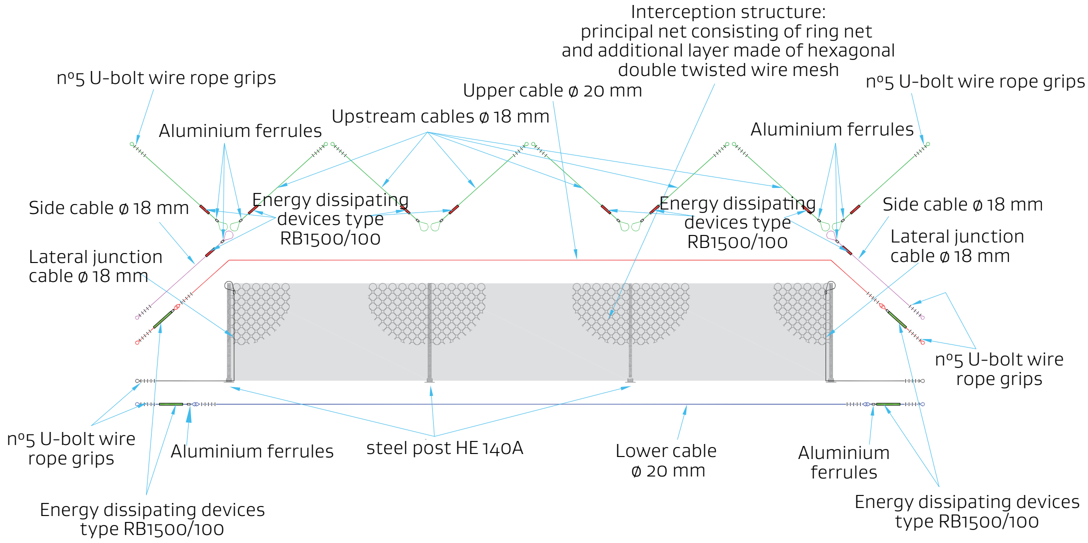
Energy dissipating devices XX



Rockfall barrier Side view

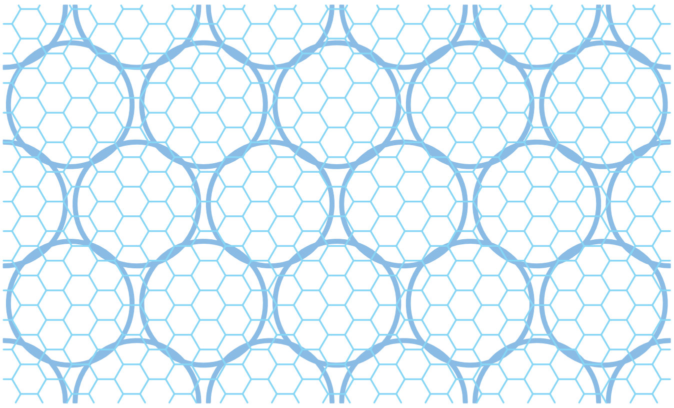


General scheme of components XX

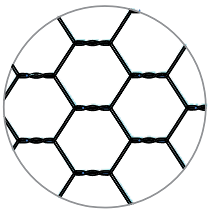


Principal and addional net layer details XX

Steel ring net consisting of rings having a diameter of approximately 350 mm and hexagonal double twisted wire mesh type 8 x 10, wire Ø 2,20 mm



Additional net detail



APPENDIX B – ATTACHMENT C. **WORKED EXAMPLE ROCKFALL FENCE DESIGN**

B–C1 Barrier height

from Piela & Ronco (2009):

$b_w := 1.7\ m$	95th%ile boulder width
$h_b := 3.3\ m$	95th%ile boulder bounce height (to centre)
$hp := \frac{b_w}{2} + h_b\ 4.15\ m$	Design interception height (bounce height + boulder radius)
$f := \frac{b_w}{2} = 0.85\ m$	Clearance height
$hi := hp + f = 5\ m$	Minimum Fence height

from Gimrod & Giacchetti (2014):

$\gamma T_r := 1.02$	Slope material properties defined on back analysis
$\gamma D_p := 1.02$	High accuracy of topographic survey
$\gamma R := 1.1$	Place frequented (post development) with high value

$$H_d := h_b \cdot \gamma T_r \cdot \gamma D_p + \frac{(b_w \cdot \gamma R)}{2} = 4.368\ m$$

Design trajectory height

$f_{min} := 0.5\ m$	Free zone
$H_{tot} := H_d + f_{min} = 4.868\ m$	Minimum barrier height

B–C2 Anchor design

Ground investigations at the site indicate a maximum thickness of approximately 1.6 m soil over Volcanic rock. For anchor design purposes, it is assumed that all anchors will be drilled into the rock to obtain adequate pullout resistance for the proposed RPS.

This worksheet calculates the maximum permissable anchor load for rockfall fence anchors.

Two cases are apparent:

a Lateral Foundation Anchors and
b Upslope Anchors.

Note that Downslope Anchors, where requried, are only to provide restraint to upslope movement of the corner post under tensioning. No specific load design has been assessed for these anchors.

B–C3 Assessment of grout to rock bond

Based on previous testing for a nearby site and previous projects, allow 500 kPa as grout to rock bond.

B–C4 Assessment of anchor length and bar diameter

a Lateral foundation anchors

$D_{gl} := 0.1\ m$	Diameter of the grout column around theanchor
$L_{al} := 3.0\ m$	Anchor length into volcanic rock
$F_{yl} := 442\ kN$	Ultimate tensile strength of the steel bar(s). GEWI 32mm bar assumed
$f_{gl} := 500 \cdot kP_a$	Grout-ground pullout resistance of the target founding strata (confirmed on the basis of sitespecific testing)

Note

This value should be supported by load testing, and should take into account the construction methodology – ie pressure grout will provide a greater resistance than gravityfed. If no testing data is available, it would be preferable to assess loading use the friction angle and at-rest earth pressure, which is outside the scope of this sheet.

$\Phi_s := 0.73$	Reduction factor on steel tensile strength (as a guide, FHWA recommends 0.55 for dead+live+earth loads, and 0.73 for seismic loads)
$\Phi_g := 0.67$	Reduction factor on grout-ground pullout resistance (as a guide, FHWA recommends 0.50 for dead+live+earth loads, and 0.67 for seismic loads. See also AS2159:1995 Tb 4.1 and Building Code B1/VM4 Tb 4)

$$\Phi_{Steel} := (\Phi_s \cdot Fyl) = 322.66\ kN$$

Calculated Bar Yield Stress

$$\Phi_{ground} := \Phi_g \cdot \pi \cdot f_{gl} \cdot D_{gl} \cdot L_{al} = 315.73\ kN$$

Permissable anchor load

Tensile Force on Lateral Anchors, for a 3000kJ barrier = 300.0 kN which is less than the permissable anchor load and bar yeild stress calculated above.

b Upslope foundation anchors

$L_{au} := 2.5\ m$	Anchor length into volcanic rock
$D_{gu} := 0.10\ m$	Diameter of the grout column around the anchor
$F_{yu} := 442\ kN$	Ultimate tensile strength of the steel bar(s). GEWI 32 mm bar assumed
$f_{gu} := 500 \cdot kPa$	Grout-ground pullout resistance of the target founding strata

$\Phi_{Steel} := (\Phi_s \cdot F_{yu}) = 322.66\ kN$

Calculated Bar Yield Stress

$\Phi_{ground} := \Phi_g \cdot \pi \cdot f_{gu} \cdot D_{gu} \cdot L_{au} = 263.108\ kN$

Permissable anchor load

Tensile Force on Upslope Anchors, for a 3000kJ barrier = 243.4kN. This is less than the permissible anchor load and bar yeild stress calculated above.

c Base anchors

The Manufacturer's base plate design calls for two anchors; one anchor inclined at approximately 60° from horizontal orientated upslope (to accomondate shear loading) and a second anchor on the downslope side (to accomodate compressive load)

c.1 Upslope Shear Anchors

$L_{au} := 2.00\ m$	Anchor length into volcanic rock
$D_{gu} := 0.10\ m$	Diameter of the grout column around the anchor
$F_{yu} := 339\ kN$	Ultimate tensile strength of the steel bar(s). GEWI 28mm bar assumed
$f_{gu} := 500 \cdot kPa$	Grout-ground pullout resistance of the target founding strata

$\Phi_{Steel} := (\Phi s \cdot Fyu) = 247.47\ kN$

Calculated Bar Yield Stress

$\Phi_{ground} := \Phi_g \cdot \pi \cdot f_{gu} \cdot D_{gu} \cdot L_{au} = 210.487\ kN$

Permissable anchor load

Tensile Force on Baseplate Shear Anchor, for a 3000kJ barrier = 207.6kN. This is less than the permissible anchor load and bar yeild stress calculated above.

C.1 Downslope Compressive Anchors

Load resistance at the tip of the anchor is ignored in the calculation below as it is likely to be insignificant compared to the shaft frictional resistance.

$L_{au} := 2.75\ m$	Anchor length into volcanic rock
$D_{gu} := 0.10\ m$	Diameter of the grout column around the anchor
$F_{yu} := 442\ kN$	Ultimate tensile strength of the steel bar(s). GEWI 32mm bar assumed
$f_{gu} := 500 \cdot kPa$	Grout-ground pullout resistance of the target founding strata

$\Phi_{Steel} := (\Phi_s \cdot F_{yu}) = 322.66\ kN$

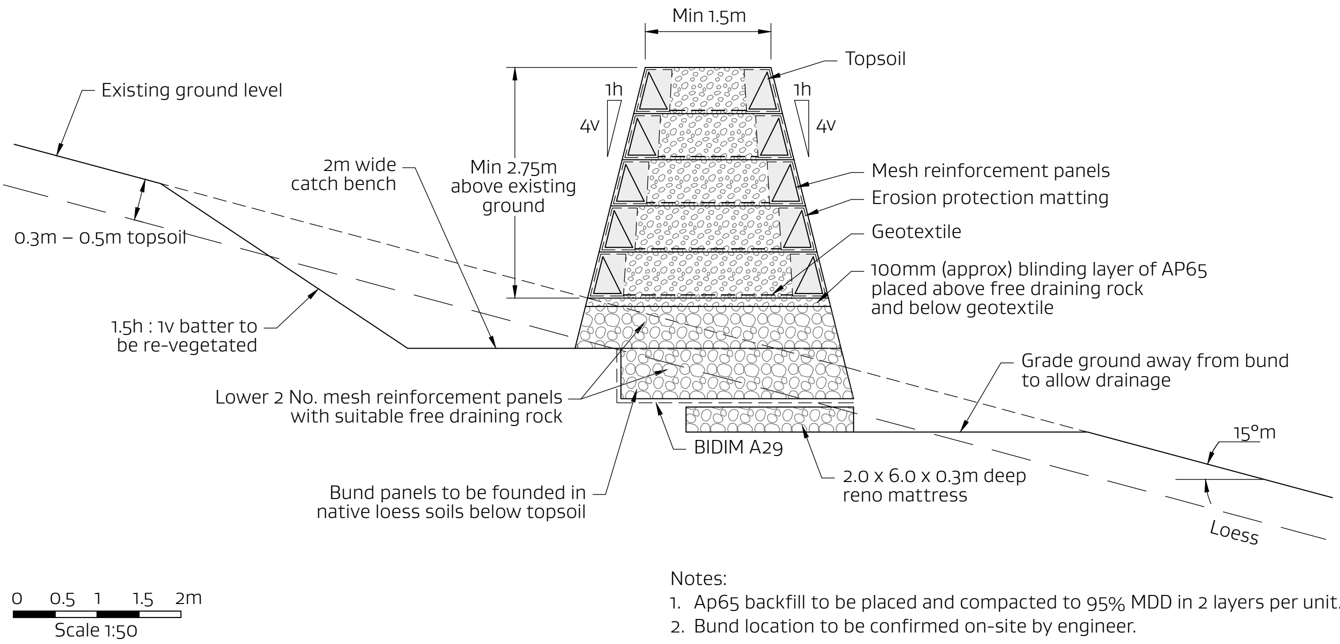
Calculated Bar Yield Stress

$\Phi_{ground} := \Phi_g \cdot \pi \cdot f_{gu} \cdot D_{gu} \cdot L_{au} = 289.419\ kN$

Permissable anchor load

Tensile Force on Baseplate Shear Anchor, for a 3000kJ barrier = 275.6kN. This is less than the permissible anchor load and bar yeild stress calculated above.

APPENDIX B – ATTACHMENT D.
TYPICAL EARTH BUND DESIGN DRAWINGS



Notes:
1. Ap65 backfill to be placed and compacted to 95% MDD in 2 layers per unit.
2. Bund location to be confirmed on-site by engineer.

APPENDIX B – ATTACHMENT E.
EARTH BUND DESIGN CALCULATIONS

B–E1 Introduction

This document includes the calculations undertaken for the detailed design of a reinforced earth bund above the site. The parameters considered in the design are presented below.

$b_w := 1.7\ m$ Boulder Width

$h_b := 1.1\ m$ Boulder bounce height (to centre of boulder)

$Đ\ h_{design} = +\ h_b\ Đ\ b_w\ 2\ 1.95\ m$
Height of design boulder trajectory (bounce height + boulder radius)

B–E2 Bund height

from Ronco et al (2009): $h_{design} - \frac{h_i}{\gamma_h} \leq 0$

$Đ_h := 1.1$ suggested as 1.1 based on Eurocode 7, Design Approach 2

$h_i := \gamma_h \cdot h_{design} = 2.145\ m$ Interception height (embankment height less upper soil layer)

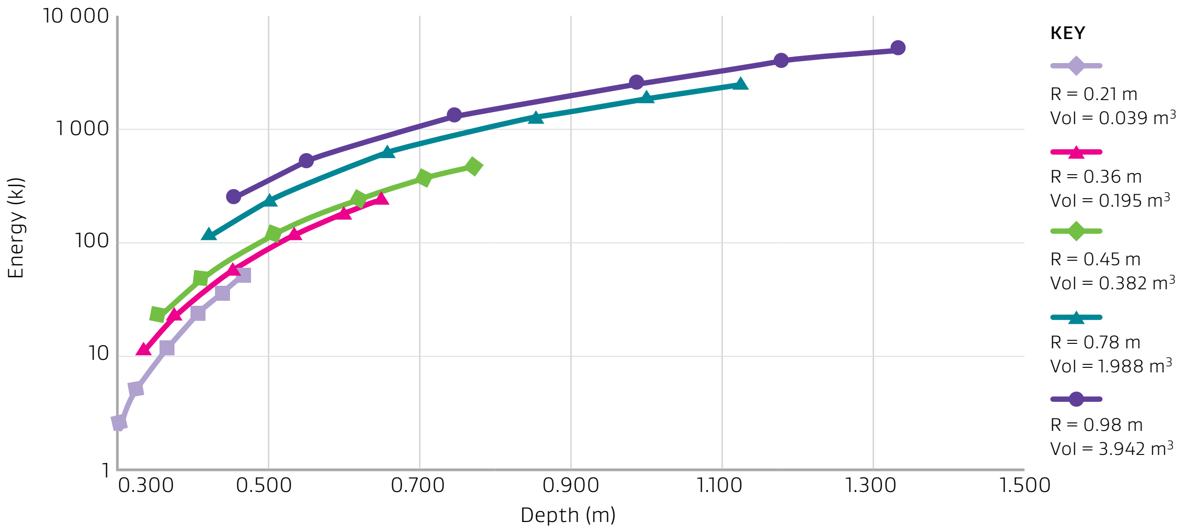
$l_s := 0.6\ m$ Soil layer thickness

$h_e := h_i + l_s = 2.745\ m$ Embankment Height

B–E3 Crest width

Figure 1 below shows the maximum penetration distance of various boulder sizes and energies.

Maximum penetration of falling boulders (ρ = 25 N/m³) (after Calvetti and Di Prisco, 2007 – redrawn)



From Figure 1, it is clear that greater penetration distances are achieved for smaller boulders assuming translational energy remains constant. Therefore we can assume penetration distances for a radius 0.98 m boulder in the diagram above. MBIE's design guidance indicates the following criteria for the minimum width of the bund:

- At least 2 times the anticipated penetration of the ULS boulder at the height of impact, and
- At least 5 times the anticipated penetration of the SLS boulder at the height of impact.

From rockfall modelling, the following energies are apparent:

$E_{ULS} := 518$ kJ

$E_{SLS} := 178$ kJ

Refering to Figure 1 and the ULS case, a line representing the ULS case intersects the curve for R = 0.78 m at approximately:

$p_{ULS} := 0.5\ m$ ULS penetration depth

For the SLS case:

$p_{SLS} := 0.42\ m$ SLS penetration depth (essentially the same penetration as ULS)

Therefore:

$w_{ULS} := 2 \cdot p_{ULS} = 1\ m$
Minimum embankment width to satisfy ULS condition

$w_{SLS} := 5 \cdot p_{SLS} = 2.1\ m$
Minimum embankment width to satisfy SLS condition

Therefore, the SLS case is the critical case and:

$u_f := h_e - h_{design} = 0.795\ m$
Distance between top of embankment and impact height

$\alpha := 70\ °$ Sideslopes of geogrid reinforced embankment

The minimum crest width of the bund will be required to be greater than or equal to the following:

$$t_E := w_{SLS} - 2 \cdot \frac{u_f}{tan(\alpha)} = 1.521\ m$$

The width of the base of the bund, tb can be calculated as follows:

$$t_b := + t_E + 2 \cdot \left(\frac{h_e}{tan(\alpha)} \right) = 3.519\ m$$

Bearing capacity

Check capacity using B1/VM4 method of the building code where:

- The bund is assumed to be 20 m long, has a 7 m base width and is founded on a 20° slope
- The subgrade is stiff to very stiff loess, use c_u = 50 kPa, or ϕ = 30 °, c' = 5 kPa

Constants in the bearing capacity calculation:

$B' := 7.0\ m$
$L' := 20\ m$
$D := 0.5\ m$
$\omega := 20°$
$D_e := 0\ m$
$\gamma' := 18\ \frac{kN}{m^3}$
$q(D,\gamma') := D \cdot \gamma'$
$q\left(0.5\ m, 18 = \frac{kN}{m^3}\right) = 9\ kPa$

For the undrained case, when $\phi = 0$

$c := 50\ kPa$
$N_c := 5.14$
$N_q := 1$
$N\gamma := 0$

The depth and load inclination factors equal 1, or very close to one and will not influence the bearing capacity calculation for both the drained and undrained case.

The shape and ground inclination factors are:

$$\lambda_{cs}\ (B', L', N_q, N_c) := 1 + \frac{B'\ N_q}{L'\ N_c}$$

$$\lambda_{cs}\ (2.6\ m, 43\ m, 1, 5.14) = 1.012$$

$$\lambda_{cg}\ (\omega, D_e, B) := 1 - \left(\omega \cdot \frac{\left(1 - \left(\frac{D_e}{2 \cdot B}\right)\right)}{150} \right)$$

$$\lambda_{cg}\ (10, 0.5, 2.6) = 0.94$$

Use the following as q.Nq is only 9kPa and $N\gamma = 0$, and therefore will not significantly affect the undrained bearing capacity:

$\lambda_{qs} := 1 \qquad \lambda_{\gamma s} := 1$

$\lambda_{qg} := 1 \qquad \lambda_{\gamma g} := 1$

$$q_u := (c \cdot \lambda_{cs} \cdot \lambda_{cg} \cdot Nc) + (q \cdot \lambda_{qs} \cdot \lambda_{qg} \cdot N_q) + \left(\frac{1}{2} \cdot \gamma' \cdot B' \cdot \lambda_{\gamma s} \cdot \lambda_{\gamma g} \cdot N_\gamma\right)$$

$$q_u := (50\ kPa \cdot 1.012 \cdot 0.94 \cdot 5.14) + (9\ kPa \cdot 1 \cdot 1 \cdot 1) + (0) = 253.479\ kPa$$

For the drained case, when $\phi = 30°$ and $c' = 5kPa$

$\phi := 30\,°$

$c := 5\ kPa$

$N_c := 26$

$N_q := 16$

$N_\gamma := 16$

$$\lambda_{cs}(B', L', N_q, N_c) := 1 + \frac{B'}{L'} \cdot \frac{N_q}{N_c}$$

$$\lambda_{cs}(7.0\ m, 20\ m, 16, 26) = 1.215$$

$$\lambda_{cg}(\omega, D_e, B) := 1 - \frac{\left(\omega \cdot \left(1 - \left(\frac{D_e}{2 \cdot B}\right)\right)\right)}{150}$$

$$\lambda_{cg}(10, 0.5, 2.6) = 0.94$$

$$\lambda qs := 1 + \frac{\left(B'\right)}{\left(L'\right)}\ tan(\phi) = 1.202$$

$$\lambda qg := \left(1 - tan\left(\omega \cdot \left(1 - \frac{D_e}{2 \cdot B}\right)\right)\right)^2$$

$$\lambda qg := \left(1 - tan\left(10\,° \cdot \left(1 - \frac{0.5}{2 \cdot 2.6}\right)\right)\right)^2\ 0.707$$

$$\lambda_{\gamma s} := 1 - 0.4 \frac{\left(B'\right)}{\left(L'\right)} = 0.86$$

$$\lambda_{\gamma g} := \lambda_{qg} = 0.707$$

$$q_u := (c \cdot \lambda_{cs} \cdot \lambda_{cg} \cdot N_c) + (q \cdot \lambda_{qs} \cdot \lambda_{qg} \cdot N_q) + \left(\frac{1}{2} \cdot \gamma' \cdot B' \cdot \lambda_{\gamma s} \cdot \lambda_{\gamma g} \cdot N_\gamma\right)$$

$$q_u := (2\ kPa \cdot 1.037 \cdot 0.94 \cdot 26) + (9\ kPa \cdot 1.032\ 0.707 \cdot 16) + \left(\frac{1}{2} \cdot (18 \cdot 2.6)\ kPa \cdot 0.976 \cdot 0.707 \cdot 16\right) = 414.102\ kPa$$

The applied load of approximately 5 m of fill at the downslope side of the bund is:

$$l_{app} := 5\ m \cdot 18\ \frac{kN}{m^3} = 90\ kPa$$

$$FoS := \frac{q_u}{l_{app}} = 4.601$$

which is acceptable under short term loading.

B–E4 Internal stability of the bund

Internal stability of geogrid reinforced earthbunds is unlikely to be a significant design concern as the bund is not subject to external soil load. Design of the geogrid reinforcement can be undertaken either in consultation with the manufacturer or with slope stability analysis software.

APPENDIX C. **SWITZERLAND REGULATORY ROCKFALL RISK MANAGEMENT PROCESS**

This document describes Switzerland's approach to natural hazard risk assessment and mitigation. The Swiss approach is outlined in its institutional framework policy.

C1 Overview on risk analysis policy for mitigating natural hazards in Switzerland

PLANAT (the national platform for natural hazards) has been developing and coordinating the Swiss policy for almost 20 years.

Motivated by the Danioth motion (1999), the Swiss Federal Council commissioned PLANAT to develop a comprehensive and interlinked strategy to improve protection against natural hazards. The Federal Council emphasised that protection against natural hazards should not only be provided for residents of the Alpine region, but for the entire Swiss population. It also aimed to ensure comparable security standards throughout Switzerland based on extensive risk management to protect people and property.

To date, PLANAT has completed the first step , which involved developing a comprehensive and interlinked strategy for improving protection against natural hazards; the second step, which involved analysing the current situation and proposing an action plan with measures; and the third step (action plan 2005–2008) and fourth step (action plan 2009–2011), which involve implementing these measures.

In order to evaluate progress made in implementing the natural hazards strategy, strategic controlling was carried out in 2013 (http://www.planat.ch/en/specialists/strategy-natural-hazards/, 21 June 2016).

C2 Integrated risk management

According to the PLANAT strategy, integrated risk management implies achieving a comparable security level for all natural hazards. Numerous actors bear responsibility for protecting against natural hazards, either because they are legally obligated or they assume individual responsibility. All responsible actors must be involved in the planning and implementation of protection measures. In this process it is relevant to consider not only one type of measure but also the full spectrum of possible measures (http://www.planat.ch/en/specialists/risk-management/, 21 June 2016).

Integrated risk management is informed by comprehensive data and information about the occurrence of hazards and the risks they pose. The measures used to control risks are diverse, need to be combined in an optimal way and cover the three phases of the risk management cycle: mitigation preparedness, response and recovery.

Definitions (from Camenzind, R.; Loat, R. (2014): risk-based spatial planning: synthesis report on two case studies at communal land use planning level. National Platform for Natural Hazards/Federal Office for Spatial Development/Federal Office for the Environment, Bern. 21p.

C3 Risk

In assessing the extent and probability of the occurrence of damage caused by natural hazards, characteristic parameters to take into account include average annual damage and the extent to which damage may reoccur. Characteristic parameters include the average annual damage and the extent to which damage may reoccur.

Risk analysis

The following process is used to characterise and quantify a risk in relation to the probability of occurrence and extent of damage. In case of a road, annual object risks and individual lethality can be calculated as: Annual object risk (collective risk CHF/year) for a road section:

$r_{i,j} = h_e \times P_{rA} \times N_F \times \alpha \times \beta$

Individual lethality is calculated by:

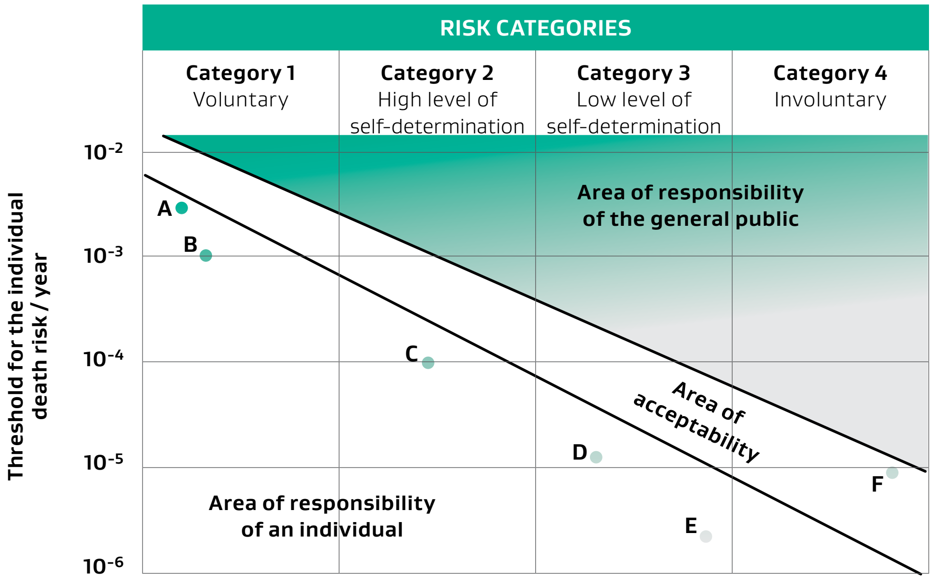
$T_{c,d\ ind} = \frac{r_{i,j}}{MDT \times \beta} \times X$

whereas

he	frequency of occurrence
prA	spatial probability of occurrence
pSp	probability of stoppage
NF	quantity of affected vehicles
λ	lethality
β	mean occupancy per vehicle
MDT	mean daily traffic
g	endangered section of road
v	mean velocity
f	conversion factor
X	1 person which passes the hazard zone X times per day

C4 Risk categories

Figure 1: Assessment of death risk (Merz et al., 1995)* Source PLANAT



C5 Risk governance

Risk governance provides answers to three key questions:

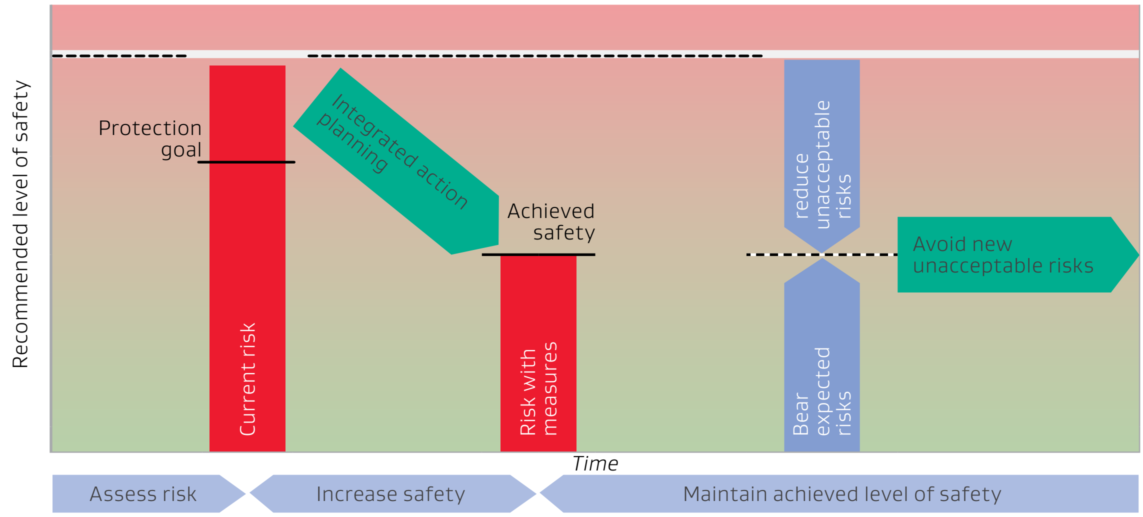
QUESTION	ANSWER
What can happen?	Risk identification and analysis is a systematic, science-based process. Both the intensity and frequency of natural hazards and the expected consequences (damage, losses) are analysed.
What is allowed to happen?	The evaluation and balancing of risks is a social process to distingush between acceptable and unacceptable risks. A risk that is considered permissible for good reason is an acceptable risk.
What has to be done?	Measures are implemented to avoid future risk, to reduce existing risks to an acceptable level and to manage the remaining risks with individual approaches.

C6 Safety levels and protection goals

Procedure for achieving and maintaining the recommended level of security:

- **Monitor and assess risks:** periodic monitoring is required to determine what action is needed.
- **Increase security:** to reduce the risk, measures are planned and implemented as part of an integrated process.
- **Maintain achieved security level:** all actors jointly maintain the achieved security level, in particular through spatial planning measures.

***Figure 2: Approaching protection by definition of protection goals and safety levels*[1]**



Recommended security level

***Figure 3: Protected objects in accordance with the PLANAT recommendation*[2]**

CATEGORY	PROTECTED OBJECT	PROTECTION OBLIGATION	WHAT IS PROTECTED?
Human beings	Person	Human life and health	The individual
Major material assets	Buildings	Property	
	Infrastructure	Swiss economy	The society
	Objects of considerable economic significance or scope	Swiss economy	
	Essential natural resources for livelihoods	Natural resources	
	Cultural goods	Cultural heritage	
Environment	Nature, environment	Nature	The environment

C7 Major material assets

Buildings have to provide a high level of protection to people and property. They have to be resilient and not pose any threat to people and property. The residual risks to people and property are acceptable to the risk carriers [. . .][2].

Swiss approach to risk analysis in natural hazard protection

Based on an ongoing protection project against rockfall, the practical use of risk analysis tools are illustrated (see figures 4 and 5 for workflow).

Objectives

Risk analysis is carried out to define cost-effective measures to respond to defined protection goals (see figure 3) having carried out field analysis.

Field work

Preliminary field work involves establishing a hazard map of a rockfall scenario for three defined probability classes (return period 0 to 30 years, 30 to 100 years, and 100 to 300 years). In this process block sizes for each return period are defined taking into account joint systems, fractures and observed rockfall blocks in the spread area (see figure 4, step 1 and 2).

Rockfall modelling and evaluation

Rockfall modelling allows the ability to define energy and bounce heights for each scenario. In order to establish intensity maps, the results are sorted within three classes:

- low intensity (0 to 30 kJ)
- medium (30 to 300 kJ). and
- high intensity (>300 kJ).

Classes correspond to rockfall energies:

- 30 kJ can be stopped by a tree
- 30 to 300 kJ by a concrete wall, and
- 300 kJ by designed rockfall protection systems (see figure 4, step 3 and 4).

Mapping of rockfall intensity and risk analysis

These maps are used as an input parameter in risk analysis containing information about probability and spatial distribution of classified rockfall intensity. Potential damage is crucial when evaluating risk. Factors include:

- type and vulnerability of construction
- number of occupying people, and
- average frequency by people.

Cost-effectiveness of measures are evaluated based on the assumption that up to 5 million Swiss Francs (CHF) is design life value. The definition of cost-effective protection measures is achieved by comparing risk reduction per year (in CHF) to the annual costs of protection measures. Establishing this ratio for each option enables the ability to define the most cost-effective protection measure (see figure 6).

1 Camenzind, R.; Loat, R. (2014): risk-based spatial planning: synthesis report on two case studies at communal land use planning level. National Platform for Natural Hazards / Federal Office for Spatial Development/Federal Office for the Environment, Bern. 21p.

2 PLANAT, (2014): Security Level for Natural Hazards. National Platform for Natural Hazards PLANAT, Bern. 15 p

Figure 4: Workflow of a natural hazard protection project in Switzerland, hazard assessment

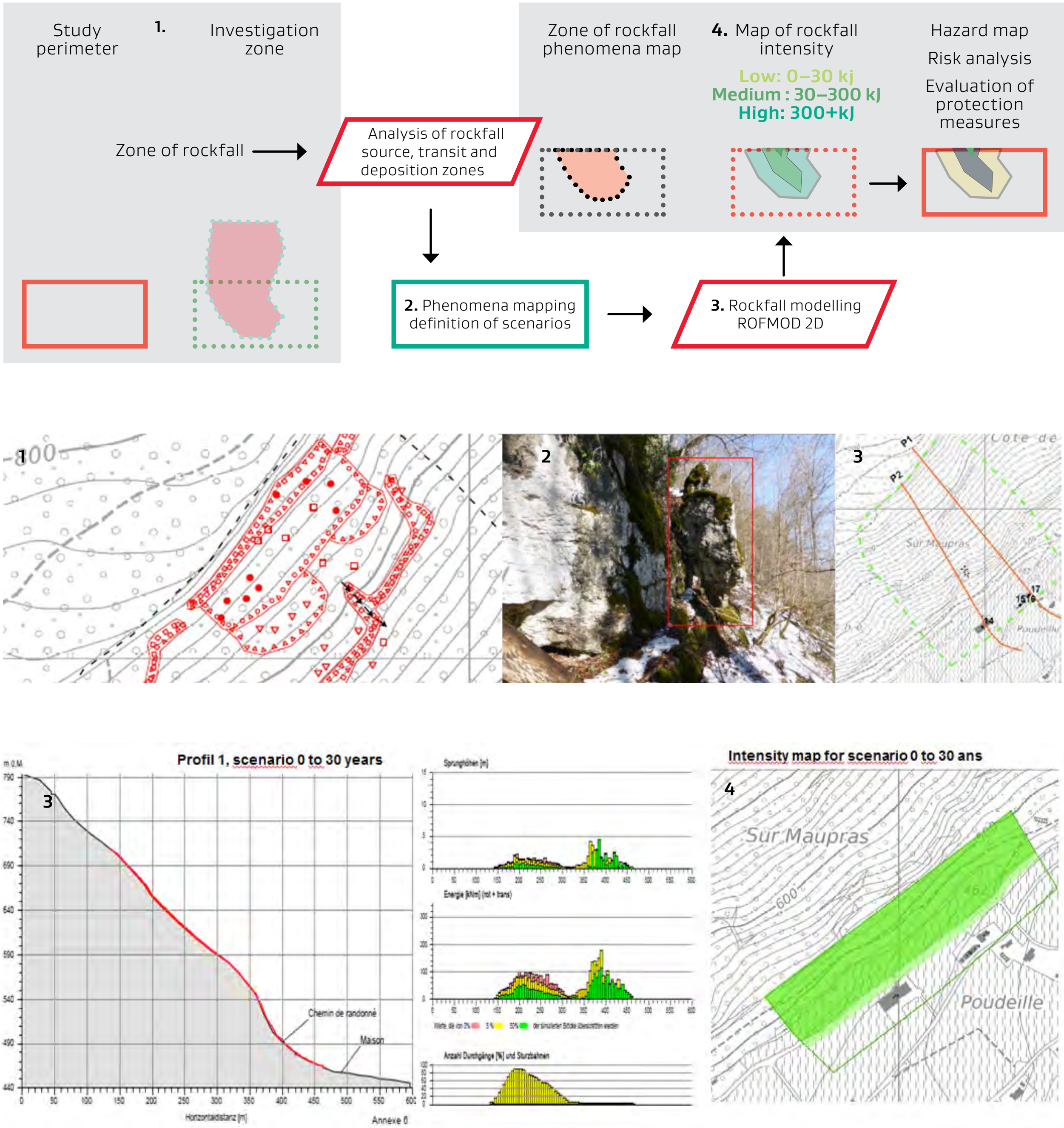
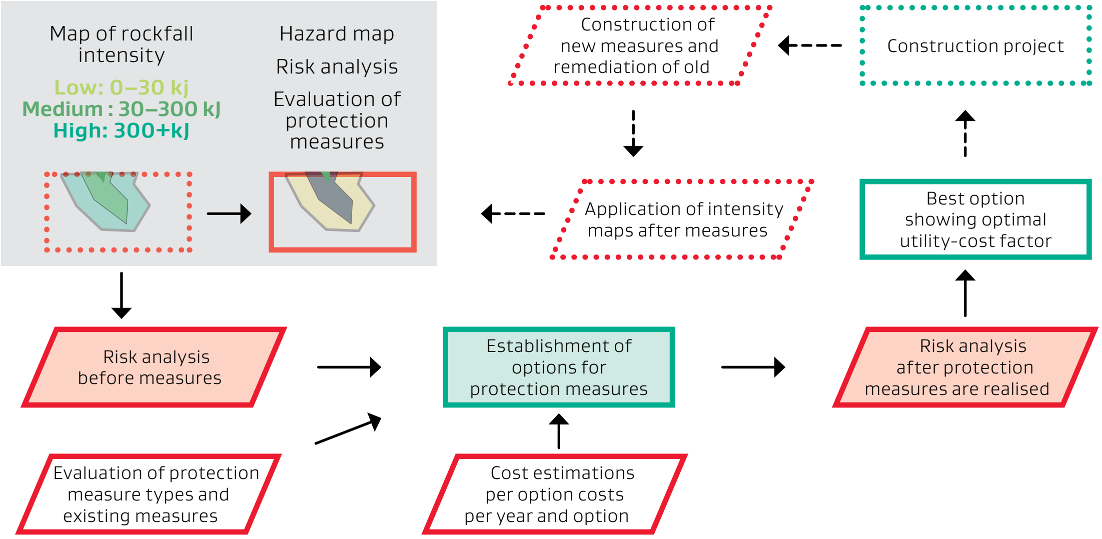


Figure 5: Workflow of a natural hazard protection project in Switzerland, risk analysis

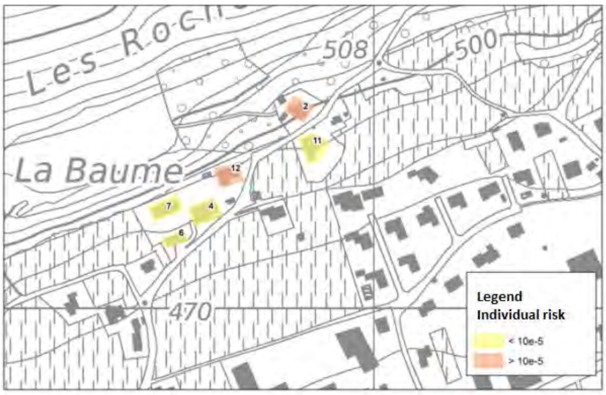


Results of risk analysis practical case study

Results of risk analysis: Figure 6 shows a map of classified individual risks probability <10–5 < probability per house as well as low collective risk per year (1,622 CHF/year). In two houses (2 and 12), individual death risks cover the threshold of 10–5. These two houses are affected by rockfall of higher promontories. Houses 4, 6 and 7 are protected from these rockfalls by the large terrace which is visible on the map near La Baume.

Figure 6: Map showing classified individual risks probability <10–5 < probability

Right: collective risk per year (1,622 CHF/year)
Below right: Individuel death risk quantified for houses 2 and 12



COMPILATION OF DAMAGE EXTENT WITHOUT AVERSION			
CATEGORY	SCENARIO 30	SCENARIO 100	SCENARIO 300
Buildings	9617 CHF	38 085 CHF	110 245 CHF
Special objects	0 CHF	0 CHF	0 CHF
Road traffic	0 CHF	0 CHF	0 CHF
Pipes	0 CHF	0 CHF	0 CHF
Mechanical ascent help	0 CHF	0 CHF	0 CHF
Agriculture, forest and parks	0 CHF	0 CHF	0 CHF
Rail traffic	0 CHF	0 CHF	0 CHF
Special objects rail traffic	0 CHF	0 CHF	0 CHF
Persons	773 CHF	3 751 CHF	219 858 CHF
Total damage extent	10 390 CHF	41 836 CHF	330 103 CHF
Damage extent persons	0.00015456	0.00075024	0.0439716
OVERVIEW INTEGRAL RISK/YEAR	**ALL SCENARIOS**		
Risks real value			846 CHF/a
Risks persons			776 CHF/a
Total risk			

NR. ECONOME	BUILDING TYPE	INDIVIDUAL RISK
2	Accommodation unit (at 224 people according to BSF)	2.504e-0.5
12	Accommodation unit (at 224 people according to BSF)	2.504e-0.5

Figure 7 shows a protection option with two rockfall barriers and an intensity map (with a return period of 30 years) after placing barriers. House 12 still faces low rockfall energy from the lower rockwall (house directly situated under a rockface). However, the resulting low energies can be tolerated and do not determine the risk situation. Main risk comes from an event with a 100 to 300-year return period.

Protection designed against these events resulting from the higher promontory. Cost- effectiveness is very low due to high costs and relative low monetarised risk per year. The barriers will nevertheless be constructed due to the non-acceptable risk situation. The study of different solutions has also shown that measures other than rockfall barriers (like rockfall dams) are technically not constructible due to steep rock flanks and relative high energies.

Figure 7: Rockfall protection option, its influence on the 0 to 30 return period intensity map and very low cost-effectiveness values for the design solution.

RISK CHF/A	RISK CONTRIBUTION SCENARIO 30 IN CHF/A	100 IN CHF/A	300 IN CHF/A	TOTAL
Before measures	242	279	1 100	1 622
After measures	112	167	140	420
Risk reduction (utility) CHF/a	130	112	960	1 202
			costs of measures	7 786
			gain/utility – rate	0,2

C8 References

1 Camenzind, R.; Loat, R. (2014): risk-based spatial planning: synthesis report on two case studies at communal land use planning level. National Platform for Natural Hazards / Federal Office for Spatial Development/Federal Office for the Environment, Bern. 21p.

2 PLANAT, (2014): Security Level for Natural Hazards. National Platform for Natural Hazards PLANAT, Bern. 15 p