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Design of rockfall protection embankments: A review

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Rockfall protection embankments are elevated massive structures designed to stop falling rock blocks. This paper discusses two specific facets of their design: their ability to control a block's trajectory and to withstand the impact. Based on a critical review of the literature and the state of the art, this article treats the main features in the design of rockfall protection embankments and highlights the limitations of the methods used today. Rockfall simulation codes lack of precision as they are generally not calibrated nor validated on the block's flying heights and velocities. And, analytical methods used to assess the response of embankments to impact globally fail in satisfactorily considering all the mechanisms involved and estimating their magnitude. These points should be considered by design engineers and could potentially lead to future improvements in the design of rockfall protection embankments.

1. Introduction

Rockfall is the most frequent natural event in mountainous areas. Every year blocks ranging from a few kilograms to tens of tons cause traffic interruptions, damage to structures and vehicles, and sometimes fatalities. Possible countermeasures against rockfall include active and passive solutions, either preventing the event from occurring or protecting at-risk structures from the rockfall event. In the latter case, embankments can be used to stop or to deviate the blocks on their route down the slope.

Embankments are massive earthworks constructed perpendicular to the slope, from 3 to 25 m high and up to a few hundred meters long. A ditch is most often associated with the embankment to collect

the intercepted blocks. Two types of ditches predominate: (1) a ditch with a dust road allowing the circulation of heavy vehicles for fallen block removal, particularly in areas where frequent or large-volume events are expected; (2) a catchment ditch made of loose material designed to dissipate the block's energy before it reaches the embankment (Peckover and Kerr, 1977; Hoek, 2007; Maegawa et al., 2011).

Embankments are appropriate when medium- to very-high-kinetic-energy events are expected, from a few hundred kilojoules to tens of megajoules. They are preferred over net fences when the design impact is higher than 5000 kJ (Descoedres, 1997). The other advantages are low maintenance costs and reduced visual impact (Brunet et al., 2009). Nevertheless, they are not appropriate on steeper slopes and their construction generally requires extensive space and accessibility for heavy vehicles. For large structures, designed for high-kinetic-energy blocks, the ditch is dug in the slope uphill of the structure and

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the cut materials are generally used to erect the embankment. Embankments are most often reinforced with horizontal inclusions such as geosynthetics, substantially reducing the volume of the structure as well as increasing the slope ratio of its mountain-side facing.

Originally, rockfall protection embankments were mainly made from compacted natural soil and were designed for rather low-impact-energy events. Most often, their cross-sectional shape was trapezoidal. Sometimes, the mountain-side facing was steepened with a concrete wall, gabions or prefabricated concrete components (Paronuzzi, 1989). At the end of the 1980s, ground-reinforced structures were developed to reach higher projection energies, up to 100 MJ (Mathieu and Marchal, 1989; Morino and Grassi, 1990).

Nowadays, a wide variety of embankments exist, differing mainly in their construction materials. A descriptive comparison of some of these structures is given in Peila (2011). Table 1 lists the possible materials for the structure's mountain-side facing, core and valley-side facing.

Various combinations can be found between these three lists, some extremely rare and others common worldwide or found only in certain countries (Figure 1). For instance, in France interconnected soil-filled recycled tires are frequently used as facing material only or as both core and facing materials.

The most common structures in the world are soil-reinforced structures. The variety of these structures has considerably increased over the last two decades, employing different types of reinforcement materials and reducing both their footprint and visual impact. Many of these structures are presented in technical publications (Mathieu and Marchal, 1989; Morino and Grassi, 1990; Gerbert and Hespel, 1999; Mannsbart, 2002; Cargnel and Nössing, 2004; Coulon and Bruhier, 2006; Jaecklin, 2006; Rimoldi et al., 2008; Brunet et al., 2009; Hara et al., 2009; Simmons et al., 2009; Lorentz et al., 2010).

The design of an embankment requires addressing issues related to its influence on the block's trajectory and structure stability. The latter concerns the slope's ability to support the earthwork mass, its ability to withstand the forces of gravity and the block's impact without collapsing. In addition, particular attention must be paid to rainwater, avalanches and debris flows so the embankment does not become an obstacle to their flow. Nevertheless, the hydraulic, slope stability and internal stability under gravity issues are not specific to rockfall protection embankments (Paronuzzi, 1989; Koerner, 2005). On the contrary, the design with respect to the block's trajectory and the structure's stability under impact are highly specific. These two aspects are successively treated in this article with the aim of identifying the potential and limitations of the methods and tools used today.

Table 1
Materials used to build rockfall protection embankments.

| Mountain-side facing | Core | Valley-side facing |
|-----------------------|---|---------------------|
| Interconnected tires | Soil reinforced with GSY ^a | Gabion ^b |
| GSY ^a | Soil reinforced with interconnected tires | Soil |
| Metallic wire mesh | Soil reinforced with metallic wire mesh | GSY ^a |
| Cast iron panel | Soil reinforced with wood and steel | Tires |
| Gabion ^b | Soil bag ^c | Timber |
| Soil bag ^c | Gabion ^b | |
| Soil | Compacted soil | |
| Timber | | |
| Concrete | | |
| Riprap | | |

^a Geosynthetics, such as geotextiles or geogrids.

^b Woven wire mesh (hexagonal) or welded wire mesh cages filled with either coarse or fine granular materials.

^c Typically a sand-filled geotextile sock.

2. Design with respect to block trajectory control

For a given area to be protected against rockfall, the optimum mitigation strategy is defined based on many criteria such as the type of element at risk, the site's topography, the block detachment frequency, and the trajectory analysis (Agliardi et al., 2009). Trajectory analysis methods and their use have been discussed in detail in Dorren et al. (2011). Trajectory analysis can provide a statistical distribution for the flying height and velocity at any given point on the slope for a given block of a certain mass (Agliardi and Crosta, 2003; Giani et al., 2004; Bourrier et al., 2009). Different protection scenarios can be envisaged, and building an embankment is one of the possible solutions (Calvino et al., 2001; Volkwein et al., 2011).

Basically, the embankment is designed to be an impassable obstacle for falling blocks. Reaching this goal involves satisfying embankment cross-sectional shape requirements, depending on its position along the slope. The literature on the efficiency of embankments in controlling the block's trajectory is rather poor. The general design method is described in the following section, with reference to several relevant publications, and the limitations of this design are identified.

2.1. General method

Fig. 2 proposes a four-step flow chart for embankment design. It provides a general framework that can be adapted depending on the context, method and trajectory analysis tool used.

The first step makes use of trajectory analysis data. The positions where the block flying height (h_f) and kinetic energy (E_{kin}) values are minimum allow determining eligible sites as embankment positions along the slope (step 1). The embankment's optimal position also depends on the number of blocks to be stopped. Indeed, the lower these values are, the smaller and less expensive the structure is. Eligible sites are most often located at the end of the transit zone, close to the deposit zone.

Step 2 consists in evaluating the accessibility of the site to earth-moving equipment. This phase may induce embankment position changes for practical and financial reasons.

For the selected position, the embankment height and mountain-side batter are established based on the statistical analysis of the trajectory analysis results (step 3). The embankment height is defined based on the reference block flying height (Masuya et al., 2009), a maximum deduced from the statistical analysis of the rock trajectory envelope (Figure 3). It is increased by a freeboard. The batter of the mountain-side is defined so as to prevent blocks from bouncing or rolling over the structure. The literature on these two parameters is rather poor due to insufficient research results. In Austria, the recommended free-board depends on the situation with a minimum of 1.5 times the block radius (ONR, 2012). In France, a batter less than 25° and a freeboard equaling the block radius are recommended (Calvino et al., 2001). Lower batter values are sometimes mentioned in specific studies (Simmons et al., 2009).

The efficiency of the embankment in stopping blocks also depends on the ditch width and mechanical characteristics. In an optimization process, these parameters may be considered along with the embankment height and batter. The altitude of the ditch is defined based on the toe of the embankment. Digging this ditch often requires modifying the uphill slope, depending on its position (Figure 3).

Step 3 may involve an optimization process to define ditch width, embankment batter and height values so as to minimize construction costs. For this purpose, and mainly for large structures, consideration may be given to the balance between the cut volume and embankment volume.

The next step consists in estimating the effective residual risk (step 4) (Paronuzzi, 1989; Agliardi et al., 2009; Kister and Fontana, 2011), carried out following a trajectory analysis conducted based



Fig. 1. Illustration of the variety of rockfall protection embankments (1: Coulon and Bruhier, 2006; 2: Lorentz et al., 2010; 3: Simmons et al., 2009; 4: Ronco et al., 2010; 5: author's collection; 6: Willye, 2003; 7 and 8: author's collection).

on the new profile to assess the structure's projected efficiency in stopping blocks.

This chart provides a general frame, with some of the steps irrelevant or unnecessary in certain situations. The position of at-risk structures often forces the embankment position. For instance, steps 1 and 2 may not be necessary for protecting transportation lines, as embankments are often placed along these lines. Steps 2 and 4 may require returning to previous steps in the process.

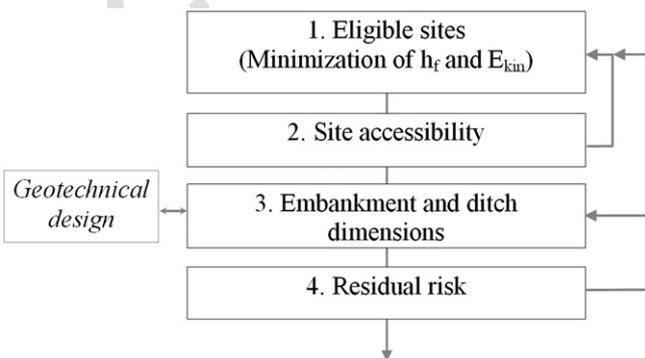


Fig. 2. Flow chart for the block trajectory control-oriented design of embankments.

Finally, budget and time constraints may affect this flow chart.

Moreover, this flow chart may be influenced by geotechnical design needs. Step 3 provides two characteristics related to the shape of the embankment: batter and height. The definition of the full embankment cross-sectional shape also requires the crest width and the inclination of the valley-side facing. These dimensions are defined based on the mechanical analysis so that the embankment can withstand impacts. The crest width depends on the impact energy and the structure type. For soil-reinforced structures, this width must be sufficient to allow compaction machinery to operate, with a minimum

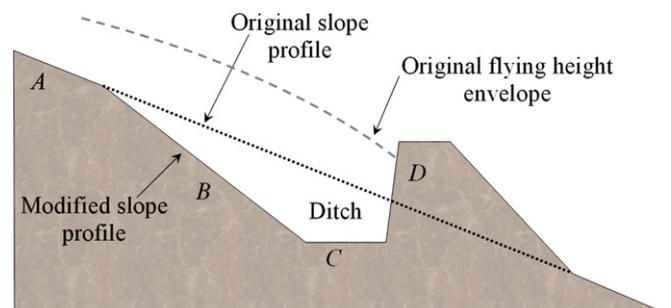


Fig. 3. Profile in an embankment with natural slope reshaping.

recommended value of 2 m. (Calvino et al., 2001). The valley-side batter may be defined depending on the proximity of at-risk structures (roads) or on valley-side visual requirements. The volume of the embankment is estimated based on the embankment geometry. The slope stability is then checked taking into consideration the embankment and the possible uphill ground removal. If the slope is not stable, the position or dimensions of the embankment will be modified (step 1 or 3).

2.2. Discussion

As for all rockfall protection structures, the trajectory analysis plays an important role in the design of embankments. For this reason, the relevance and reliability of the methods and currently used tools should be addressed. In particular, questions concerning the statistical analysis of the results, rebound modeling and the characteristics attributed to the materials should be raised.

The trajectory analysis provides statistical distributions for the flying heights and kinetic energy of the boulder, which are used to identify possible construction sites, defining the main geometrical characteristics of the embankment and assessing its efficiency (steps 1, 3 and 4). For these purposes, statistical estimators of these distributions are considered. In practice, the different estimators used are the mean value, quantiles values – e.g., corresponding to a 99% or 95% probability that an event occurs – or the maximum value reached. Although the estimator strongly influences the design, no consensus exists on the choice of a relevant design. In particular, using the maximum value as an estimator may be misleading because it depends to a large extent on the number of simulations with an influence on the occurrence of the reference event. The maximum value is also the indicator the more affected by limitations of rockfall simulation codes in traducing the physics of the rock propagation: this value is often non-physical.

At the end of the process, the assessment of the projected structure is based on the residual risk, which depends on the occurrence of quite rare to extremely rare events resulting in embankment overtopping. These critical cases, whose identification requires a large number of trajectory simulations, are characterized by large flying heights, high energies or high rotational velocities in the vicinity of the embankment. The relevance of the trajectory analysis tools in accurately modeling the block's trajectory thus appears to be even more legitimate. Rockfall trajectory analysis results show high variability from one tool to another (Labieuse et al., 2001; Berger and Dorren, 2006), most particularly due to the rebound model considered, the soil and vegetation cover parameters and the number of runs (Bourrier and Hungr, 2011). This variability is even more pronounced for the velocity and flying height values as most models were calibrated based on the block stop distance (Dorren et al., 2011). Moreover, the rebound models often do not account for mechanisms that seem essential in estimating the trajectory in the vicinity of an embankment, where the deformability of the surfaces and impact orientations differ from more classical trajectory analysis contexts for which parameters are calibrated (Lambert et al., 2013). The coupling between the translational velocity and the rotational velocity as well as the shape of the block are generally roughly accounted for in these models even though they seem to play an important role in the block's trajectory (Chau et al., 2002). The rotational kinetic energy is often neglected because it is generally less than 10–15% of the total block energy (Chau et al., 2002; Bourrier and Hungr, 2011), even though it has been shown to be sufficient for the block to overtop the embankment (Plassiard and Donzé, 2010). In addition, the spatial resolution of trajectory analysis tools may not be appropriate to satisfactorily account for the slope profile in the vicinity of the embankment, mainly for 3D codes (Lambert et al., 2013).

For all these reasons, caution is required when using the flying height for design purposes in any embankment shape optimization process as well as when calculating residual risk. Given that the

uncertainties associated with these values are higher than classical values in geotechnical engineering, safety factors such proposed by the Eurocodes (CEN, 2004) may be considered insufficient and specific ones should be considered as proposed by a recently published Italian standard (UNI, 2012).

Constructing the embankment modifies the slope profile as well as the materials impacted by the block, with consequences on the soil-block interaction, which in turn governs the block's trajectory.

The profile is modified by digging the ditch, with probable reshaping of the natural slope uphill of the ditch, and erecting the embankment. All these changes affect the block's flying height and velocity. In particular, steepening the slope uphill of the ditch tends to increase the block's kinetic energy by increasing the falling height and decreasing the number of potential impacts on the soil. The designer engineer can consider these changes, providing the spatial resolution of the trajectory analysis tool can account for them satisfactorily.

In contrast, accounting for the change in surface materials is more complex, and this has never been mentioned in the literature. In lieu of one natural soil cover (soil A, Figure 3) three materials should be considered with very different mechanical characteristics (soils B, C and D, Figure 3).

Slope reshaping leads to exposing a denser soil without vegetation (B). Sometimes, huge blocks that cannot be removed outcrop. This leads to a decrease in the soil energy dissipation capability. Qualification of this soil before the earthworks is difficult and generally conducted by empirically changing the soil parameters used for the rockfall simulation.

The ditch soil characteristics (C) depend on the ditch type. A dust road has a high bearing capacity and thus exhibits low energy dissipation capacities. In this case, the soil parameters (e.g., restitution coefficients) may be determined by comparison with other dust road materials. If this ditch is designed as a catchment ditch, it is made of loose material and the soil parameter values are substantially reduced to account for the potentially high energy dissipation.

The last surface to be impacted (D) clearly constitutes the most problematic one. Table 1 shows that the embankment mountain-side facing may be made up of different materials and structures: concrete, riprap, tires, soil-geosynthetic composites, gabions, etc. exhibiting very different characteristics. The soil parameters considered for these facings are arbitrary as there are very few specific data concerning their impact response. After a rebound in the ditch, the block has an upward movement before touching the embankment with an inclined velocity with respect to the embankment facing. This is the most critical situation in terms of embankment overtopping. This risk is increased if the embankment is impacted close to its crest or if the block's rotational velocity is high (Plassiard and Donzé, 2009).

Fig. 4 shows a 3-m-tall embankment made up of gabion cages with a vertical facing. After an impact in the middle of the ditch, a 1.5-m block hit the face at a height of 1.8 m. As a result of its upward trajectory and probably the rotational velocity, the block overtopped the structure and crossed a railway and a highway, with only minor damage to the roadway. It is clear in this case that the facing deformation significantly contributed to the embankment clearing. A more rigid facing would have reflected the block uphill. This suggests that the actual response of the structure should be taken into account, depending on its mechanical characteristics.

This discussion shows that apart from the limitations of the currently used trajectory analysis tools, whose consequences still require investigation, the main problem assessing a projected embankment's efficiency in stopping blocks is related to the characterization of the impacted surfaces. Improving the assessment of an embankment's efficiency requires developing further research on the impact response of the different materials and structures used. This will alleviate building costly oversized structures and will substantiate the embankment's height and mountain-side batter requirements.



Fig. 4. This 3-m-tall gabion embankment was cleared by a block after an impact more than 1 m below its crest (Ste Marie de Cuines (73), France).

In most critical cases, designers should also account for the actual response of the embankment and its possible deformation. The efficiency of an embankment in stopping blocks therefore also depends on the mechanical design, investigated in the next section.

3. Design with respect to impact

After addressing issues related to the slope and the supporting soil, the geotechnical engineer checks the internal stability of the embankment and then the embankment impact response. The design of embankments exposed to such a specific dynamic loading is very complex because of (i) the large and irreversible deformations induced, (ii) the nonlinear soil stress-strain behavior, and (iii) the interaction between the different components (reinforcing components and soil).

In the following, research studies on the impact response of embankments are briefly mentioned to then analyze the embankment response. Then the current design methods are described. Finally, the discussion highlights important features that should be considered by design engineers and issues requiring further research and development are identified.

3.1. Specific research

The increasing need for efficient protection structures against high-energy block impacts has motivated research for more than 30 years. These are based on experimental, analytical and numerical developments. The main experimental research is presented in this section, in chronological order.

The first experiments were conducted in the 1980s in view of constructing a huge embankment with a 200-MJ capacity based on small-scale structures (Lepert and Corté, 1988; Corté et al., 1989). The 1/100 scale embankments made of sand were exposed to centrifuge impact tests at a 100-g acceleration. The impact force and acceleration within the embankment were recorded. In practice,

these experiments confirmed that the planned earth fill would be effective against a 200-m³ volume block with a velocity of 26 m/s.

The first real-scale impact experiments involved a structure type developed by the Colorado Department of Transportation in view of ensuring safety along an important road through a canyon (Burroughs et al., 1993; Hearn et al., 1995, 1996). The tests concerned a structure made of soil reinforced with geosynthetics, 24 m long, 3 m high and either 1.8 or 2.4 m thick, with vertical sides (Figure 5). Eighteen rock blocks of different shapes and masses were rolled down a hill against the barrier. The maximum block energy before the impact was 1400 kJ. The valley-side facing deformation increased nonlinearly with the boulder's kinetic energy, following a power law (Hearn et al., 1996). It decreased with the structure thickness: for a 1400-kJ impact, this deformation was 0.75 m and 0.35 m for a 1.8-m- and 2.4-m-thick structure, respectively (ref. A1/A3 in Table 2).

In the end of the 1990s, the behavior of a new type of structure was investigated through real-scale experiments (Yoshida, 1999). This structure consisted of a soil-reinforced earthwork protected by two layers of soil bags, placed uphill (Figure 6). The soil bags were exposed to the impact to act as an energy dissipater to reduce the impact force on the reinforced embankment. Blocks were rolled down the hill, resulting in impact energies ranging from 58 to 2500 kJ (ref. C1/C3 in Table 2). All the blocks were stopped without collapsing the structure. The deformation on the mountain-side facing was measured. More recent experiments were conducted on similar structures, but no significant data are available (Protec Engineering, 2011).

Since the end of the 1990s, an important study has been conducted by the University of Torino on geosynthetic reinforced embankments (Peila et al., 2000, 2002, 2007; Ronco et al., 2009; Peila, 2011). This research is the most exhaustive work on the topic to date, including experiments and numerical modeling aiming to develop both design charts and numerical tools to be used for design purposes (Brunet et al., 2009; Ronco et al., 2010) and allowing back analysis of natural impacts (Peila, 2011). Real-scale structures were tested on a site dedicated to rockfall protection structure testing (Figure 6). The equipment can convey blocks up to 10 tons at a velocity of 30 m/s. A total of eight impact tests on five different embankments were performed. The impacted structures were reinforced with geosynthetics, except for one structure that was made of compacted soil (ref. B1 to B3 in Table 2). The parameters investigated were the geosynthetic type, the soil type and the block mass. These reinforced embankments were demonstrated to be efficient against successive 4900-kJ impacts.

The Technical University of Vienna conducted small-scale experiments with the main objective of investigating the influence of the geotextile (Blovsky, 2002; Brandl and Blovsky, 2004). Twenty model tests were carried out on embankments scaled at 1:50. The parameters investigated were the soil compaction, the mountain-side dip and the geotextile deployment scheme. Embankments with and without geotextiles were tested (Figure 7). In the former, different geotextile anchoring lengths were considered. Impacts were repeated several times, measuring the impact force. Comparisons were made based on the sum of impulses after three successive impacts.

In Japan, three real-scale structures differing in their construction materials were exposed to impacts (Aminata et al., 2008; Sung et al., 2008). Impact tests were either performed rolling blocks down a 17-m hill or using a boulder suspended from a crane released from a height of 11 m, reaching a maximum energy of 110 kJ. The impacted structures were a geogrid-reinforced wall, 2 m high, and two walls made of ductile cast iron panels containing boulders (Figure 8). Block penetration and velocity and impact force were derived from the measurement of block acceleration.

Cellular rockfall protection structures have recently been investigated (Heymann et al., 2010; Bourrier et al., 2011; Lambert et al., 2011). These structures are made of gabion cages filled with different materials depending on their location in the structure. The aim is to

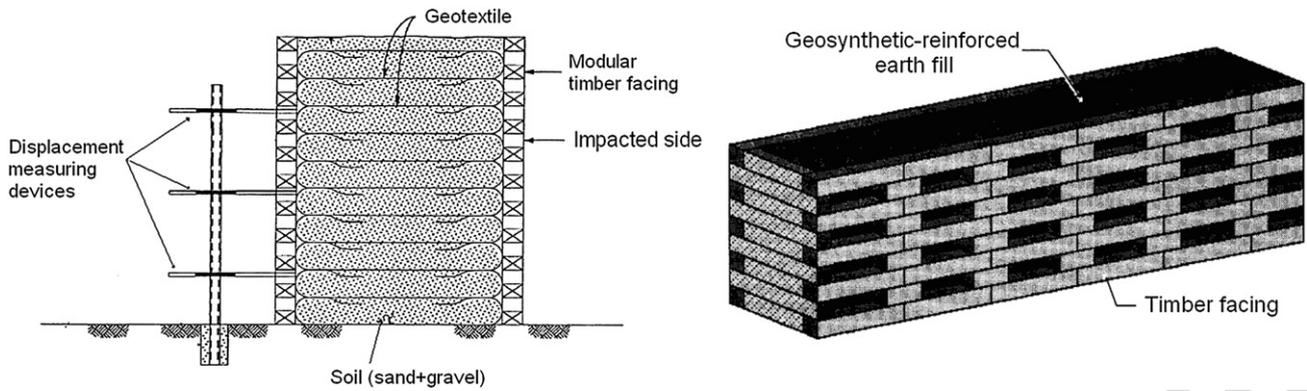


Fig. 5. Structure tested by the University of Colorado.
(Adapted from Burroughs et al., 1993; Hearn et al., 1995).

reduce the stresses transmitted within the structure, increasing the diffusion of stress as well as dissipation of the impact energy. Both half-scale and real-scale structures were impacted (Figure 9). Different fill materials were considered, including shredded tire-sand mixtures and ballast. Three different real-scale structures were subjected to impact by a 6500-kg spherical boulder, with a maximum velocity of 25 m/s (Lambert et al., 2011). The measures concerned acceleration of the boulder and within the structure, the transmitted forces and the structure deformations. Cellular walls 4 m in height and 3 m in thickness didn't collapse after a 2100 kJ impact.

Maegawa et al. (2011) investigated the concept of sandwich structures as pioneered by Yoshida (Yoshida, 1999) but employing geocells filled with gravel instead of soil bags for the structure's facing. Impacts ranging from 786 to 2709 kJ were obtained rolling a block down a 37-m-high slope onto the structure. Eight tests were performed on two types of structure. Deformation on the mountain-side facing and the block acceleration were measured (ref. D1 to D3 in Table 2). All the blocks were stopped.

Finally, small-scale experiments were recently conducted on 1/33 scale embankments made of sand (Mölk and Hofman, 2011; Hofmann and Mölk, 2012). The tested embankments varied in geometry, some had a rip-rap facing and others were reinforced with a geotextile. Tests were conducted considering various block energies and diameters, rotational velocities and impact angles. This study resulted in design charts considered in the Austrian standard concerning rockfall protection structures (ONR, 2012).

Numerical models have been developed in parallel to real-scale experiments, and rather fair agreements were obtained. Finite element methods (FEM) have been used by Burroughs et al. (1993), Peila et al. (2002, 2007) and Sung et al. (2008). FEM-based approaches require an explicit method and a re-meshing algorithm taking rockfall dynamics into account and modeling large deformations (Peila et al., 2007). On the contrary, discrete element methods (DEM) that naturally allow modeling large deformations have been used by many authors (Hearn et al., 1995, 1996; Carotti et al., 2004; Nicot et al., 2007; Plassiard and Donzé, 2009, 2010; Bertrand et al., 2010; Lorentz et al., 2010; Bourrier et al., 2011). More recently, Breugnot et al. (2010) and Breugnot (2011) have proposed using a combined discrete-continuum approach in order to derive advantages of the discrete approach in the impact area and from the finite difference approach in the far field. Finally, in order to provide design engineers with a much simpler tool, Bourrier et al. (2011) have proposed a quantitative approach based on a mass-spring model accounting for both non-linearities in stress-strain material behaviors and energy dissipation.

Despite the great variability of testing conditions and structure types, even when performed by the same authors, these experiments globally provide a valuable database for understanding the main mechanisms involved in the embankment response. Table 2 reports some experimental results obtained on real-scale structures with impact energies higher than 1000 kJ.

The capacity of the embankment, in terms of the impact energy it can withstand without collapsing, is the most important information

Table 2
Some experimental data concerning impact experiments with energies higher than 1000 kJ.

| Ref. | Structure type | Height (m) | Thickness (crest/base) (m) | Block mass (kg) | Translational velocity (m/s) | energy (kJ) | Impact height (m) | Deformation | |
|------|--|------------|----------------------------|-----------------|------------------------------|-------------|-------------------|-------------------|-----------------|
| | | | | | | | | Mountain-side (m) | Valley-side (m) |
| A1 | MSE wall + wood ^a | 3.05 | 1.82 | 5300 | 19.5 | 1010 | 1.5 | 0.6 | 0.2 |
| A2 | | 3.05 | 1.82 | 8300 | 18.3 | 1400 | 1.4 | 0.9 | 0.7 |
| A3 | | 3.7 | 2.4 | 8400 | 18.3 | 1410 | | | 0.34 |
| B1 | Reinforced dam ^b | 4.2 | 0.9/5 | 5000 | 31.7 | 2500 | 3 | 0.6 | 0.23 |
| B2 | | 4.2 | 0.9/5 | 8780 | 31.3 | 4350 | | 1 | 0.9 |
| B3 | Unreinforced dam ^b | 4.2 | 0.9/6 | 8780 | 31.3 | 4350 | 3 | 1.5 | Collapse |
| C1 | GeoRockwall ^c | 4 | 3.3/5.3 | 3300 | 24 | 970 | 2 | 0.22 | 0 |
| C2 | | 4 | 3.3/5.3 | 7700 | 24 | 2000 | 2-3 | | 0.09 |
| C3 | | 4 | 3.3/5.3 | 17000 | 17.7 | 2700 | 3-4 | | 0.5 |
| D1 | MSE wall + geocell facing ^d | 4.2 | 2.2/4.3 | 10100 | 15.7 | 1243 | 2.31 | 1.56 | 0.266 |
| D2 | | 4.2 | 3/5.1 | 17100 | 13.9 | 1567 | 2.85 | 1.44 | 0.091 |
| D3 | | 4.2 | 2.2/4.3 | 17100 | 14.4 | 2037 | 2.55 | 1.73 | 0.239 |

^a Hearn et al., 1996.

^b Peila et al., 2002, 2007.

^c Yoshida, 1999.

^d Maegawa et al., 2011.

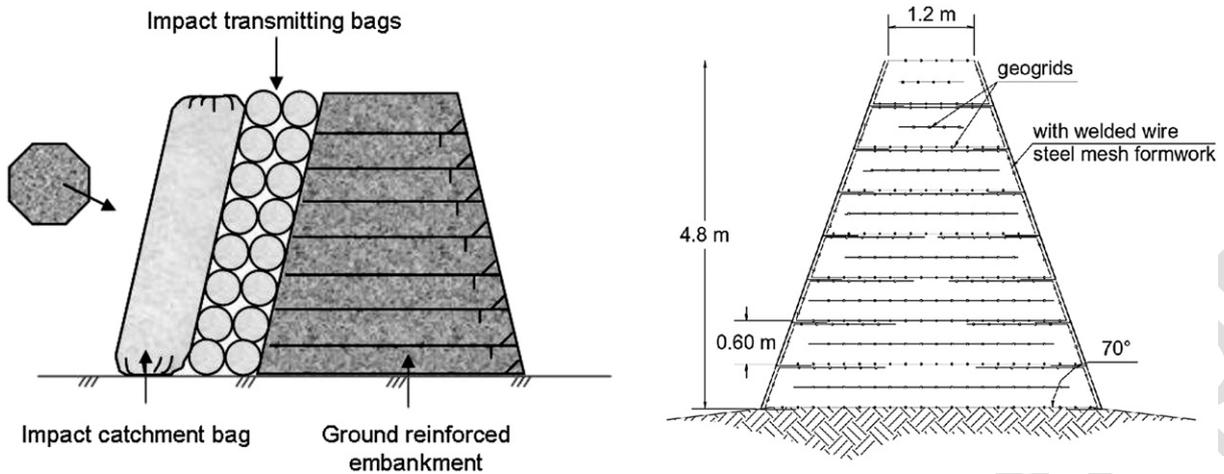


Fig. 6. Embankments tested by Yoshida (1999) (left) and by the University of Torino (Peila et al., 2002) (right).

obtained. The analysis of the response of impacted structures may also be based on the residual deformation on both mountain-side and valley-side facings of the embankment after an impact. In particular, the valley-side deformation can be considered a simple design criterion, as it governs post-impact structure stability (Ronco et al., 2009).

3.2. What can be learned about the response of an embankment to block impact?

The impact of a boulder, whose typical velocity ranges from 5 to 30 m/s and whose mass ranges from a few thousand to tens of hundreds of kilograms, results in a dynamic localized loading lasting between 0.03 and 0.2 s and generating strains in the impact direction at a rate higher than 0.1 s^{-1} . The embankment's response to this impact can be schematically separated into three phases.

During the first phase, the block's penetration only affects materials in the vicinity of the impacted area. It results in compaction of granular materials with possible particle crushing, depending on the embankment fill material characteristics: the narrower the soil grading, the larger the particle size and the higher the stress, the greater the grain crushing is. The rest of the structure exhibits almost no significant changes. It basically acts as a buttress allowing compaction and particle crushing on the mountain side. The compression wave travels within the structure, within a truncated cone from the impact point and with stress amplitude attenuation (Ronco et al., 2009).

The second phase starts after the compression wave has reached the back of the structure. This second phase is characterized by a global embankment acceleration. Indeed, as the valley-side facing is a free boundary, the compression wave results in a downhill-oriented movement of this facing, with possible soil bulking or projection (Brandl and Blosky, 2004; Hofmann and Mölk, 2012). Its efficiency as a buttress vanishes with time, allowing overall embankment displacement.

During the last phase, the block and the mountain-side facing movement progressively stop while the valley-side facing keeps moving, due to inertia. Cracks parallel to the facing may develop within the structure as a result of these two antagonistic motions (Peila et al., 2002). The embankment may collapse if the valley-side facing displacement is substantial.

The occurrence of these different phases and the magnitude of the associated mechanisms depend on the impact energy as compared to the embankment's capacity to absorb energy.

For low impact energies, both the penetration required to stop the block and the stress generated within the embankment are small (ref A1, B1, C1 in Table 2). The residual valley-side facing deformation is small compared to the block penetration. The impact energy is dissipated by compaction and grain crushing in the impact area while a small part propagates through compression wave.

With a higher impact energy, both the block penetration and stress within the embankment increase. The volume of material undergoing plastic strains increases, similar to what is observed for ground compaction (Mayne and Jones, 1983). The valley-side facing displacement increases and progressively tends toward the value of



Fig. 7. Post-impact top views of small-scale embankments showing the influence of the geotextile on the valley-side facing deformation: without/with geotextile, left/right. (From Brandl and Blosky, 2004).

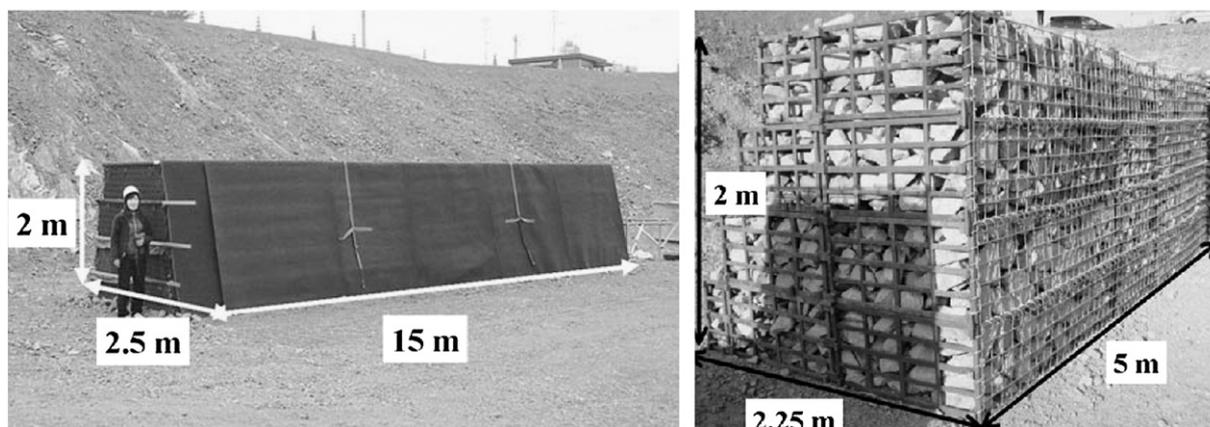


Fig. 8. Two of the three structures recently investigated in Japan (Aminata et al., 2008; Sung et al., 2008).

the mountain-side facing displacement (e.g., ref. A1 and A2 in Table 2). Moreover, the thicker the embankment the lower the valley-side displacement is (e.g., ref. D1 and D2 in Table 2).

Depending on the point in the structure, prevailing mechanisms are either compaction (including crushing) or an almost rigid body movement. Compaction is predominant close to the impact, on the mountain side, while rigid body movement is predominant on the opposite side. Due to these large displacements, part of the impact energy is dissipated through friction along rupture planes. Nevertheless, energy dissipation through soil compaction remains predominant, with a numerically estimated proportion of about 80% of the block's kinetic energy (Ronco et al., 2009).

Finally, when the impact energy equals the capacity of the embankment, collapse occurs as a result of a large valley-side facing displacement.

The structure's response versus time thus reveals the influence of the different parameters related to the material characteristics: the parameters associated with their compressive response, friction angle and unit mass. The influence of these parameters on the whole structure's behavior depends on the impact energy, the structure's dimensions and the structure's boundary conditions. For example, the more the valley-side facing is free to move and the greater the volume of soil displaced with the block, the more influential the unit mass is. Additionally, the friction angle influence increases with the whole structure displacement, thus with the impact energy.

Moreover, the ability of the embankment to withstand the impact is improved by reinforcement layers (ref. B2/B3 in Table 2). Reinforcement

layers distribute the load in the longitudinal direction (Peila et al., 2007). The deformation of the embankment in the vicinity of the impact point results in a tension in the layer oriented along the longitudinal axis of the embankment. This differs from the static case accounting for gravity loads, where the reinforcement layer is loaded along the transverse axis of the embankment direction (Peila et al., 2002; Brandl and Blovsky, 2004). The impact load is thus distributed to soil masses at a distance on both sides of the impacted area. In the impact vicinity, the resulting confining effect by the reinforcement layer increases the penetration resistance of the embankment. Moreover, the layer restrains the displacement of the valley-side facing and thus increases the ability to withstand the impact while the impact force is increased (Blovsky, 2002) (Figure 7). The negative counterpart is that horizontal planar reinforcement layers may offer a preferential plane for shear rupture (Figure 10).

3.3. Current design methods

In spite of their important protective role, the design of embankments is generally based on simplistic approaches. In most of the concerned countries, there are neither recommendations nor guidelines concerning their design. In Italy and Austria standards have been recently published, covering most of the issues to be addressed (UNI, 2012; ONR, 2012). Even if in some cases, specific and rather complex studies have been conducted (Lepert and Corté, 1988; Brandl and Blovsky, 2004; Lorentz et al., 2010), the vast majority of existing structures have been designed with basic approaches, considering dynamics only to a minor extent.

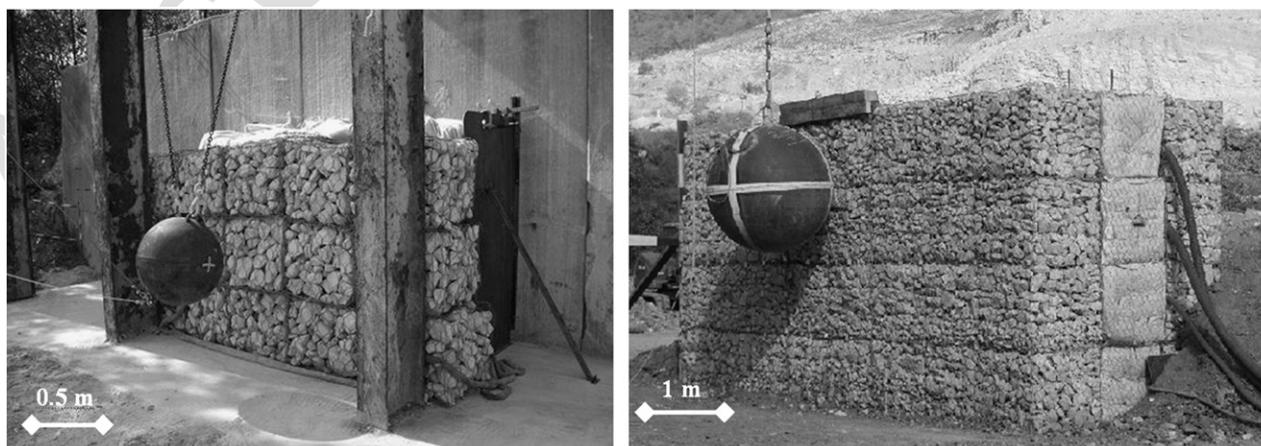


Fig. 9. Half-scale and full-scale cellular sandwich structures tested by Heymann et al. (2010) and Lambert et al. (2011).

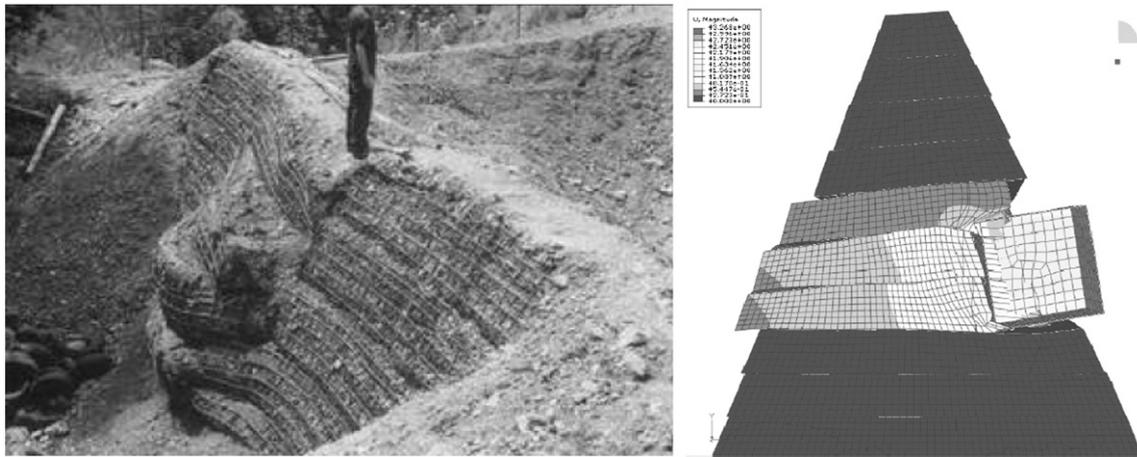


Fig. 10. Reinforcement layers offer a preferential plane for shear rupture (Ronco et al., 2009).

The approaches in use today differ in their level of complexity and their ability to account for the dynamics:

- Type 1: based on its mass, the embankment is considered to be able to stop the block and withstand the impact.
- Type 2: penetration criteria-based approaches estimate the penetration of the block in the embankment, which is multiplied by a factor of 2–3 to obtain the minimum embankment thickness (Brunet et al., 2009).
- Type 3: pseudo-static approaches consider a load that is statically equivalent to the dynamic impact load for designing the embankment (Jaecklin, 2006; Kister and Fontana, 2011). A safety factor may be considered, expressing the uncertainties associated with the hypothesis of a statically equivalent loading (Brandl and Adam, 2000). The structure's static stability is checked considering this load in combination with gravity loads.
- Type 4: energy balance analytical approaches use analytical methods to compare the incident translational kinetic energy of the block with the energy dissipated within the embankment during the impact. The first dissipative mechanism accounted for is friction along shear planes assuming that a section of the embankment moves as a rigid body (Tissières, 1999; Brandl and Adam, 2000). Different shear planes can be considered, notably along the reinforcement layers (Carotti et al., 2000; Subrin, 2006; Ronco et al., 2009). Energy dissipation through soil compaction can also be considered (Jarrin, 2001; Ronco et al., 2009). The design consists in assessing that the structure deformation required to dissipate the block's kinetic energy is consistent with the embankment dimensions. For this purpose, the mountain-side deformation due to the block penetration may be deduced from the impact force (Ronco et al., 2009).
- Type 5: approaches based on numerical modeling uses specific numerical tools to model the impact and evaluate the deformation of the embankment, using either finite or discrete element methods (FEM or DEM) (Peila et al., 2007; Lorentz et al., 2010).

Types 2–4 generally involve determining the block penetration or the impact force. Different methods have been used for these purposes. The penetration can be estimated using the expression given by Kar (1978) (Paronuzzi, 1989) with possible adaptation to account for reinforcement layers (Cargnel and Nössing, 2004). Brunet et al. (2009) suggest using the formulations given by Calveti and di Prisco (2008). The impact force can be evaluated using the expressions given by Montani (1998), by Mayne and Jones (1983) or by Labiouse et al. (1996) (Jaecklin, 2006; Peila et al., 2007; ASTRA, 2008; Ronco et al., 2009; respectively). It should be noted that the soil parameters considered in these expressions are derived from

static tests and mainly concern the elastic response of soils. Other methods that have been specifically developed for determining the penetration of rock blocks in soils are not mentioned in the literature concerning embankments (Pichler et al., 2005; Wang and Cavers, 2008).

The block penetration, δ , can also be estimated from the impact force (Carotti et al., 2004; Peila et al., 2007; Ronco et al., 2009):

$$\delta = \frac{mv^2}{F_{\max}} \quad (1)$$

where F_{\max} is the peak impact force (kN), v the block velocity just before impact (m/s) and m its mass (kg). This relation indicates that for a given velocity the higher the penetration the lower the impact force.

3.4. Discussion

The analytical methods described in the previous section provide design engineers with easy-to-use tools for estimating the impact force and block penetration. Nevertheless, their relevancy is limited due to the uncertainty associated with each method. For instance, the penetration is overestimated by up to a factor of 2 using the expression proposed by Kar (1978) (Cargnel and Nössing, 2004; Carotti et al., 2004), whereas it is underestimated down to about 50% using the models proposed by Labiouse et al. (1996) and by Mayne and Jones (1983) (Carotti et al., 2004; Oggeri et al., 2004; Peila et al., 2007). Considering Eq. (1), the opposite trends are observed when estimating the impact force. For a specific case, great variability in the impact force values has been obtained using the different existing methods, with a maximum ratio of 1:5 between the highest and the lowest values (Kister and Fontana, 2011). In fact, these models were derived from experiments in specific contexts. Kar's model was developed for cohesive nonfrictional soils impacted by projectiles with a velocity ranging from 200 to 800 m/s. Mayne and Jones's model was developed for heavy fine soil tamping. Labiouse's model was developed for granular strata, a 1-m maximum thickness, lying on a rigid support. Compared to the impact of a block on an embankment, these contexts differ in the projectile velocity and shape as well as the impacted structure's mechanical characteristics, dimensions and boundary conditions.

More globally, the approach consisting in considering a statically equivalent load for modeling the impact force may not be fully appropriate. The main reason is that this approach neglects some of the mechanisms at work during the impact, which governs the actual response of the whole structure. In particular, this response depends on the stress transmitted over time within the structure, which in turn

depends not only on the impact force, but also on other geometrical and mechanical parameters related to the embankment. It has been shown that for a given impact force the granular material characteristics may strongly affect the maximum value of the transmitted stress as well as its spatial distribution (Laue et al., 2008; Lambert et al., 2009; Heymann et al., 2010; Calvetti and di Prisco, 2011). The shape of the block has also been shown to have an influence on the impact force and on the transmitted stress (see the state of the art as described in Montani, 1998; Degago, 2007). The impact force is not only difficult to estimate, but is also partly inappropriate to account for what occurs within the embankment.

The reliability of analytical approaches based on the energy balance (Type 4) depends on the quantification of the different energy dissipation terms: soil compaction first and then friction along shear planes. These methods are attractive in the sense they are based on phenomenological considerations. Nevertheless, they require defining a priori the embankment volume and surfaces concerned by compaction and friction, respectively, whereas these strongly depend on the loading conditions (impact energy and impact point location) and embankment material characteristics, which constitute the limitations of energy balance analytical methods.

Undoubtedly, the analytical expressions available have moved a step forward in accounting for the dynamics and are useful for preliminary design (Peila et al., 2007). Nevertheless, complementary investigations are recommended in view of an optimized design. These could concern the mechanisms associated to energy dissipation, which depend in particular on both the embankment geometry and the mechanical characteristics of the materials.

In this context, employing numerical tools to model the actual response of these structures seems to be more satisfactory (Peila et al., 2007; Lorentz et al., 2010). Indeed, even if the numerical methods are not yet within the reach of all design engineers and are time-consuming to develop and use, they allow going further in accounting for the dynamics of the impact.

Whether a numerical or analytical method is considered, the material characteristics to take into account raise major problems. In some cases, the models were calibrated based on real-scale impact tests on granular layers or structures (e.g., Carotti et al., 2000; Plassiard and Donzé, 2009; Ronco et al., 2009). Generally, classical geotechnical data are used even though they result from static tests. The relevancy of these data for satisfactorily modeling the structure response under impact loading is questionable. For instance, the interaction between the soil and the reinforcing geosynthetic, in terms of both interface friction and confining effect, has never been addressed under localized impact loading. The friction between these materials is modeled using friction angles and cohesion values derived from static tests, whereas this interface has been clearly shown to govern embankment collapse (Peila et al., 2007; Ronco et al., 2009). Complementary investigations on the impact response of the different materials are therefore necessary. This will improve numerically based extrapolations to high-impact-energy events. The dynamic interaction between the geosynthetic and the soil should be investigated as a priority, focusing first on the friction and then on the confinement effect by the geosynthetic material in cases of localized impact.

The various types of reinforced-soil embankments differ in the nature of the reinforcing component, which can be a geotextile, geogrid, wire-mesh or interconnected tires. Embankments also differ in the reinforcement deployment scheme. In some cases, only the mountain-side facing is reinforced, while in others both the mountain and valley sides are reinforced. Valley-side reinforcement is increasingly frequent throughout the world. A gentle downhill slope is sometimes preferred for reasons of the valley-side visual impact. On the other hand, the downhill slope is often steepened for embankments protecting transportation lines at the toe of steep slopes. Beyond such site-specific requirements, reinforcing the downhill face significantly improves the

embankment's ability to withstand the impact as it reduces valley-side facing displacement (Blovsy, 2002; Peila et al., 2007; Lambert et al., 2011). Compared to a massive backfill having the same buttress effect, this solution has the advantages of a limited footprint and a reduced overall risk of slope instability.

An increasing consideration is given to post-impact behavior and characteristics of embankments in developing new design methods or new building technologies (Brandl and Blovsy, 2004; Ronco et al., 2009; Heymann et al., 2010). The basic issue concerns the residual ability of the structure to intercept blocks that may fall again in the vicinity of a previously impacted area. Damage to the structure increases with the impact energy, until the structure collapses. Whatever the damage level, the most important requirement is that the block is stopped. Compared to the possible consequences of an unstopped block, the cost of reconstruction is moderate. If there is no visible damage, the block is removed and the embankment characteristics are considered unchanged. On the other hand, the embankment has to be reconstructed after collapsing. The embankment's residual characteristics are questioned when there is intermediate damage. Some structures have been experimentally proved to be efficient even after several high-energy impacts despite significant deformations (Peila et al., 2007). The post-impact stability of the embankment can also be assessed taking into account the deformation of the structure (Ronco et al., 2009). However, the relevance of this issue in a practical context is lessened by technical and human considerations. First, it is not easy to assess the residual characteristics of an embankment after a natural event, whose energy is difficult to estimate. Computations can reproduce the observed deformations numerically and geophysical surveys can identify the damaged zones. Nevertheless, these investigations will certainly be insufficient for guaranteeing the serviceability of the structure. This is first and foremost the engineer's responsibility, but also a problem of visual perception of the structure. The impression of inefficiency resulting from visible damage is not acceptable to the general public. It is therefore not simply a technical matter. For this reason, structures having an easy-to-repair mountain-side facing may be an alternative to classical embankments (Yoshida, 1999; Brunet et al., 2009; Lambert et al., 2009; Maegawa et al., 2011).

From a practical standpoint, the design of rockfall protection embankments challenges geotechnical engineers with the principles of currently used methods such as the Eurocodes. Passive rockfall protection structures are not within the scope of specific Eurocodes and should be considered special structures. Moreover, the impact load should not be calculated as an accidental action according to EN 1991-1-7 (CEN, 2007), as the expressions given were developed for very different impacts and may lead to asymptotic values. The impact loading may be considered either accidental or variable, which influences the safety factors used for the design. Besides, the service and ultimate limit states are not clear to define, as discussed earlier. The straightforward application of the Eurocodes principles thus appears not possible. It requires an interpretation by the design engineer accounting for the peculiarities of this type of structure. For this reason, some countries have recently proposed partial safety factors through national standards (UNI, 2012; ONR, 2012). Besides, Durville et al. (2010) have proposed a two-step procedure evaluating first the ability of the structure to intercept a block and second its post-impact residual characteristics. An ultimate limit state verification is conducted considering the impact as an accidental situation and a block of maximal energy. It ensures that the block is stopped by the embankment, even if this structure collapses. Then a serviceability limit state verification is conducted to assess the post-impact embankment characteristics and considering an impact of medium energy. It allows checking that the structure is able to intercept another block of the same energy. Even if this approach is more suitable for embankments, questions concerning the parameters defining the loadings may rise. The embankment response depends on both the energy of the block before

impact and on the impact location. A low-energy event on the top of the structure may be more detrimental than a high-energy impact at its toe (Bertrand et al., 2010). Therefore, various combinations of these two variables should be considered, using their respective detailed statistical distributions.

4. Conclusion

In order to provide engineers with cost-effective rockfall protection embankment design tools, different methods have been developed over the last two decades to predict their ability to stop blocks and withstand impacts. Although embankment design with respect to these two facets has been significantly improved, a number of limitations have been identified and should be accounted for by design engineers.

The main limitations concerning the functional design of embankments result from the rockfall simulation codes. In the design process, these codes are used to determine the rock passing heights and velocities even though they are not calibrated for this purpose. Rockfall simulation codes are generally calibrated and/or validated from previous rockfall events or rockfall experiments that are compared solely on the rocks' stopping distance. Trajectory analysis tools therefore provide relevant results for rock stopping distances: however their accuracy in estimating the rock kinematics is generally more limited. In addition, 3D rockfall simulation codes are currently used and calibrated with a spatial resolution that does not satisfactorily model the embankment shape. Consequently, the spatial resolution of the models does not allow one to obtain a good estimate of the residual risk. Modeling the mechanical interaction between the rock and the embankment also has limitations. In particular, models for the interaction between the block and the embankment as well as with the surfaces in its vicinity lack realism mainly due to limited knowledge on the actual impact response of the materials concerned.

As for embankment design with respect to impact, satisfactorily accounting for the dynamics requires taking into account the complexity of the mechanisms at work during the impact and the difficulty estimating their amplitude with time, which depends on the impact energy, embankment material characteristics and dimensions. As a consequence, analytical models are not yet satisfactory for this purpose. They fail to give good estimates of impact forces, block penetration, and energy dissipation terms. Numerical models are costly but may be identified as relevant tools for the design of embankments, providing a validation based on real-scale experiments involving impact energies of similar amplitude. In a research process, these methods could be used jointly with experiments to improve current analytical models for different loading cases.

More globally, a useful perspective would be gained from reliability analysis so that the structure's failure probability could be calculated given the distribution of the rock impact points and kinematics. Embankment design with respect to impact stability generally considers a few loading cases, whereas no extensive research has been conducted on the variability of embankment responses considering the wide variety of impact scenarios.

All these points should be considered so as to improve the design of embankments. In addition, and in a more formal context, a common base for the application of the Eurocodes is necessary.

References

Agliardi, F., Crosta, G., 2003. High resolution three-dimensional numerical modelling of rockfalls. *International Journal of Rock Mechanics and Mining Sciences* 40, 455–471.

Agliardi, F., Crosta, G., Frattini, P., 2009. Integrating rockfall risk assessment and countermeasure design by 3D modelling techniques. *Natural Hazards and Earth System Sciences* 9, 1059–1073.

Aminata, D., Yashima, A., Sawada, K., Sung, E., 2008. New protection wall against rockfall using a ductile cast iron panel. *Journal of Natural Disaster Science* 30 (1), 25–33.

ASTRA, 2008. Directive 12006: Actions de chutes de pierres sur les galeries de protection. OFROU, Bern, Switzerland (22 pp.).

Berger, F., Dorren, L.K.A., 2006. Objective comparison of rockfall models using real size experimental data. Conference on Disaster Mitigation of Debris Flows, Slope Failures and Landslides – Interpraevent 2006, Nigata, Japan, pp. 245–252.

Bertrand, D., Lambert, S., Gotteland, P., Nicot, F., 2010. Les merlons à technologie cellulaire. In: Lambert, S., Nicot, F. (Eds.), *Géo-mécanique des instabilités rocheuses: du déclenchement à l'ouvrage*. Hermès, Paris, pp. 369–402 (in French).

Blovsky, S., 2002. Model tests on protective barriers against rockfall. 15th European Young Geotechnical Engineers Conference, Dublin, Ireland.

Bourrier, F., Hungr, O., 2011. Rockfall dynamics: a critical review of collision and rebound models. In: Lambert, S., Nicot, F. (Eds.), *Rockfall Engineering*. John Wiley & Sons, New York, ISTE Ltd, London, pp. 175–210.

Bourrier, F., Eckert, N., Nicot, F., Darve, F., 2009. Bayesian stochastic modeling of a spherical rock bouncing on a coarse soil. *Natural Hazards and Earth Systems Sciences* 9, 831–846.

Bourrier, F., Lambert, S., Heymann, A., Gotteland, P., Nicot, F., 2011. How multi-scale approaches can benefit cellular structure design. *Canadian Geotechnical Journal* 48, 1803–1816.

Brandl, H., Adam, D., 2000. Special applications of geosynthetics in geotechnical engineering. Proceedings of the 2nd European Geosynthetics Conference and Exhibition-Eurogeo II, Bologna, Italy, pp. 305–310.

Brandl, H., Blovsky, S., 2004. Protective barriers against rockfall. Proceedings of the 3rd European Geosynthetics Conference-Eurogeo III, Munich, Germany, pp. 95–100.

Breugnot, A., 2011. Approche numérique couplée discret-continu appliquée aux ouvrages cellulaires impactés. PhD thesis, University Joseph Fourier, Grenoble, France, 247 pp.

Breugnot, A., Gotteland, P., Villard, P., Garcin, P., 2010. Modelling of block impacts with a combined discrete-continuum approach. Proceedings of the 3rd Euro Mediterranean Symposium on Advances in Geomaterials and Structures – AGS 2010, Djerba, Tunisia, pp. 289–295.

Brunet, G., Giacchetti, G., Bertolo, P., Peila, D., 2009. Protection from high energy rockfall impacts using Terramesh embankment: design and experiences. Proceedings of the 60th Highway Geology Symposium, Buffalo, New York, pp. 107–124.

Burroughs, D.K., Henson, H.H., Jiang, S.S., 1993. Full scale geotextile rock barrier wall testing, analysis and prediction. Proceedings of Geosynthetics '93, Vancouver, Canada, pp. 959–970.

Calveti, F., di Prisco, C., 2008. Linee guida per la progettazione di gallerie paramassi. *Starrylink*, Italy (in Italian, 184 pp.).

Calveti, F., di Prisco, C., 2011. A new design method for rockfall shelters covered by granular layers. In: Lambert S., Nicot, F. (Eds.), *Rockfall Engineering*. John Wiley & Sons, New York, ISTE Ltd, London, pp. 343–373. ISBN 978-1-84821-256-5.

Calvino, A., Dumont, P., Durville, J.-L., Dussauge, C., Effendiantz, L., Evrard, H., 2001. *Parades contre les instabilités rocheuses*. Guide technique, Collection environnement, Les risques naturels, LCPC, Paris, 143 pp. (in French).

Cargnel, O., Nössing, L., 2004. Comportamento di opere passive interessate da scoscedimenti massivi. Proceedings of Convegno Bonifica di versanti rocciosi per la protezione del territorio, Trento, Italy, pp. 419–434.

Carotti, A., Peila, D., Castiglia, C., Rimoldi, P., 2000. Mathematical modelling of geogrid reinforced embankments subject to high energy rock impact. Proceedings of the 2nd European Geosynthetics Conference and Exhibition – Eurogeo II, Bologna, Italy, pp. 305–310.

Carotti, A., di Prisco, C., Vecchiotti, M., Recalcati, P., Rimoldi, P., 2004. Modeling of geogrid reinforced embankments for rockfall protection. Proceedings of the 3rd European Geosynthetics Conference, Munich, Germany, pp. 675–680.

CEN, 2004. EN 1997-1, Eurocode 7: Geotechnical design – part 1: general rules. European Committee for Standardisation, Brussels (168 pp.).

CEN, 2007. EN 1991-1-7, Eurocode 1 – actions on structures – part 1-7: general actions – accidental actions. European Committee for Standardisation, Brussels. (62 pp.).

Chau, K.T., Wong, R.H.C., Wu, J.J., 2002. Coefficient of restitution and rotational motions of rockfall impacts. *Rock Mechanics and Mining Science* 39, 69–77.

Corté, J., Lepert, P., Rochet, L., 1989. Design of a protective system against rock falls; example of RN 90 highway. *Revue Générale Des Routes Et Aéroports* 669, 114–117.

Coulon, E., Bruhier, J., 2006. Les merlons renforcés de protection. Proceedings of Rencontres Géosynthétiques 2006, Montpellier, France, pp. 367–372 (in French).

Degago, S.A., 2007. Impact tests on sand with numerical modeling, emphasizing on the shape of the falling object. Masters Thesis, NTNU, Trondheim, Norway, 140 pp.

Descoedres, F., 1997. Aspects géomécaniques des instabilités de falaises rocheuses et des chutes de blocs. Publications De La Société Suisse De Mécanique Des Sols et Des Roches 135, 3–11.

Dorren, L.K.A., Domaas, U., Krohnolm, K., Labiouse, V., 2011. Methods for predicting rockfall trajectories and runout zones. In: Lambert, S., Nicot, F. (Eds.), *Rockfall Engineering*. John Wiley & Sons, New York, ISTE Ltd, London. ISBN: 978-1-84821-256-5, pp. 143–173.

Durville, J.-L., Guillemin, P., Berthet-Rambaud, P., Subrin, D., 2010. Etat de l'art sur le dimensionnement des dispositifs de protection contre les chutes de blocs. Collection Études et recherches des LCPC – série Géotechnique et risques naturels, Paris (84 pp.).

Gerbert, S., Hespel, F., 1999. Geotextile's reinforced wall for natural risk protection works. Proceedings of Rencontres Géosynthétiques, Bordeaux, France, pp. 147–150.

Giani, G.P., Giacomini, A., Migliazza, M., Segalini, A., 2004. Experimental and theoretical studies to improve rock fall analysis and protection work design. *Rock Mechanics and Rock Engineering* 37 (5), 369–389.

Hara, T., Tsuji, A., Yashima, A., Sawada, K., Tatta, N., 2009. Dynamic interaction between pile and reinforced soil structure – piled geo-wall. Proceedings of the International Symposium on Prediction and Simulation Methods for Geohazard Mitigation, Kyoto, Japan, pp. 457–463.

- Hearn, G., Barrett, R.K., Henson, H.H., 1995. Development of effective rockfall barriers. *Journal of Transportation Engineering* 121 (6), 507–516.
- Hearn, G., Barrett, R.K., Henson, H.H., 1996. Testing and modelling of two rockfall barriers. *Transportation Research Record* 1504, 1–11.
- Heymann, A., Lambert, S., Haza-Rozier, E., Vincelas, G., Gotteland, P., 2010. An experimental comparison of real-scale rockfall protection sandwich structures. *Proceedings of the 11th International Conference on Structures Under Shock and Impact*, Tallinn, Estonia, pp. 15–26.
- Hoek, E., 2007. Practical rock engineering. Available online: <http://www.rocsience.com/>.
- Hofmann, R., Molk, M., 2012. Bemessungsvorschlag fur Steinschlagschutzdamme. *Geotechnik* 35, 22–33 (in German).
- Jaecklin, F., 2006. Innovative design for repairing Gondo mudslide by 20 m high geogrid wall. *Proceedings of the 8th International Conference on Geosynthetics*, Ozaka, Japan, pp. 1223–1228.
- Jarrin, P., 2001. Trajectographie des blocs rocheux. Master thesis, ENSMP, Paris.
- Kar, A.K., 1978. Projectile penetration into buried structures. *Journal of Structural Division*, ASCE 104 (1), 125–139.
- Kister, B., Fontana, O., 2011. On the evaluation of rockfall parameters and the design of protection embankments – a case study. *Proceedings of Interdisciplinary Workshop on Rockfall Protection – Rocexs 2011*, Innsbruck, Austria, pp. 31–32.
- Koerner, R.M., 2005. Designing with Geosynthetics, 5th edition. Prentice Hall (796 pp.).
- Labieuse, V., Descoedres, F., Montani, S., 1996. Experimental study of rock sheds impacted by rock blocks. *Structural Engineering International* 3, 171–175.
- Labieuse, V., Heidenreich, B., Desvarreux, P., Viktorovich, M., Guillemin, P., 2001. Etudes trajectographiques, Prevention des mouvements de versants et des instabilites de falaises. *Programme Interreg II C*, pp. 155–205 (in French and in Italian).
- Lambert, S., Gotteland, P., Nicot, F., 2009. Experimental study of the impact response of geocells as components of rockfall protection embankments. *Natural Hazards and Earth Systems Sciences* 9, 459–467.
- Lambert, S., Heymann, A., Gotteland, P., 2011. Real-scale experimental assessment of cellular rockfall protection structures. *Proceedings of Interdisciplinary Workshop on Rockfall Protection – Rocexs 2011*, Innsbruck, Austria, pp. 36–37.
- Lambert, S., Bourrier, F., 2013. Improving three-dimensional rockfall trajectory simulation codes for assessing the efficiency of protective embankments. *International Journal of Rock Mechanics and Mining Sciences*. <http://dx.doi.org/10.1016/j.ijrmms.2012.12.029i>.
- Laue, J., Chikatamarla, R., Springman, S., 2008. Do we need coded load assumptions for rockfall protection measures. *Proceedings of Interdisciplinary Workshop on Rockfall Protection*, Morschach, Switzerland, pp. 55–57.
- Lepert, P., Corte, J., 1988. Centrifuge Modeling of the Impact of Large Blocks of Rock on a Protection Structure. *Centrifuge* 88, Paris, France, pp. 457–465.
- Lorentz, J., Plassiard, J.-P., Muquet, L., 2010. An innovative design process for rockfall embankments: application in the protection of a building at Val d'Isere. *Proceedings of the 3rd Euro Mediterranean Symposium on Advances in Geomaterials and Structures – AGS 2010*, Djerba, Tunisia, pp. 277–282.
- Maegawa, K., Tajima, T., Yokota, T., Tohda, M., 2011. Slope-rockfall tests on wall embankments reinforced with geosynthetics. *Proceedings of the 6th International Structural Engineering and Construction Conference*, Zurich, Switzerland, pp. 641–646.
- Mannsbart, G., 2002. Geosynthetic reinforced protection structures in mountainous regions – examples of safe and cost-effective alternatives to conventional structures. *Proceedings of the 7th International Conference on Geosynthetics*, Nice, France, pp. 299–301.
- Masuya, H., Amanuma, K., Nishikawa, Y., Tsuji, T., 2009. Basic rockfall simulation with consideration of vegetation and application to protection measure. *Natural Hazards and Earth System Sciences* 9, 1835–1843.
- Mathieu, Y., Marchal, J., 1989. Deviation de la route nationale 90 à Aigueblanche – realisation d'un massif renforce par geotextile et parement Pneusol: le Pnutex. *Bulletin des Laboratoires des Ponts et Chaussees* 162, 77–80 (in French).
- Mayne, P.W., Jones, S.J., 1983. Impact stresses during dynamic compaction. *Journal of Geotechnical Engineering* 109, 1342–1346.
- Molk, M., Hofman, R., 2011. The Austrian standard ONR 24810: design of rock-fall protection measures. Partial factor of safety-approach and best practice for the design of rock-fall embankments. *Proceedings of Interdisciplinary Workshop on Rockfall Protection – Rocexs 2011*, Innsbruck, Austria, pp. 45–46.
- Montani, S., 1998. Sollicitation dynamique de la couverture des galeries de protection lors de chutes de blocs. PhD thesis, EPFL Lausanne, Switzerland (in French).
- Morino, A., Grassi, P., 1990. Design and construction of a reinforced earth embankment for protection against rock falls. *Proceedings of the 4th International Congress on Geotextiles, Geomembranes and Related Products*. The Hague, The Netherlands, p. 124.
- Nicot, F., Gotteland, P., Bertrand, D., Lambert, S., 2007. Multi-scale approach to geo-composite cellular structures subjected to impact. *International Journal for Numerical and Analytical Methods in Geomechanics* 31, 1477–1515.
- Oggeri, C., Peila, D., Recalcati, P., 2004. Rilevati paramassi. *Proceedings of Convegno Bonifica di versanti rocciosi per la protezione del territorio*, Trento, Italy, pp. 191–232 (in Italian).
- ONR, 2012. ONORM 24810: Technical protection against rockfall – Terms and definitions, effects of actions, design, monitoring and maintenance. Austrian standard institute, Vienna (draft 21 August 2012, in German).
- Paronuzzi, P., 1989. Criteri di progettazione di rilevati paramassi. *Geologia Tecnica* 1, 23–41 (in Italian).
- Peckover, F.L., Kerr, W.G., 1977. Treatment and maintenance of rock slopes on transportation routes. *Canadian Geotechnical Journal* 14 (4), 487–507.
- Peila, D., 2011. Ground reinforced embankments for rockfall protection: From real scale tests to numerical modelling. In: Lambert, S., Nicot, F. (Eds.), *Rockfall Engineering*. John Wiley & Sons, New York, ISTE Ltd, London. ISBN: 978-1-84821-256-5, pp. 393–426.
- Peila, D., Castiglia, C., Oggeri, C., Guasti, G., Recalcati, P., Sassudelli, F., 2000. Full scale tests on geogrid reinforced embankments for rock fall protection. *Proceedings of the 2nd European Geosynthetics Conference and Exhibition*, Bologna, Italy, pp. 317–322.
- Peila, D., Oggeri, C., Castiglia, C., Recalcati, P., Rimoldi, P., 2002. Testing and modelling geogrid reinforced soil embankments to high energy rock impacts. *Proceedings of the 7th International Conference on Geosynthetics*, Nice, France, pp. 133–136.
- Peila, D., Oggeri, C., Castiglia, C., 2007. Ground reinforced embankments for rockfall protection: design and evaluation of full scale tests. *Landslides* 4 (3), 255–265.
- Pichler, B., Hellmich, C., Mang, H.A., 2005. Impact of rocks onto gravel – design and evaluation experiments. *International Journal of Impact Engineering* 31, 559–578.
- Plassiard, J.-P., Donze, F.-V., 2009. Rockfall impact parameters on embankments: a discrete element method analysis. *Structural Engineering International* 19 (3), 333–341.
- Plassiard, J.-P., Donze, F.-V., 2010. Optimizing the design of rockfall embankments with a discrete element method. *Engineering Structures* 32 (11), 3817–3826.
- Protect Engineering, 2011. www.proteng.co.jp/english/dike_testing.html.
- Rimoldi, P., Lorzio, R., Pettinau, D., Rocallo, C., Secci, R., 2008. Impressive reinforced soil structures in Italy. *Proceedings of the 1st Pan American Geosynthetics Conference*, Cancun, Mexico, pp. 789–798.
- Ronco, C., Oggeri, C., Peila, D., 2009. Design of reinforced ground embankments used for rockfall protection. *Natural Hazards and Earth System Sciences* 9, 1189–1199.
- Ronco, C., Oggeri, C., Peila, D., Bertolo, P., Ferraiole, F., Giacchetti, G., 2010. Numerical modeling of ground reinforced embankments used for rockfall protection. *Proceedings of the 3rd Euro Mediterranean Symposium on Advances in Geomaterials and Structures-AGS 2010*, Djerba, Tunisia, vol. 2, pp. 269–276.
- Simmons, M., Pollak, S., Peirone, B., 2009. High energy rock fall embankment constructed using a freestanding woven wire mesh reinforced soil structure. *Proceedings of the 60th Highway Geology Symposium*, Buffalo, New York, pp. 290–301.
- Subrin, D., 2006. Modelisation analytique et numerique pseudo-statique des merlons de protection contre les chutes de blocs rocheux. *Proceedings of Journees Nationales de Geotechnique et de Geologie*, Lyon, France, pp. 145–152 (in French).
- Sung, E., Yashima, A., Aminata, D., Sugimori, K., Sawada, K., Inoue, S., Nishida, Y., 2008. Numerical assessment of the performance of protecting wall against rockfall. *Proceedings of the 5th International Symposium on Earth Reinforcement*, Kyushu, Japan, pp. 861–867.
- Tissiers, P., 1999. Ditches and reinforced ditches against falling rocks. *Proceedings of the Joint Japan-Swiss Scientific Seminar on Impact Load by Rock Fall and Design of Protection Structures*, Kanazawa, Japan, pp. 65–68.
- UNI, 2012. UNI 11211-4 – Opere di difesa dallacaduta massi. Parte 4: Progetto definitivo ed esecutivo. (in Italian).
- Volkwein, A., Schellenberg, K., Labieuse, V., Agliardi, F., Bourrier, F., Dorren, L.K.A., Gerber, W., Jaboyedoff, M., 2011. Rockfall characterization and structural protection – a review. *Natural Hazards and Earth Systems Sciences* 11, 2617–2651.
- Wang, B., Cavers, D.S., 2008. A simplified approach for rockfall ground penetration and impact stress calculations. *Landslides* 5 (3), 305–310.
- Willye, D.C., 2003. Rockfall mitigation study – Nigata and Kanazawa – Japan. Available online: www.wnrockeng.com/.
- Yoshida, H., 1999. Recent experimental studies on rockfall control in Japan. *Proceedings of the Joint Japan-Swiss Scientific Seminar on Impact Load by Rock Fall and Design of Protection Structures*, Kanazawa, Japan, pp. 69–78.