4 ROCKFALLS

F. Descoeudres - S. Montani Stoffel
A. Boll and W. Gerber
V. Labiouse

4.1 INTRODUCTION

In order to assess the technical possibilities of individual rockfalls or catastrophic falls of large rock masses, it is first necessary to briefly define which types of phenomena will be considered, as well as their intensity and probability of occurrence. According to a generally admitted classification, the generic term *landslides* includes ail gravity induced movements of a soil or a rock mass along a slope (WP/WLI 1990). Five major mechanisms can be distinguished:

fall topple slide flow spread.

The major characteristic for rockfalls is the suddenness of their occurrence, associated with the high particle velocity. Block velocities of 20 to 40 m/s are commonly observed on landslide sites.

Rockfalls consist of free falling blocks of different sizes which are usually detached from a steep rock wall or a cliff, after an initial block *toppling* (block overturning) or a *local* ***slide,*** associated with gravity, water pressure in the joints or adjacent block thrust. The block movement also includes bouncing, rolling and sliding with rock block fragmentation during slope impact (Fig. 4.1).

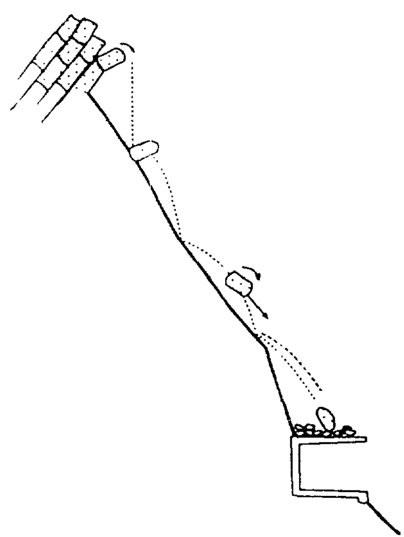


Fig. 4.1 Different particle positions during **a rockfall.**

In Switzerland, an important research programme called *Matterock* has been accomplished recently (Rouiller et al 1998). It deals with the study of the structural pattern of the cliff, confronted with the local topography and with the assessment of the probability that a rockfall can occur, for a given volume and shape.

Rock block *fall* analysis *methods* are used in order to predict the block path and the block energy during movement (Giani 1992). A block detached from a rock face may have the following types of movement during flight: fret: falling, bouncing, rolling, or sliding. Analytical procedures for the mathematical description of the rockfall phenomenon, that consider the geometrical and mechanical characteristics, have been set up by several researchers in the last twenty years. The analytical formulations can be divided into two categories: rigorous methods and lumped mass methods. In the ***rigorous method analysis,*** the size and shape of the blocks are assumed to be known "a priori" and all the block movements, including those involving the block rotation, are considered. In the ***lumped mass method,*** however, the single block is considered to be a simple point of mass m and velocity **v**. Therefore, the rotational moments are not taken into account.

When focusing on the relation between the dangerous natural phenomenon and the man-made structures to be protected (buildings, roads, lifelines), it is **worth** noting that the disaster resilient infrastructures have to be designed mainly in the ***slope*** exposed to rockfalls, because **no** relevant protection can be taken in the source area, except some attents of local stabilisation in the cliff zone. This is often hard to attain and dangerous to modify.

4.2 ROCKFALL RESILIENT INFRASTRUCTURE

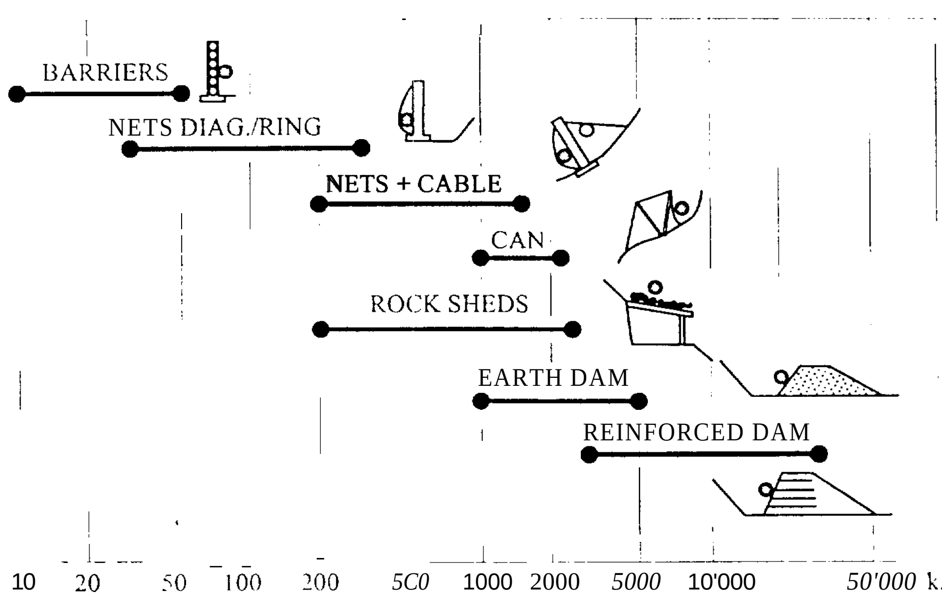
4.2.1 Stabilisation methods

The modification of the cliff geometry by drilling and blasting is an hazardous solution, associated with difficulties in *controlling* the fall of the blasted rock itself as well as in ***assessing*** the stability of the remaining rock masses.

However, the face of the cliff can be protected by bolting and shotcreting in order to reduce the rate of weakening of the rock mass or of the weathering process. The *remedial **works*** are not easy to execute and their effectiveness difficult to quantify by means of stability analyses or visual observations.

4.2.2 Protecting measures (Fig. 4.2)

The *design* of protecting measures involves the evaluation of the rockfall characteristics and the slope geometry (Descoeudres 1997). Rockfall modelling allows the designer to compute the maximum possible length of the path of a flying block, the distances between the bounces, the elevation of the block trajectory above ground, the velocities and the energy assumed by the block at any time of the movement. In situ observations of past rockfall damages allows calibration of the model.



Fir. 4.2 Energy dissipated by different protection structures

Modification of slope geometry

The creation of benches or ditches in a slope to stop failing blocks can be effective. The position of the *benches* is designed by simulating a large **number** of **rockfalls** in a computer model **where** the benches are incorporated. The rock exposed on the plane of the **beams** can be covered with uncompacted rockfill or earthmaterial to absorb a large **part of** the impact energy.

Slope *ditches* are used to catch the blocks after a fall to prevent rolling or to change the block movement from falling to rolling. Rockfall modelling can also be applied for the best ditch positioning and for the ditch geometry design (depth and **width).**

Barriers and wire net systems

Rail walls and other *stiff barriers* are often used, either individually or in combination with ditches. Their capacity of energy absorption is low. The kinetic energy of deformation is about 10 to 50 kNm or kJ.

Flexible wire net systems, supported by hinged steel posts, have been extensively developed during the last ten years. **A** detailed description is given in section 4.3. The energy absorbing capacity has been improved from about 250 kJ in the 1980ies to more than 2'000 kJ nowadays with ring net constructions.

Rock Sheds

Rockfall shelters are usually concrete structures covered on the roof by an absorbing material such as soil backfill used as a shock absorbing cushion. These protecting structures are expensive but efficient, and consequently used in areas with serious rockfall problems. **A** detailed description and design approach is given in section 4.4.

Reinforced earth retaining structures

Earthdams, often reinforced at the upstream-impact slope with **strip** / sheet metallic or wire elements, can absorb the largest kinetic energies of failing **rocks,** up to 30'000 kJ. The impacts of blocks of about 20 to 30 tons with velocities of 30 to 40 m/s create important deformations in the earth dam. **A** periodic control of the works is therefore required. Reparations are possible after **a** major event.

4.3 WIRE NET ROCKFALL BARRIERS

4.3.1 Introduction

For the last ten years, flexible wire net systems have become an integral part in the protection against rockfall. These modern structures consist of steel wire nets supported by hinged steel posts, which are tied back by wire ropes that contain rope brakes (Fig.4.3).

In order to stop a rock, the maximum kinetic energy of the rock has to be smaller than the energy absorbing capacity of the rockfall barrier. For the last ten years, this energy absorbing capacity has been considerably improved by the use of more sophisticated structural components and structural alterations. In 1985, rocks with a kinetic energy of about 250 kJ could be stopped; nowadays, ring net constructions capable of withstanding more than 2000 kJ are possible (Gerber and Haller 1997). These results could not have been achieved without co-operation between industrial firms **and** research institutes. This applies particularly to the testing of structural components and complete systems. Rockfall barriers have been tested in different countries (e.g. USA, Japan, Taiwan, China, France, Italy, Switzerland) and world-wide contacts between researchers are quite close. In Switzerland, extensive full-scale testing has been carried out by the Swiss Federal Institute for Forest, Snow and Landscape Research (WSL) and two industriai firms, one of the latter having a world-wide reputation for its flexible rockfall barriers. In the following, some major results will be presented.

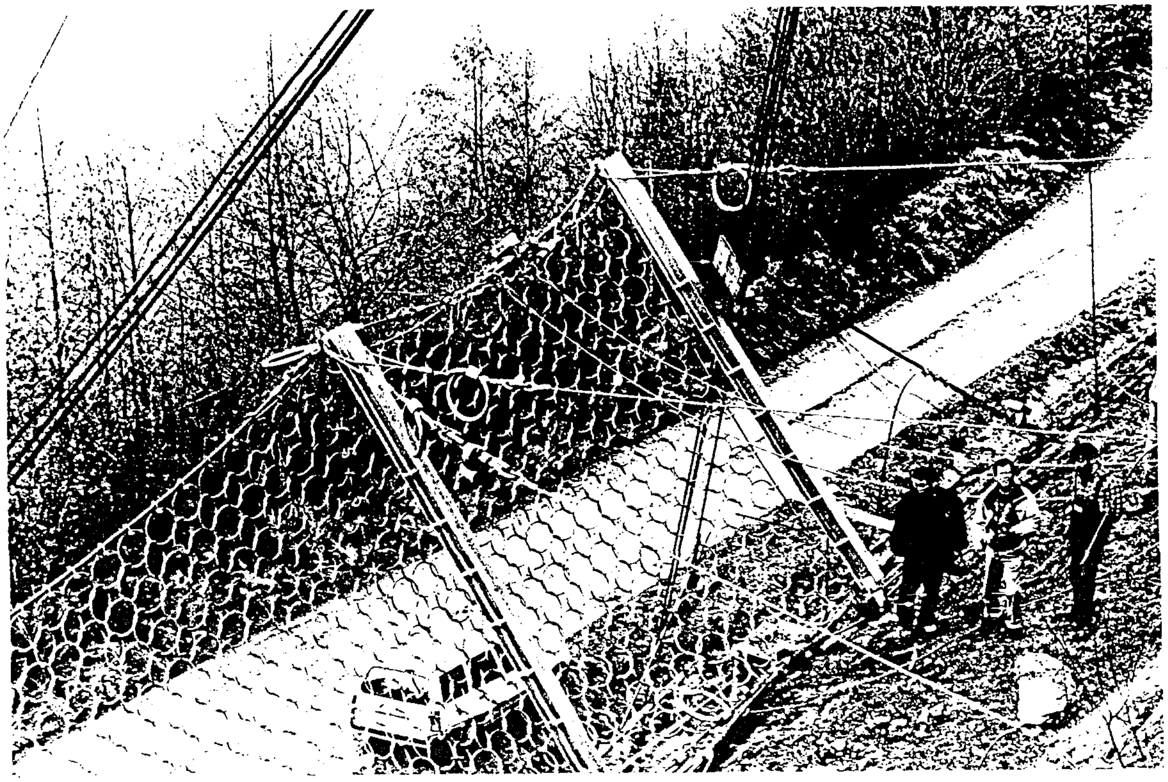


Fig.4.3 Rockfall Barrier with Brake Rings and Strain Gauges.

4.3.2 Full-Scale Testing of Rockfall Barriers

The testing site at Beckenried in Switzerland is characterised by a fairly stable rock surface with a slope angle of about 45°. In the first tests (Test Series 1) a cable crane was used to move the rocks up to the top of the slope from where they were set into motion, accelerated down the slope and - after several impacts with the ground - ended in the barrier, which was to be tested. Due to this set-up, the energies acting onto the barrier differed considerably. All the same, in 1990 a first important step in the development of rockfall barriers was reached, namely, the improvement in energy absorbing capacity from about 250 kJ to about 400 kJ. This was, so to say, the birth of the highly flexible type of rockfall barrier, internationally employed ever since.

The aim then was to obtain higher values of a rock's kinetic energy and, accordingly, of the barrier's energy absorbing capacity. To this purpose, a new cable crane was installed. In Test Series 2, the rocks remained suspended from the cable crane during part of its downward motion, were released in full flight some distance above the barrier and hit the ground prior to rolling or bouncing into it. The impact velocities were mostly lower than 20 ms^{-1}. In Test Series 3, the rocks were aimed directly from the downward moving cable crane at the rockfall bamer with a given velocity of about 26.5 ms^{-1}. This set-up made it possible to not only calculate the energy of a rock in advance, but also to hit specific points of the barrier quite accurately. In 1992 Test Series 3 raised the energy level to about 1000 kJ (Gerber and Böll 1993) and by the end of 1997, 2000 kJ could be absorbed.

Comparing international results, it is essential to stress the fact, that all the maximum values of impact energy mentioned in the context with our tests, were fully absorbed by the rockfall bamers themselves. That means, that no ground contact occurred during the deceleration phase of the rock in the net, and that the systems suffered no damage.

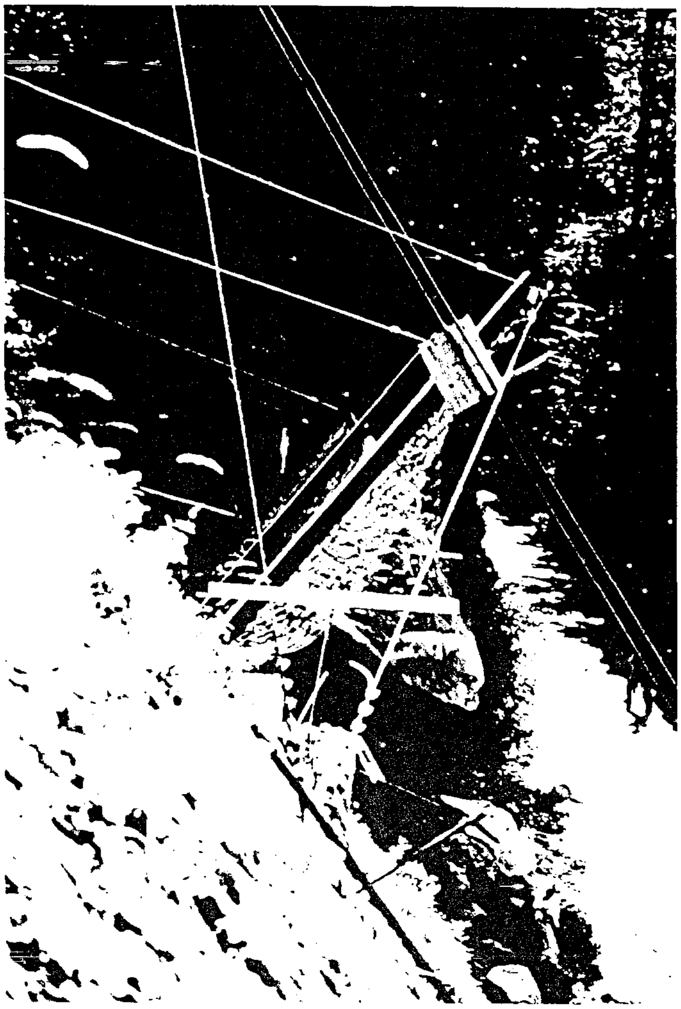


Fig. 4.4 Rockfall Barrier at impact.

4.3.3 **Forces arid** Design Criteria

From a structural engineer's point of view, the energy based design method, where a rock's kinetic energy is compared with the energy absorbing capacity of the rockfall barrier, has its severe drawbacks. It can, among other things, not provide information about important safety aspects such as the factors of safety against partial or total collapse. It is quite clear, that, as long as a more sophisticated engineering design method is not available, all the different types of rockfall barriers have to be tested individually in full-scale tests. We are quite sure that full-scale tests will always be necessary to a certain extent. Considering the high costs of such tests, it would be highly desirable to minimise their number. Accordingly, we started to concentrate our efforts on the determination of forces acting during impact – a first step towards proper engineering design based on calculations rather than tests only. Detailed studies on the relationship between forces, bending moments and energy dissipation resulted, when in one test a steel post was deliberately struck (Böll 1995).

Gerber and Haller (1997) report that a fast frame film camera was used to establish velocities, how the relationship between velocity and time allowed the calculation of the deceleration of each rock of given mass, and how it was possible to compute the effective forces acting on the rocks and the barriers, respectively. For two years, tensile strain gauges have also been used to record the forces acting on wire ropes, namely, guy ropes and net supporting ropes.

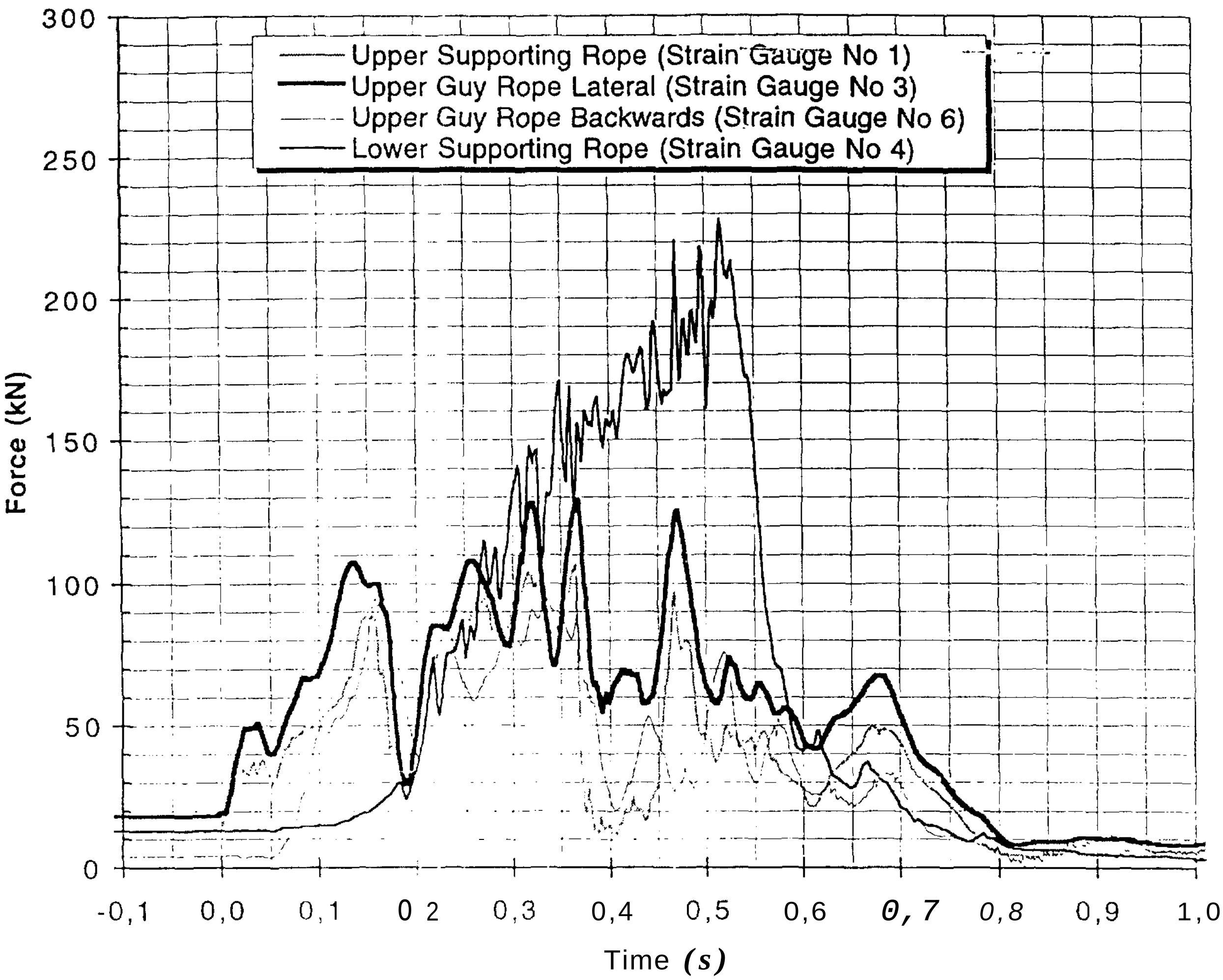


Fig. 4.5 Strain Gauge Readrngs (1500 kJ – Test; 500 **Samples per** Second)

The maximum peak forces occur at about 0,2 to 0,5 seconds after contact between the rock and the barrier. The sooner they occur, the higher they are; thereby representing rather stiff and fairly flexible systems, respectively.

From the point of view of maintenance, a stiff system, that can still safely absorb the required energy, is obviously ideal. The forces, on the other hand, have to be kept within reasonable limits. It is therefore essential to optimise the stiffness of a system and its elements according to the required energy capacity. By judicious design of each different barrier, the maximum peak forces in the ropes could be held at well under 250 kN, more or less independent on the specific maximum impact energy for which that particular system had been designed. In a barrier designed to withstand an impact energy of 2000 kJ, for example, this energy can be dissipated within 0,5 seconds.

4.3.4 Summary and Outlook

The last ten years of research, development and testing have yielded interesting and impressive results. The energy absorbing capacity of rockfall barriers has been raised by a factor of eight, and the scientific and technical knowledge has improved considerably. Despite these efforts and results there remain, alas, still many design problems unsolved. Only current arid future work can provide answers.

In Switzerland, as well as in many other countries, Standards on rockfall barriers and unified testing procedures are about to be issued. This, of course, is an important step forward. We have to make sure, however, that individual research, development and testing will still be carried on to assure future progress.

4.4 ROCK SHEDS

4.4.1 Introduction

in mountainous areas, highways frequently follow steep slopes. Because exposed to avalanches and falling rocks, they are usually protected at hazardous places by rock sheds **(Fig. 4.6).** These structures are characterised by a highly reinforced concrete roof slab covered by a **soil** layer used as a shock absorbing cushion.

To have a better knowledge on the damping abilities of the covering cushion, and thus to acquire a reasonable estimation of the impulsive load due to a rockfall, an experimental study was carried out at the rock mechanics laboratory of the Swiss Federal Institute of Technology Lausanne (Labiouse et al. 1996). This research work formed **part** of a specifications for the design of rock sheds (Montani Stoffel 1998).

4.4.2. Description of problem

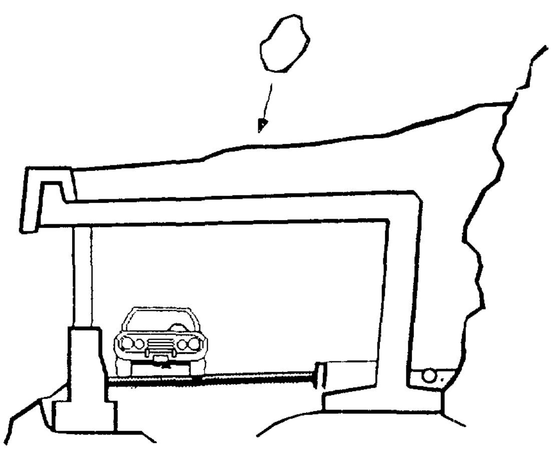


Fig. 4.6 Rockshed.

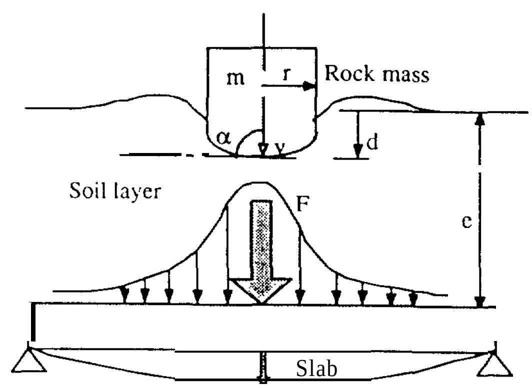


Fig. 4.7 Definition of **problem.**

The impulsive force F on the structure and the penetration depth d are mainly governed by three factors (Fig.4.7) :

- Mass m, equivalent sphere radius r, velocity v and impact angle α of rock blocks;
- Slope, thickness e and material properties of covering cushion;
- Structural characteristics of rock shed (scheme, strength, stiffness, natural frequencies).

4.4.3 Test device

The tests were conducted in a 5 m diameter and **8** m deep shaft (Fig. **4.8).** At its bottom, a reinforced concrete slab **(3.4** m **x 3.4** m **x** 0.2 m) on four supports was covered by a soil layer. The experiments were conducted by block impacts on this set-up, **varying** the parameters according to Table 4.1.

Table 4.1 Range of test parameters

Parameter	
Mass of block m [t]	0.1, 0.5, 1
Radius of block r [m]	0.21, 0.36, 0.45
Impact velocity v [m/s]	4.4 ÷ 14
Soil layer thickness e [m]	0.35, 0.5, 1.0
Soil modulus M_E [kPa] (whether compacted or not)	600÷43'000
Impact angle α [°]	44, 67, 90

The shape of the falling weights was cylindrical with a spherical bottom, made of steel shells filled with concrete. Three kinds of soil materials were used: (1) gravel 3/32, (2) filling materials that can be economically laid on real structures: (3) materials from alluvial fans or scrap rocks from tunnel excavations.

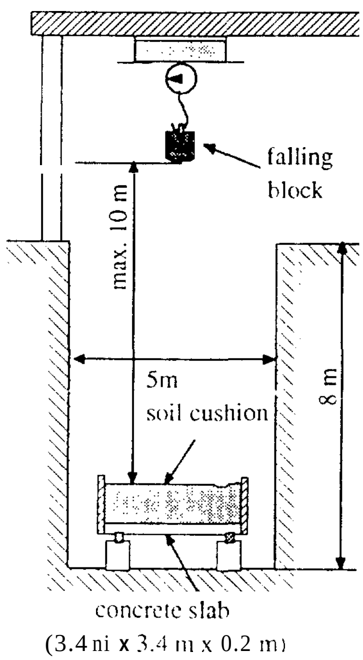


Fig. 3.8 Elevation of test shaft.

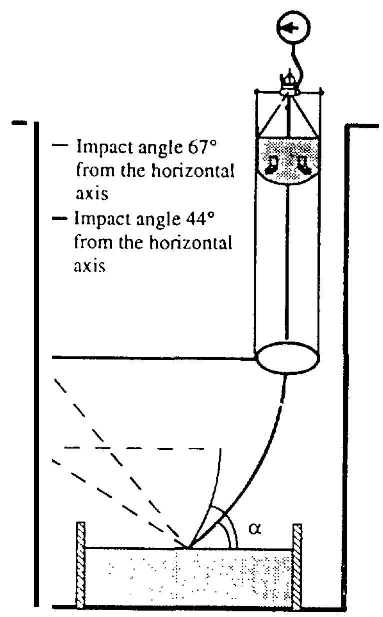


Fig.4.9 Elevation of testing shaft and for inclined impacts.

During an impact test, data were either directly measured or indirectly calculated from other measurements. From the accelerometer located on the falling block, the deceleration of the weight during the impact was measured. Then a first integration enabled to determine the decrease of velocity with time, and a second integration the block penetration into the soil layer. The *impulsive force* F_{acc} is defined as the deceleration of the falling weight multiplied by its mass.

Assuming a perfectly centred impact (above P1 pressure meter) and an axisymmetrical distribution of earth pressure acting upon the surface of the concrete slab, the so-called *integrated force* as the resultant interaction force acting on the slab was determined (Fig.4.7).

To evaluate the significant parameters for the slab design, a qualitative interpretation was performed analysing systematically the influence of each parameter. The important parameters of this preliminary study are:

- weight of block and its falling height,
- thickness of soil cushion and its compaction degree.

4.4.4 Quantitative evaluation of forces

From the experimental results, the relationship between the work due to the block penetration $F_{acc} \cdot d$ and the potential energy of the block E_{pot} was analyzed. From simple models, this relation was established *to* be linear as

$$F_{acc} \cdot d = 1.6 \cdot E_{pot} \quad . \tag{4.1}$$

It **is** worthwhile *to* notice that the M_E-Modulus (Modulus **of** subgrade reaction obtained from a standardised plate bearing test on the soil layer) has no influence on this relation.

The modulus has **an** effect on the relation between impulsive force and penetration of the block, however. The penetration follows approximately an ***elastic Hertz*** *law* with a small correction due to the influence of the internal friction angle **as**

$$F_{acc} = 1.05 \cdot R^{0.5} \cdot \exp\left(\frac{R}{1.2 \cdot e}\right) \cdot M_E \cdot (\tan\varphi)^{0.5} \cdot d^{1.5} \quad . \tag{4.2}$$

From Eqs. (4.1) and (4.2) results for the *impulsive force*

$$F_{acc} = 1.35 \cdot R^{0.2} \cdot \exp\left(\frac{R}{3 \cdot e}\right) \cdot M_E^{0.4} \cdot (\tan\varphi)^{0.2} \cdot E_{pot}^{0.6} \quad . \tag{4.3}$$

This dimensionally homogeneous formula is to that developed by the Japan Road Association [1978]. The differences are:

- the layer thickness e has a similar influence for an infinite layer only,
- the internal friction angle is not included in the Japanese expression. This influence could not be assessed from the experiments and the exponent 0.2 of $\tan\varphi$ originates from numerical studies by Genchi et al. (1996), Montani et al. (1997), and Donzé et al. (1999).

A formulation for the *integrated force* has been developed in a similar way as

$$F_{int} = 0.13 \cdot R^{0.8} \cdot e^{-0.1} \cdot M_E^{0.4} \cdot \sqrt{\frac{k}{(m+M) \cdot g}} \cdot E_{pot}^{0.6} \quad . \tag{4.4}$$

This equation is similar to Eq.(4.3) with respect to the influence of the M_E modulus and the potential energy E_{pot}. Differences for the other parameters include:

- Although an influence of the friction angle on the integrated force seems to be obvious, it has not been introduced here, owing to divergent results.
- As expected, the force acting on the slab is inversely proportional to the layer thickness, with the borderline case of a zero force for a layer of infinite thickness.
- The integrated force is proportional to the square root of the slab stiffness **k** divided by the oscillating mass after impact (mass of the block m + equivalent mass of the soil cushion and the slab M). This term can also be deduced from simple energy considerations (Tonello 1988).

For both, the impulsive force F_{acc} and the integrated force F_{int} agreement between calculated and measured values was observed. Although the impact energies of the test campaign were important for laboratory experiments (100 W), they remain much smaller than on real rock sheds (up to 2000 kJ). For this range of high energies, it is expected that the influence of the plastic characteristics of the soil cushion (i.e. internal friction angle) becomes important, and the effect of the elastic characteristics thus less important.

The validity range of the proposed equations related to a minimal layer thickness is: e≥50 cm and e ≥ **2d.** Otherwise, the required damping conditions are not satisfied and the integrated force is acting nearly as a single load. For instance, for impacts on a 35 cm thick layer, it has been observed that the measured forces are larger than the calculated.

4.4.5 Inclined impacts

To evaluate the influence of the impact angle, some additional tests were conducted by changing the testing device as shown in Figure 4.9. A ring and a strap were attached by an articulation to the wall of the testing shaft. During the fall, the block was dropped in the ring, the attachments stabilising the system horizontally break, the whole system block-ring-strap described a circle by turning around the articulation to produce an inclined impact. The impact angle was varied by changing the attachment point and the length of the strap. The main results were:

- For blocks completely stopped after first impingement, the impulsive force doesn't change. However, a reduction of this force occurs when the block keeps a part of its kinetic energy after impact.
- The reduction of the integrated force as a function of the impact angle α is

$$F_{\mathrm{int},inc} = F_{\mathrm{int},normal} \cdot (\sin\alpha)^2 \tag{4.6}$$

4.4.6 Conclusions

A qualitative interpretation of the expenmental results allowed to analyse systematically the influence of each parameter. The most important factors with regard to the design of rock sheds are:

- the weight of the block and its falling height,
- the thickness of the soil cushion and its compaction degree.

The several forces measured during the testing campaign were compared qualitatively. Then, statistical analyses resulted in mathematical expressions for the *impulsive force* and the *integrated force*. Some of the results may be influenced by the specific test program. Although the impact energies were large for laboratory experiments (100 kJ), they remain much smaller than on real rock sheds (up to 2000 kJ). For that reason, an *in-situ testing program* should be undertaken in the future.

References

Böll, A. (1995). Tragsicherheit von Stahlstützen in Steinschlagverbauungen. *Schweizer Ingenieur und Architekt* 113(45): 1035-1039.

Descoeudres, F. (1997). Aspects géomécaniques des instabilités de falaises rocheuses et des chutes de blocs. Société *Suisse de mécanique des sols et des roches* **135:** 3-11.

Donzé F. V., Magnier S. A., Montani S. and Descoeudres F. (1999). Numerical simulation of rock block impacts on soil-covered sheds by a discrete element method (to be published).

Genchi R., Calvetti F., Nova R. (1996). Studio degli effetti dell'impatto di massi su una struttura di protezione rigida, Politecnico di Milano, Italy.

Gerber, W., Boll **A.** (1993). Massnahmen zum Schutz gegen Rutschungen und Steinschlag. In: Eidgenossische Forschungsanstalt für Wald, Schnee und Landschaft **(Hrsg.)**: *Naturgefahren, Forum für Wissen*: 33-38.

Gerber, W., Haller, B. (1997). Safe and economical rockfall protection barriers. In: LEE, H.K.; Yang, H.S.; Chung, S.K. (eds) Proceedings of the ***1st** Asian **Rock** Mechanics **Symposium:*** ARMS '97. **A** regional Conference of ISRM/Seoul/Korea/13-15 October **1997.** Environmental and Safety Concerns in Undergound Construction 2: 915-920. Balkema: Rotterdam.

Giani, G.P. **(1992).** *Rock slope stability analysis.* Balkema: Rotterdam.

Japan Road Association (1978). *Handbook of prevention against rockfalls.* **Tokyo:** Japan (in Japanese)

Labiouse V., Descoeudres F., Montani S. **(1996).** Experimental study **of** rock sheds impacted by rock blocks. *Structural Engineering International* **IABSE 3: 171-176.**

Montani S., Descoeudres F., Bucber K. M. **(1997).** Numerical analysis **of** rock **blocks** impacting a rock shed covered by a soil layer, S. Pietruszczak, G.N. Pande, eds., *Numerical Models* in *Geomechanics*, NUMOG VI, Montréal: 641-646. Balkema: Rotterdam.

Montani Stoffel S. (1998). Sollicitation dynamique de la couverture des galeries de protection lors de chutes de blocs, *PhD-Thesis* 1899. EPFL: Lausanne, Switzerland.

Rouiller J.-D., Jaboyedoff M. (1998). *Pentes instables dans le Pennique valaisan - Matterock.* Vdf ETH: Zurich.

Tonello J. (1988). Généralités et approche de modèles simples, *Stage paravalanches,* **A** et B, (E.N.P.C.).

WP/WLI (1990). Int. Geot. Soc. UNESCO W.P. on World Landslide Inventory. Bull. *Int. Ass. of Eng. Geol.* **41**: 5-12.