SIXTH FRAMEWORK PROGRAMME

Project no. 018412

IRASMOS

Integral Risk Management of Extremely Rapid Mass Movements

Specific Targeted Research Project

Priority VI: Sustainable Development, Global Change and Ecosystems

D2.2 – Detailed performance study of countermeasures in selected test areas

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SIXTH FRAMEWORK PROGRAMME
PRIORITY VI
Sustainable Development, Global Change and Ecosystems

SPECIFIC TARGETED RESEARCH PROJECT

irasmos

INTEGRAL RISK MANAGEMENT OF EXTREMELY RAPID MASS MOVEMENTS

WORK PACKAGE 2:
COUNTERMEASURES

DELIVERABLE D2.2
DESIGN CRITERIA, COST/BENEFIT ANALYSIS

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DECISIONAL PROCESS

1.1 Decision making process

Introduction

Natural hazards represent an enormous threat to the development of the alpine regions, hence, since many centuries ago, the administrators have provided defense structures to protect the territory and life of people. Even though the technology and construction techniques of countermeasures have developed, increasing their effectiveness and durability, in the recent years we have experienced enormous natural disasters, with lots of damages and loss of people. On the one hand this can be due to the increased seriousness and intensity of natural events and to specific anthropogenic factors (e.g. road cut, deforestation), which augmented the propensity to instability. On the other hand, the tourist and economic growth of the territory has strongly increased its vulnerability, amplifying the results and damages of each event.

At national and even regional scales, it's difficult to accurately assess economic damage form landslide because most countries and provinces do not compile comprehensive records of landslides and related damage, and data collection methods vary from area to area. In the following table are resume estimates damages in some nations caused by natural hazard:

Table 1: *e*stimated average annual costs (in US$) of landslide in various nations [Sidle and Hirotaka 2006]

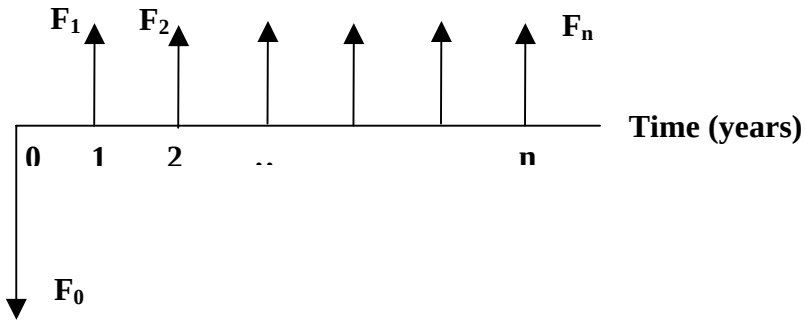
Nation	Average annual direct costs	Average annual total costs
Japan	1.5 billion $	4 billion $
United States	1.2 billion $	1.6-3.2 billion $
Italy		2.6-5 billion $
Canada		70 million $
Nepal	19.6 million $	

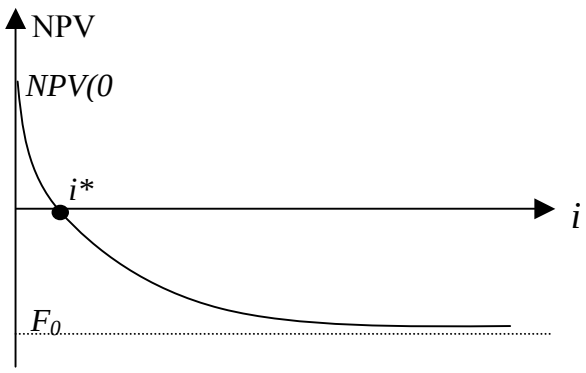
Economics factors are calling for an ever-stronger request for security in the mountain regions, which could permit a safe development of the territory. Yet, due to the increasingly limited financial resources of the public sector, it is important to consider not only the capability of the measure to mitigate the danger, but also its overall economic efficiency. In practice, the derived utility should exceed or be equal to the associated costs [Fuchs, McAlpin 2005].

How to take into account the benefit of a countermeasure? What are the costs involved? The objective of the cost-benefit analysis (CBA) is thus to give an answer to these questions and to address an evaluation approach that is suitable to all natural hazards. In a simplified scheme, the analysis should take into account the scenarios ante and post the construction of the countermeasure (namely with and without countermeasure), and evaluate the associated costs and benefits.

The decisional process

A project of investment can be defined as any program or action that may result in an investment and that may be described by a cash flow (see Figure 1). The projects of investments may be dependent or independent, mutually exclusive or conditioned and, when the budget of expenditure is limited, may have additional dependencies. In order to propose a general decision support method, it is necessary that all projects of investments be combined in a way that the decision involves just mutually exclusive alternatives. Then for every alternative the applicability is evaluated and the corresponding cash flow calculated.

Figure 1: *cash flow example*	
Economic equivalence methods	The next step is to define a basis for the comparison of the alternatives that contains detailed information about expenditures and incomes and takes into account the time value of money (economic equivalence).
Net Present value (NPV)	The net present value is the difference between the equivalent of expenditures and incomes of a cash flow for an investment, discounted by an interest rate to account for the time value of money [Thuesen, Fabrycky 1993]. A given amount of money is always more valuable sooner than later since this enables one to take advantage of investment opportunities. Being Ft the cash flow at the time t, i the interest rate and n the duration (in years) of the investment, the net present value (NPV) can be calculated as follows: $NPV(i) = \sum_{t=0}^{n} \frac{F_t}{(1+i)^t}$ If the interest rate is expected to change during the payback period it is common to use the different interest rate estimates for the future time periods.
Annual equivalent (AE)	The annual equivalent is based on the assumption that every cash flow can be transformed in a series of equal annual payments. It can be calculated as follows: $AE(i) = \left[\sum_{t=0}^{n} \frac{F_t}{(1+i)^t}\right] \cdot \left[\frac{i(1+i)^n}{(1+i)^n - 1}\right]$
Internal rate of return (IRR)	The internal rate of return (IRR) for a project is the interest rate that equalizes incomes and expenditures of a cash flow (see Figure 2). It can be calculated as reported hereafter:

	$i^* \mid NPV(i^*) = \sum_{t=0}^{n} \frac{F_t}{(1+i^*)^t} = 0$ This method should be avoided in case of cash flows with multiple sign changes, in which case multiple internal rates could be obtained.
***Figure 2**: internal rate of return*	
Decision criteria	A feasible and often unrecognized alternative is always the "no investment" alternative, which means that the investor won't do anything about the prospected projects. This implies that the available money will be funded in other investments, most likely "trusted" investments that can guarantee an acceptable rate to the investor. This rate is referred to as the MARR (minimum attractive rate of return). In other words, for the "no investment" alternative, we can put: $i^*_{A_0} = MARR$. Furthermore it is important to evaluate investment alternatives with the same temporal duration; otherwise the missing cash flows should be integrated. Once the cash flows of all the alternatives are available, it is possible to find the best (most profitable) one. Given two mutually exclusive alternatives, for example A0 (usually defined as the "no investment" alternative) and A1, two approaches can be adopted: Comparison on the incremental investment The alternatives have to be arranged in order of increasing initial investment. Then, on the basis of the differences between their cash flows, one of the following criterion can be used, depending on the chosen economic equivalence method:

	NPV: if $NPV(i)_{A_1-A_0} > 0$: Accept A1, otherwise accept A0 IRR: if $i*_{A_1-A_0} > MARR$: Accept A1, otherwise accept A0 Comparison on the total investment The objective is to choose the alternative with the highest present value. The rule can be outlined as follows: AE: if $AE(i)_{A_1} > AE(i)_{A_0}$: Accept A1 , otherwise accept A0 NPV: if $NPV(i)_{A_1} > NPV(i)_{A_0}$: Accept A1 , otherwise accept A0 Once all the alternatives have been analyzed, the one with the highest value will be chosen. Special attention should be taken if using the IRR: the alternatives should be arranged in order of increasing IRR and, if the highest IRR of all projects is superior to MARR, that alternative should be chosen. Yet this method, under the total investment conditions, presents some drawbacks: it can be adopted just in case of mutually exclusive alternatives and unlimited expendable capital, furthermore, the choice of the highest IRR couldn't lead to the alternative that maximizes the NPV [Thuesen, Fabrycky 1993]. For this reason it is preferable to use the IRR method only under the incremental approach.
Methods of analysis for decision making	While private bodies are evaluated according to profit, public administrations are usually assessed in terms of general welfare. How to measure the general welfare? And furthermore, is there an objective way of measurement? These questions are difficult to answer for a number of reasons [Thuesen, Fabrycky 1993]: - multi-perspective of judgment: citizen side (again characterized by multi-opinions) and private side often have different judgment criteria; - target of the project: it is difficult to differentiate between who receives the benefit (geographic or social community) and who pays for it;

- multi-purpose projects: it is difficult to quantify the aggregated overall benefit, as often the scopes of the projects are conflicting;
- monetization: some of the benefits and disadvantages associated with each project can be expressed in monetary terms, for others it is very difficult: for example a building can be evaluated in monetary terms, but the value of security or clear water is very hard to be assessed. However it's equally important that also the benefits without a market price be included in the analysis. If the reverse occurs and economists have to work with non-market goods, like human lives or the value of recreation, the willingness to pay (WTP) concept is generally used to determine the monetary value of certain impacts;
- interest rate: as a benefit at the present is not equal to a benefit in the future, an interest rate should be taken into account. This should at least reflect the cost of money for the administrations, and in general be comparable to the rate used in private activities. As outlined by [Gamper et al. 2006], the choice of the appropriate discount rate is still contentious and can therefore differ i.e. between 4 and 12 percent (Boardman et al.; 2001, Hackl and Pruckner, 1994). For discount rates used in the applications of natural hazards, see e.g. Wilhelm (1997 or 1999)

In order to maximize the use of public resources for the overall benefit, several techniques have been put in place to quantitatively assess the desirability of different projects.

Cost/Benefit Analysis (CBA)

Cost/Benefit Analysis (CBA) is one of the most popular methods that allows finding the economic justification for a public project. It is based on the analysis of the ratio between the equivalent benefits for the user and the equivalent costs for the promoter (see Table 2):

$$BC(i) = \frac{equivalent\_benefits}{equivalent\_costs}$$

***Table 2**: equivalent costs and benefit*

	Point of view	**Sign (+)**	**Sign (-)**
benefits	User	incomes, advantages, savings	expenditures, disadvantages, losses
costs	Promoter	expenditures, losses	incomes, savings

	If, among the costs, we differentiated between the initial capital invested (I) and the annual operation and maintenance costs (C), we would obtain the following: $BC(i) = \frac{B}{I+C} > 1$ where B is the equivalent benefit and i the interest rate. In order for a project to be financed, the benefits must exceed the costs, i.e. the ratio must be grater than one. In case the public administrator had to choose between several projects with different levels of benefits and costs, it is necessary to interpret the corresponding B/C ratios. The alternatives should be arranged in order of increasing denominator and then the B/C ratio should be calculated according to the incremental procedure: $BC(i)_{A_1 - A_0} > 1$ => Accept A1 , otherwise accept A0 When all the alternatives have been mutually compared, the choice will go to the last remained alternative. This assures that the general net welfare be maximized: $BC(i) = \frac{B}{I+C} > 1$ $B - (I + C) := welfare > 0$ as can be demonstrated by the NPV or AE of welfare based on the total or incremental approach [Thuesen, Fabrycky 1993]. In terms of legitimacy of CBA in natural hazard studies, the method is, among other countries, required by law for major expenditure decisions on protective measures in Austria.
Cost/ Effectiveness	CEA was firstly introduced for the economic evaluation of complex defense and spatial activities. It can be accomplished if the three

Analysis (CEA)	following steps are followed: 1. define clearly the objectives of the system, in order to have a coherent basis of evaluation; 2. delineate at least two alternatives and describe them on the basis of their optimal configuration; 3. outline the criteria relative to cost and efficiency evaluation; often it is more difficult to define the criteria for efficiency (e.g. utility, value, advantage, gain…) than for cost; 4. choose the method of evaluation: fixed cost or fixed efficiency. In the first case, the alternative with the highest efficiency will be chosen, in the latter, the alternative with the minimum cost.
Multicriteria Analysis (MCA): description, appropriate use, key input and output	"Multicriteria Analysis can be defined as the study of methods and procedures by which concerns about multiple conflicting criteria can be formally incorporated in a decision making process" [International Society on MCDM, 2004]. MCA describes any structured approach used to determine overall preferences among alternative options, where the options accomplish several objectives. Explicit recognition is given to the fact that a variety of both monetary and non-monetary objectives may influence policy decisions. In MCA, desirable objectives are specified and corresponding attributes or indicators are identified. The actual measurement of indicators doesn't need to be in monetary terms, yet they are often based on the quantitative analysis (through scoring, ranking and weighting) of a wide range of qualitative impacts. Different environmental and social indicators may be developed side by side with economic costs and benefits. Multicriteria analysis is particularly applicable to cases where a single-criterion approach (such as cost-benefit analysis) falls short, especially where significant environmental and social impacts cannot be assigned monetary values. MCA allows decision makers to include a full range of social, environmental, technical, economic, and financial criteria. The key output can be a single most preferred option, ranked options, short list of options for further appraisal, or characterization of acceptable or unacceptable possibilities. The key input is the criteria of evaluation as well as relevant metrics for those criteria.
CBA and MCA	Cost Benefit Analysis (CBA) and Multicriteria Analysis (MCA) are often used in the environmental economic area. The emphasis of these tools lies

	in providing a sound basis for decision makers' information to consider the alternative projects and to choose the optimal solution. As Gamper et al. [2006] outlines, there are similarities and differences between the two methods. Establishing the decision context: a shared understanding of the decision context that is the political, social, economic and administrative structure, is essential because the impacts can be manifold and a lot of people may be affected whose preferences and perceptions need to be recognized (Omann, 2004). Identify objectives and criteria: an overall objective can be broken down into a subset of objectives, thus higher level goals are dependent on lower level ones: in natural hazard management, a higher level of security might be the main aim when installing protective measures against avalanches, but at the same time a sub-goal could be to minimize the environmental impact (i.e. the loss of biodiversity incurred through the building of high altitude feeder roads). The finalization of the chosen criteria requires assessing them against a range of qualities: criteria should be complete, operational, and decomposable (two factors should not be in opposition in a single criterion), non-redundant, minimal and defined in terms of time [Belton and Stewart, 2002]. MCA can be seen as complementing monetary evaluation or decision making methods such as CBA, rather than looking at it as a different method as a whole. [Garrod and Kenneth, 1999]. Multidisciplinary perspectives and approaches are needed for an integrated assessment of natural hazard management. This issue is also claimed by Steininger and Weck-Hannemann [2002], Ammann [2001] and Kienholz et al. [2004]. Consequently, a complexity of knowledge and data is arising that is difficult to arrange for a single final decision maker. The various interests involved in natural hazard management add on to this complexity.
Performance	
Costs	
Effectiveness	

	1.2 Cost Analysis
Introduction	Determining the economic feasibility of mitigating natural hazard can be viewed according to the prospective of who has an economic interest in the outcome. Hence, economic analysis approaches are covered for both public and private sectors as follows. Investing in public sector mitigation activities: evaluating mitigation strategies in the public sector is complicated because involves estimating all of the economic benefits and costs regardless of who realizes them, which could potentially be a large number of people and economic entities. Furthermore, some benefits cannot be evaluated monetarily, but still affect the public in profound ways. Economists have developed methods to evaluate the economic feasibility of public decisions that involve a diverse set of beneficiaries and non-market benefits Investing in private sector mitigation activities: private sector mitigation projects may be mandated by a regulation or standard, or may be economically justified on its own merits. A building or landowner, whether a private entity or a public agency, required to conform to a mandated standard, may consider the following options: 1) request costs sharing from public agency 2) dispose of the building or land by sale or demolition 3) change the designated use of the building or land and change the hazard mitigation compliance requirement 4) evaluate the most feasible alternatives and initiate the most cost effective hazard mitigation alternative.
Costs of the disaster: direct and indirect costs	We can divide the costs implied by a natural hazard in two great categories: the costs related to the disaster occurrence and the costs of the countermeasure. According to [Sidle 2006], the former can be divided in direct and indirect costs: Direct costs: examples of property damage resulting from landslide near urban or residential areas include damage to homes, offices, and industrial complexes; displaces infrastructure such as roads, railways, power lines, gas lines, communication lines and sidewalks; severed water lines, sewage systems, storm water drains and irrigation system; and damage to other urban facilities or personal possession (car, parks, street lights etc.). In rural areas, property damage from landslide is typically less because of more dispersed investment; however, losses are general more difficult to quantify [DeGraff, 1991]. Other property damage that could be classified as direct costs includes loss of storage capacity of a reservoir due to

	siltation, blockage of navigation in rivers and damage to underground and surface mining operations [Li,1989]. For urban and rural environments alike, significant direct costs are associated with the repair and maintenance of damaged properties Indirect costs: indirect costs resulting from landslides are difficult to assess and typically ignored. In some cases indirect costs may exceed direct costs [Schuster and Highland 2001]. Indirect costs may include: loss of industrial productivity, loss of human productivity (injuries or death), loss of site productivity, depreciated real estate values in the vicinity of damage and in areas threatened by future landslides, costs incurred through the obligation of the public sector to obtain title to property that has recurring landslide problems, loss of tax revenue on devalued property or in the mandatory conversion of private property to public property, loss of tourism revenues, expenses for emergency services and public health workers, impacts on water quality and aquatic habitat. The costs of the disaster are very sensitive to the cost of the human life. The monetary value of human fatalities for a given event can be determined using various approaches: 1) human capital approach [Linneroot 1979]. The average discounted present value of a person is calculated using the average remaining working years, derived from the average age and average retirement age, and the average annual salary; 2) annual collective risk: it allows to calculate the amount of money which has been spent so far to protect human life (implied cost of averting a fatality).This amount of money depends whether people are able to influence the risk or not, and the ability to reduce the risk by own decisions can be defined in four categories. The amount of money for reducing risk is increasing from category 1 to 4 by a factor of one thousand [Bründl et al. 2005]; 3) taxes revenue approach: examines distributional implications of the investments in avalanche defense structure by comparing the total future revenue from taxes of persons living in endangered areas with the cost of the measures. In fact those residents could not live without mitigation measures and thus would not be liable for income tax.
Other impacts of landslides	Aside from the indirect environmental cost of lost site productivity following landslide, other environmental damages can occur that are

	significant but difficult to quantify: Deterioration of aquatic habitat [Platts and Megahan 1975, Swanson et al 1987, Allison et al 2004]: high levels of suspended sediment in streams following landslides can damage the gills of fish and cause mortality [Herbert and Merkens 1961] suspended sediment can also block the transmission of light in water and thus reduce the depth to which photosynthesis occurs. Change on stream channel: landslide can inflict direct damage or drastic change in channel and aquatic habitat by scouring and depositing large quantities of sediment and wood with changes in primary and secondary production [Swanson et al. 1987]. Impact on water users: sediment generated by mass movements can adversely affect downstream municipalities, industries, agriculture, recreational users and domestic water system. When water is diverted for agricultural use, the life of irrigational pumps can be shortened by turbid water and the infiltration capacity of land continually irrigated with such water may be reduced. Furthermore many industries incur additional treatment or production costs when using turbid water. Whilst these damages are significant, they are difficult to quantify in an economic analysis of landslide impacts and thus are usually ignored. Besides their negative consequences, landslide may benefit the environment in a number of ways as natural sources and supplies of sediment to stream. Such debris flow deposits in low-energy environments may enhance aquatic habitat by forming ponds, increasing channel complexity via large wood, releasing nutrients from buried organics, augmenting off-channel habitat, replenishing spawning graves and altering the composition of riparian forests [Nakamura et al, 2000]
Costs of the countermeasure	The costs of the countermeasure, as far as the time of expenditure is concerned, can be divided in: - initial cost, i.e. the initial investment for the realization of a structure (building, bridge, countermeasures etc.) - maintenance costs, also called operation costs because they rerun with continuity during all the life of the work - environmental costs: these costs are related to the modification of the habitat of animals of vegetation, due to the countermeasure construction

Determination of benefits	The benefits of hazard mitigation can be divided into two categories: – Benefits dependent on the occurrence of the natural disaster and its destructive power on structures, people and activities (e.g. buildings-infrastructures damages avoided, relocation and disruption expenses of economic activities avoided, loss of lives avoided, psychological factors). These effects are highly dependant on the particular hazard under consideration (snow avalanche, rock avalanche and debris flow) and can be estimated using observed prices, costs and engineering data. Yet the difficult part is to correctly determine the effectiveness of the hazard mitigation and the resulting reduction in damages and losses; – Benefits independent of the disaster occurrence (e.g. building-land values and insurance availability and rates)

	1.3 Impact Analysis
Introduction	Impact analysis of countermeasures is often a thorny task. It requires the analyst to describe and thereby make assumptions on the effectiveness (i.e., the expected impact) of countermeasures. However, when designing countermeasures we often don't know much about their specific risk reduction impact, but must rely on past observations (often also rare) with similar types of measures. Particularly, their behavior under design load can hardly be observed in reality as most countermeasures are designed to sustain hazard events with return periods of more than 30 years. A further challenge in designing countermeasures is the assessment of combinatorial effects, which emerge from joining multiple measures into one mitigation strategy. Artifacts such as redundancy and impact uncertainty determine the effectiveness of a mitigation strategy and must been regarded in the impact analysis.
	1.3.1 Impact assessment
First Order Reliability Method (FORM)	For the last 30 years, structural engineers have used the First Order Reliability Method (FORM) in order to assess if a structure can sustain a certain load. In its classical notion, FORM traces back to the works of Basler (1961) and Cornell (1969). While its basic principles are presented below, the interested reader is referred to benchmark books in reliability engineering (Madsen et al. 1986; Schneider 1997; Stewart and Melchers 1997) for an in-depth. Though FORM has been advanced to higher order reliability methods, we think that for assessing the uncertain risk reduction impact of a countermeasure the use of FORM is sufficient. The principle of FORM is based on a limit state function $G = R - S$, whereby G, R, and S are random variables. R denotes the resistance of a measure with the probability density function (PDF) $f_R(x)$, S denotes the load upon the measure with the PDF $f_S(x)$, and G is referred to as the limit state defining the failure of a structure as $G < 0$. To begin with, one determines the probability that R is smaller than the load x, which is obtained by the cumulative distribution function (CDF) $F_R(x) = P(R < x)$. The probability that $S = x$ is obtained by the PDF $f_S(x) = P(S = x)$. From this, the failure probability P_f of the mitigation

measure given the uncertain load S may be determined by the integral (see Figure 3):

(1.1) $$P_f = P(R \le S) = P(R - S \le 0) = \int_{-\infty}^{\infty} F_R(x)\, f_S(x)\, dx.$$

Based on the limited state function, one defines the safety margin $M = R - S$ as depicted in Figure 4. If R and S are normally distributed, it follows that M is also a normally distributed random variable whose mean and variance obtains from:

(1.2) $$\mu_M = \mu_R - \mu_S; \quad \sigma_M = \sqrt{\sigma_R^{\ 2} + \sigma_S^{\ 2}}.$$

These two parameters allow defining the reliability index $\beta = \mu_M / \sigma_M$ as illustrated in Figure 4. The failure probability may now be determined by use of the standard normal distribution function as $P_f = \Phi(-\beta)$.

In the general case, R and S are non-normally distributed; one is forced to approximate their real PDF f_i by a normal PDF f_i^N. Rackwitz (1977) proposed the so-called normal tail approximation, which requires that $f_i^N(x_i^*) = f_i(x_i)$ and $F_i^N(x_i^*) = F_i(x_i)$ hold at the design point x_i^*. However, the mean and standard deviation are not equal ($\mu_i^N \neq \mu_i; \sigma_i^N \neq \sigma_i$). The normalized PDF and CDF can be written as:

(1.3) $$f_i^N(x_i^*) = \frac{1}{\sigma_i}\varphi\left(\frac{x_i^* - \mu_i^N}{\sigma_i^N}\right),$$

and

(1.4) $$F_i^N(x_i^*) = \Phi\left(\frac{x_i^* - \mu_i^N}{\sigma_i^N}\right).$$

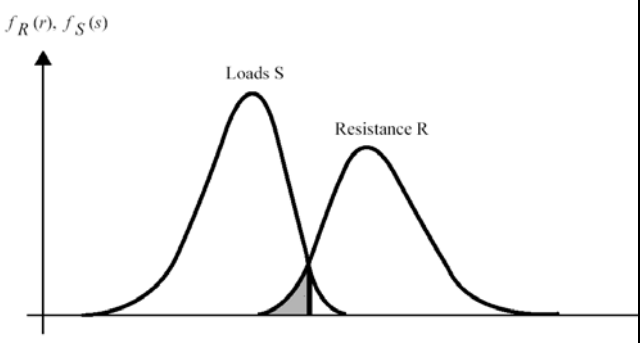
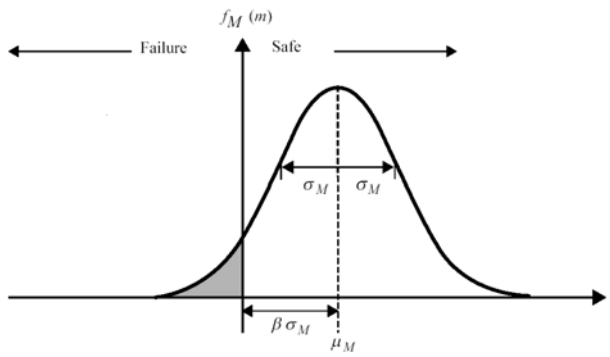
Solving Equations (1.3) and (1.4) with respect to μ_i^N and σ_i^N yields:

(1.5) $$\mu_i^N = x_i^* - \sigma_i^N(x_i^*) \times \Phi^{-1}\left(F_i(x_i^*)\right),$$

and

(1.6) $$\sigma_i^N = \frac{\varphi\left(\Phi^{-1}\left(F_i(x_i^*)\right)\right)}{f_i(x_i^*)}.$$

whereby φ and Φ denote the parameters of the standard normal distribution. If we have determined μ_i^N and σ_i^N at $x_i^* = \mu_S$, we can insert them into Eq. (1.2) and proceed as described above to find the

	failure probability.	
Figure 3		Integration over the realizations of *R* and *S* (adapted from Faber 2005).
Figure 4		PDF for a normally distributed safety margin *M* (adapted from Faber 2005)
Example application	Now, we illustrate the approach by an example from avalanche mitigation. Let us assume an avalanche gallery, which according to the Swiss structural codes must sustain the pressure imposed by a 30-year design load (ASTRA 1994; SIA 2003). We are interested in the expected risk reduction impact *I* caused by the countermeasure *m*, which can be written as (1.7) $I[m] = 1 - P_f(m \mid x > X_{\mathrm{DL}}) \times P(x > X_{\mathrm{DL}})$. Consistent with the Swiss hazard mapping guidelines (Salm et al. 1990), the 30-years design load X_{DL} for an avalanche path is determined at 90 kN/m2 using modeling techniques such as AVAL 1-D (Christen et al. 2002). The probability of one or more avalanche releases to exceed this design load during the lifespan of an avalanche gallery equals one minus the probability that the design load is not exceeded $P(x > X_{\mathrm{DL}}) = 1 - P(x \le X_{\mathrm{DL}})$. It can be approximated by the binomial distribution: (1.8) $P(X_{\mathrm{DL}} \le x) = (1 - (1/T_{\mathrm{DL}}))^n$, whereby the exceedance probability of the design load equals the inverse of the return period T_{DL} = 30 years. The expected lifespan of the gallery *n*	

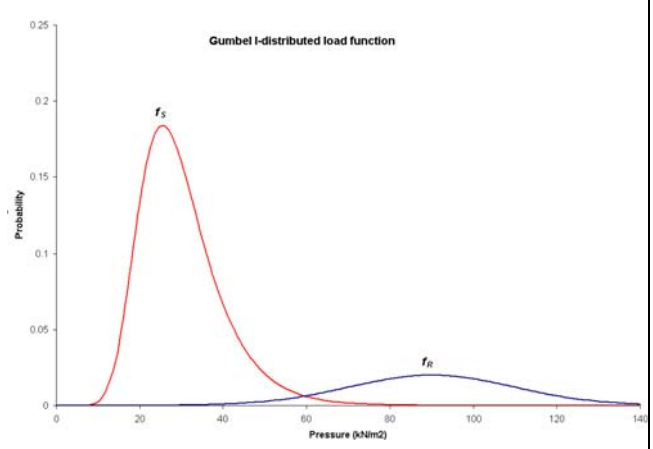
is assumed 50 years (SIA 2003). Thereof, the probability of a design load release within the lifespan becomes $P(x > X_{\mathrm{DL}}) = 0.816$.

The probability of structural failure $P_f(m|x > X_{\mathrm{DL}})$ is assessed applying the FORM concept. One generally assumes the resistance function $f_R(x)$ of a structure to be normally distributed (Schneider 1997), whereas the avalanche load $f_S(x)$ is best described by a Gumbel I- or by a Generalized Pareto distribution (Keylock 2005). For this example, we assume a normally distributed resistance function $f_R \sim N(\mu_R = 90\,\mathrm{kN/m^2}; \sigma_R = 20\,\mathrm{kN/m^2})$ that might be obtained from the construction plan. For the avalanche pressure, a Gumbel I-distributed load function $f_S \sim G(\mu_S = 30\,\mathrm{kN/m^2}; \sigma_S = 10\ \mathrm{kN/m^2})$ is assumed (see Figure 5), which is approximated by the normally distributed function f_S^N using Eq. (1.3). The tail-approximation yields $\mu_S^N(x^* = \mu_S) = 29{,}57\,\mathrm{kN/m^2}$ and $\sigma_S^N(x^* = \sigma_S) = 2{,}45\,\mathrm{kN/m^2}$. By inserting these values into Eq. (1.2), we obtain the reliability index and the corresponding failure probability:

$$\beta = \frac{\mu_M}{\sigma_M} = \frac{\mu_R - \mu_S^N}{\sqrt{\sigma_R^{\ 2} + \sigma_S^{N\,2}}} = 3.09 \quad \Rightarrow \quad P_f = \Phi(-3.09) = 0.001. \tag{1.9}$$

From this, Eq. (1.7) can be solved yielding the expected risk reduction impact $I[m] = 0.999$ of mitigation measure m. The use of this measure reduces the initial risk R_0 to the residual risk $R_1 = R_0 \times (1 - I)$.

Figure 5



PDFs of the Gumbel I-distributed load function $f_S(x)$~G(μ = 30 kN/m2; σ = 10 kN/m2) and the Normal-distributed resistance function $f_R(x)$~N(μ = 90 kN/m2; σ = 20 kN/m2).

1.3.2 Uncertainty analysis

What can we say about the risk reduction impact of a countermeasure if

we don't know much? We here propose a Bayesian approach to specify the probability of frequency distribution for the uncertain or not well-known impact of a mitigation measure. We exemplify the procedure on the risk reduction impact of road closing-policies (RCP). Analyses of roads protected by RPC show that failure frequencies are about 40% (Blattenberger & Fowles 1995), and decrease to less than 25% when successfully supported by artificial avalanche releases (Margreth et al. 2003).

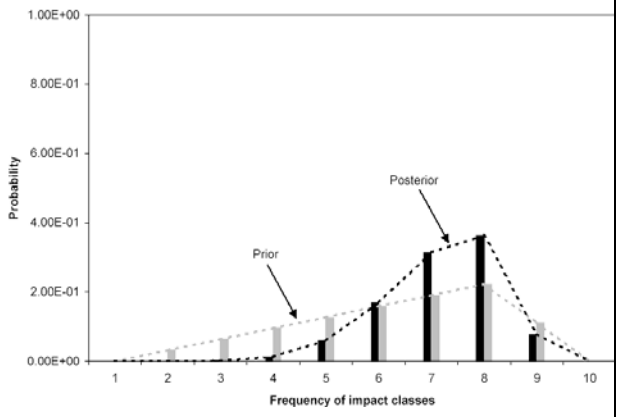
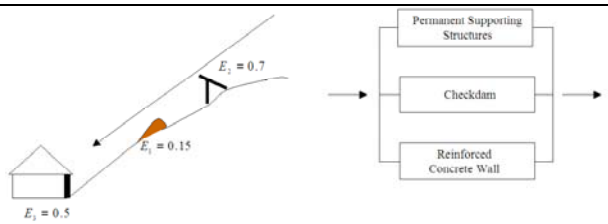
Though we may define the effectiveness in risk reduction of RPC by a single impact value, there is an inherent uncertainty on the performance of road closures affected by organizational aspects of avalanche services such as the attitude toward failures, precautionary behavior, or the communication in ambiguous situations (see Pate-Cornell 1990). As it is difficult to observe this uncertainty, we describe it using the anticipated frequency of risk reduction impacts (Kaplan et al. 1981). We therefore construct a triangular distribution to specify the prior probability of frequency $p(\theta_i)$ that the risk reduction impact belongs to the impact class $i \in \{1, \ldots, 10\}$ defining risk reductions from 0–10% (i = 1, worst case) to 90–100% (i = 10, best case), with i = 8 (70–80%) being the most frequently expected impact. As the impact of RPC is of the form {x successful closings on n avalanche events}, its probability given that the frequency is θ_i may be written as

$$(1.10) \qquad p(I_{\mathrm{OMS}}|\theta_i) = \binom{n}{x} \times \theta_i^{\,x} \times (1-\theta_i)^{n-x}.$$

Using Bayes' Theorem, the posterior probability of frequency given the information on the average of successful closings obtains from

$$(1.11) \qquad p(\theta_i|I_{\mathrm{OMS}}) = \frac{p(\theta_i)\,p(I_{\mathrm{OMS}}|\theta_i)}{\sum_{\theta j} p(\theta_j)\,p(I_{\mathrm{OMS}}|\theta_j)}.$$

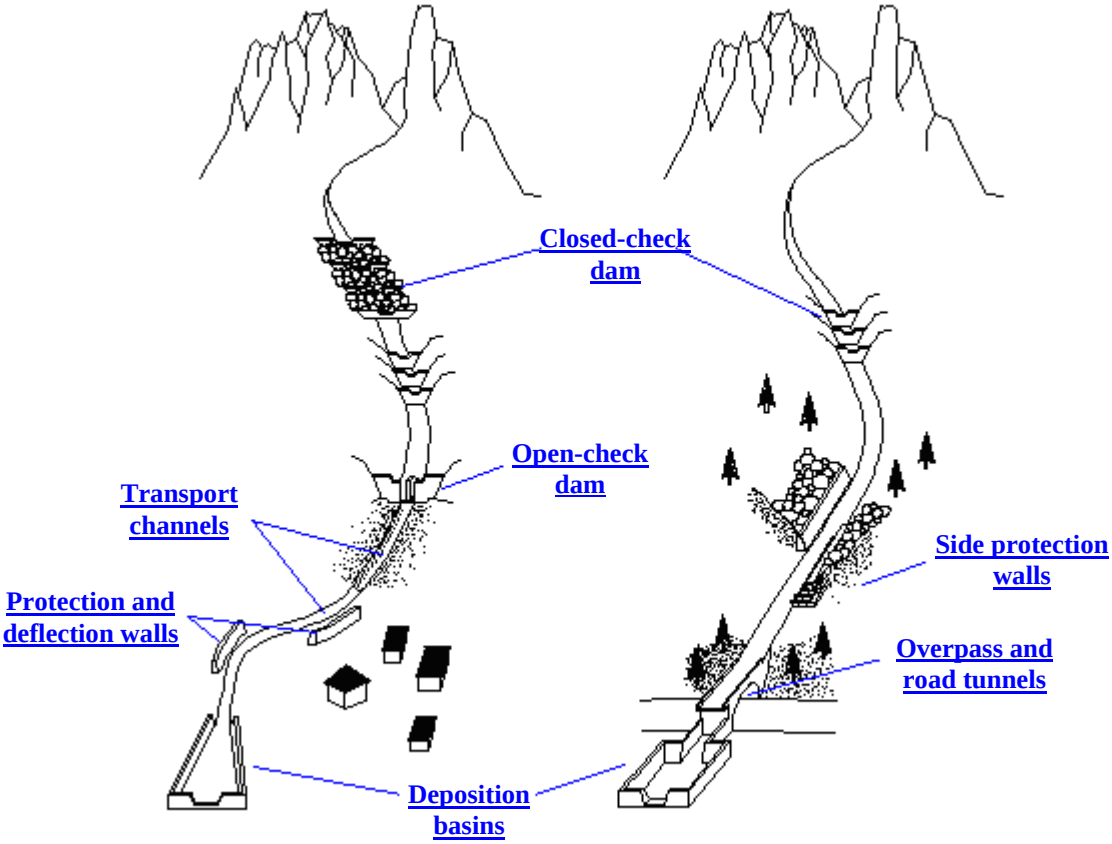
The prior and posterior probabilities of frequencies are depicted in Figure 6. The standard deviation of the posterior (σ = 0.10) allows to specify the expected risk reduction impact E[I_{OMS}] = 0.72, and reasonable minimum ($I^{-}_{\mathrm{RPC}} = 0.62$) and maximum ($I^{+}_{\mathrm{RPC}} = 0.82$) impacts. I^{-}_{RPC} can consequently be used as a precautionary decision criteria representing the lowest impact to be assumed in the risk analysis.

Figure 6		Prior and posterior probability of frequency distributions for the risk reduction impact of RPC.
	1.3.3 Redundancy analysis	
	If different countermeasures are jointed into a risk mitigation strategy, redundancy effects occur. Figure 7 illustrates these redundancy effects by means of a simple example. E.g., a house at the foot of an avalanche prone slope might be protected by: permanent protection structures in the release area (i_1 = 0.5); a small check dam in the transition zone (i_2 = 0.15); and a reinforced concrete wall in the run-out area (i_3 = 0.7). For the sake of simplicity, we focus on the expected impacts and neglect their PDFs. Employing this strategy, damage may only occur if the three components of mitigation fail simultaneously. In accordance with Eq. (1.7), the impact of this risk mitigation strategy can be written as: (1.12) $I_{1,2,3} = \prod_{x=1,2,3} i_x = (1-0.7)\times(1-0.15)\times(1-0.5) = 0.13.$ As impacts do not sum up ($I_{1,2,3} \neq i_1 + i_2 + i_3$), but the costs do ($C_{1,2,3} = c_1 + c_2 + c_3$), it is the crucial task of designing risk mitigation strategies to decide if the increase in cost implied by redundant measures is justified by the increase in risk reduction (Pate-Cornell et al. 2004).	
Figure 7		Illustration of the parallel configuration of a risk mitigation strategy against snow avalanches

Chapter 2

DEBRIS FLOWS

2.1 Countermeasures against Debris Flow



Kind of structure: **Closed check dam**

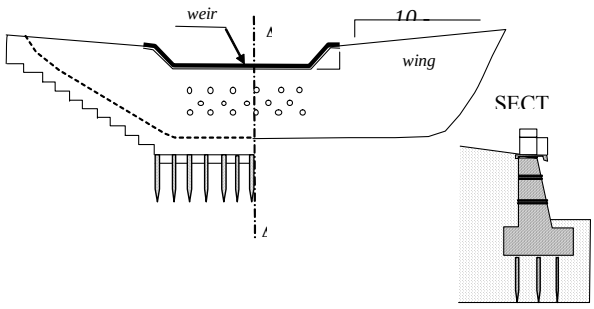


Figure 8: *Closed check dam.*

Description of the countermeasure:	A closed check dam is a structure placed transversally to the torrent, from one bank to the other one and backfilled. It is designed to control the solid and the liquid discharges and volumes, the velocity and the level of a debris flow (photo gallery). Closed check dams are composed of a weir, two wings and a strong foundation, which must be designed for the hydraulic purposes and must be verified for different conditions of stability.

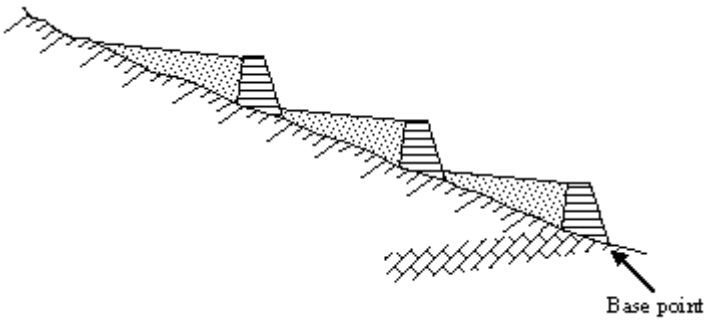


Figure 9: *Scheme of a series of closed check dams.*

Purpose:	A check dam has the main purpose to control directly the depth of the debris flow. Indirectly it changes the slope of the bed and as a consequence it reduces the flow velocity and the erosion capacity on the bed and on the banks. A secondary effect is a possible action on the water content of the debris flow. The change in slope normally determines sedimentation of part of the debris. This effect induces a reduction of granular concentration.

Hydraulic design:	Height and number of the dams: it depends on the designed change in slope. Height of the dam: it depends on the form of the cross section of the torrent, and in particular on the height of the banks. Weir dimension: it depends on the peak (total) discharge; practical rules are proposed for the weir thickness, and the wings slopes (example). Downstream erosion: it depends on height of the dam, peak discharge and downstream condition (flow depth and grain size distribution) (example).
Static/Dynamic design criteria:	Global stability: translation, rotation, bearing capacity, state of stress and piping. Dynamic impact. (example).
Structural design:	During its intended life a closed check dam is exposed to different types of static and dynamic actions from the foundation soil, the backfill material and from the flow. Some of these action values vary greatly from one situation to the other. The first step consists in defining all the situations to which the structure will be exposed. Then, combinations of actions are calculated before verifying that the structure remains stable whatever the situation. The stability is assessed concerning the risk of translation, rotation and material foundation resistance. Special attention must be paid to the risk of downstream erosion (scouring) or lateral erosion (flood bypass). In the case of freestanding structures this stability validation is always followed by the design of the concrete structure. (example).
Materials:	The classical dams, especially those used for debris flow purposes, are built by reinforced concrete. Gabions, wire nettings, geotextiles and timber are also used for part of the structure or for the whole structure. The weir is generally protected from abrasion using a specific material (granite or steel section mainly). On high-mountains it is possible to find dams built with ecologic material (wood) that nevertheless has the defect of scarce durability and in the case of debris flow defense-works they are not recommended because of their scarce resistance. The dams in Palvis gabions are built quickly, relatively cheap but the weir must be covered by strong materials in order to prevent the cutting of the metallic net. All these materials are exposed to abrasion, oxidation and site specific ageing agents (moisture, freeze/thaw cycles., etc.). In some cases the torrent water quality may affect the dam.
Costs analysis:	The evaluation of the costs of a dam in reinforced concrete is based on the prices list of the own country (p. 49).
Software available:	Both geotechnical and hydraulic design criteria are specific to torrents. Largely available software's are not appropriate for their design and oriented tools are necessary. In France, some automatic design tools are

	used.
References:	
Photo gallery	 
	Figure 10: Series of gravity check dams (Cudam)

Figure 11: Dam in Palvis Gabions (Maccaferri)

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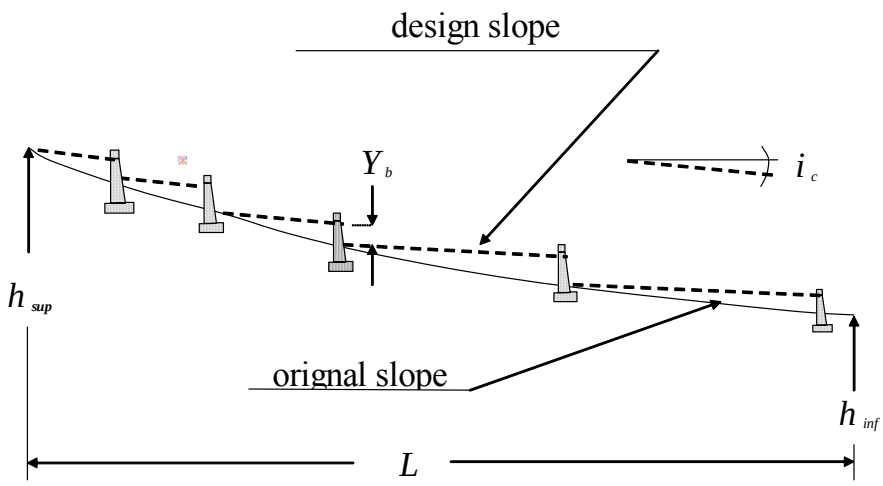
Hydraulic design:	**General design: number, distances, height** The aim of a check dam is to modify the slope of the channel. With this purpose, series of check dams are generally constructed. Depending on the longitudinal torrent slope and on the design slope, the number and height of dams is defined. The distance between two dams is generally determined depending on the equilibrium slope. For a single dam, the main characteristic is its height, which is basically linked to its function. To fulfill this function, different types of structure can be built: - free standing dam, mainly made of reinforced concrete with or without abutment, but also made of prefabricated reinforced concrete elements or of prefabricated metallic structures; - gravity dam: made of concrete, rocks, masonry, prefabricated concrete blocks, gabions, geotextile reinforced soil; - arch-dams; - other types, such as wire netting fence and expanded metal fences, are generally used for small height dams and in gullies. The choice between the different types of structures mainly depends on the height of the dam, the torrent cross-section, the bed and bank soils mechanical characteristics, the site accessibility and the cost. For example, the choice of a free standing dam in place of a gravity dam (the two main types) can be justified by economical reasons when the height is more than 3 m or in case of poor quality of the foundation soil. The structural design of check dams depends on the type of structure, following the same frame. It accounts for mechanical interactions between the soil and the structure, the flow and the structure, but also for interactions with possible snow avalanches.  ***Figure 12**: Number, distances, height of closed dams. h_{sup} and h_{inf} are respectively the elevations of the bed in the initial and final section of the torrent.* If the average height of the dams Y_b is given, the number of the dams can be calculated as:

	$$n = \frac{h_{\text{sup}} - h_{\text{inf}} - i_c L}{Y_b}$$ where h_{sup} and h_{inf} are the elevations of the bed in the initial and final section of the torrent respectively, Y_b is the height of the dam and L in the length of the reach and i_c is the compensation slope as defined in the next paragraph.

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Hydraulics: slope design	<u>Slope according to bed load condition:</u> There are two criteria to estimate the new design slope of the torrent: – **compensation slope**; – **equilibrium slope**. In the first case the slope is estimated with the Shields' criterion, assuming the condition of incipient motion for the bed material: $$\theta = \theta_{cr}$$ The definition of θ, the Shield mobility parameter, is: $$\theta = \frac{u_*^2}{g\Delta d}$$ where: $u_* = \sqrt{\tau_o / \rho}$ [*m/s*] is the shear velocity. τ_o is the bed shear stress $\left[N/m^2\right]$, ρ is the density of the fluid $\left[kg/m^3\right]$, ρ_s is the density of the sediments and $\Delta = \frac{\rho_s - \rho}{\rho}$ is the relative submerged density of sediments and *d* is the diameter of the sediments. θ_{cr} represents the critical mobility parameter for the case of incipient motion: for rough bed, as normally occurs in torrents and mountain rivers, $\theta_{cr} \cong 0.06$. The local value of τ_o is generally given by the mathematical models. However in uniform flow condition the local value of τ_0 is given by the expression $\tau_o = \rho h_o i_o$. Here h_o is the flow depth and i_o the bed and free surface slope (which are coincident in uniform flow conditions). After substitution it is possible to deduce the value of the **compensation slope**: $$i_c = \theta_{cr} \Delta \frac{d}{h_o}$$ In the second case the **equilibrium slope**, i_θ**,** is the design bed slope and it is obtained under the hypothesis of the presence of sediment transport (bed load). In this case also the bed load, Q_s $\left[m^3/s\right]$ must be known. By using the Meyer-Peter Müller's formula, it is possible to deduce the equilibrium slope, corresponding to the design sediment transport load, Q_s [m^3/s]: $$i_\theta = \left(\frac{\Delta}{h}\right)\left(\theta_{cr} d + 0.25\left(\frac{Q_s}{B\sqrt{g\Delta}}\right)\right) = i_c + 0.25\left(\frac{\Delta}{h}\right)\left(\frac{Q_s}{B\sqrt{g\Delta}}\right)^{\frac{2}{3}}$$ where B is the width of the channel.

	<u>Slope according to debris flow peak discharge (Takahashi, 1991):</u> Takahashi's formula gives the value of slope of the torrent that mobilizes a layer of bed of thickness d, corresponding to the flow depth h: $$tan\,\alpha = tan\,\varphi \frac{C^*(\Delta - 1)}{C^*(\Delta - 1) + (1 + h/d)}$$ where C^* is the volumetric concentration of sediment in static conditions, $C^* = (1 - n)$ with n the porosity of the material. If the value of the thickness d is too small (in comparison with h) to allow a uniform dispersion of the solid particles in the fluid above, the movement of the particles initiates under the combined effect of the dynamic action of the fluid and due to the gravity. This situation is an intermediate stage between that of the intense bed load and that of a fully developed debris flow, and it is named *immature debris flow*. According to Takahashi, if $d \geq 0.7h$ a debris flow develops. For $d = 0.7h$ the threshold slope α_1 is obtained; for slopes greater than α_1 the debris flow develops: $$tan\,\alpha_1 = tan\,\varphi \frac{C^*(\Delta - 1)}{C^*(\Delta - 1) + 2.43}$$

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Hydraulics: weir design	The hydraulic behavior of a dam before it's filling is assumed to be that of a horizontal-crested weir. The discharge is given by the Belànger's formula: $$Q=\sqrt{\frac{4}{27}}\left(B_g+\frac{4}{5}nh_m\right)\sqrt{2g}h_m^{1.5}$$ where n is the reciprocal of the slope of the inclined sides of the weir and h_m is the hydraulic head with respect to the weir base. It is possible to calculate the flow depth h_m upstream of the dam, if the discharge Q and the width B_g of the weir are assigned, or to calculate the width if the discharge and the flow depth are assigned.

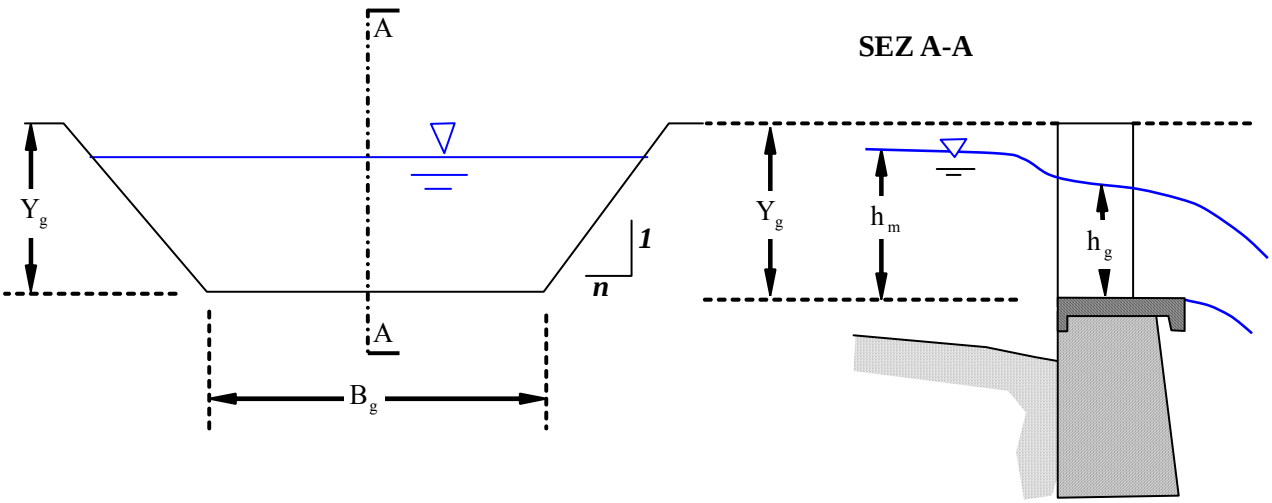


Figure 13: *Scheme of the weir of a dam.*

	Assuming that a critical flow condition takes place on the weir, the depth h_g inside the weir is given by: $$h_g=h_c=\sqrt[3]{\frac{Q^2}{gB_g^2}}=\sqrt[3]{\frac{\left(0.385\sqrt{2g}h_m^{1.5}\right)^2}{g}}=\frac{2}{3}h_m$$ The height of the weir Y_g has to satisfy the following condition: $$Y_g\geq\frac{3}{2}h_g$$ Instead, in the case of a completely filled dam and under the hypothesis of absence of sediment transport, the character of the flow upstream of the dam is typically supercritical and the conditions on the weir are obtained from the conservation of the volume of the flow and of the mechanical energy between the section above the dam and the section of the weir and is given by:

$$h_g + \frac{Q^2}{2gh_g^2\left(B_g + nh_g\right)^2} = h_m + \frac{Q^2}{2g\left(h_m B_m\right)^2}$$

The critical condition in this situation represents a safer condition.

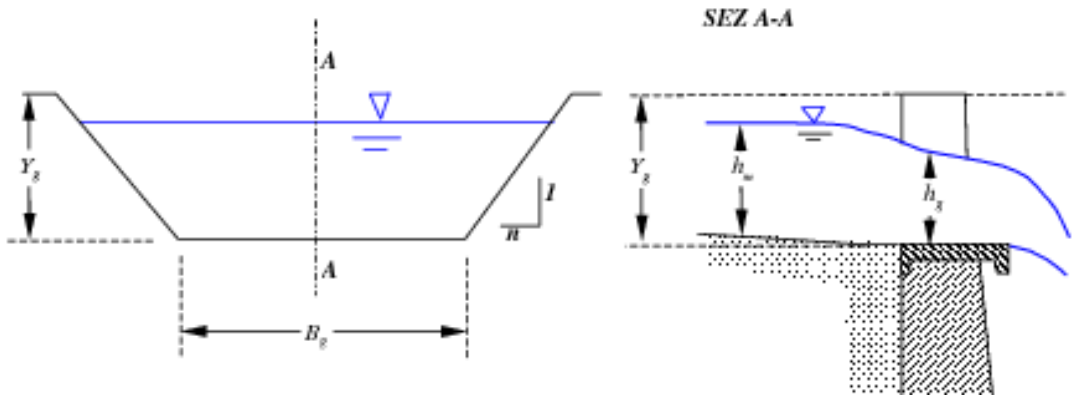
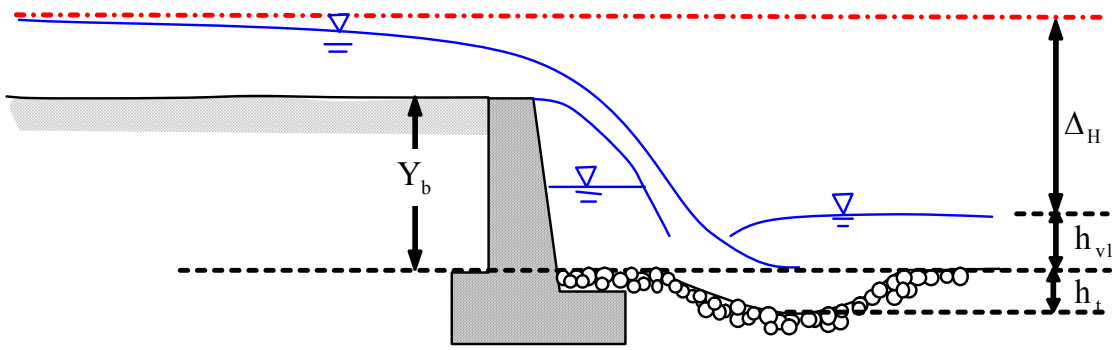


***Figure 14**: Scheme of the weir of a dam completely filled.*

A numeric example of the design of the weir is given in Appendix (p. 197).

Downstream localized scouring:	The flow that slumps from the weir can cause erosion on the foot of the dam that can decrease the stability of the structure if it is not contained within some limits. The entity of the excavation can be calculated as a function of the diameter d_{90} of the bed material, according to the two next formulas by Schoklitsch and by Veronese, respectively: $$h_t = 4.75 \frac{\Delta_H^{0.2} q^{0.57}}{d_{90}^{0.32}} - h_{v1}$$ $$h_t = 3.68 \frac{\Delta_H^{0.225} q^{0.54}}{d_{90}^{0.42}} - h_{v1}$$ where Δ_H (the difference of the energy between the section of the weir and the section downstream) is expressed in [m], h_{v1} and h_t in [m], d_{90} in [mm] and q in [m^3/(sm)].
 ***Figure 15**: Scheme of the erosion down the dam.*	
	To calculate h_{v1} we proceed in the following way: 1) by imposing the balance of the energy between the section of the weir and the section down the stream, we have: $$Y_b + \frac{3}{2}\sqrt[3]{\frac{Q^2}{gB_g^2}} = h_{v_1} + \frac{Q^2}{2g\left(B_{v1}h'_{v1}\right)^2}$$ 2) h_{v1} is usually know by imposing the uniform flow condition in the reach downstream the dam. Using the Gauckler-Strickler uniform flow formula, where K_S $[m^{\frac{1}{3}}s^{-1}]$ is the friction coefficient according to Strickler, and under the hypothesis of rectangular wide bed, we have: $$Q = K_S B_{v1} h_{v1}^{5/3}\ i_f^{1/2}$$

The length L_s of the erosion can be calculated by the following formula:

$$L_s = 1.8\left(h_{v1} + h_t\right)$$

The protection of the erosion can be obtained in these ways:

1) by predisposing a filter in loose stones with suitable d_{90} proper to reduce the entity of the erosion. To deduce the value of d_{90} we can use Schoklitsch's or Veronese's formula, imposing to h_t the value of residual erosion that is considered acceptable.

2) by the insertion of a sill downstream the dam (counterdam) in order to have a submerged jump with the purpose of reducing the entity of the erosion.

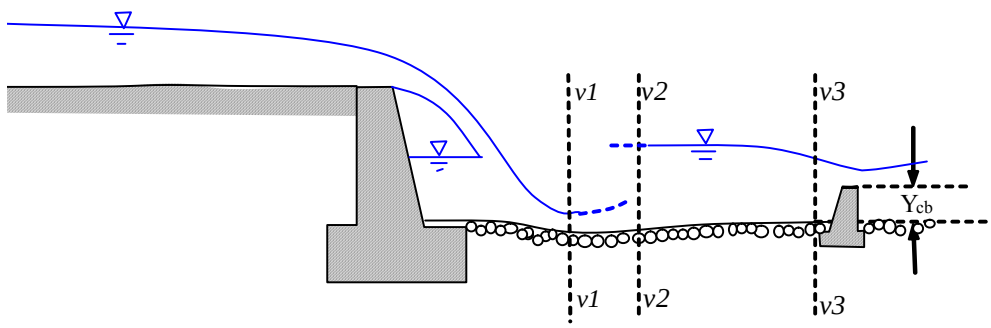


***Figure 16**: Hydraulic scheme of dam and counterdam.*

Solving the system between the total momentum balance equation in the section v_1 and in the section v_2 and the energy balance between the section v_2 and the section of the counterdam, we obtain the height Y_{cb} of the counterdam:

$$\frac{1}{2}\gamma h_{v_1}^2 B + \rho Q u_{v_1} < \frac{1}{2}\gamma h_{v_2}^2 B + \rho Q u_{v_2}$$

$$h_{v_2} + \frac{Q^2}{2gB^2 h_{v_2}^2} = Y_{cb} + \frac{3}{2}\sqrt[3]{\frac{Q^2}{gB_{cb}^2}}$$

A numeric example of the design of the erosion is given in Appendix (p. 199).

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Static/Dynamic design criteria:	The structure of the classical dam is that of a gravity wall subjected to its own weight, forced on above hangings and water pressure under the wall at the base of the foundation.

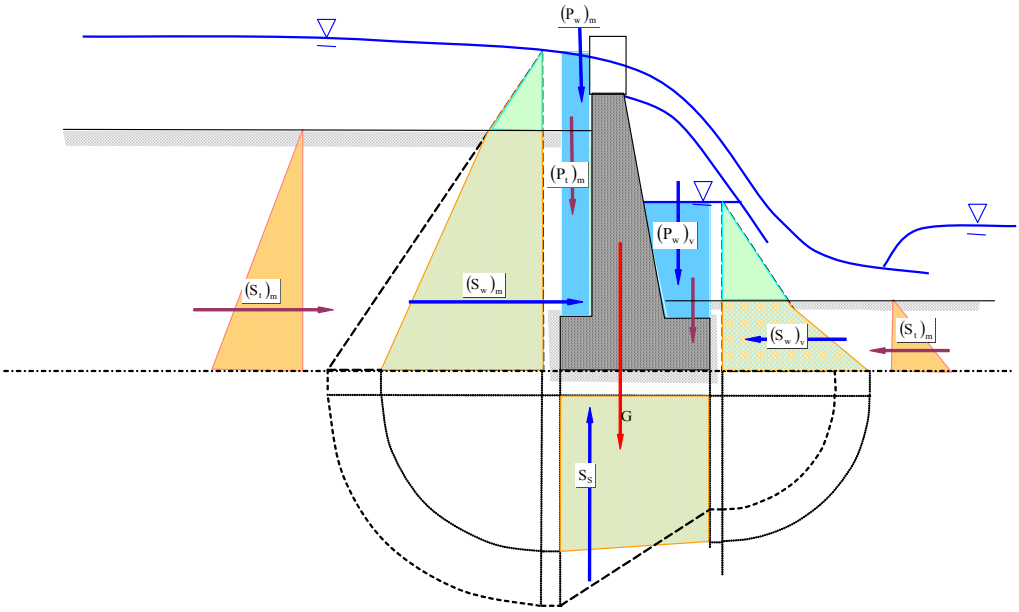


***Figure 17**: Static equilibrium of a dam: scheme of the agent forces.*

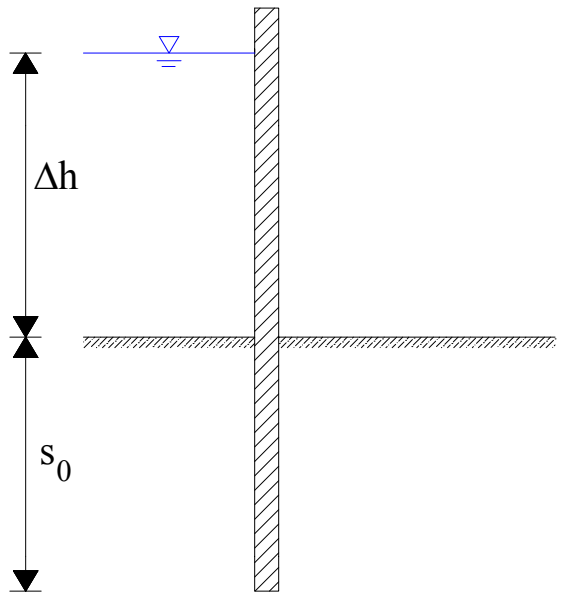
	The conditions of loading are: before and after the burying and, with or without drainages in the dam. For each one of these cases we will make the verifications:
Sliding stability analysis:	$$\eta_{sliding} = \frac{f(F_V - SP)}{F_o} > 1.2$$ where F_V is the resulting of the vertical forces (own weight of the dam and the weight of the soil over the structure), SP the under-pressure force, F_O the resulting of the horizontal forces and $f = \frac{2}{3} tg\varphi$ with φ internal frictional angle of the soil.
Rotation stability analysis:	$$\eta_{ribalt} = \frac{M_s}{M_r} > 1.2$$ with M_s stabilizing moment and M_r active moment calculated in the most depressed point of the foundation.

Verification of round bearing analysis:	We must verify that there is no breaking of the complex soil-foundation due to limit loading. This loading is not a characteristic of the soil, but its value can be calculated in function to the characteristics of the foundation and of the physical- mechanic proprieties of the soil. From the theory of plasticity it is possible to make the calculation for the rectangular elongated foundation subjected to centered loadings and horizontal foundations. The basic formula can be extended to other conditions through the introduction of corrective coefficients. The basic expression of limit loading in drained conditions is: $$q'_{lim} = c' N_c + q' N_q + \frac{1}{2}\gamma' B N_\gamma$$ while in un-drained conditions: $$q_{lim} = c_u N_c + q N_q + \frac{1}{2}\gamma' B N_\gamma$$ where $\gamma' = \gamma_{sat} - \gamma$, c' and c_u are the cohesion of the soil in drained and un-drained conditions, N_c, N_q and N_γ are dimensionless coefficients deduced from the theory of plasticity and function of internal frictional angle φ of the soil under the foundation and B the base of the foundation. The procedure of verification in drained conditions is reported hereafter: – all the horizontal and vertical forces that act and produce moment must be calculated. The eccentricity of the loading is given by: $$ecc = \frac{\sum M}{\sum N}$$ and it determines a reduction of the base to $B' = B - 2 \cdot ecc$. – The vertical effective tension q' on the base of foundation and q'_{lim} are calculated. – The limit loading for linear meter of depth is equal to: $$Q'_{lim} = q'_{lim} \cdot B'$$ – The admissible loading of the foundation for linear meter of depth is calculated:

	$$Q'_{amm} = \frac{Q'_{lim}}{3}$$ – It is verified that the total loading that acts under the foundation is greater than the resulting of all loadings that act over the pit: $$\frac{Q'_{amm} + SP}{Q_{app}} > 3$$ where SP is the hydraulic heave that acts over the foundation.
Strength resistance of material:	We must impose that the admissible tensions in compression and in traction of the material, in the most stressed zone, is not exceeded. The maximum and the minimum value of the tension in the section of the beginning of the plinth of the foundation are σ_1 (the minimum at the extremity above) and the maximum σ_2 of compression down. We must consider the tensions over the foundation, too. In a generic section the tensions are: $$\sigma_{1,2} = \frac{N}{A} \pm \frac{M}{W}$$ with N the normal force, M the moment of the forces calculated in comparison to the barycentre of the section, A the area of the section and W its resistance module.

Fine Sand or lime	8.5
Fine sand	7.0
Middle sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Middle gravel	3.5
Coarse gravel with cobbles	3.0
Cobble and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Stiff clay	1.8
Very stiff clay	1.6

***Table 3**: Lane's rule; factors of security F^* a function of the nature of the soil.*

Piping verification:	We must consider that the dams can be subjected to a phenomenon of instability due to the process of water filtration, and it can cause the so-called piping. This phenomenon can lift a part of the soil in the zone at the bottom of the structure when the local slope i_e is greater than the critic slope i_c. So it is possible to define a coefficient of security for the piping (Terzaghi): $$F = \frac{i_c}{i_e} \geq 4 \div 5$$ with $i_c = \frac{\gamma_{sat} - \gamma}{\gamma} \approx 1$ and $i_e = \frac{\Delta h}{s_0}$ for the case in Figure 18.  ***Figure 18:*** *Scheme of calculation according to Terzaghi.* An empirical method commonly used for piping verification is the Lane's method, according to which there isn't piping until: $$F = \frac{\frac{1}{3} L_O + L_V}{\Delta h} \geq F^*$$ with L_O and L_V the horizontal and vertical dimensions of the work in the soil, Δh difference of water-depth and F^* factor of security variable in function to the nature of the soil (Table 3).
A numeric example of the static design is given in Appendix (p. 202).	

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Structural design:	Classically, the structural design steps are: – Choice of shape and type of the dam. – Definition of phenomenon to whitch the dam is exposed : torrent flow, snow avalanche, bank thrust. – Definition of the different situations. – Identification and quantification of actions related to each situation. – Calculation of the combinations of actions. – Checking of the global stability and calculate the structure. The actions normally considered are: – Permanent actions: weight of the dam, weight and active earth pressure of the backfill material. – Pseudo-permanent actions: weight and active earth pressure of the backfill material with buoyancy, uplift force. – Variable actions: weight of the flow, hydraulic force due to the flow directly on the wall and increase of lateral earth pressure due toe the flood weight. – Accidental actions: in case of mud/debris flow, weight of the flow, impact force due of the flow directly on the wall and increase of lateral earth pressure due to the flood weight. And others are not considered: – Toe abutment: as it may disappear due to excessive scouring. – Overload punctually or uniformly distributed on the backfill. – Seismic solicitations: as regulations do not concern this type of structure. Actions calculation: – Active earth pressure are calculated considering either the Rankine's model or the Coulomb's model, depending on the shape of the structure, backfill characteristics and height. – In the case of buoyancy, the backfill is considered fully saturated and the drainage thought the dam is supposed to be efficient. – The dynamical action of a debris flow on the wall of the dam is calculated as the product of the hydrostatic pressure by a coefficient of 3 to 5. – The foundation soil reaction is considered trapezoid. – Actions during execution concern the case when the concrete is fresh and the dam is not yet backfilled and exposed to a debris flow or a sudden water level.

	Structural design limits: – Foundation related limit states verification is confronted to the characterization of the soil. The soils in the torrents generally consist of strongly heterogeneous and highly variable granular materials (Tacnet et al., 2000 a). On-site and in lab geotechnical classical method are often not suitable for the characterization of torrents bed soils. This appears to be particularly problematic for the determination of the compressive strength of the soil. More over, the bed soil is not uniform across the torrent. To circumvent this difficulty, soil characteristics are defined by comparison with more uniform soils. This is detrimental to structure optimization. – Calculations are generally performed on 1m width structure sections. Sections considered are generally under the weir and on the wings. This method is not optimized as it does not account for transverse actions on and in the structure. For example, these actions may be due to banks thrust or to changes of action values across the torrent (Tacnet et al., 2000 b). – Impacts solicitations due to rocks or timber, mainly on the upper part of the wings, is not well known. Calculations are simplistic. More over, the mechanical response of the structure submitted to impact is not taken into account.
In Italy:	– Analysis of the loads considering all the possible operative conditions that can happen during the life of project of the structure. – Pre-dimensioning and choice of the materials. In this stage they have recourse to the method of admissible tensions (D.M. 14 February 1992, C.M. 24 June 1993 n.37406/STC and Eurocode 2) and to the method of ultimate state limit and serviceability state limit (D.M. 16 January 1996, C.M. 4 July 1996 n.156 AA.GG./STC and Eurocode 2). The dam is schematized as an element fixed at the base and free on the top. – Structural checks.
In France:	The structural design of closed check dams is based on two main documents: – 'CCTG - Fascicule 62, Titre V' concerning the design of foundations in civil engineering. – 'CCTG- Fascicule 62, Titre 1 – BAEL 99' concerning the design of concrete structures.
A numeric example of the structural design is given in Appendix (p. 210).	

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Materials:	List of the possible material used to construct the structure or parts of the structure, and some explanations (*pro and contra*): – Blocks (gravity): Rock blocks are used to build load-bearing structures, with or without jointing. It is sensitive to dislocation. – Concrete and mortar: Concrete is used for the construction of gravity dams. Note that even if not absolutely required, steel reinforcement elements like welded wire mesh are often placed in this type of structure. The problems to be considered with this material are : - Abrasion: when exposed to run-off with loaded flows - Quality of the water for the concrete processing. Unreinforced concrete and mortar are also sometimes used in masonry or large blocks jointing. Specific mixture allows using this type of material in zones prone to abrasion (concrete 'alag', aluminous cement, and special mortar). This mixture is specifically designed for this purpose and generally resist to impact. The problem with these products is their processing that requires skilled staff and specific site conditions. Concrete is sensitive to physical and chemical deterioration, mainly when in contact with water (cracking, oxidation, decalcification, alkali-aggregate reaction…). This phenomenon can lead to considerable damages. Dams made of concrete are sensitive to cracking when exposed to excess of tension within the structure. – Reinforced concrete: Reinforced concrete is mainly used for the construction of freestanding dams. The durability of reinforced concrete structures is confronted to the problem mentioned for their two components : steel and concrete. – Steel: Steel is used in different forms and for different purpose in check dams. Welded wire meshes and bars reinforce concrete. Sections can be used to protect concrete exposed to abrasion (U or L shaped). Steel is sensitive to corrosion. In abrasion protection applications the thickness is a key point as well as the problem of fixation on the structure. See *gabions and wire netting* for other applications. – Wood: Some dams are constructed assembling timbers. This type of structure is not really suitable in case of frequent debris/mud flows as it does not resist to strong abrasion or shocks. In other cases, the advantages can be the cost (construction and maintenance) when adequate tree species are available and

	landscape integration. Wood can also be used as abrasion surface. Timbers placed longitudinally to the flood in the weir offer dam a shock resistant protection against abrasion. It requires regular but rather cheap maintenance and it is not suitable for torrents with frequent debris/mud flows. – Gabions and wire netting: Gabions are sometimes used for the construction of gravity dams. The weir is then protected against abrasion. Wire netting are used to build barriers in gullies, creating low height dams. Durability depends on the same phenomena as those mentioned for steel.

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Cost analysis	Example of the evaluation of the cost based on the prices list of the Provincia Autonoma di Trento.

VOICE	UNIT PRICE	QUANTITY EVALUETED	COST
Excavation with open section (with transport)	7.03 €/m^3		
Excavation with open section (with temporary deposit)	4.31 €/m^3		
Rent and transport of a tracked excavator with power above 110 kW and 155kW	82.49 €/h		
Rent and transport of a dump-truck	56.70 €/h		
Conferment in consented place of the material of the excavation evaluated useless	7.03 €/m^3		
Conglomerate Rck30 for foundation	105.10 €/m^3		
Conglomerate Rck30 for elevation	126.20 €/m^3		
Steel of armature FeB44K or FeB38K	0.78 €/kg		
Electric welded net steel FeB44K or FeB38K	0.70 €/kg		
Drained tube in pvc of diameter of 200 mm	14.72 €/kg		
Stones for upset	42.05 €/m^3		
Stone of construction site	31.54 €/m^3		
Skilled worker	27.59 €/h		

***Table 4**: Const analysis of a closed check dam.*

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Software available:	BARTO – Design of gravity and free standing check dams. 2006. Developed by Cemagref and property of ONF-RTM. (in French)

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References:	Armanini A. (2005). Mountain Streams. Encyclopedia of Hydrological Sciences, John Wiley and Sons Ltd. Armanini A., Fraccarollo L., Larcher, M. (2005). Debris flows. Encyclopedia of Hydrological Sciences, John Wiley and Sons Ltd. Armanini A. and M. Larcher (2001). Rational Criterion for Designing Opening of Slit-Check Dam, J. of Hydr. Enginer., ASCE, Vol. 127, No. 2, Feb., 94-104 Armanini A. and Scotton P. (1993). On the dynamic impact of a debris flow on structures. Proc. XXV IAHR Congress, Tokyo, vol. B, paper n. 1221. Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005. Couvert, B., Lefebvre, B., Lefort P., Morin, E. (1991) Etude générale sur les seuils de correction torrentielle et les plages de dépôt, La Houille Blanche n°6-1991. Deymier, C. ; Tacnet, J.M. ; Mathys, N. (1995) Conception et calcul de barrages de correction torrentielle, Etudes du Cemagref, Série Equipements pour l'eau et l'environnement, n° 18. 1995. Cemagref éditions 287 p. Jäggi, M.N.R. and Pellandini, S. (1997). Torrent Check Dams as a Control Measure for Debris Flows. Lecture Notes on Earth Sciences, Vol. 64, Recent Developments of Debris Flows, Armanini & Michiue (Eds.), Springer-Verlag: 186-205. Nicot, F. ; Tacnet, J.M. ; Flavigny, E. Torrent control dams : assessment of the thrust of the banks 15th international conference on soils mechanics and geotechnical engineering, Istanbul, TUR, 27-31 august 2001 p. 1223-1227 Provincia Autonoma di Trento (2002). http://www.sistemazionemontana.provincia.tn.it/ Provincia Autonoma di Trento (1991). Per una difesa del territorio, Edizioni Arco Trento. Tacnet, J.M. ; Gotteland, P. ; Bernard, A. ; Mathieu, G. ; Deymier, C. (2000 a) Geotechnical characterizing of coarse grained soils : application to torrent soils. International Symposiun Interpraevent 2000, Villach, AUT. p. 307-320. (In French) Tacnet, J.M. ; Garin, L. ; Cheruy, O. (2000 b). Global design of torrent control dams : soil-structure interactions analysis. International Symposiun Interpraevent 2000, Villach, AUT, 26 juin 2000. p. 295-

306. (In French)

Takahashi, T. (1991). Debris flow. IAHR Monograph. Rotterdam: Balkema.

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Kind of structure: **Open check dam**

Description of the countermeasure:	Open check dams are check dams having an opening in their central part, often provided with grids or bars, with the function of regulating the solid discharge. In fact open check dams temporarily retain the sediment transport intercepting coarser material while letting finer grain-size sediment pass trough. According to the historical development of these structures, and in connection with their specific purpose, the open check dams can be divided into two large categories, *beam-dams* and *slit-dams* characterized by a different management of the sediment transport: • *Beam-dam* with wide openings, whose purpose is mostly *filtering* sediments and logs: the retention effect is due to a selective mechanical sieving of the bigger particles. Beam-dams were therefore originally conceived assuming the material to be retained by a mechanical sieving exerted by the grid. The opening of the dam was designed as large as possible, in order to avoid any contraction of the torrent section and to increase as much as possible the surface of the sieve. The distance between the grids varies in the literature: from 1.2-1.5 times (Zollinger 1984; Mizuyama 1984) to 3 times (Üblagger 1972) the diameter of the material to be retained. Quite quickly the material accumulated upstream the grid obstructs the opening of the check dam, hindering the attempted sieving and filtering effect. The presence of a considerable amount of vegetal material might accelerate the clogging process. Therefore this type of check dam, after a certain time from the beginning of the hydrological event, tends to clog and starts working as a traditional closed check-dam. • *Slit-dams* are based on a completely different criterion. This kind of check dam presents one or more narrow, vertical openings, going from the dam's base up to the weir. The effect is mostly *dosing* the sediment transport rate and it is obtained by means of a backwater effect that allows most of the particles to deposit upstream the dam (*hydrodynamic sorting*). Open check dams are composed of a large opening between two wings, a strong foundation, and a weir. Open check dams are mainly free standing structures. (photo gallery) From a functioning point of view any type of open check dam can be related to that of a slit-check dam.

Hydraulic design	The flow velocity is reduced upstream the dam, due to the backwater effect allowing part of the material to deposit. The final goal is to obtain the reduction of the peak solid discharge. If the dam is correctly design the material deposited during the flood is progressively released during the minor events in order to keep the volume upstream of the dam free for the next disastrous event. (An example of calculation of the capacity of an open check dam is given in Appendix).
Structural design:	During its intended life an open check dam is exposed to different types of static or dynamic actions from the foundation soil, the backfill material and from the flow. Some of these action values vary greatly from one situation to the other and over the dam service life. At the beginning of its service life, an open dam is exposed to the action of flow, including the impact of transported solids. The height of the backfill increases the importance of active earth pressure. The first step consists in defining all the situations to which the structure will be exposed. Then, combinations of actions are calculated before verifying that the structure remains stable whatever the situation. The stability is assessed concerning the risk of translation, rotation, material foundation resistance. Special attention must be paid to the risk of downstream erosion (scouring) or lateral erosion (flood bypass). In the case of free standing structures this stability validation is always followed by the design of the concrete structure.
Materials:	Open check dams are mainly made of concrete, steel (section or reinforcement) and rock. Gabions, wire nettings, geotextiles and timber may also be used for part or whole structure. The weir is generally protected from abrasion using a specific material (steel section, granite or steel section mainly) All these materials are exposed to abrasion, oxidation and site specific ageing agents (moisture, freeze/thaw cycles..). In some cases the torrent water quality may affect the dam.
Environmental effects of the open-check dams:	The first advantage of the open-check is the capacity to reduce the peaks of the total discharge Q or of the solid discharge Q_s. Another effect, connected to the first, is the prevention of the erosions of the valley zones of the rivers, thanks to the fact that the check-dam releases the sediments later. Also the check-dam reduces the jump that the fishes can find in the ascent of the river.
Costs analysis:	
Software available:	Both geotechnical and hydraulic designs are specific to torrents. Largely

	available software's are not appropriate for their design and oriented tools are necessary. In France, some automatic design tools are used.
References:	

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Photo gallery:	
	Figure 19: Example of *b*eam-dam with too small openings
	
	Figure 20: Example of slit-dam
	
	Figure 21: Example of open check dam with multiple vertical openings

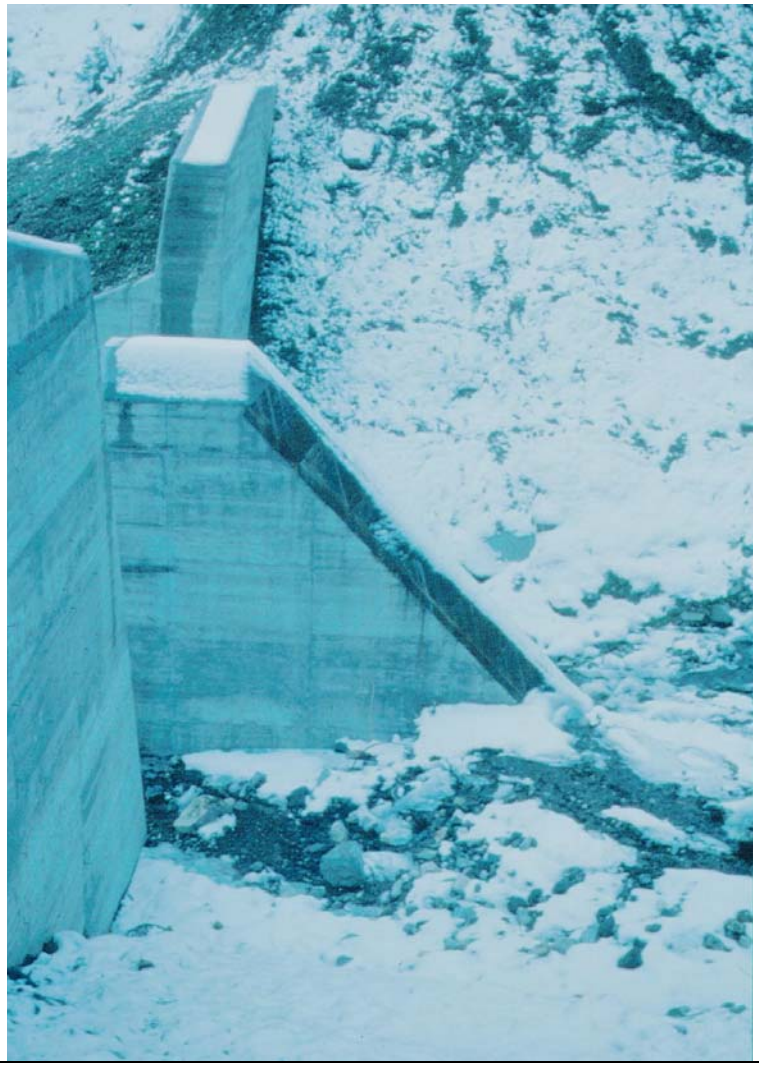

Figure 22: Examples of debris flow breaker

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Materials:	List of the possible material used to construct the structure or parts of the structure, and some explanations (*pro and contra*): – Blocks (gravity): Rock blocks are used to build load bearing structures, with or without jointing. It is sensitive to dislocation. – Concrete and mortar: Concrete is used for the construction of gravity dams. Note that even if not absolutely required, steel reinforcement elements like welded wire mesh are often placed in this type of structure. The problems to be considered with this material are : - Abrasion: when exposed to run-off with loaded flows - Quality of the water for the concrete processing. Unreinforced concrete and mortar are also sometimes used in masonry or large blocks jointing. Specific formulations allow using this type of material in zones prone to abrasion (concrete 'alag', aluminous cement, and special mortar). These formulations are specifically designed for this purpose and generally resist to impact. The problem with these products is their processing that requires skilled staff and specific site conditions. Concrete is sensitive to physical and chemical deterioration, mainly when in contact with water (cracking, oxidation, decalcification, alkali-aggregate reaction…). This phenomenon can lead to considerable damages. Dams made of concrete are sensitive to cracking when exposed to excess of tension within the structure. – Reinforced concrete: Reinforced concrete is mainly used for the construction of free standing dams. The durability of reinforced concrete structures is confronted to the problem mentioned for their two components : steel and concrete. – Steel: Steel is used in different forms and for different purpose in check dams. Welded wire meshes and bars reinforce concrete. Sections can be used to protect concrete exposed to abrasion (U or L shaped). Steel is sensitive to corrosion. In abrasion protection applications the thickness is a key point as well as the problem of fixation on the structure. See *gabions and wire netting* for other applications. – Wood: Some dams are constructed assembling timbers. This type of structure is not really suitable in case of frequent debris/mud flows as it does not resist to strong abrasion or shocks. In other cases, the advantages can be the cost (construction and maintenance) when adequate tree species are available and

	landscape integration. Wood can also be used as abrasion surface. Timbers placed longitudinally to the flood in the weir offer dam a shock resistant protection against abrasion. It requires regular but rather cheap maintenance and it is not suitable for torrents with frequent debris/mud flows. – Gabions and wire netting: Gabions are sometimes used for the construction of gravity dams. The weir is then protected against abrasion. Wire netting are used to build barriers in gullies, creating small dams. Durability depends on the same phenomenon as those mentioned for steel.

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Software available:	BARTO – Design of gravity and free standing check dams. 2006. Developed by Cemagref and property of ONF-RTM. (in French).

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References:	Armanini A. (2005). Mountain Streams. Encyclopedia of Hydrological Sciences, John Wiley and Sons Ltd. Armanini A., Fraccarollo L., Larcher, M. (2005). Debris flows. Encyclopedia of Hydrological Sciences, John Wiley and Sons Ltd. Armanini A. and M. Larcher (2001). Rational Criterion for Designing Opening of Slit-Check Dam, J. of Hydr. Enginer., ASCE, Vol. 127, No. 2, Feb., 94-104 Armanini A. and Scotton P. (1993). On the dynamic impact of a debris flow on structures. Proc. XXV IAHR Congress, Tokyo, vol. B, paper n. 1221. Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005. Armanini, A., Benedetti, G. (1996). "Sulla larghezza di apertura delle briglie a fessura." *XXV Convegno di Idraulica e Costruzioni Idrauliche*, Torino 16-18 Settembre 1996, Atti-Volume III, pp. 13-24. Armanini, A., Dellagiacoma, F. & Ferrari, L. (1991). "From the check dam to development of functional check dams." *Fluvial Hydraulics of Mountain Regions. Lecture Notes on Earth Sciences*, n.37, Springer-Verlag, pp.331-344. Deymier, C. ; Tacnet, J.M. ; Mathys, N. (1995) Conception et calcul de barrages de correction torrentielle, Etudes du Cemagref, Série Equipements pour l'eau et l'environnement, n° 18. 1995. Cemagref éditions 287 p.

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Hydraulic design:	The functioning of the open check dam is based on a hydrodynamic selection and a optimal (design) of the slit induces the sedimentation of the bigger material faraway from the slit so it isn't obstruct.

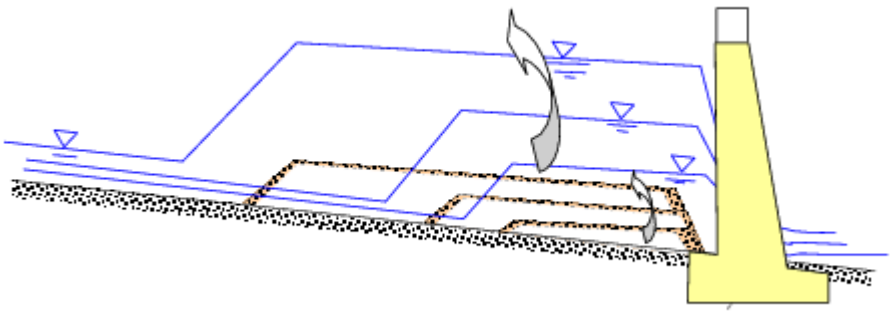
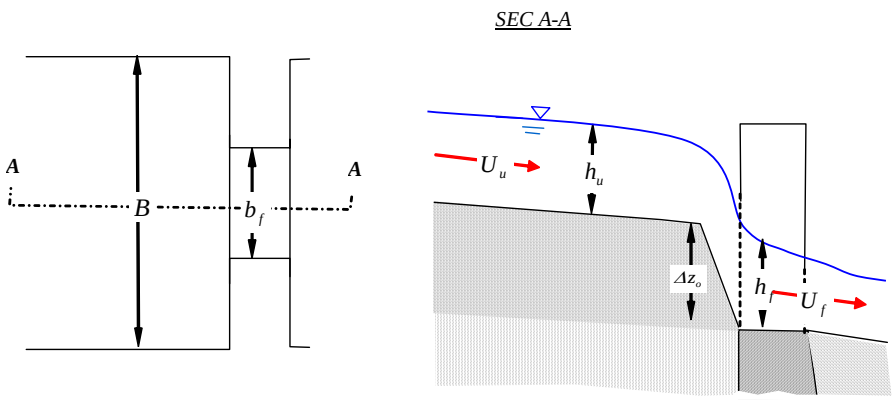


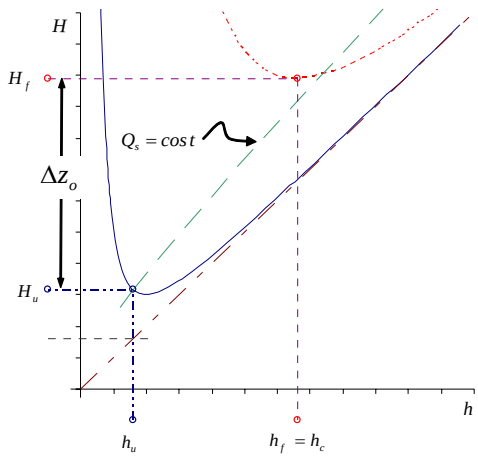
Figure 23: *Scheme of the mechanism of formation of the deposit when the liquid discharge and solid discharge increase.*

The rational criterion for designing the opening is based on the conservation of the mass of water and sediments and on the energy balance between an upstream section and the one of the opening of the check dam (Armanini & Larcher, 2001).
For the case of ordinary sediment transport, the flow regime in the slit can take place according to two different schemes: if the slit is relatively wide, the critical velocity inside the restriction ($u_c = \sqrt{gh\cos\alpha}$) is too small in order to warrant the conservation of the sediment discharge through the dam, therefore a supercritical flow takes place in the slit; on the contrary, if the slit is quite narrow, the critical velocity in the slit is higher than the transport velocity (the uniform flow velocity corresponding to the equilibrium slope) and in the slit there will be a critical flow condition. In this case the solid discharge is smaller than the transport capacity, but it is conserved assuming that the bed inside the slit is unerodible.



The narrow slit scheme for bed-load transport, can be easily extended with minor modifications to massive sediment transport under the simplifying hypothesis that a debris flow can be treated like an homogeneous fluid. On

the height of the deposit upstream the check dam can be calculated imposing the critical flow condition in the slit, the conservation of the total discharge Q and an energy balance between section "u" immediately upstream the check dam and section "f" of the slit:



$$Q = U_u h_u B = U_f h_f b_f$$

$$U_f = \sqrt{g h_f} = \sqrt[3]{\frac{Qg}{b_f}}$$

$$\Delta z_0 = H_f - H_u + \Delta E = \left(h_f + \frac{U_f^2}{2g} \right) - \left(h_u + \frac{U_u^2}{2g} \right) + \Delta E$$

where Δz_0 represents the deposit height, U_u and h_u represent the flow velocity and the depth on top of the deposit respectively, U_f and h_f the flow velocity and depth in the slit, B and b_f the width of the undisturbed channel and of the slit, H_u and H_f the specific energy on the deposit and in the slit respectively and ΔE the localized energy loss due to the presence of the check dam.

The localized energy loss can be expressed with the formula proposed by Borda, provided the flow in the slit is critical:

$$\frac{\Delta E}{h_u} = \frac{F_u^2}{2} \left[1 - \frac{2}{3} \left(F_u R \right)^{-\frac{2}{3}} \right]^2$$

where $F_u = U_u / \sqrt{g h_u}$ is the Froude's number on top of the deposit and $R = B / b_f$ the contraction ratio.

Substituting this expression in the third equation of the system, after some algebra, the following expression is obtained (Armanini & Larcher, 2000):

$$\frac{\Delta z_0}{h_u} = \frac{3}{2} \left(F_u R \right)^{2/3} - 1 - \frac{F_u^2}{2} \left\{ 1 - \left[1 - \frac{2}{3} \left(F_u R \right)^{-2/3} \right]^2 \right\}$$

	Neglecting the energy loss induced by the slit, the expression of Δz_0 can be rewritten as follows: $$\frac{\Delta z_0}{h_u} = \frac{3}{2}\left(F_u R\right)^{2/3} - 1 - \frac{F_u^2}{2}$$
Other typologies of open-check:	*Grid dam*: it modulates the solid capacity (solid discharge) withholding the material of greater dimension and leaving to pass the finer fraction. *Beam dam*: It is calculated as support or embedded beams. *Beam dam with two or three openings*: the openings are separated by buttresses. *Comb dam*: it is constituted from vertical beams calculated as embedded consoles. *Rope dam* *Window dam*: it is constituted from a concrete reticulum. *Reticular dam*: it is formed from one reticular structure. These open-check dams have the same behavior of the slit-check dams and also have an effect of sieve for the presence of the grid.
Costs analysis:	This evaluation of the costs of a dam in reinforced concrete is based on the prices list of the own country.
References:	
A numeric example of the design of a slit check dam is given in Appendix (p. 217).	

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Cost analysis	Example of the evaluation of the cost based on the prices list of the Provincia Autonoma di Trento.

VOICE	UNIT PRICE	QUANTITY EVALUETED	COST
Excavation with open section (with transport)	7.03 €/m^3		
Excavation with open section (with temporary deposit)	4.31 €/m^3		
Rent and transport of a tracked excavator with power above 110 kW and 155kW	82.49 €/h		
Rent and transport of a dump-truck	56.70 €/h		
Conferment in consented place of the material of the excavation evaluated useless	7.03 €/m^3		
Conglomerate Rck30 for foundation	105.10 €/m^3		
Conglomerate Rck30 for elevation	126.20 €/m^3		
Steel of armature FeB44K or FeB38K	0.78 €/kg		
Electric welded net steel FeB44K or FeB38K	0.70 €/kg		
Drained tube in PVC of diameter of 200 mm	14.72 €/kg		
Stones for upset	42.05 €/m^3		
Stone for construction site	31.54 €/m^3		
Skilled worker	27.59 €/h		

***Table 5**: Const analysis of a wall of slit-check dam.*

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Kind of structure: **Side protection walls**

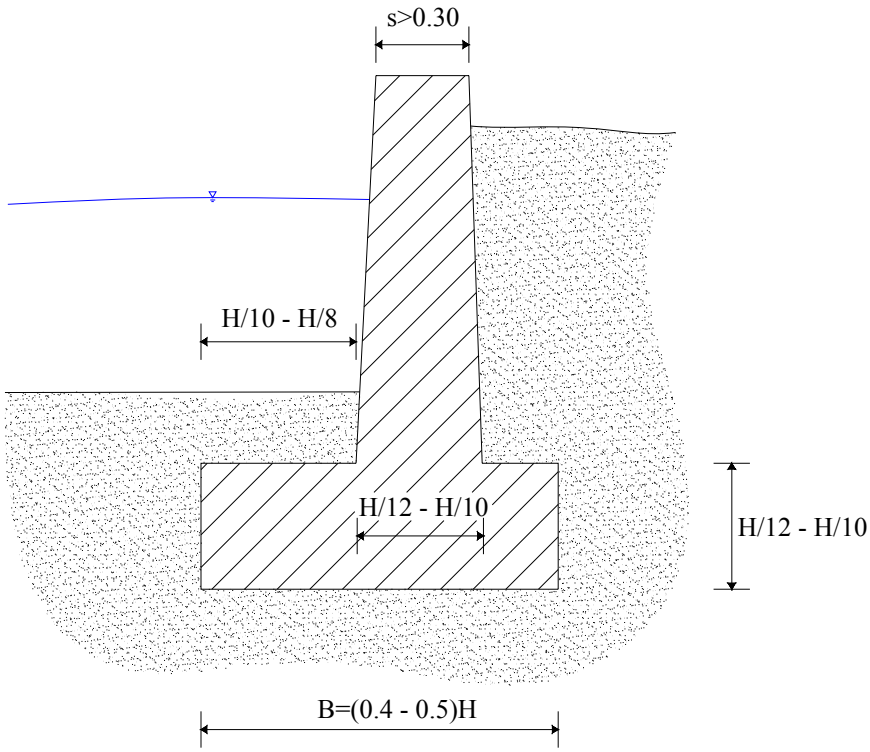

***Figure 24**: Structure of a side protection wall.*

Description of the countermeasure:	Side protection walls found the protection of banks about 2.00 – 2.50 m under the line of thalweg, and someway at least 1.00 m under the level of maximum excavation; they must be protected at the base by stones to reduce the excavation. Often the walls have some holes of drainage that reduce the driving force on the wall. Sometimes, to protect a settlement against floods, the elevation of existent walls may be necessary.
Purpose:	The works of defense of banks have principally the purpose to avoid the erosion of the versants. Side wall protections are longitudinal defenses and are executed fixing, with a wall, the line of slope that we want to maintain. Side wall protections are necessary in the places where the scarce availability of adjacent sites does not consent the use of versants with mild slopes.

Specific design criteria:	Suggested dimensions for a side protection wall are in the scheme above. For a side protection wall, all classical geotechnical verifications must be done, and those must be done in more exacting terms (e.g. torrent without water and soils behind the wall with a vein). One problem for the walls of bank is the depth of their foundation, especially in the curves but also in the straight length where we can find some bars. The foundation must be found at 1 m under the quota of maximum erosion (see example in Appendix).
Cost analysis:	The evaluation of the costs of a dam in reinforced concrete is based on the prices list of the own country.
References:	

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Costs analysis:	This evaluation of the costs of a dam in reinforced concrete is based on the prices list of the Provincia Autonoma di Trento.

VOICE	UNIT PRICE	QUANTITY EVALUETED	COST
Excavation with open section (with transport)	7.03 €/m^3		
Excavation with open section (with temporary deposit)	4.31 €/m^3		
Rent and transport of a tracked excavator with power above 110 kW and 155kW	82.49 €/h		
Rent and transport of a dump-truck	56.70 €/h		
Conferment in consented place of the material of the excavation evaluated useless	7.03 €/m^3		
Conglomerate Rck30 for foundation	105.10 €/m^3		
Conglomerate Rck30 per elevation	126.20 €/m^3		
Steel of armature FeB44K or FeB38K	0.78 €/kg		
Electric welded net steel FeB44K or FeB38K	0.70 €/kg		
Drained tube in pvc of diameter of 200 mm	14.72 €/kg		
Stones for upset	42.05 €/m^3		
Stone for construction site	31.54 €/m^3		
Skilled worker	27.59 €/h		

***Table 6**: Const analysis of a side protection wall.*

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References:	Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005.

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Kind of structure: **Protection and deflection walls**

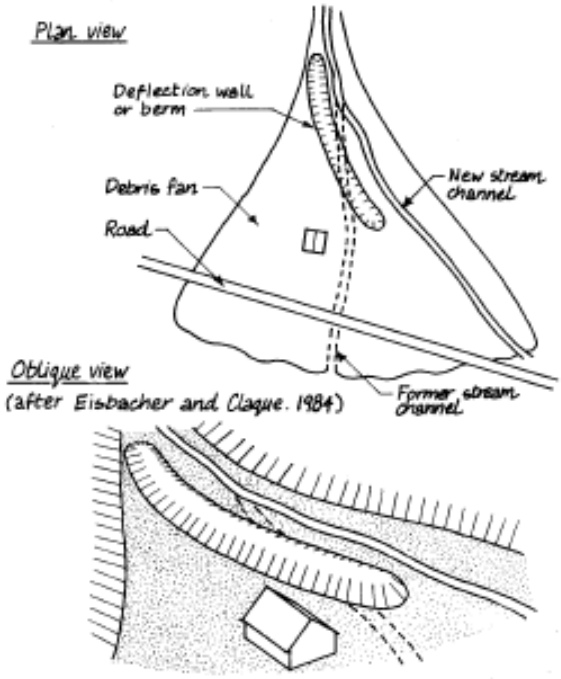

***Figure 25**: Plan and oblique view of deflection wall or berm.*

Description of the countermeasure:	Deflection walls are similar to lateral berms in that they are usually built immediately downslope from the apex of the debris fan, and parallel to the desired path of the debris flow whose lateral movement they are used to constrain. They differ from lateral walls or berms in that they deflect the flow path and prevent it from going straight. They can be used to protect a structure, deflect the flow to another area of the fan, or increase the length of the flow path, thereby decreasing the overall gradient and encouraging deposition . Walls are usually constructed of reinforced concrete; berms are usually constructed from local materials, but can be a composite. As for lateral walls or berms, the main design consideration for deflection structures is the maximum discharge and flow depth of the debris flow past the location of the structure. In addition, because of the curvature of the stream, potential impact forces, run-up and superelevation must be considered. To take these into account, the front face of the structure is designed for stability and with an appropriate slope and height. The freeboard heights discussed for lateral walls or berms can be used as well, but an additional height for superelevation is required. Some form of erosion protection or armoring must also be included in the design of these structures to minimize the addition of material from the structure to the debris flow mass. Where deposition is encouraged, the likely flow path of the fine-grained sediment, water from the debris flow, and subsequent water flows must be considered. If deposition does occur, the coarse-grained debris must be removed from the stream channel. These works can have problems about bend superelevation, dynamic

	impact of the debris flow in the normal direction to the wall and tangent stress transmit by debris flow in the parallel direction to the wall. The bend superelevation can be calculated with this formula: $$\Delta h = \alpha \frac{bu^2}{rg}$$ where b is the width of the channel and r the radius of the curvature and the coefficient α equal to 10. The impulse of the debris flow on the wall can be calculated by the following formula, which can be projected in the normal or parallel direction: $$S_d = \alpha \rho_{df} u^2 h^2$$ where the coefficient α is equal to 0.5, ρ_{df} is the relative density of the debris flow, u the velocity of the debris flow and h the depth of the debris flow. (photo gallery)
Purpose:	Deviate the debris flow to one side to protect some places.
References:	

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Photo gallery:	

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References:	Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005. VanDine D. F. 1996. Debris Flow Control Structures for Forest Engineering. Research Branch, British Columbia Ministry of Forests, Victoria, B. C., Working paper 08/1996.

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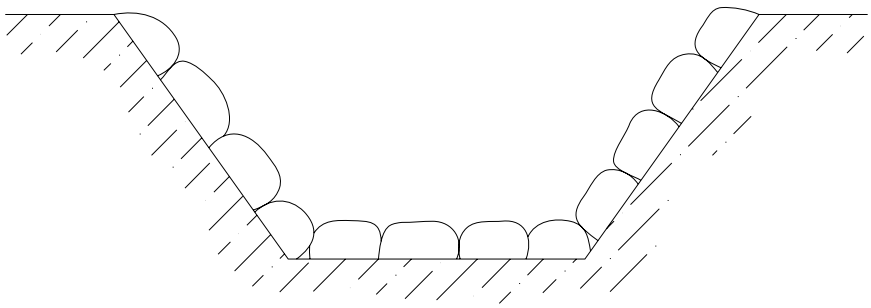
Kind of structure: **Transport channel**

	 ***Figure 26:*** *Transport channel.*
Description of the countermeasure:	Transport channels are generally installed on the alluvial fans of torrents, to ensure the correct transit of torrential floods across urbanized zones. They are channelized reaches, more or less consolidated to limit erosion of the banks or of the bottom. A crucial point is the size of their cross section, which will determine their hydraulic capacity (depending on the slope and nature of the flow). They are often built between sediment traps upstream and the confluence with the main river downstream. (photo gallery). In another section we examine an example of a transport channel design.
Purpose:	The main purpose of transport channel is to ensure the flowing of a given flow without flooding or divagations in the surrounding zones. A secondary purpose is to avoid excessive erosion in that specific reach, especially in the case they are built downstream of sediment trap that will catch the main part of the sediment load and deliver "clear" erosive water. They are «passive» countermeasures, in the sense that their purpose is not to reduce the flow but to limit the possible negative effect of the flows on exposed areas.
Specific design criteria:	The main aspect for their design is to warranty some hydraulic capacity corresponding to the nature of the flow and the desired level of protection. When they are built downstream of a sediment trap, they are mainly concerned by water flows, eventually transporting sediments by suspension and/or bedload transport. If there is no sediment trap upstream or if the capacity of this latter is limited, it is necessary to consider the eventuality of debris flow transit in the channel.

Structural design:	Basically, a "transport channel" is a channel offering a certain cross section to the liquid flow. Depending on the situation, this section can be obtained digging on the natural soil or by constructing lateral dykes. Lateral banks of the channel are often to be consolidated to resist erosion, by the use of rip-rap, gabions, and masonry or concrete walls. The protection against erosion is particularly necessary downstream of deposition basins. Indeed, in this case, the flow has a high erosion potential as it is free of transported sediments. Different techniques are employed to limit the effect of erosion on the river bed. The most frequent consists in building series of weirs regularly spaced that fix the global longitudinal bed profile. Erosion can however occur in the reaches between the weirs. Bottom erosion control can also be achieved covering the torrent bed bottom with rip-rap, masonry or concrete apron.
Materials:	Banks are generally made of natural soil, rip-rap, gabions, masonry or concrete walls. Bottoms are generally made of torrent bed natural soil and weirs, rip-rap, masonry, reinforced concrete.
Costs analysis:	The evaluation of the costs of a dam in reinforced concrete is based on the prices list of the own country.

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Hydraulics design criteria:	The calculation of the necessary cross section depends on the nature of the fluid: Classical hydraulics flow rules for lightly loaded, for example downstream a deposition basin. In that case, a special attention should be given to the value of the Froude number. Indeed, for Froude number values around 2 or higher, self-generated instabilities called "roll waves" occur, that increase drastically the maximum flow depth observed for a given liquid discharge. In consequence, the real hydraulic capacity is significantly reduced in comparison with theoretical hydraulic capacity deduced from classical steady flows equations. One will therefore avoid too smooth channels at steep slopes which can lead to high velocity flows with high Froude number values. For debris flows, design criteria are similar with the main difference that classical hydraulics formulas are not valid. One will then prefer specific expressions giving flow cross section and velocity versus discharge, slope angle and debris flow material specific parameters. These latter are always difficult to estimate and some calibration in reference to historical events, for example, is often necessary. Furthermore, debris flows are even more sensitive than clear water flows to the occurrence of roll-waves. Theoretical analyses have demonstrated that this phenomenon can develop with values of the Froude number as low as 0.5 in specific cases.

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Weir design criteria:	The design of weirs is similar to the design of check dams/sills.

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Kind of structure: **Example of a transport channel design**

Description of the countermeasure:	The transport channels are relatively small channels, designed to carry the residual flow discharge downstream of a check dam. (photo gallery). The side slope is often 1:1 to guarantee the stability of total heap of stones-soil, and the minimum width of the base to work smoothly is 80 cm. The banks of the transport channels are often covered by stones carefully posed. Since the slope of the torrent is elevated, it is often necessary to insert a series of sills to reduce the global slope. (example).
Purpose:	Transport channels are build downstream of retention works in order to canalize the torrent and to maximize the flow velocity of the debris flow, in order to avoid overflooding. In fact the smooth bed and the compact section of the transport channel enhances the process of transport. Transport channels are often built if the torrent flows through a settlement and when the available volumes for debris deposition is exiguous.

Specific design criteria:	The transport channel must resist high velocities and tangential stresses, hence, it must have a more stable bed. The stability of the bed can be obtained with the filling of the voids between the stones with concrete, but also adopting stones. In order to obtain a more friendly solution from the ecological point of view it is much better to use natural stones without any concrete filling. In this case the verification of stability of the bed of transport channel can be made with the incipient motion condition according to Shields criterion, taking into account of the effect of gravity: $$\theta_{cr} = \frac{u_*^2}{g\Delta d} = \theta_0 \cdot \left(\cos\alpha - \frac{\rho_s}{\rho_s - \rho} \frac{\sin\alpha}{\tan\varphi} \right)$$ where $\theta_0 = 0.04 \div 0.06$ is a suitable choice for the critical mobility parameter, α is the angle of longitudinal inclination of the transport channel, $\tan\varphi$ in the internal friction angle. If the stability results insufficient, the stones must be fixed with concrete. Curves along the torrent induce a superelevation of the water surface at the extrados, which makes it necessary to rise of the banks. This rise is calculated with regard to the kinetic energy of the debris flow: $$\Delta h = \alpha_{df} \frac{B}{r} \frac{U^2}{g}$$ where B and r are respectively the width of the channel and the radius of curvature. The coefficient α_{df} is equal to 2 (because Froude is greater than 1). Nevertheless some regional torrent control authorities recommend a coefficient $\alpha_{df} = 10$.
Costs analysis	The evaluation of the costs of a dam in reinforced concrete is based on the prices list of the Provincia Autonoma di Trento.
References:	

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Costs analysis:	This evaluation of the costs of a dam in reinforced concrete is based on the prices list of the Provincia Autonoma di Trento.

VOICE	UNIT PRICE	QUANTITY EVALUETED	COST
Excavation with open section (with transport)	7.03 €/m^3		
Excavation with open section (with temporary deposit)	4.31 €/m^3		
Rent and transport of a tracked excavator with power above 110 kW and 155kW	82.49 €/h		
Rent and transport of a dump-truck	56.70 €/h		
Conferment in consented place of the material of the excavation evaluated useless	7.03 €/m^3		
Stones for upset filter	42.05 €/m^3		
Stone for construction site	31.54 €/m^3		
Skilled worker	27.59 €/h		

***Table 7**: Const analysis of a transport channel.*

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References:	Armanini A. (2005). Lecture notes of the course: "S*istemazione dei bacini idrografici"*, Part III, Università degli Studi di Trento, 2005.

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Kind of structure: **Deposition basin**

Control structures:	There are two types of control structures: – Open debris flow control structures: • *Unconfined deposition areas*. • *Impediment to flow - baffles*. • *Deflection walls – berms*. • *Terminal walls – barriers*. – Closed debris flow control structures: • *Debris racks, slit dams*. • *Debris barriers and storage basin with debris - straining structures incorporated into the barrier*.

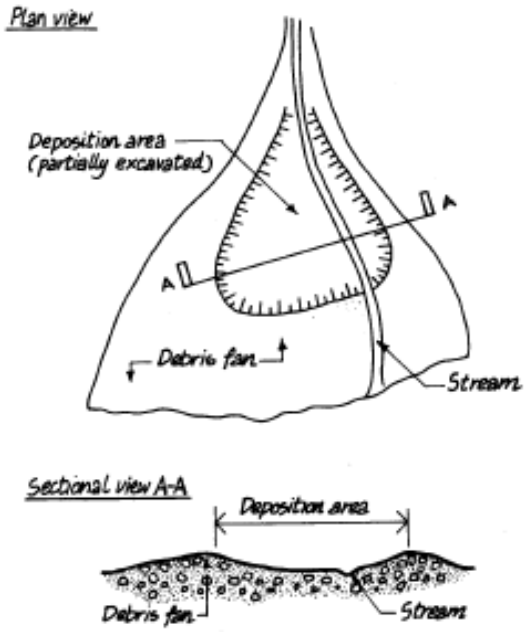
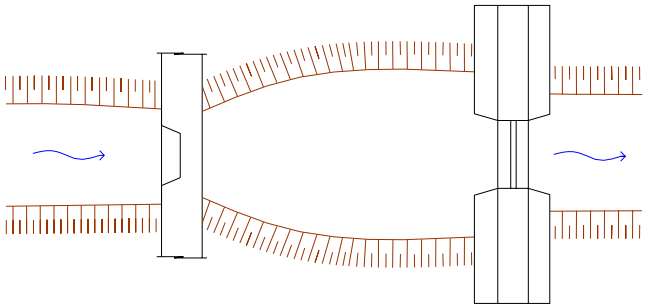
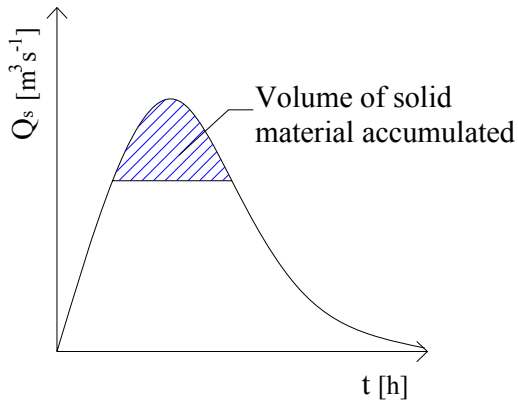


Figure 27: *Plan and oblique view of typical components of a debris barrier and storage basin.*

Description of the countermeasure:	Unconfined deposition areas, located on the debris fan, are designed and prepared to receive a portion or all of the debris from a channellized debris flow. To encourage the coarse-grained debris to deposit, the gradient of the fan is reduced or the debris is allowed to spread out and lose its confinement. It is possible to optimize available volume remembering that the angle of widening of a supercritical flow depends on Froude number of the flow upstream. (photo gallery).
Purpose:	If the deposition basin is along the torrent it has the purpose to reduce the solid discharge (Figure 28), while if it is at the end of the torrent it accumulates the transported solid material (Figure 33).
Design considerations:	Include the design magnitude or volume of the debris flow; the likely flow paths, including length to width ratio of the flow on the fan; the potential

	runout distance (see Rickenmann [2005] for details); and the probable storage angle. This method of debris control is best suited to larger debris fans that have relatively low gradients and few artificial structures. The geometry and morphology of the debris fan can be used to optimize the location of the area. An excavated, or partially excavated, deposition area can be prepared and shaped to further decrease the gradient and thereby decrease the potential runout distance and increase the potential storage volume. This form of control can be accompanied by some form of flow impediment within the deposition area or by a terminal berm or barrier at the downstream end (refer to "Impediments to Flow (Baffles)" and "Terminal Walls, Berms, or Barriers"). Some method of channeling the fine-grained sediment and water from the debris flow, and from subsequent stream flows downstream of the area, may be required. After a debris flow has occurred, the coarse-grained debris that has collected in the deposition area must be cleaned out in preparation for subsequent flows.
 ***Figure 28:*** *Deposition basin along the torrent.*	
Correct working of the deposition basin:	 ***Figure 29:*** *Q_s goes in the deposition basin.*

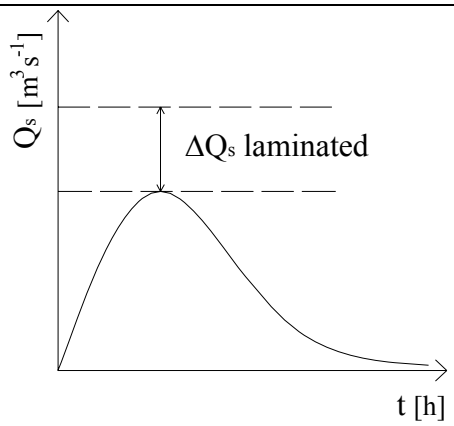


Figure 30: *Q_s goes out the deposition basin.*

Wrong working of the deposition basin:

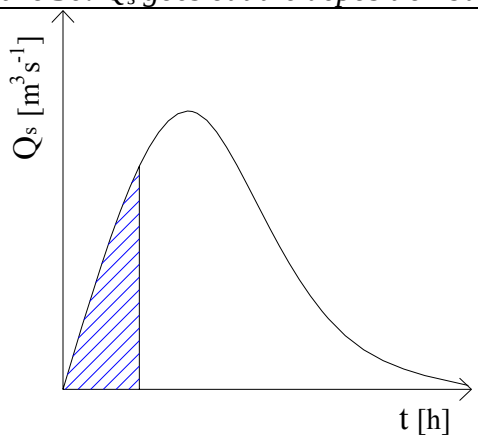


Figure 31: *Q_s goes in the deposition basin.*

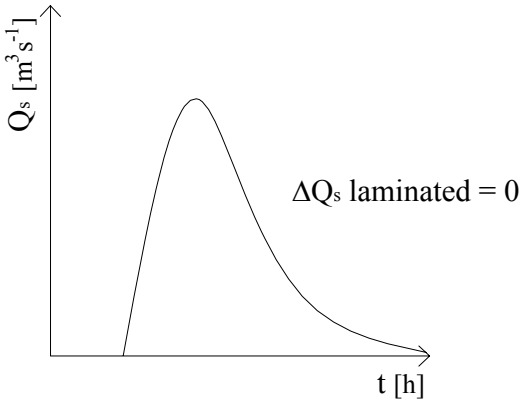


Figure 32: *Q_s goes out the deposition basin.*

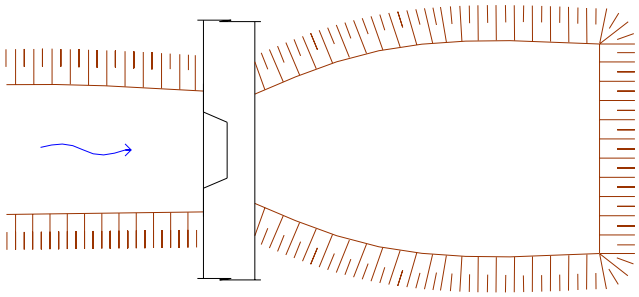


Figure 33: *Deposition basin at the end of the torrent.*

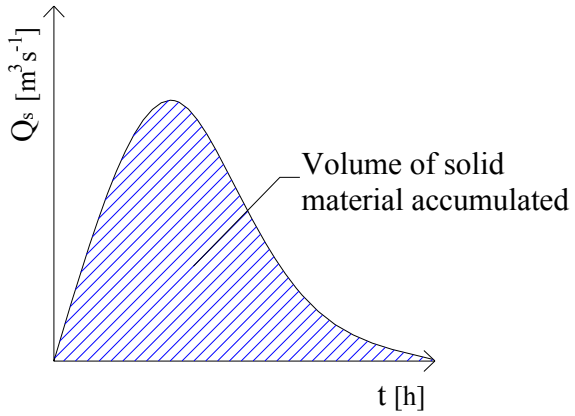


Figure 34: *Qs goes in the deposition basin.*

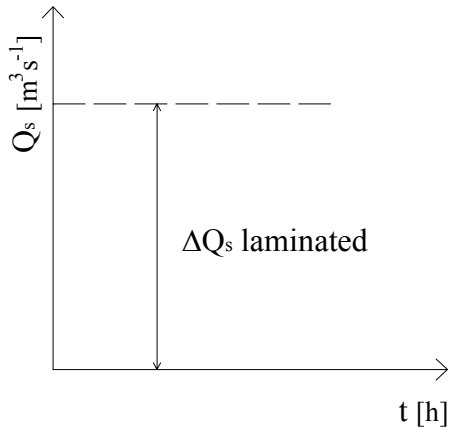


Figure 35: *Qs goes out the deposition basin.*

References:

Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005.

Rickenmann D. 2005. Runout prediction methods in Jakob M. & Hungr O. Debris-flow Hazards and Related Phenomena. SPRINGER-PRAXIS book in geophysical sciences.

VanDine D. F. 1996. Debris Flow Control Structures for Forest Engineering. Research Branch, British Columbia Ministry of Forests, Victoria, B. C., Working paper 08/1996.

Zollinger, F. 1985. Debris detention basins in the European Alps. *In* Proc. Int. Symp. Erosion, debris flow and disaster prevention, Tsukuba, Japan, pp. 433-438.

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Kind of structure: **Impediment to flow – Baffles**

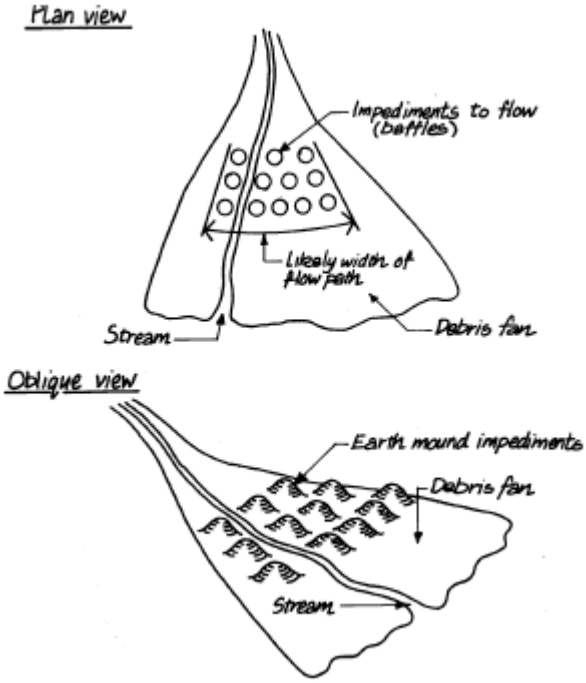

***Figure 36**: Plan and oblique view of impediments to flow - baffles.*

Description of the countermeasure:	Impediments to flow, or baffles, are used primarily to slow down a debris flow and thereby encourage it to deposit. In some instances they are used to deflect the flow. Impediments can be either natural or artificial. When trees are used, they have been referred to as "debris flow dispersing forest zones". Artificial impediments can be constructed of earth berms, timber, or steel, and function in much the same way as snow avalanche retarding structures. They can be placed as single units, in lines or staggered (Figure 36). Although they can be used by themselves, they are more commonly used in concert with other forms of control, often in unconfined deposition areas (described above). Impediments to flow should not be confused with debris-straining structures (discussed below).
Design considerations:	Include the design magnitude or volume of the debris flow, likely flow path, potential runout distance, impact forces, and run-up. Although they are often designed to be sacrificial, and replaced or rebuilt after use, they should be designed so that they do not add to the mass of the debris flow.

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Kind of structure: **Terminal Walls – Barriers**

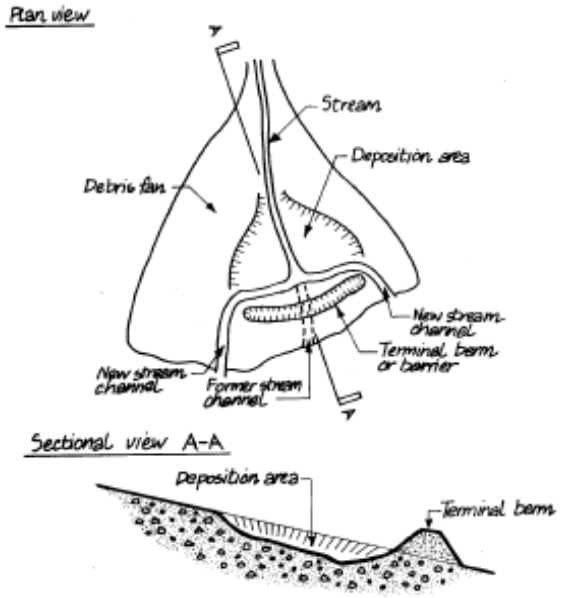

***Figure 37**: Plan and section of a terminal berm or barrier.*

Description of the countermeasure:	Terminal walls, berms, or barriers are constructed across the path of a debris flow to encourage deposition by presenting a physical obstruction to flow. They do this by increasing the length of the flow path. They are built with a finite length so that normal water flows and fine-grained sediment and water from the debris flow can find their way around either end of the berm (Figure 37). Once a debris flow has been deposited upstream of a terminal structure, the coarse-grained debris must be removed from the area.
Design considerations:	Design considerations include the design magnitude or volume of the debris flow, the likely flow paths, potential runout distance, impact forces, run-up, and probable storage angle. Terminal walls, berms, or barriers are usually located as far as possible downstream from the apex of the fan to maximize the runout distance and deposition area, and to minimize the impact forces and run-up. These structures are often built with a deposition area or partial deposition area upstream. The excavated deposition area artificially lowers the gradient, increases storage capacity, and decreases runout distances, impact forces, and run-up. Terminal structures have usually been constructed as massive gravity earth structures, so as to withstand the impact forces and the external forces of sliding and overturning. Impact forces and run-up can be reduced by decreasing the slope angle of the front face and by installing a sand cushion in front of the structure. For smaller magnitude debris flows, terminal walls, built as concrete walls, soldier pile walls, and soil and rock gravity walls, including gabions, can be used.

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Kind of structure: **Debris racks, slit dams**

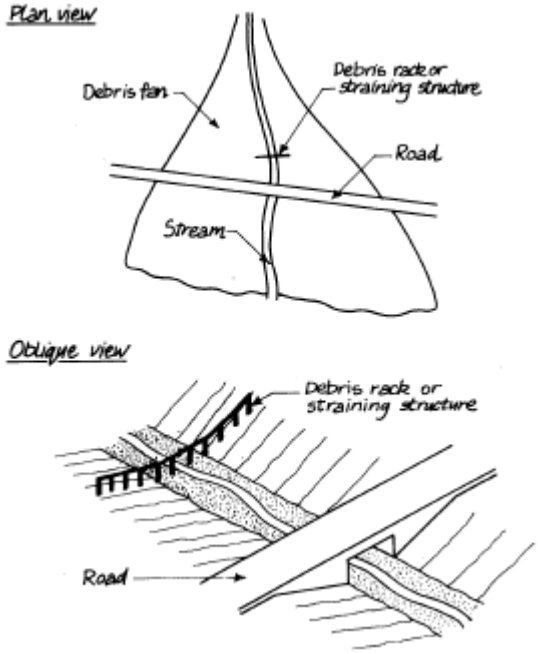

***Figure 38**: Plan and oblique view of a debris rack or straining structure.*

Description of the countermeasure:	Debris racks, slit dams, or other forms of debris-straining structures are used to separate the coarse-grained debris from the fine-grained debris and water of the debris flow, thus encouraging the coarse-grained portion to be deposited. Often used to prevent culvert openings and bridge clearances from becoming blocked with debris, debris racks are also often used as an integral component of debris barriers. To remain effective, the coarse grained debris must be removed from behind the straining structure on a regular basis. (photo gallery)
Design considerations:	Design considerations include the design magnitude or volume of the debris flow; the likely flow path so that the debris flow remains in the channel until it reaches the structure; the size and gradation of the debris; potential impact forces; and probable storage angle. When located within a stream channel (with its storage volume constraints), this system of control is limited to small volumes of debris. Debris-straining structures located within a channel must be designed to allow the normal water flows and stream bedload to pass at all times, and should redirect fine-grained sediment flows and water from the debris mass back into the channel after the coarse-grained debris has been stopped. For this purpose, a weir is often incorporated into the design of the straining structure. As a rule of thumb, Austrian practitioners design the slit interval at about 3 times the maximum diameter of the boulders

	Debris racks can be constructed of a wide variety of materials: – railroad rails; – structural steel sections, such as I-beams; – timbers; – pre-cast concrete beams; – cables; – culvert pipes; and – fencing materials. Zollinger (1985) found that when the structural members were placed vertically rather than horizontally, inorganic coarse-grained debris jammed more easily. Organic debris was found to jam more easily with structural members placed horizontally rather than vertically. Numerous examples of debris-straining structures are included in Hübl & Fiebiger (2005).

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Kind of structure: **Debris barriers and storage basin with debris - straining structures incorporated into the barrier**

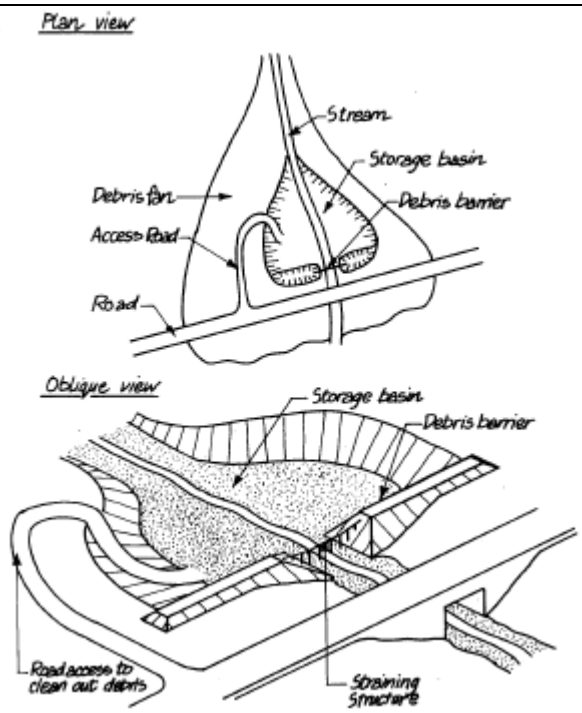

***Figure 39**: Plan and oblique view of typical components of a debris barrier and storage basin.*

Description of the countermeasure:	Debris barriers and storage basins, with some form of debris-straining structure incorporated into the barrier, are also referred to as retention basins. This system of debris flow control is similar to that achieved by a terminal berm or barrier, in that both are located across the debris flow path and designed to encourage deposition. Unlike terminal berms or barriers, however, debris barriers are designed as a closed barrier, or "dam" so that all the coarse-grained debris is contained within the storage basin located upslope of the barrier. The debris-straining structure must be designed so that during normal conditions, stream water and bedload can travel through the structure and, after a debris flow, the water that was in the flow and some of the fine-grained sediment can escape. As for a terminal berm or barrier, the area upstream of the debris barrier can be excavated to reduce the gradient and to increase storage capacity. Depending on the site, an inlet structure may be constructed upstream of the storage basin to minimize erosion of the streambed. Different types of impediments (stacks, teeth, racks…) can also be used inside the basin to optimize lateral spreading of the flow, deposition, and energy dissipation. After a debris flow has occurred, the coarse-grained debris trapped behind the debris barrier must be removed. This form of debris flow control is generally considered to be the most sophisticated and generally the most costly.

Design considerations:	Include design magnitude or volume of a debris flow, size and gradation of the coarse-grained debris (pertinent to designing the straining structure), potential runout distance (see Rickenmann [2005] for details), impact forces, run-up, and probable storage angle. Properly located, designed, and constructed, a debris barrier and storage basin, with an appropriate form of debris-straining structure incorporated into the barrier, is probably the most positive form of debris flow control. As well, this form of control structure is best suited to a larger debris fan with a relatively low gradient. The geometry and morphology of the debris fan can be used to optimize design and minimize construction costs. Most of the storage basins have been built as variations of earth berms or earth dams. Concrete and reinforced concrete gravity and arch-dam type structures have also been used. Unlike traditionally designed water-retaining structures, debris barriers are usually designed with the curve downstream to maximize the volume of debris storage. It is better to incorporate a weir or spillway into the structure to allow debris, fine-grained sediment, and water to safely overtop the structure should the storage basin be filled when a subsequent debris flow occurs. Other design considerations include external forces such as sliding, overturning uplift, and foundation and abutment loadings.
Shape of the retention basin:	– Angular shapes are not convenient because corner are not adapted to water and sediment flows; – extremely long or wide shapes do not optimize the sediment storage; – shape can be optimized for sediment retention or self-cleaning but it is difficult to achieve both objectives at the same time; – when self-cleaning is privileged, the pear-shape with the narrow extremity upstream will be preferred; – when sediment storage is privileged, the longitudinal slope of the retention basin has to be designed as low as possible, the deposition slope of debris flows being generally more difficult to assess and more gentle than the one for bed-load transport.
Volume of the retention basin:	It is not possible here to present all methods of volume assessment. Consequently, we give here only a few ones. Given the large uncertainty associated to each of them, it is better to combine the results of these approaches: – assessment of erosion rate; – assessment on the basis of the alluvial fan volume; – historical data on floods; – assessment of volumes of debris flow prone areas of the basin; – empirical formula based upon global geomorphic parameters.
Scouring of downstream channel:	Self-cleaning being very often limited, scouring very often takes place in the channel downstream the retention basin. It is therefore necessary to protect this channel by the use of check dams, lateral walls or armoring of the channel bed.
Sediment control:	Sediment control structures are used by themselves or in concert with

	debris control structures to control the movement of fine-grained material across a debris fan or alluvial fan, thereby minimizing the amount of fine-grained sediment entering a neighboring body of water. In general, their design can be divided into two types: – energy dissipation or settling basins; and – sediment control fences constructed of natural or artificial materials. The location of sediment control structures is very important. Located too close to the apex of the debris fan, they are subject to a large volume of coarse-grained debris and large impact forces. When located at the distal end of a debris fan or on the alluvial fan, they must face the possibility of an avulsion of the stream channel and flow path farther up the fan, which might result in the structure being bypassed. Because the emphasis of this study was on debris flow control structures, sediment control structures are not discussed here further.
Self cleaning:	To avoid erosion of the channel downstream of the retention basin, it is interesting to facilitate self-cleaning. This is possible when the dam closing the retention basin presents a slit down to the basin bottom and the downstream channel can receive the solid transport without damage. Self-cleaning requires the following conditions: – important water flow compared to the debris volume; – the deposit presents a large fraction of fine but non-cohesive material; – retention basin is narrow and with a steep slope; – absence (or removal) of woody debris blocking the exit. Self-cleaning is difficult to achieve. However, several examples show a self-cleaning efficiency of 30 to 60%. Most of the time, artificial cleaning is required and is costly.
References:	

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References:	Hübl J. and Fiebiger G. 2005. Debris-flow mitigation measures in Jakob M. & Hungr O. Debris-flow Hazards and Related Phenomena. SPRINGER-PRAXIS book in geophysical sciences. Rickenmann D. 2005. Runout prediction methods in Jakob M. & Hungr O. Debris-flow Hazards and Related Phenomena. SPRINGER-PRAXIS book in geophysical sciences. VanDine D. F. 1996. Debris Flow Control Structures for Forest Engineering. Research Branch, British Columbia Ministry of Forests, Victoria, B. C., Working paper 08/1996. Zollinger, F. 1985. Debris detention basins in the European Alps. In Proc. Int. Symp. Erosion, debris flow and disaster prevention, Tsukuba, Japan, pp. 433-438.

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Kind of structure: **Overpass and road tunnels**

Description of the countermeasure:	If the places to protect by debris flow are crossed by important way of communications like streets and railways, an overpass or a road channels can be built to guarantee a good protection for the infrastructure. (photo gallery)
References:	

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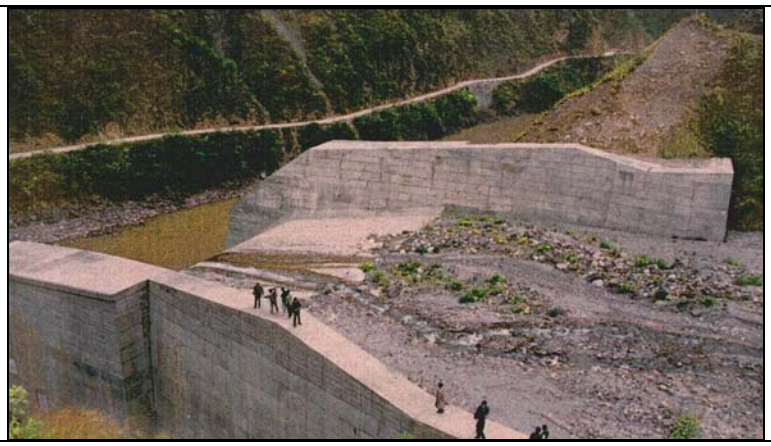
Photo gallery:	
	Tunnel to protect the railway Baoji-Chengdu (China) to the debris flow.
	
	Detail of the shoot over the same tunnel.

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References:	Armanini A. (2005). Lecture notes of the corse: "S*istemazione dei bacini idrografici"*, Part III, Università degli studi di Trento, 2005.

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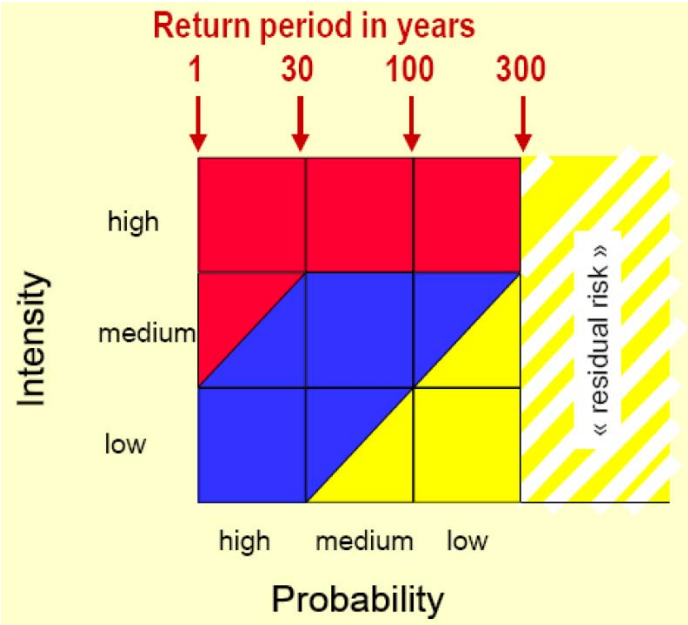
	2.2 Case Study Debris Flow
Explanation of the hazard and classification	In Italy every area is evaluated in relation to its hazard propensity. Given the previous natural disasters occurred, the territory is mapped and classified according to the values of certain parameters, e.g. velocity, height of deposit, return time. These maps are then taken into consideration by the urbanization offices who, given the various degree of intensity and probability, will regulate the construction permissions on the territory and consequently the monetary value of land and buildings. In the province of Trento (Italy), for debris flow, we can distinguish four categories of hazard level: red zone, characterized by a high level of danger, blue zone, with moderate level, yellow zone with low danger and white zone with negligible level of hazard.
Figure 40: Hazard rating system: red = high, blue = medium, yellow = low, white = no hazard. The hazard degree (three colors) is a function of the intensity and probability of an event	

Table 8:

Hazard classification in Trentino, with danger scale, relationship

Class	Danger level	Intensity-probability	Maximum expected damages	Urbanizing restrictions
Red	High (H4)	Extremely intense, independent from probability	Loss of human lives, destruction of buildings	Any anthropogenic construction is banned, infrastructures are allowed only if proper alarm and procedures are put in place
Blue	Medium (H3)	high probability of occurrence, medium-low intensity	Human settlement is banned and proper emergency procedures has to	Medium damage for people (especially outside of buildings)

between intensity and probability, maximum expected damage and urbanizing restriction.

			be implemented	
Yellow	Low (H2)	Low intensity events with medium probability	Small damages to buildings and infrastructures and functionality not stopped for long	Construction allowed under special verification of active and passive protection measures, especially for sensible buildings (e.g. school, church)
White	Neglegible (H1)	Irrelevant intensity and probability	Absent or irrelevant damages	None

Territory affected

Sauris, a little village on the northeastern Italian Alps, is affected by many natural hazards. In particular a campground is located in proximity to a debris flow site, where the zone has been classified as red. In the surroundings there are also commercial and touristic activities like a sport center, market and restaurant (*Figure 41*)

Figure 41: orthophoto of study zone. In yellow the main debris flow channel. On the right side of channel is visible campground and sport center (lower part of channel).

Hazard mapping and classification

Hazard maps can be based either on an inventory of past events or else using simulation models. The principal assumption of the first method (most common in Italy) is that future events will start and develop according to the same rules as past events. The second method calculates the susceptibility to hazard using specific hydrological and geotechnical models, on the basis on various territory information (e.g. geological characteristics, type of terrain, position of water table, slope, etc.) and land use maps.

In this case study the propensity to hazard has been studied using a model developed by the University of Trento. Four different scenarios have been taken into consideration, one without mitigation measure (present situation) and the others based on three different sizes of the countermeasure. The object is to obtain the most convenient size using the CBA approach.

In the first simulation (Figure 42) the presence of the mitigation measures is neglected, so the run out distance is calculated at its full extent. This result represents the probable maximum loss, i.e. the area where the largest potentially assumable loss is expected.

For the hazard classification, three methods are used:

Table 9: classification's parameters according to BUWAL (Rickenmann)

	Water depth (m)	Velocity (m/s)	Scour S (m)	Deposit D (m)
3	-	v>1.5	-	morph>1
2	-	v>0.4	-	morph>0.4
1	-	v>0	-	morph>0

Table 10: classification's parameters according to parameters of Trentino province

	Water depth (m)	Velocity (m/s)	Scour S (m)	Deposit D (m)
3	h>1	v>1	S <-2	D >1
2	h>0.5	v>0.5	S <-0.5	D >0.5
1	h>0	v>0	S <0	D >0

Table 11: classification's parameters according to Trentino province without taking into account the morphology

	Water depth (m)	Velocity (m/s)	Scour S (m)	Deposit D (m)
3	h>1	v>1	-	-
2	h>0.5	v>0.5	-	-
1	h>0	v>0	-	-

The classification criteria used in the case study is the second, as the most precautionary.

As can be seen in the map, the simulation creates small very irregular spots and this creates some difficulties for the final classification. In order to have a more homogeneous map, the resolution of the grid was increased, determining a mediation of the parameters.

The second figure is a comparison between simulation and the PAI classification.

In the third images the resolution is 10x 10 meters with an average value of each parameter for each cell.

Figure 42: hazard map without countermeasures and with a resolution grid of 2x2 meters

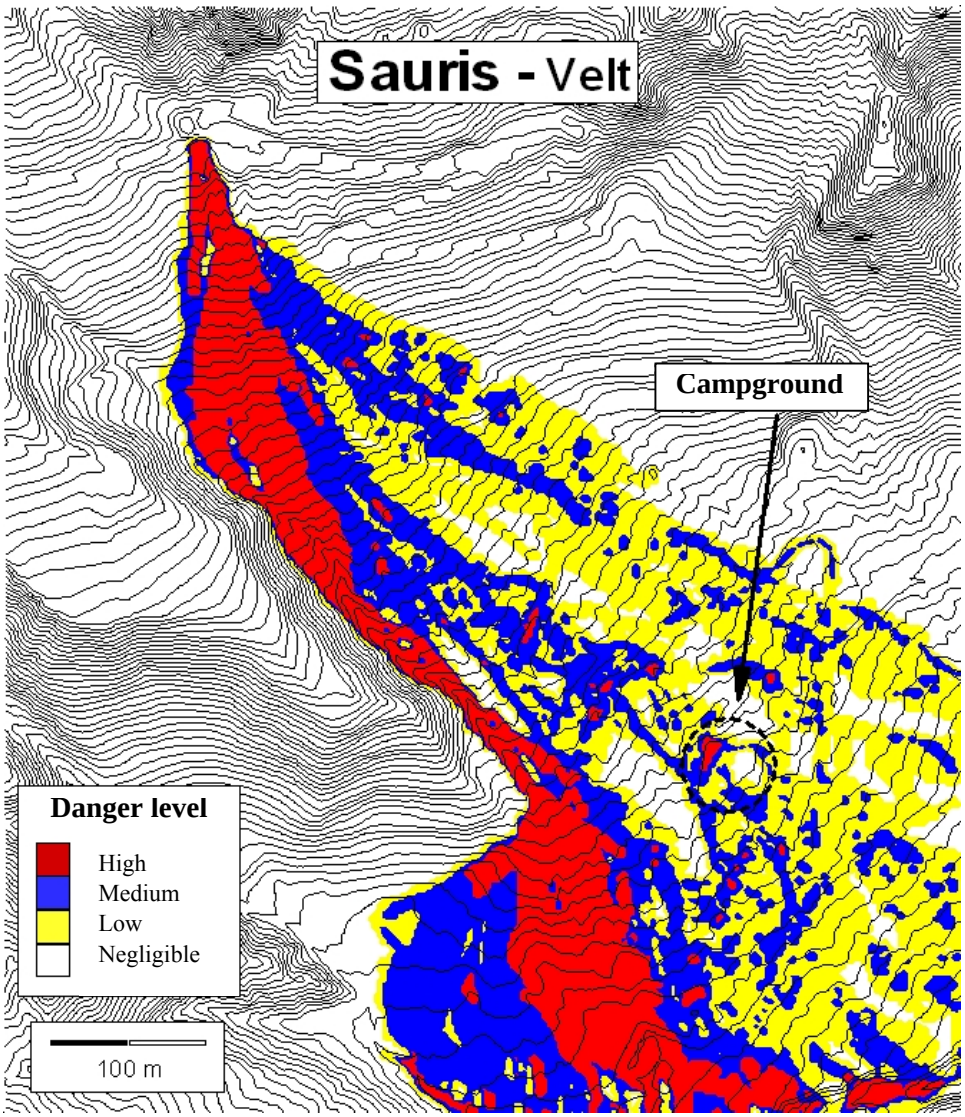
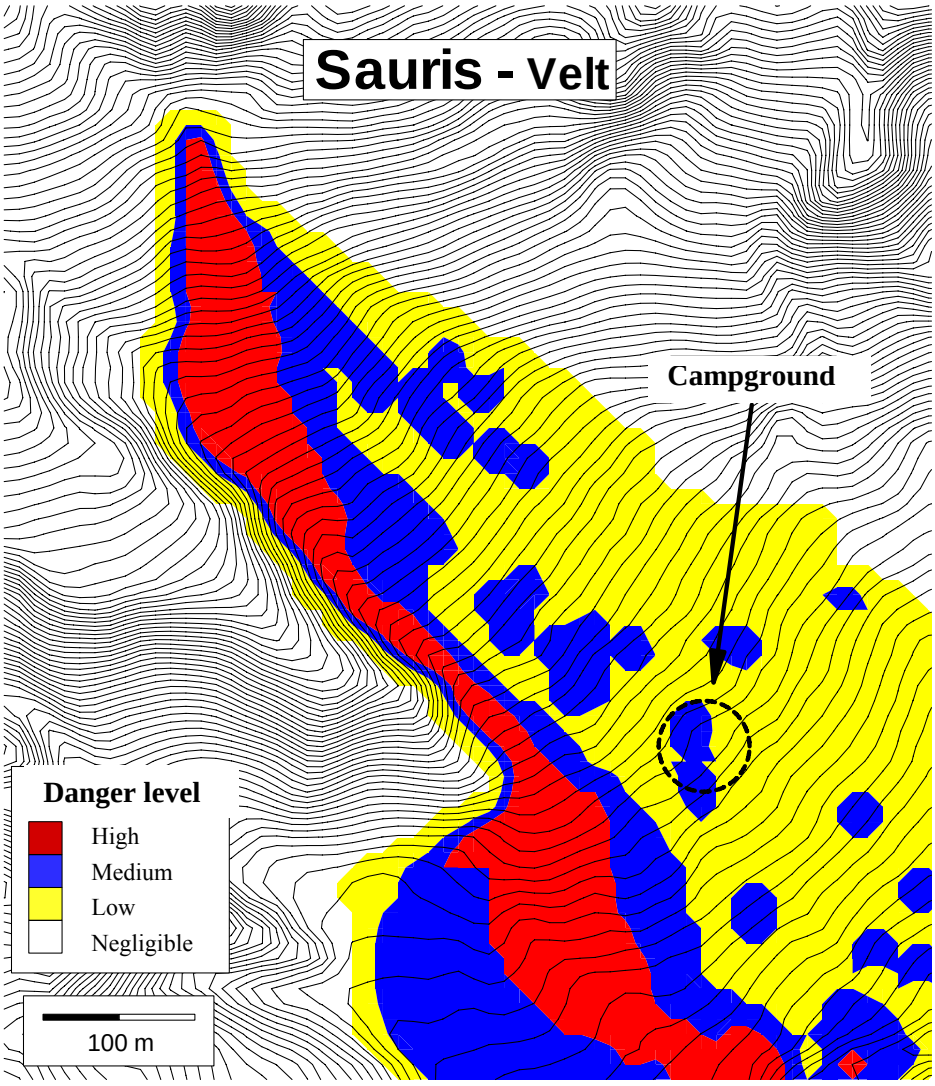
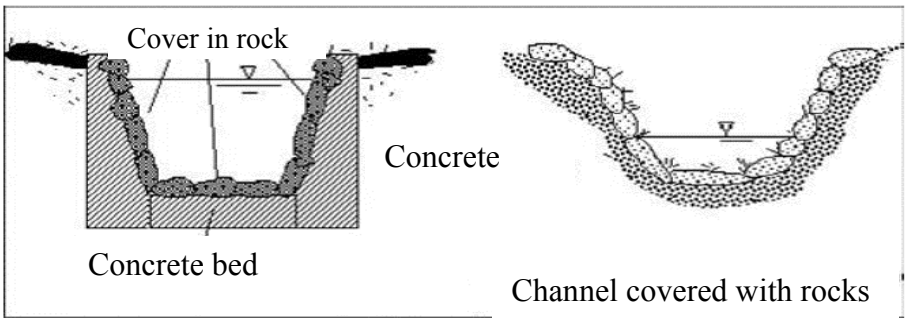


Figure 43: hazard map without countermeasures and with a resolution grid of 10x10 meters	
Description of the countermeasures	As a countermeasure, in the case study we have chosen the realization of a channel covered with rocks (see following figure), in order to ease the flowing of the solid transport. As the tangential stress at the bottoms is very high, a stability analysis has to be accomplished, to evaluate if concrete has to be used to retain the single rocks.
Figure 44: scheme of mitigation measure	

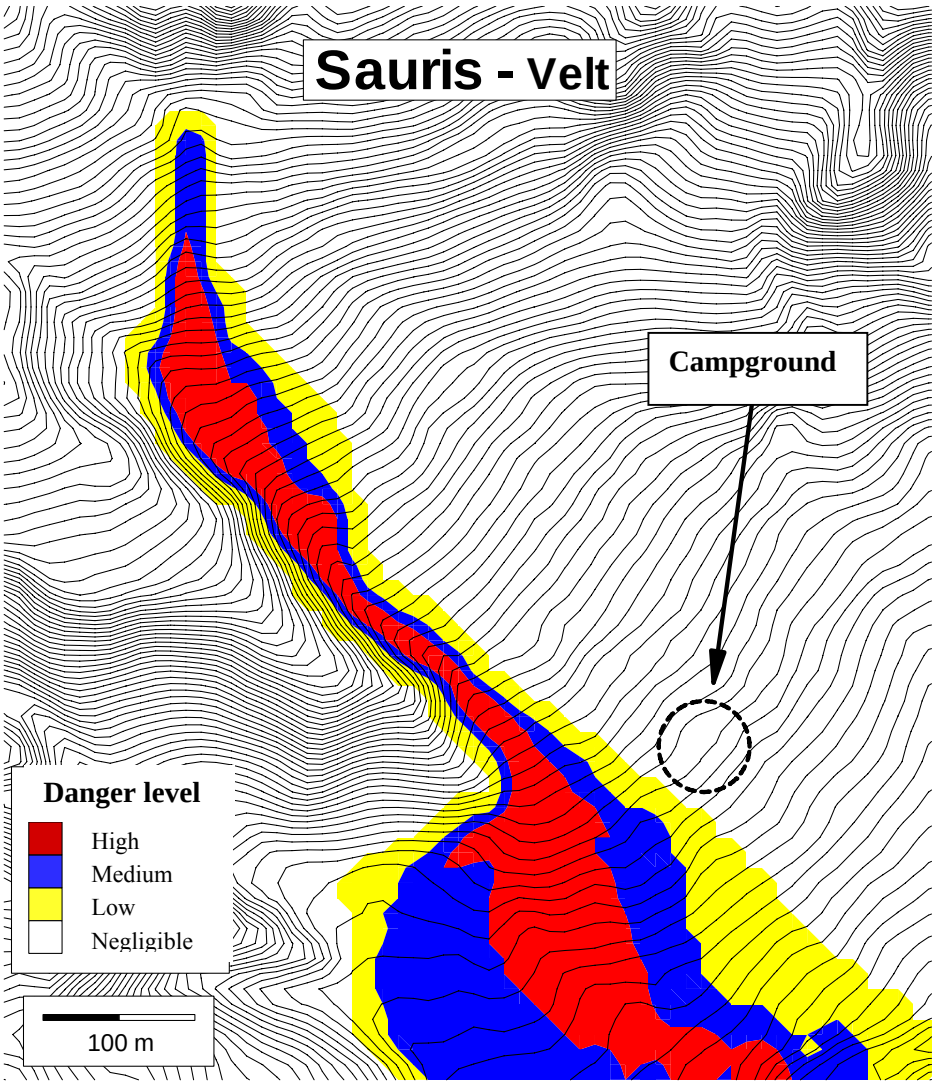
New hazard mapping and classification	In order to evaluate the efficiency of the mitigation measure, a new simulation with the new hydraulic conditions (fixed bed) was accomplished. This assumption decreases the erosion power of the debris flow, resulting in a diminished affected area. A great part of the blue and yellow zone turned in yellow and furthermore the campground fell out of the affected area.
Figure 45: hazard map with mitigation measures (length 80 m)	

Figure 46: hazard map with mitigation measures (length 170 m)

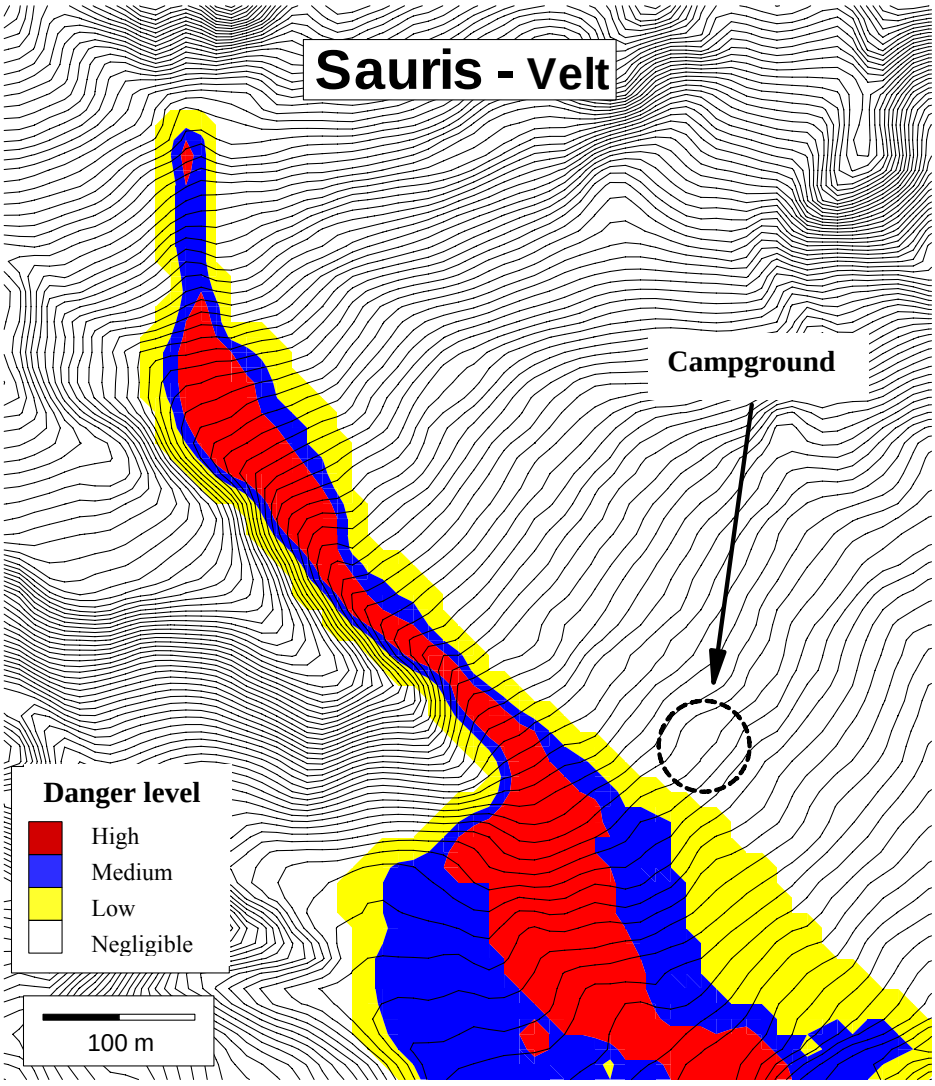
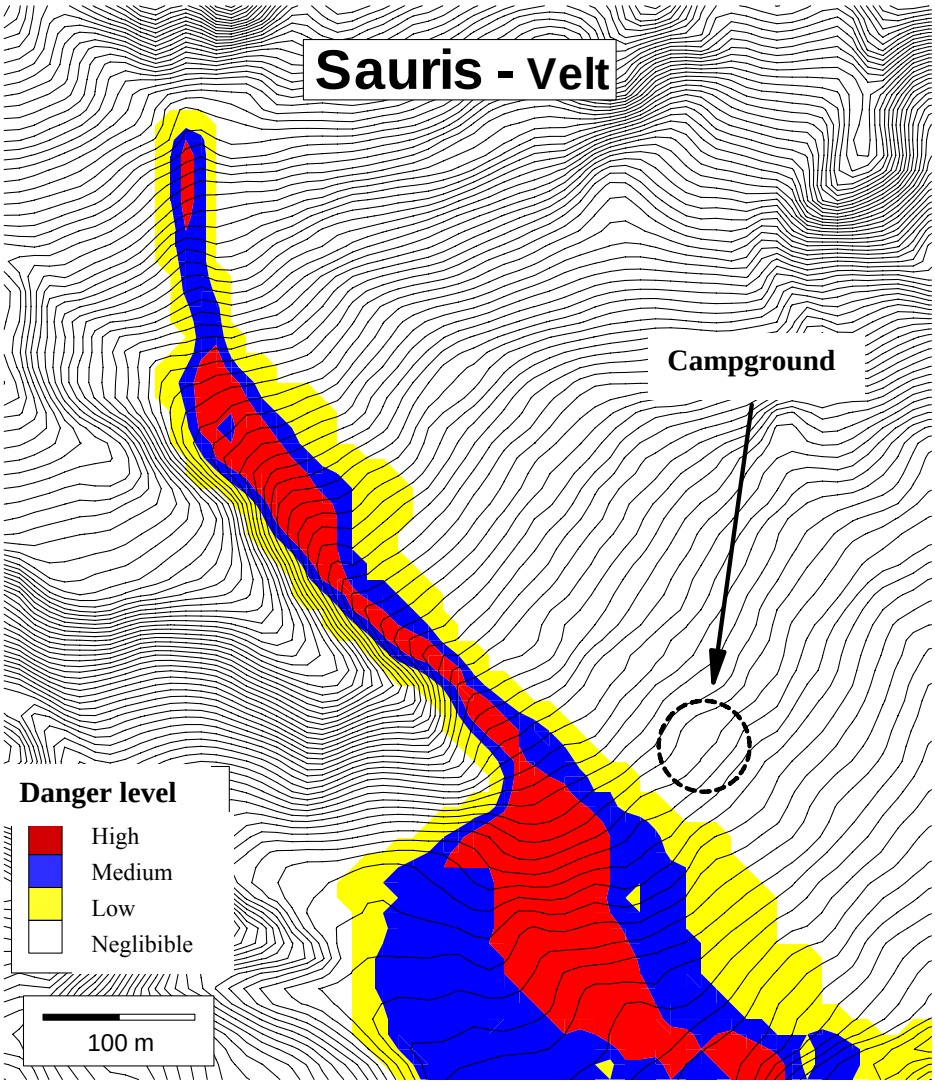


Figure 47: hazard map with mitigation measures (length 260 m)



Costs and benefits involved

Three simulations at different channel length (80m, 170m and 260m) were made, in order to calculate the optimal length, i.e. the length that maximizes the cost/benefit ratio.

The costs considered are the cost of construction of the countermeasure. According to the prices given by the Province of Trento (Italy), a simple relation cost-length has been retrieved (see table).

Table 12: realization costs of mitigation measure (channel rock

Cost of countermeasure (for meter)	822 €/m
Company gain (10%)	82 €/m
Security (5%)	41 €/m
vat (20%)	164 €/m
TOTAL (for meter)	**1,110 €/m**

covered)

solution A (80 m)	**88,776 €**
solution B (170 m)	**188,649 €**
solution C (260 m)	**288,522 €**

The benefit induced by the countermeasure could be summarized as follows:

- Increasing value of land: the market value of land decreases with the increase of the hazard level of the zone. In fact a land in white or yellow zone maintains its value unchanged, whereas in the blue and red zone the reduction is significant. Literature values in Italy report a 20% reduction coefficient in blue zone and 100% reduction in the red zone. With the countermeasure it is expected to have less area in red and blue zone and more in yellow and white, with a consequent monetary benefit.
- Increasing value of buildings: also buildings decrease their value if they are located in a high hazard area. Also in this case it is possible to find in the literature proper reduction coefficient.
- Avoided damage to buildings: for buildings replacement value is considered, hence, for the benefits the costs of reconstruction have to be considered.
- Avoided fatalities;
- Avoided damage to traffic: in the case study, the main flow affects some mountain roads. With the countermeasures great part of these will not be damaged, resulting in a benefit;
- Suspension of commercial activity: the accumulation of sediments on the road obstructs the access to the campground and other activities in the zone and thus a suspension of the activity.

In this case study, we consider only the benefits deriving from the first point, the increasing value of land. The other points are neglected as they require a more complex field analysis. Furthermore, as the benefit is immediate, a short period analysis was done and so the interest rate was not taken into consideration.

As can be seen in the classification maps, we can notice a substantial increase of land value, as some areas in the red and blue zone become white zones, outside the danger.

In absence of specific data on the land value in Sauris, we considered the

land price and corresponding reduction based on the hazard category, as was performed in the town of Cogne and Rovereto in Italy.

Table 13: land value without mitigation measures (simulation)

		without mitigation measures				
land		white	yellow	blue	red	
	area	277900 m^2	108000 m^2	34300 m^2	26200 m^2	
	value	38.75 €/ m^2	38.75 €/m^2	38.75 €/m^2	38.75 €/ m^2	
	value reduction	0%	0%	40%	90%	
	effective value	38.75 €/m^2	38.75 €/m^2	23.25 €/m^2	3.88 €/m^2	TOT
	total value	10768625 €	4185000 €	797475 €	101525 €	**15852625 €**

Table 14: land value with mitigation measures (solution A)

		length A (80 m)				
land		white	yellow	blue	red	
	area	372800 m^2	29300 m^2	23300 m^2	21000 m^2	
	value	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	
	value reduction	0%	0%	40%	90%	
	effective value	38.75 €/m^2	38.75 €/m^2	23.25 €/m^2	3.88 €/m^2	TOT
	total value	14446000 €	1135375 €	541725 €	81375 €	**16204475 €**

cost	88,800 €
benefit (without)	351,850 €

B/C (without)	3.96

Table 15: land value with mitigation measures (solution B)

		length B (170 m)				
land		white	yellow	blue	red	
	area	371600 m^2	29300 m^2	23700 m^2	21800 m^2	
	value	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	
	value reduction	0%	0%	40%	90%	
	effective value	38.75 €/m^2	38.75 €/m^2	23.25 €/m^2	3.88 €/m^2	TOT
	total value	14399500 €	1135375 €	551025 €	84475 €	**16170375 €**

cost	188,700 €
benefit (without)	317,750 €

B/C (without)	1.68

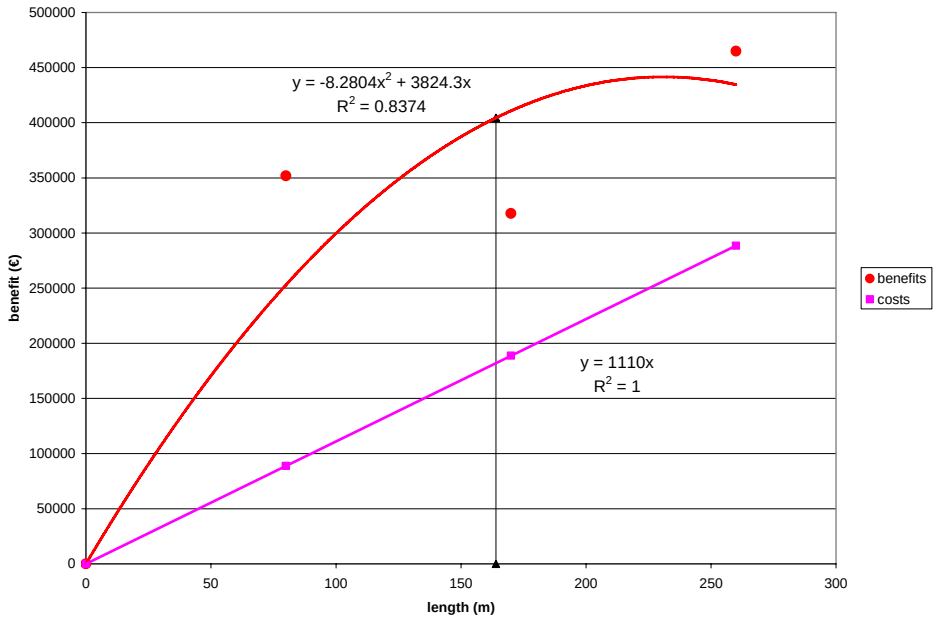
Table 16: land value with mitigation measures (solution C)

		length C (260 m)				
land		white	yellow	blue	red	
	area	377600 m^2	28300 m^2	22300 m^2	18200 m^2	
	value	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	38.75 €/m^2	
	value reduction	0%	0%	40%	90%	
	effective value	38.75 €/m^2	38.75 €/m^2	23.25 €/m^2	3.88 €/m^2	TOT
	total value	14632000 €	1096625 €	518475 €	70525 €	**16317625 €**

cost	288,600 €
benefit (without)	465,000 €

B/C (without)	1.61

Figure 48: comparison between benefit curve (in red) and cost curve (in violet). The red points are the experimental data



The optimal length of the structure

According to the marginal cost theory, the optimal length coincides with the point that maximizes the difference between the two curves, i.e. where the derivate of the benefit curve equals the derivate of the cost curve. In this case the optimum is 160m.

Applying the incremental C/B criterion, we can verify the optimal solution. In this case we have four possible scenarios (no countermeasures and three lengths for the channel). According to Table 17, A-0 is the incremental condition passing from the scenario 0 without countermeasure to the scenario A, where the corresponding incremental

costs and benefits are highlighted. When the B/C ratio is below 1, passing to the new alternative is not appropriate. In this case, the optimal solution is B.

Table 17: the incremental C/B analysis.

0 without countermeasure,

A, B, C incremental length of the countermeasure

Scenario	Incremental cost	Incremental benefit	B/C
A-0	88,800 €	252,949 €	2.8485297
B-A	99,900 €	157,878 €	1.5803604
C-B	99,900 €	23,736 €	0.2375928

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Chapter 3

ROCK AVALANCHES

3.1 Countermeasures against Rock Avalanches

Introduction

As the experience with rock avalanche countermeasures is sparse it is difficult to give reliable information about cost and efficiency of rock avalanche countermeasures. Additionally, general cost estimates are hard to give as there are great differences in the extension of the unstable rock masses and the accessibility will vary from one place to another.

To meet these constraints, a description of the main cost elements for the ongoing work of the monitoring system at Åknes in Norway will serve as an example. Constraints in the reliability of this monitoring system as a basis for early warning will also be described.

The unstable rock mass at Åknes covers an area of about 1.5 km^2, the depth to the gliding surface is 40-90 m, and the vertical height difference is 800 m. The estimated sliding volume is 30-90 millions m^3.

3.1.1 Physical countermeasures

As there are very few practical examples of physical measures and the cost for the measures will greatly differ depending on volume and accessibility to the unstable rock mass, it is impossible to give general cost estimates for this type of measures. Anyway, undoubtedly the cost will typically be very high, in the order of several millions euros for a case of the same size as Åkneset.

	The efficiency of the measure is also hard to evaluate. If the greater part of the unstable rock mass is successfully removed, the efficiency is of course good. However, drainage measures will not be efficient if water pressure is not the governing factor controlling the stability of the rock mass. One should therefore perform detailed sub-surface surveys before drainage measures are implemented to make sure that the role of water pressure is of vital importance.
	3.1.2 Costs of monitoring systems and early warning
	The main cost elements in the monitoring system at Åknes are: • Basic maps and surveys (topographical and geological maps, boreholes, geophysical investigations) • Planning of what kind of instruments to include in the monitoring system • Purchasing costs • Installation costs • Cost for power supply • Data acquisition system • Data transfer system to the early warning unit • Maintenance costs In addition costs related to the operational phase of the early warning must be taken into account. The costs related to planning, investigations, establishment, maintenance and operational early-warning will highly depend on the site-specific conditions and the total risk. Especially important is the complexity of the slide conditions and the practical challenges related to power and data transfer. Experience from operational early-warning systems in Italy, Switzerland, Canada and Norway may indicate that the costs will be in the following order: <u>Planning and investigations.</u> This will include all geological and geotechnical investigations of the individual sites (e.g. field investigations, drilling, periodically monitoring, stability analysis). Total costs will probably be in the order of 1-5 million Euro. <u>Establishment of monitoring systems.</u> This will include all costs related to installation of different monitoring systems, power supply and data transfer. Total costs will probably be in the order of 1-5 million Euro. <u>Establishment of a monitoring centre.</u> This will include costs related to establish a monitoring centre with integrated monitoring systems,

servers/databases and all issues related to warning and evacuation procedures (planning, physical warning systems). Total costs is estimated to be in the order of 1-5 million Euro.

Maintenance and operational early-warning. This includes all issues related to maintenance and operational work, including personnel costs. The early-warning centre in Lombardia in Italy has 15 employees, and the total yearly budget is in the order of 2-3 million Euro. The total annual costs for an early-warning centre is estimated to be in the range of 1-3 million Euros. It has to be underlined that this can include the monitoring of several risk sites.

3.1.3 Reliability of monitoring systems

In order to optimize the reliability of the monitoring systems there are several important aspects that is needed to be taken into account:

- The need for several different monitoring systems in order to cope with problematic situations where one or several system may be out of order
- Large changes in weather and atmospheric conditions may occur that may affect the efficiency of the instrumentation
- Snow-avalanche and rock-fall hazard in the monitored slope may cause damage the instrumentation
- Optimal and reliable system for power and data transfer

The experience from some of the large international monitoring projects may indicate that the different systems can be grouped according to their importance and reliability in operational early-warning systems, which is often site-specific:

Primary sensors: Reliable and robust

- Surface crack meters/extensometers
- Surface tilt meters
- Single lasers (needs caution during bad weather)
- Borehole inclinometers
- Borehole extensometers

Secondary: Not yet reliable for full operational monitoring

- Laser Ranging (EDM),
- GPS
- Ground-based radar
- Microseismic sensors

Tertiary: Support/Information Sensors

- Meteorological station
- Piezometers

	• Weir A fully operational early-warning systems (monitoring center) needs to include the following important aspects: • Reliable monitoring network, including stable power supply and data transfer • The monitoring includes the critical factors controlling stability • Effective warning and preparedness management, including suggested evacuation routes, information dissemination and implemented warning systems.

Chapter 4

SNOW AVALANCHES

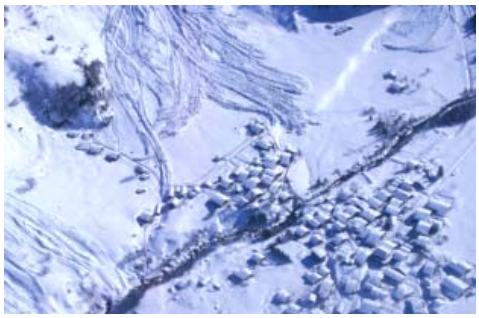
4.1 Countermeasures against Snow Avalanches

Introduction

For several centuries, countermeasures against snow avalanches have been implemented to protect settlements, roads, and infrastructures in all mountainous regions of Europe. In this chapter, we aim at giving a brief survey of the countermeasures most commonly used, the costs they incur, and the risk reduction impact they provide.

Here, we focus on structural countermeasures, notwithstanding that organizational mitigation measures such as road-closing polices or evacuation policies have gained in importance in avalanche risk management over the last twenty years (Wilhelm 1999; Bründl et al. 2004; Rheinberger et al., submitted). However, these organizational measures do largely depend on the human decision making, which is cannot be grasped in a sufficient manner by a simple efficiency measure such as a cost-effectiveness criterion (Rheinberger & Rhyner, in preparation).

Describing structural countermeasures against snow avalanches is best done by distinguishing the measures based on their position within the avalanche path. We therefore divide the avalanche path into three zones (Fig. 4.1): (1) the starting zone where the avalanche is released; (2) the transition zone within which the avalanche accelerates; and (3) the run-out zone where the avalanche typically gush out in a finger-formed shape. Depending on these positions, the mitigation measures to be applied have

	different modes of operation, which will be described below.	
Figure 4.1 (Photos © SLF)		Starting zone: avalanches are release due to weak or instabil snowpack. Countermeasures in this zone are aimed at avoiding or reducing the probability of releases. Thus, they are not designed to bear large dynamic forces.
		Transition zone: avalanches take up speed and mass. Countermeasures have a limited impact on the avalanche energy. Measures are aimed at either redirecting or decelerating and damping avalanche energy by catching a part of their mass.
		Run-out zone: avalanches gush out often endangering settlements, roads, or infrastructure. Countermeasures in this zone are constructed to reduce impact forces on values at risk, such as buildings, traffic, or people situated in the avalanche run-out.
	4.1.1 Short description of countermeasures commonly used	
Measures in the starting zone	Within the starting zone, avalanches are released. Thus, structural countermeasures in this zone aim at averting releases or at least reducing the probability of them. Consequently, they are not constructed for bearing large dynamic forces but for sustaining the static pressure of snow. There are different kinds of structures in use (Fig. 4.2), including: **Permanent supporting structures**: these measures are either rigid sustaining only small elastic deformation (snow rakes with upright crossbeams or snow bridges with horizontal crossbeams) or rocking being capable to support a limited deflection (snow nets). They are built for supporting and ponding snow in the starting zone. Thus, their height depends on the expected snow heights at the specific site. **Snowdrift regulations**: these measures are constructed to inhibit cornice formation and wind drifting, which causes accumulations of snow in the	

	starting zone. One differentiates wind baffles (a trapezoidal board facing the prevailing wind direction), jet roofs (a wing-like structure constructed directly below the ridgelines to prevent the formation of cornices and accumulation of snow), and snow fences (a perpendicular structure built on ridges to accumulate drifting snow).	
Figure 4.2 (Photos © SLF & CEMAGREF)		Snow rakes are the traditional supporting measures, which are built to stabilize and pond snowpack in steep terrain liable to avalanche releases.
		Snow nets are rocking structures that support and pond snow in the starting zone reducing avalanche releases.
		Wind baffles are trapezoidal boards facing the prevailing wind direction. They hinder the accumulation of large snow masses reducing the probability of weak snowpack.
Measures in the transition zone	Within the transition zone, avalanches take up speed and mass. In this area, countermeasures have a limited impact on the avalanche energy. Measures are aimed at either redirecting or decelerating and damping avalanche energy by catching a part of their mass. Most effectively, they are implemented in the lower part of the transition zone or in the upper part of the run-out zone. The following structures are used to this end are (Fig. 4.3): **Avalanche breakers**: these are earthwork or concrete wedges heaped up to detach the avalanche within the flow process. By being divided into several flow arms, the avalanche loses mass energy.	

	Catching and retention dams: these are dams to catch and retain at least parts of the avalanche mass reducing thereby mass-related energy. **Deflection dams**: these structures are built above settlements or single buildings in order to redirect the flow of avalanches into run-out arms that do not pose a threat.
Figure 4.3 (Photos © SLF)	 Avalanche breakers are wedges built in the lower part of the transition zone or in the upper part of the run-out zone to divide an avalanche into several flow arms lowering thereby the avalanche energy.
	 Deflection dams are built to protect settlements by redirecting the avalanche flow direction.
	 Catching dams are built to retain at least a part of the avalanche mass reducing thereby the avalanche energy.
Measures in the run-out zone	In the run-out zone, avalanches gush out often endangering settlements, roads, and infrastructure. Countermeasures in this zone are constructed to reduce impact forces on values at risk, such as buildings, traffic, or people situated in the avalanche run-out. As these values at risk show varying degrees of vulnerability (people being hit outdoor are mostly killed, while the vulnerability of buildings to small avalanches is comparably low), different measures are used to protect them (Fig. 4.4): **Avalanche galleries or tunnels**: these measures are covering structures designed to protect roads and railways against both slab and powder avalanches. Avalanches float over or are deposited on the roof of the gallery or tunnel. To withstand loading forces, the gallery has a concrete

roof, with strong foundations, that must be designed to sustain the hydraulic forces of the avalanches. When appropriately designed to larger avalanche forces, galleries can also be implemented in the transition zone.

Reinforcements of buildings: these measures are used to protect single buildings. Among the feasible reinforcement measures, there are: reinforced walls, deposition of earth walls behind the building (a special form of this measure is the so-called "Ebenhöch", which is a building whose roof is smoothly trued into the uphill terrain), reinforcement and adapted design of roofs, and protection of vents.

Figure 4.4

(Photos © SLF)

Avalanche galleries are constructions, which cover roads to protect them against both powder and slab avalanches.

Reinforcing buildings becomes necessary, if a building stands in the run-out zone. Depdnging on the position toward the slope, concrete reinforcements or earth work is used to minimize the damage potential.

4.1.2 Cost analysis

The different countermeasures incur different costs for implementation, operation, maintenance, and repair. In the following Table 4.1, these costs are given for the measures discussed above. The costs are quoted as benchmarks from Switzerland (Wilhelm, 1999), but have to be adapted to site-specific conditions and regional prices. Based on these benchmark values and with the help of the annuity cost method, one can calculate the annual costs *AC* for a specific mitigation measure by:

$$AC = (IC + OC + MC + RC) \times \frac{i(1+i)^n}{(1+i)^n - 1} \qquad [€/\text{Year}]. \tag{4.1}$$

The annual costs can then be compared to the annually avoided risk in order to calculate the cost-effectiveness of the countermeasure under analysis.

Table 4.1

	Determine		Assess		Estimate		
		Investment cost ***IC***	Operation cost ***OC***	Maintenance cost ***MC***	Repair cost ***RC***	Interest rate ***i***	Duration ***n***
	Countermeasure	[€ per unit]	[% of IC]	[% of IC]	[% of IC]	[%]	[Years]
	Avalanche Gallery	15'000/meter	0.2	0.5	0.25	2	50
	Permanent supporting structures	1'000/meter	0	0.25	0.25	2	70
	Snowdrift regulations	Type specific - Jet roofs: 500€/meter - Wind baffles: 3'000€/#	0	0.25	0.25	2	30
	Avalanche breakers	50-100€/m3	0	0	0.2	2	50
	Catching and retention dams	5-10€/m3	0	0.5	0.2	2	100
	Deflection dams	5-10€/m3	0	0.5	0.2	2	100
	Reinforcements of buildings	Type specific - Concrete work: 150€/m3 - Earth work: 5€/m3	0	Variable	Variable	2	50

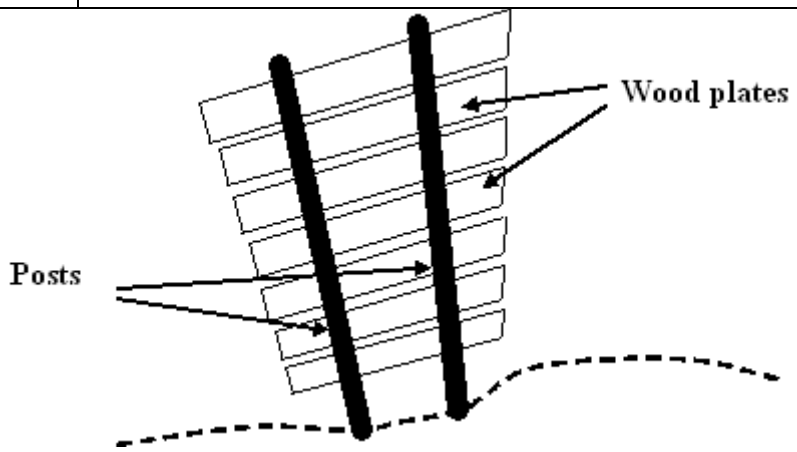
4.1.3 Impact analysis

Depending on site-specific conditions, the different countermeasures provide variable risk reduction impacts. In Table 4.2, we list some experiences with the countermeasures described above.

Table 4.2

Countermeasure	
Avalanche Gallery	**95-100%**: if design requirements are met (in Switzerland the guidelines for avalanche galleries by ASTRA (1994) and the SIA 260 norm (2003) for engineered structures have to be fulfilled), avalanche galleries are likely to reduce avalanche impacts on the covered road sections almost completely. Most avalanche accidents are happening at the entrances of galleries indicating that the specific structure was too short. Thus, the length of the gallery is a crucial issue to be carefully analyzed in the design phase.

	Permanent supporting structures	**60-90%**: if design requirements as described in Margreth (2007) are met, permanent supporting structures are likely to reduce avalanche impacts by 60-90%. E.g., if the supporting structures are designed to sustain a snow depth with a return period T = 100 years (Eq. 4.2a) or T = 300 years (Eq. 4.2b) respectively, the likelihood of an avalanche event within the life expectancy of the supporting structures (n = 50 years) can roughly be estimated by: (4.2a) $1-P(X=0)=1-(1-1/T)^n=1-(1-1/100)^{50}=0.4$ (4.2b) $1-P(X=0)=1-(1-1/T)^n=1-(1-1/300)^{50}=0.15$ In order to be effective, it is most important that the measures cover the whole starting zone (Margreth & Romang 2006). Additionally, the height of the supporting structures as well as the horizontal distance between the rows of structure are decisive for their risk reduction impact. After heavy winters, foundations have to be controlled in order to guarantee full retention capacity.
	Snowdrift regulations	**10-20%**: snowdrift regulations at neuralgic points do reduce the accumulation of snow and thereby reduce the probability of avalanche releases. However, it is difficult to say how effective they are. Commonly, their risk reduction impact is estimated at 10-20 % depending on the site conditions and the representative wind direction. However, these structures are foremost implemented in combination with supporting structures and their own contribution to risk reduction is hardly evaluated.
	Avalanche breakers	**Max. 30%**: avalanche breakers reduce the energy of avalanches. Thus, their risk reduction impact is a function of their cubature, their arrangement, and the energy and speed of the avalanche under consideration. As a rule of thumb, 30% is the maximum risk reduction impact to be achieved by such structures.
	Catching and retention dams	**30-60%**: the risk reduction impact of catching and retention dams depends on their respective capacities. According to Rackwitz (2002), the exceedance probability of an overflow P_f (h, r) can be modeled by a Weibull distribution, if the retention capacity r and the height h are known. Commonly, one expects these structures to primarily reduce frequent events. Their risk reduction impact is often between 30 to 60%. However, they become less effective or even ineffective in protecting against follow-up avalanche occurrences, once they have been filled by the first avalanche event.
	Deflection dams	**90-100%**: the risk reduction impact of deflection dams depends on their respective heights h. If constructed in accordance with the expected avalanche dimensions, deflection dams may reduce the risk almost completely. However, deflection dams become less effective or even ineffective in protecting against follow-up avalanche occurrences, once they have been filled by the first avalanche event.
	Reinforcements of buildings	The risk reduction impact of reinforcements varies largely due to site-specific variability. Thus, it is impossible to specify benchmark values for their risk reduction impact.

	4.2 Detailed Technical Description of Countermeasures
	4.2.1 Wind Baffles
 ***Figure 49:*** *Wind baffle.*	
Description:	A wind baffle is a discrete system consisting of one trapezoidal board held by one or two posts, and facing the prevailing wind direction. Cross-wind baffles are also used: they consist of two perpendicular boards that make their operation less dependent on the wind direction. (photo gallery) (example)
Purpose:	Control of wind-transported snow to prevent avalanche formation in leeward zones. The wind baffle creates a disturbance in the wind stream which is accelerated in the vicinity of the wind baffle. Firstly the snow cannot settle around it and therefore prevents the creation of cornices. Secondly snow settling around the cleared zone acquires a great cohesion due to the modification in the snow crystals, creating a kind of crater. This crater constitutes a discontinuity and an anchoring for the surrounding snow cover and fights against the formation of great wind slabs.
Main design criteria:	– Location: avalanche starting zone; – the larger the wind baffle is, the larger and longer the zone of influence is (the height H of a wind baffle is typically about 3 or 3.5 m); – an increase of the porosity and of the bottom gap up to a certain limit seems to increase the area of the snow cover submitted to the effect of the wind.
Materials:	– The board of a wind baffle is generally made of wood plates; – the posts are made of steel, but they also exist in aluminum.

	Anchoring system solutions are described in MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne: conception, réalisation, contrôle. Guide technique.
Costs analysis (without taxes):	Investment: cost of a cross-wind baffle about 3000 euros in 1992 (Switzerland), including construction, which costs from 500 to 700 euros (1992, Switzerland).
References:	

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Example:	The wind baffle is particularly adapted to fight against the formation of cornices. It is also used to limit any local overload on traditional avalanche structures. It can sometimes be used\ to prevent the development of avalanches from wind slabs, by generating a heterogeneous structure in the snow cover. Examples of a complete design procedure: Case studies from Switzerland (Valais) are available in Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. 205 p. Manuel EPFL / Service des Forêts et du Paysage du Valais. Morgex (Italy): new protection works including cross-wind baffles in the starting zone to supplement snow cover supporting structures (after the avalanche of 1999).

Characteristics:	The wind baffles can have three purposes: – Stabilization of the slope. In this case they are located under the drop in slope, inside the avalanche starting zone (case of Grimselboden in Switzerland where the wind baffles are located 20 m under the slope-line or Illhorn where they are located 60 m under the peak). The property of the wind baffle to modify locally the structure of the snow cover is used provided that the slope is not longitudinally too long; one "nails" the snow cover by the action of the snow cones or craters at the base of the wind-baffles. This system is rarely used and should not be used alone to protect directly an inhabited zone located downstream. In the case of Grimselboden, this set-up was associated with downstream dams shortening the avalanche run-out. – Wind baffles are more classically used to prevent the formation of cornices and more generally to avoid snow accumulations on supporting structures located downstream in the avalanche starting zone. In this case, the wind baffles are located at the top of the starting zone (examples: Mont-Gond and Mont-Cauille in Switzerland) so that the crater, which forms, removes the appearance of cornices. – A third way to use wind baffles is to split the avalanche tracks into two (case of Larche in France). In this case, the wind baffles are located on small transverse peaks that delimit the avalanche tracks between them. The wind baffles constitute a kind of discontinuity as a "prepunched" material. The possible release of one track will propagate until the transverse peak and will follow the weakness line without involving the close track.

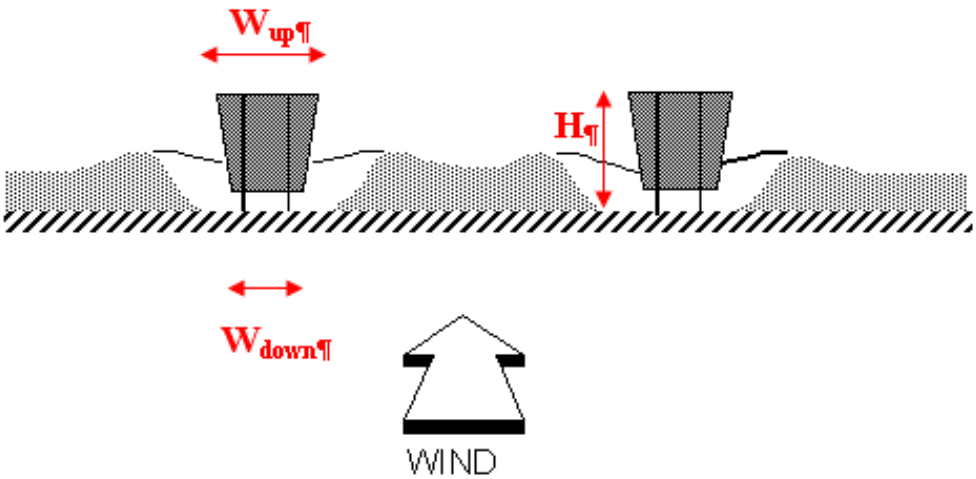

Figure 50: *Design of wind baffles.*

	– Board shape: a wind baffle has a trapezoidal shape turned up side down. It is therefore wider on the top than on the bottom; – board size: the board is generally 1.5 m wide at the bottom (W_{down}) and 3 m wide at the top (W_{up}); – height: the height H of a wind baffle is often about 3 or 3.5 m; – porosity: the porosity is defined as the percentage of the board's voids; – bottom gap: the bottom gap is the distance between the lower side of the board and the ground.

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Detailed design criteria and efficiency:	– The height H of a wind baffle is typically about 3 or 3.5 m but its size varies, given that the larger the wind baffle is, the larger and longer the zone of influence is; – concerning the porosity and the bottom gap of a wind baffle, there exist no comparative investigations of their influence on the wind. However, an increase of these two parameters up to a certain limit seems to increase the area of the snow cover subjected to the effect of the wind; – as an example, for the typical following values H = 3 m, W_{down}=1.5 m and W_{up}=3 m (see above characteristics), with a porosity of 23 % and a bottom gap of 50 cm, the wind crater can measure up to 8 m in diameter and 30 m long (in cases of favorable local topography) It has been noticed that wind baffles with low porosity (10 %) and low

	bottom gap (20 cm) worked very well. The zone devoid of snow was smaller, but clearer. The objective of a wind baffle is to clear a given zone where the amount of snow, due to snow transport by wind, is too large. Therefore, it is very important to place the baffle as close to this zone as possible (a drop in a slope, avalanche defense structure…) Nevertheless, this rule should take into account the local topography. Actually, a wind baffle should not be located in a wind protected zone. Because in this case, the wind baffle would be quickly covered by snow and thus would loose its effectiveness. It would therefore be necessary to setup the wind baffle further away from the wind protected zone, in a zone with more wind, and replace the wind baffle with a larger one, to compensate for the increased distance from the top. If the wind baffle consists of one board only, this board should be orientated perpendicular to the wind direction that causes snow accumulation. Note that it's necessary to keep in mind that each site represents a specific case. The effectiveness of a wind baffle depends on its location. It is therefore indispensable to have an accurate knowledge of how the site works in winter, acquired by local observations (main direction of the wind, typical value of the snow cover depth). These observations should continue after the structure has been set up to check that it has been correctly positioned. In fact, since the sites for the wind baffles are marked out when there is no snow, the local topography may change when there is a snow deposit. That is the reason why it is important to be able to remove the wind baffles easily, once that the project and the budget were drawn up.

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Costs analysis:	Investment (level without taxes): – the cost of a cross-wind baffle is about 3000 euros in Switzerland (1992), including construction, which costs from to 500 to 700 euros (1992, Switzerland); – the cost of a 4 m wide cross-wind baffle is 2000 euros in India (2002), without taking into account installation.

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Design-specific References:	Vinay Chaudhary, Praveen Mathur and Gurpreet Singh. 2002. Formation zone avalanche control scheme at D-10 avalanche site on National Highway, Jammu-Srinagar (India): A case study. International

Snow Science Workshop (2002: Penticton, B.C.).

MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne : conception, réalisation, contrôle. Guide technique.

Naaim F., Sivardière F. 1995. Local protection against snow accumulations due to wind structures and afforestation. Université européenne d'été sur les risques naturels – Neige et avalanches. Ed. G. Brugnot. Actes-Chamonix, 14-25 septembre 1992, p. 255-271.

Naaim F., Brugnot G. 1992. Transport de la neige par le vent. Connaissances de base et recommandations. 350 p. Division nivologie (Cemagref) / Délégation aux Risques Majeurs (Ministère de l'environnement).

Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. 205 p. Manuel EPFL / Service des Forêts et du Paysage du Valais.

Tabler and associates. 1991. Snow Fence Guide. 61 p. Strategic Highway Research Program, National Council, Washington, DC.

Taillandier, J-M., « Ouvrages à vent paravalanches » TRACE3 2003 Les moyens de prévention des avalanches par la maîtrise du vent, Edition R. Bolognesi- E. Basseti, p. 101-111.

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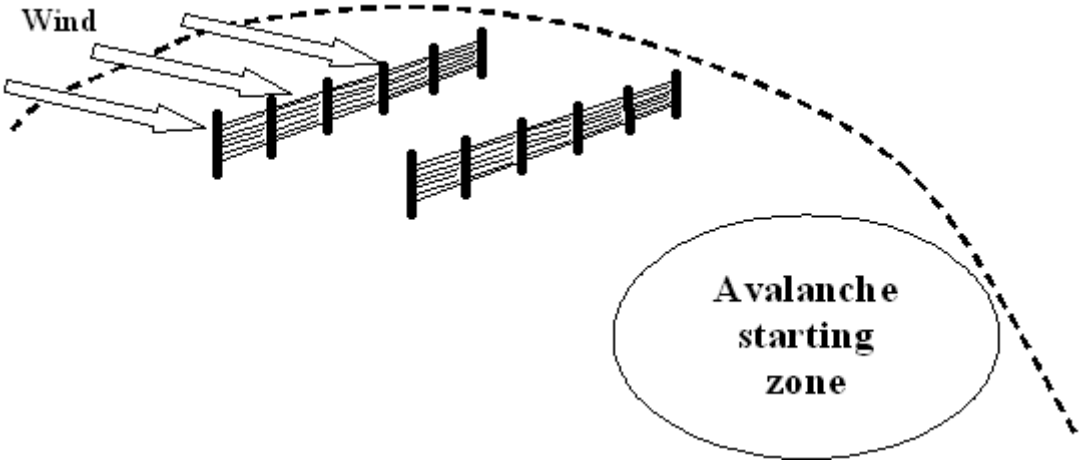
	4.2.2 Snow Fences
 ***Figure 51**: Design snow fences.*	
Description:	A snow fence is a linear system that is designed to accumulate or hold snow at a certain location while acting on the wind flow. It consists of a panel and posts ensuring its anchorage in the ground. (example) (photo gallery)
Purpose:	Control of wind-transported snow to prevent avalanche formation in leeward zones. A snow fence may be designed upstream of avalanche starting areas to limit cornices formation and to supplement snow supporting structures. Note that snow fences are generally used to fight against snowdrifts formation on roads ("plateau" context). Here we only describe snow fences that are used in avalanche defence engineering ("high mountain" context).
Main design criteria:	– Location: upstream of an avalanche release area; – collector fences are used on open windward slopes of low inclination (typically less than 15°). In steep terrain they are less effective; – typical mesh size: between 5 and 20 cm; – fence should consist of about 50% solid surface and 50% opening surface; – the distance from the ridgeline should be about 20 to 30 times the height H (H ranges generally from 3 to 4 meters); – the fence should have a gap at base ranging from 0.15H to 0.25H where H is the height of the fence).
Materials:	The first walls were made of stones but now they are mainly made of: – wood; – steel; – aluminum. The choice of the material depends on the climatic conditions snow fences

	are exposed to. The easiness of maintenance is also important. Thus, steel and wood are mainly used in mountains (whereas synthetic materials can be used for temporary snow fences on plateaus or to manage the snow cover in ski resorts: these synthetic materials snow fences can be easily relocated). Pressures due to wind exerted on snow fences can be calculated (see page 8 in AFNOR Norme française NF P95-305 "Equipements de protection contre les avalanches : barrières à neige / spécifications de conception » 13 p.). Anchoring system solutions are described in MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne : conception, réalisation, contrôle. Guide technique.
Costs analysis (without taxes):	Investment: about 600 euros per linear meter (1992, Switzerland), in high mountains context.
References:	

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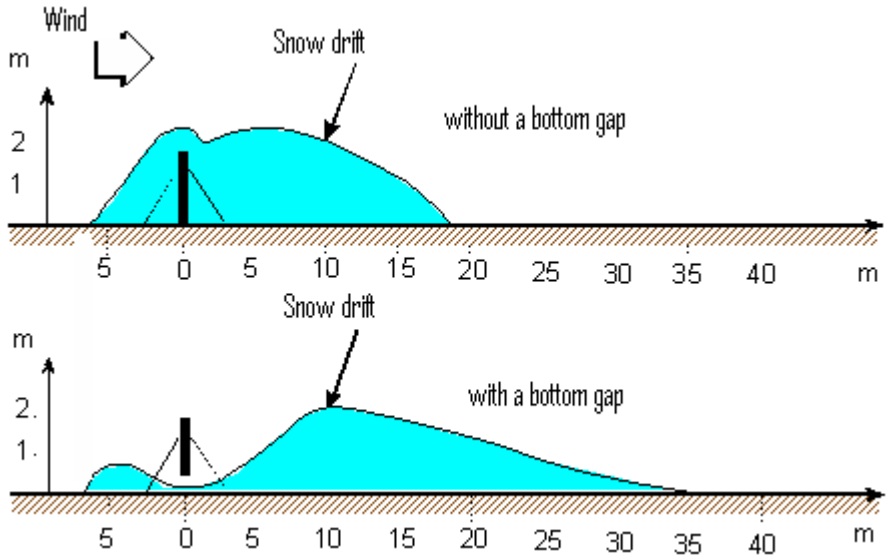
Characteristics:	
 ***Figure 52:*** *Scheme snow fences.*	
	The main characteristics of a snow fence are the following ones: – *Length*: the length is the distance between the two ends of the fence; – *porosity*: the porosity is defined as the percentage of the board's voids; – *bottom gap*: the bottom gap is the distance between the lower side of the board and the ground; – *height*: the fence height is the distance between the top of the fence and the ground (perpendicular to the ground); – *inclination*: the inclination is the angle (expressed in degrees from vertical direction) that the fence tilts upwind. This angle generally ranges from 0 to +15°, the sign "+" indicating that the inclination is in the wind direction.

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Detailed design criteria and efficiency:	– The length has to be sufficient in order to limit ends effects. It should reach at least 20 times the height of the fence. – An increase up to 45-60% in the porosity of a fence lengthens the deposit, diminished its height and moves it away from the fence. Nevertheless, the increase has a general effect to raise the volume of the stored snow. – The bottom gap produces wind acceleration at the bottom of the fence and leaves it clear. This voids the bottom of the fence being damaged by snow settlement, and also increases the intercepted snow volume. The bottom gap should be around 40 to 50 cm in mountainous areas (20 to 30 cm on a plateau). It is generally equal to 15 to 20% of the total fence height. Some kinds of fences have an adjustable bottom gap that can be increased when filled with snow. – The fence height determines the fence storage capacity that is the maximum snow quantity that the fence can accumulate. This capacity has the following form: $C = KH^2$ (m^3 / m), where K depends on the porosity and the bottom gap. It generally ranges from 15 to 25 depending on the installation type. The fence height can be defined as the snow quantity there is to accumulate during the winter. This quantity depends on the intensity and the duration of the snowfall, on the quantity of relocated snow, and the fetch (distance within which the wind can pick up snow and transport it). The French Transport Minister uses the following equation (Naaim F. and Sivardière F., 1995): $C = \frac{\alpha.P.L}{d}$, where α is the proportion of P relocated and transported by wind, P represents the snowfall expressed by meter of water, L is the fetch, d is the snow density. Some kinds of fences can be made higher during the winter, which is very useful when snowdrifts have a tendency to reach the top of the fences at the end of the season (only along roads because maintenance is needed in this situation). The height ranges generally between 3 and 4 m in mountainous areas (whereas it ranges from 1 to 2 m on the roads) – Inclination: the top of the fence can be inclined in the downwind direction up to 15° from vertical direction without affecting its performance adversely. For optimal effectiveness: – the fence should consist of about 50% solid surface and 50% opening surface; – fences should have a gap at base equal to 0.15-0.25H, where H is the fence height; – the typical mesh size should be between 10 and 20 cm to avoid obstruction by rime ice.

	Location: collector fences are used on open windward slopes of low inclination (typically less than 15°). In steep terrain they are less effective. The main rules for location are the following ones: – the storage capacity of a snow fence increases as the angle between the wind direction and the perpendicular axis to the fence decreases. Thus the snow fences should be orientated perpendicular to the prevailing wind direction. If the angle between the snow fence and the prevailing wind direction is higher than 45°, the snow stored by the barrier is negligible. – The snow fences (porous and with a bottom gap) should be placed 25 H away from the zone to be protected from snowdrift or from the next fence (where H is the fence height). When the amount of snow to be stored is high and therefore leads to the use of a fence higher than 4 meters, it is better to set up several fences. The value of 25 H varies according to the characteristics of the fence and to the topography. – Fences should be as long as possible and without interruption. Furthermore, the base of the structure has to follow the slope line in order to make the bottom gap efficient. Note that it's necessary to keep in mind that each site represents a specific case. The effectiveness of a snow fence depends on its location. It is therefore indispensable to have an accurate knowledge of how the site works in winter, acquired by local observations (main direction of the wind, typical value of the snow cover depth). These observations should continue after the structure has been set up to check that it has been correctly positioned. In fact, since the sites for the snow fences are marked out when there is no snow, the local topography may change when there is a snow deposit. That is the reason why it is important to be able to remove the snow fences easily, once that the project and the budget were drawn up. In a high mountain context, it's obviously very difficult (in comparison with a plateau context).

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Example:	A snow fence creates a disturbance in the wind flow, which decreases the local wind velocity. As a result a snow deposit appears mostly just behind the fence (a small deposit may also appear in front of the fence). Thus a snow deposit can be built at a chosen place according to the fence location. This counter-measure prevents snow from being transported elsewhere where it could be dangerous. The snow that is deposited in this way has a great cohesion. Therefore, it is very difficult for the wind to remove the snowdrift if the wind's direction changes. Examples of a complete design procedure: Case studies from Switzerland (Valais) are available in Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. 205 p. Manuel EPFL / Service des Forêts et du Paysage du Valais.

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Costs analysis:	– The price of a snow fence strongly depends on the accessibility of the site, which can be very high in "high mountain" context. – Investment (level without taxes): • the cost of snow fence is between 15 and 100 euros per linear meter in France (1986); • the cost of snow fences is about 600 euros per linear meter in Switzerland (1992) in high mountains context; • the cost of snow fences is 15 euros per linear meter in India (2002), without taking into account installation.

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Design-specific References:	AFNOR Norme française NF P95-305 "Equipements de protection contre les avalanches : barrières à neige / spécifications de conception » 13 p. Vinay Chaudhary, Praveen Mathur and Gurpreet Singh. 2002. Formation zone avalanche control scheme at D-10 avalanche site on National Highway, Jammu-Srinagar (India): A case study. International Snow Science Workshop (2002: Penticton, B.C.). MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne : conception, réalisation, contrôle. Guide technique. Naaim F., Sivardière F. 1995. Local protection against snow accumulations due to wind structures and afforestation. Université européenne d'été sur les risques naturels – Neige et avalanches. Ed. G. Brugnot. Actes-Chamonix, 14-25 septembre 1992, p. 255-271. Naaim F., Brugnot G. 1992. Transport de la neige par le vent. Connaissances de base et recommandations. Division nivologie (Cemagref) / Délégation aux Risques Majeurs (Ministère de l'environnement). 350 p. Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. Manuel EPFL / Service des Forêts et du Paysage du Valais. 205 p. Tabler and associates. 1991. Snow Fence Guide. Strategic Highway Research Program, National Council, Washington, DC. 61 p. Taillandier, J-M., « Ouvrages à vent paravalanches » TRACE3 2003 Les moyens de prévention des avalanches par la maîtrise du vent, Edition R. Bolognesi- E. Basseti, p. 101-111.

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	4.2.3 Jet Roofs

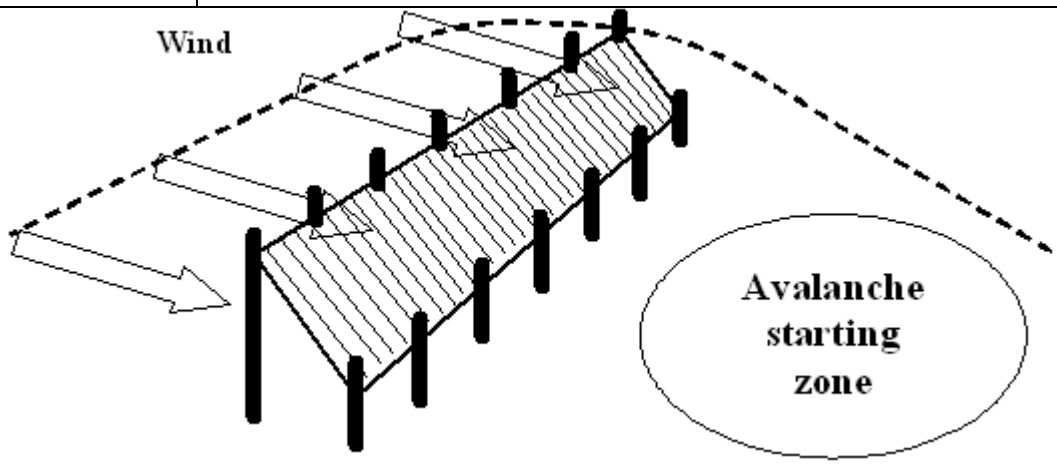

Figure 53: *Design jet roof.*

Description:	A jet roof is a linear system used to prevent the formation of cornices and an undesirable accumulation of snow that appears immediately under the ridgelines. It consists of an inclined panel, held by several posts. (photo gallery) (example)
Purpose:	Control of wind-transported snow to prevent avalanche formation in leeward zones by removing cornices. Note that jet roofs have also other applications, namely to protect buildings, ski lifts and roads. According to the principle of Venturi, a jet roof accelerates the wind: the wind coming out from the small opening under the inclined panel has a speed superior to the speed it had coming into the large opening. The result is a zone cleared from snow behind the jet roof but with a possible increase of snow accumulations in the starting zone! Jet roofs are often associated with downstream supporting structures to prevent avalanche formation.
Main design criteria:	– location: avalanche starting zone; – the surface of a jet roof panel is equal to some square meters consisting of separate boards, inclined at about 40°; – the distance between the lower side of the panel and the ground, named the "small opening", is about 1 m high; – a jet roof is usually made of wood.
Materials:	– Jet roofs are generally made of wood (panel and posts). – The posts are sometimes made of steel. Anchoring system solutions are described in MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne : conception, réalisation, contrôle. Guide technique.

Costs analysis:	Investment: between 450 and 600 euros per linear meter (1984, France), including installation.
References:	

Example:	A jet roof is used to accelerate and deflect strong wind underneath. Thereby, deposition takes place away from the ridgeline (but in the avalanche staring zone). Examples of a complete design procedure: Case studies from Switzerland (Valais) are available in Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. 205 p. Manuel EPFL / Service des Forêts et du Paysage du Valais.

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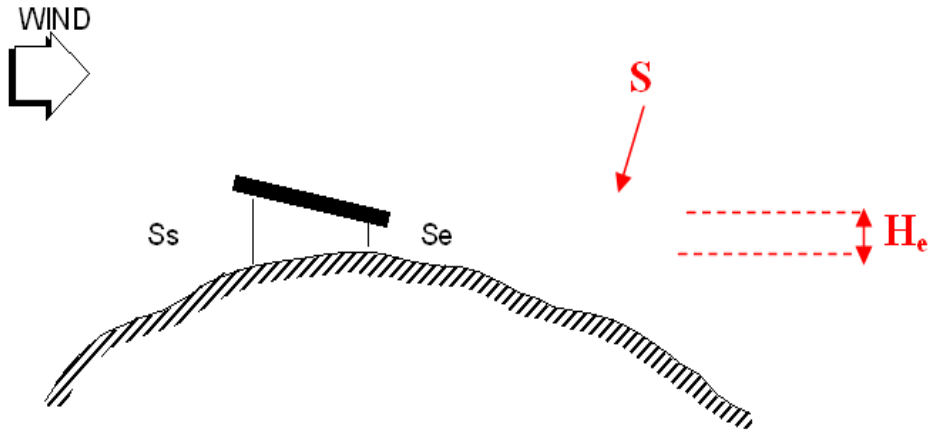
Characteristic:	 ***Figure 54:*** *Scheme jet roof.*
	Here we give the main characteristics of a jet roof: – Surface of a jet roof: a jet roof panel has a surface (S) of some square meters consisting of separate boards, inclined at about 40° – Small opening: the small opening (H_e) is the distance between the lower side of the panel and the ground. It is generally about 1 m. The velocity increment can be estimated by mass flow rate (Q) conservation: $Q = cste\ i.e.\ \ u_s S_s = u_e S_e$ where Us, Ue are the incoming (respectively the out coming) wind velocities in the large (respectively small) opening and S_s , S_e are the flow sections at the large (respectively the small) opening. Advantages: the jet roofs are the most efficient structures to fight against the formation of cornices. Drawbacks: the cornice formation is often avoided but the snow is transported further in the slope, which can increase the amount of snow in the starting zone! Therefore it's generally necessary to add, at least, a row of supporting structures to support downstream snow accumulations.

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Detailed design criteria and efficiency:	A jet roof should be placed on the ridgeline or at the top of a slope, before a cliff edge (or on the side of the zone to be protected, in case of snowdrifts on a road). This rule should take into account the local topography. Its placement should take into account the wind direction since it uses the wind force. The large opening faces the wind, the small one serves as outlet. Note that, it's necessary to keep in mind that each site represents a specific case. The effectiveness of a jet roof depends on its location. It is therefore indispensable to have an accurate knowledge of how the site works in winter, acquired by local observations (main direction of the wind, typical value of the snow cover depth). These observations should continue after the structure has been set up to check that it has been correctly positioned. In fact, since the sites for the jet roofs are marked out when there is no snow, the local topography may change when there is a snow deposit. That is the reason why it is important to be able to remove the jet roofs easily, once that the project and the budget were drawn up.

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Cost analysis:	Investment (level without taxes): – the cost of a jet root is between 450 and 600 euros per linear meter in France (1984), including installation – the cost of the structure itself is about 130 euros per linear meter in India (2002), without taking into account installation

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Design-specific References:	Vinay Chaudhary, Praveen Mathur and Gurpreet Singh. 2002. Formation zone avalanche control scheme at D-10 avalanche site on National Highway, Jammu-Srinagar (India): A case study. International Snow Science Workshop (2002: Penticton, B.C.). MEDD-DPPR / CEBTP Lyon / Cemagref Grenoble. 2004. Protection contre les risques naturels. Ancrages passifs en montagne : conception, réalisation, contrôle. Guide technique. Naaim F., Sivardière F. 1995. Local protection against snow accumulations due to wind structures and afforestation. Université européenne d'été sur les risques naturels – Neige et avalanches. Ed. G. Brugnot. Actes-Chamonix, 14-25 septembre 1992, p. 255-271. Naaim F., Brugnot G. 1992. Transport de la neige par le vent. Connaissances de base et recommandations. 350 p. Division nivologie (Cemagref) / Délégation aux Risques Majeurs (Ministère de l'environnement). Sivardière F., Castelle T. 1992. Ouvrages à vent en montagne : inventaire et diagnostics en Valais. 205 p. Manuel EPFL / Service des Forêts et du Paysage du Valais. Tabler and associates. 1991. Snow Fence Guide. 61 p. Strategic Highway Research Program, National Council, Washington, DC. TAILLANDIER, J-M., « Ouvrages à vent paravalanches » TRACE3 2003 Les moyens de prévention des avalanches par la maîtrise du vent, Edition R. Bolognesi- E. Basseti, p. 101-111.

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	4.2.4 Temporary supporting structures

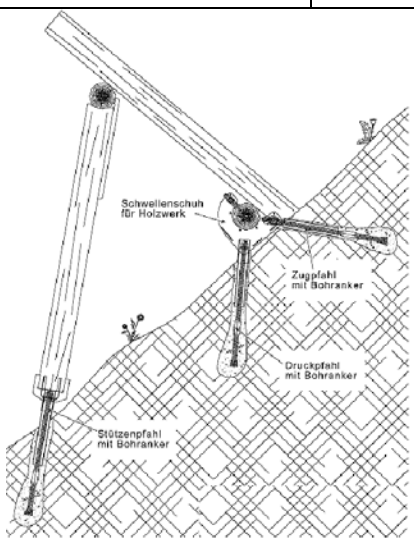

***Figure 55**: Cross-section of a Timber rake*

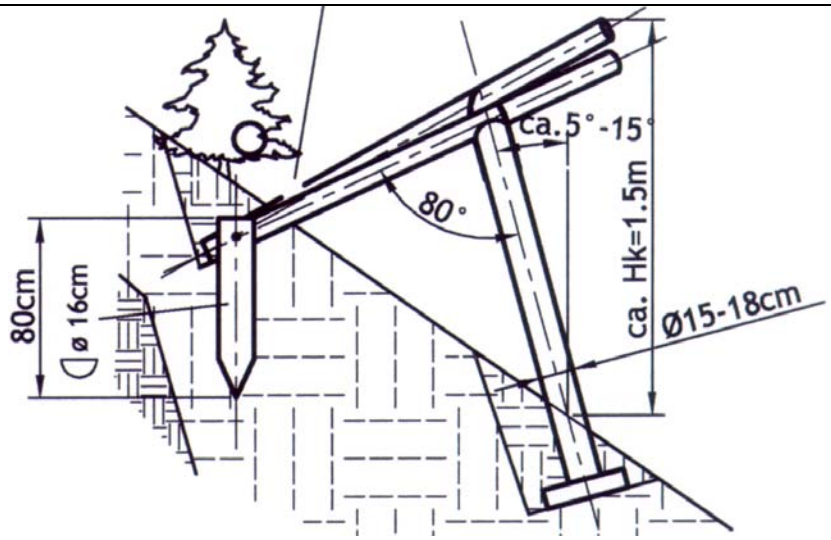

***Figure 56**: Cross-section of a tripod*

Description:	Temporary supporting structures have the same functionality as permanent supporting structures. They are erected more or less perpendicular to the slope and are well anchored in the ground and act as barriers against creeping and gliding motions. Under the term temporary supporting structures, we subsume timber rakes and tripods. While timber rakes are connectively built in a row, tripods are spaced at uniform distances. (photo gallery) (example)
Purpose:	Temporary supporting structures are used to stabilize the snow cover and reduce avalanche release in areas of reforestation. Within the lifespan of temporary supporting structures, the regenerating forest will become able to provide this protection function, as soon as the planted trees exceed in average the height of the maximal snow cover at the site. To serve this purpose, temporary supporting structures are designed for a lifetime of roughly 50 years depending on the wood used for construction. (advantages) (drawbacks)
Main design criteria:	Height of temporary structures: depends on the observed maximal snow depth at the site of application. In general, timber rakes are between 2.5 m and 3.5 m in height, while tripods are 1.5 m in height. Foundation of temporary structures: in general, there are three types of foundations in use for this type of structure. 1st variant: waling anchored by a wire rope used on solid rock; 2nd variant: steel pedestal to avoid ground striking of wood used on bedrock; 3rd variant: waling anchored with micro piles used on dense soil (clay).

	Arrangement of structures: depends on the release area, the slope angle, and the distance between rows of structures (which again depend on the average and the maximal snow cover). Static/Dynamic design criteria: static design criteria of supporting structures have to warrant the withstand against snow loading. Supporting structures are not designed to withstand large dynamic forces. Thus, only slides and sluffs have to be considered for dynamic design criteria.
Materials:	Temporary supporting structures are made of timber. For the foundation of the structures rope wire anchors, micro piles, and steel pedestals are in use. Each timber species shows a different vulnerability towards decomposition. Therefore, the choice of timber should be according to SIA norm 265/1:2003, Sect. 5. General experience with different timber species show lifespan (Leuenberger 2003) of: – 10-20 years for Larix deciduas (untreated) – 10-25 years for soaked wood (e.g., Picea abies, Abies alba, Pinus sylvestris) – 25-40 years for Castanea sativa, Pseudorobinia acazia, Quercus ssp. (untreated).
Cost analysis:	This evaluation of the costs is based on the prices list of the own country.
References:	

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Advantages:	– Timber structures are in general much less expansive than steel structures; – good experience with reforestation on different sites; – in general, protection forest is one of the most cost-effective mitigation measures against snow avalanches.

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Drawbacks:	Success of temporary supporting structures depends on the success of reforestation, which is impacted by other factors such as ungulate browsing, choice of plants, vegetation period, snow height, light conditions (see Ott 1997).

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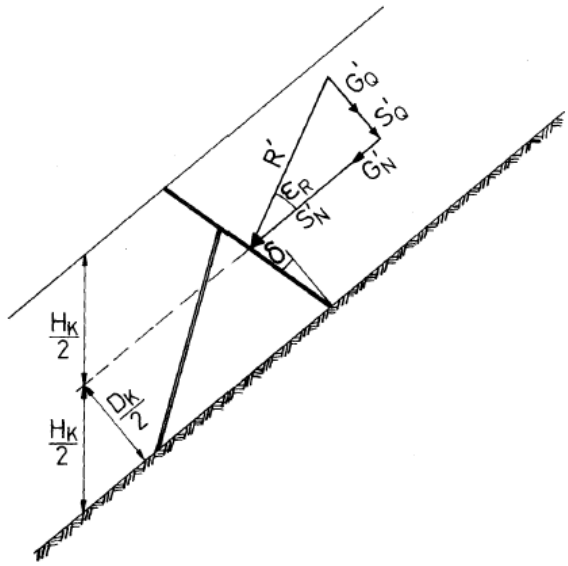
Design criteria:	**Snow loading on the supporting structure:** Component of snow pressure parallel to the slope: $S'_N = H_K^2 N\ f_C$ [kN/m'] with H_K = vertical height of the structure; N = glide factor depending on the roughness of the ground surface (for determination see Margreth 2006); f_C = altitude factor expressing the general increase in snow density with altitude (for determination see Margreth 2006). Component of snow pressure perpendicular to the slope: $S'_Q = S'_N \cdot (a/N \tan\psi)$ [kN/m'] with ψ = slope angle; choose α = 0.35 or α = 0.5 whichever gives the most unfavorable load for the particular part being considered. Component of the weight of the snow prism for a rigid supporting plane, allowing for a higher snow density near the supporting plane: $G' = 1.50\, D_K^2 \tan\delta$ [kN/m'] with D_K = effective height of the structure [m], whereby $D_K = H_K \cos\psi$; δ = angle between the supporting plane and the slope distance. End-effect forces (additional forces per unit length of the line of structures operating parallel to the slope): $S'_R = f_R\, S'_N$ [kN/m'] with $f_R = (0.92 + 0.65 \cdot N) \cdot A/2 \leq (1.00 + 1.25 \cdot N)$ being the end-effect factor. Resultant of all snow pressures are derived by vectorial addition of the above components (Figure 57): $R' = \sqrt{(S'_N + G'_N + S'_R)^2 / (S'_Q + G'_Q)^2}$ [kN/m'], which is applied in direction $\tan\varepsilon_R = R'_Q/R'_N$ whereby ε_R = angle between the resultant and the parallel to the slope. Overall design of the structure has to acknowledge the resultant and should be completed in accordance to SIA norms 261, 262, 265 (SIA 261, 262, 265: 2003).  ***Figure 57**: The resultant of the snow pressures*

Transverse loading on posts due to accumulation of snow:

This pressure is relatively small and may be expressed by means of a regularly distributed linear force:

$$qs = \eta H_K{}^2 N f_C \frac{Diameter\ of\ post}{Length\ of\ post}\ [kN/m^3]$$

with: η = efficiency in dependence on the gliding factor N (normally taken as 1.0); H_K = vertical height of the structure [m]; Diameter and length of post [m]. The direction of q_S is perpendicular to the axis of the post, and directed downhill. Posts with an asymmetric cross section should not be free to rotate about their long axis. The line of application of q_S goes through the center line of the post. Permanent weight of the structures is normally not considered.

Snow loading parallel to the supporting plane:

As the state of nature of the distribution of pressures among the supporting plane is often unstable, sterner assumptions for specific loading on the elements of the plan have to be applied. The Figure 58 shows the basic load as

$$P' = R'\cos(\delta - \varepsilon_R)\ [\text{kN/m}']$$

with: P´ = component of R´ vertical to the plane; ε_R = angle between the resultant and the parallel to the slope. Consequently, the specific transverse loading parallel to the supporting plane is $p_h = P'\cos\delta/(0.77 D_K) = P'/(0.77 B_K)$ [kN/m'] and the linear transverse loading acting upon a bar is $p'_B = p'_h \cdot b$ [kN/m'] with b = width of loading (width of bar plus share of the neighboring clearance).

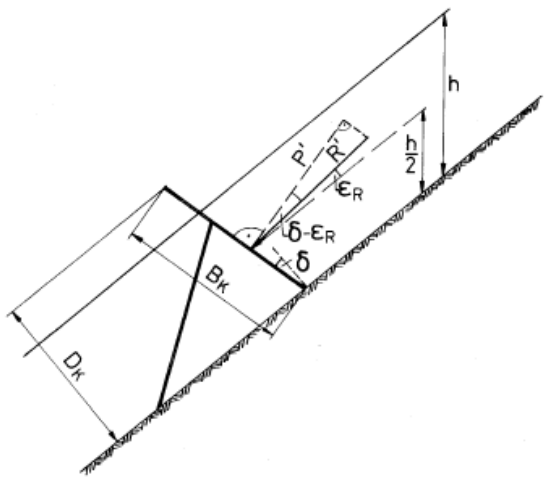


***Figure 58**: Loading parallel to the supporting plane*

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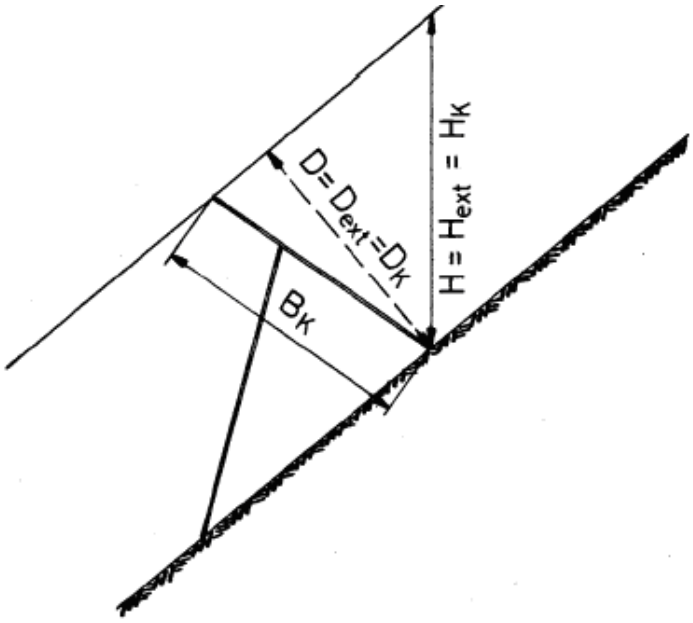
Height of structures:	The vertical height of the structure H_K is defined as the mean vertical distance between the upper border of the supporting plane and the ground. The vertical height of the structures H_K must correspond to the expected extreme snow depth H_{ext} at the location of the structure: $H_K \geq H_{ext}$ [m]. This fundamental requisite must be recognized for effective protection against avalanches during catastrophic events and for the design of the structures. It should be noted that the structures, depending on their features and on site-specific wind conditions, may themselves considerably influence snow depositions. The slant height of the structure B_K is defined as the mean height of the supporting plane measured parallel to the supporting plane of the structure. The lower limit is formed by the ground. The effective height of the structure D_K is defined as the mean distance between the upper border of the supporting plane and the ground measured perpendicular to the slope, which is analogous to the thickness of the snow.  ***Figure 59**: Slant height of the supporting plane*

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Foundation of structures:	One task of the project manager is to thoroughly analyze the ground conditions for foundations (SIA 260: 2003). Thereby, he has to consider: Geological ground conditions (thickness of ground, nature and joint density of the ground, nature of overlap, humidity and frost conditions, solifluction, soil chemistry) Determination of the ground resistance (e.g. by anchor pulling test) Choice of the structure type: as they have different requirements for the foundation, foundation conditions have to be analyzed before this choice (e.g. by tentative anchor) Type of foundation There are two types of foundation normally used on solid soil. Depending on the type of loadings described above, ground anchors, micro piles, base plates, and/or concrete fundaments are used to fix the structures on the soil. In general, two separate foundations are used for this type of structure. One is an upper or uphill foundation (so-called beam/ground foundation), the other is a lower or downhill foundation (so-called support foundation). In case of unstable or phreatic ground, it is advisable to connect the upper and lower foundations by a member called a pressure bar, which accepts tensile and compression stress. In these special cases SIA norm 267 should be consulted (SIA 267: 2003).

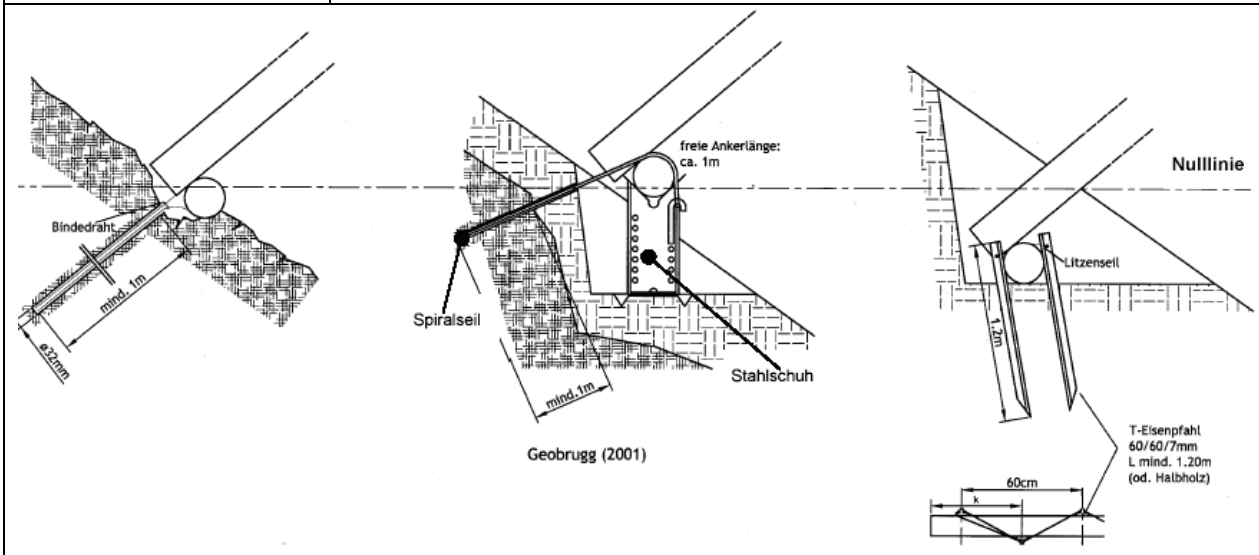

***Figure 60**: scheme of foundation of structures*

1st variant: waling anchored by a wire rope used on solid rock; borehole filled with mortar.	*2nd variant: steel pedestal to avoid ground striking of timber (-> extension of life experience); used on bedrock; minimal need of excavation.*	*3rd variant: waling anchored with micro piles used on dense soil (clay). In loose soil a timber pile can be used (never on eroding soil).*

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Arrangement of structures:	Additionally to the overall requirements of supporting structures, the slope distance between structures and lines of structures has to be designed fulfilling three conditions: (1) the structures must not be damaged by maximal snow load; (2) dynamic stress by snow creeping and drift has to be accepted; (3) the velocity of snow creeping and drift in between the structures must not exceed a defined threshold. That way, the momentum responsible for damages below the structures is limited to moderate scale. The distances between the rows of structures parallel to the slope is given by $L = f_L \cdot H_K$ [m] with the distance factor f_L depending on the friction angle φ between soil and snow, the glide factor N, and the height of the structures H_K. The Figure 61 shows f_L (φ, H_K) and allows to derive the standard measure for f_L. to acknowledge for soil surfaces and special safety demands, the following benchmarks are valid: $N > 2 \Rightarrow \tan\varphi = 0.50 - 0.55$; $N < 2 \Rightarrow \tan\varphi = 0.60$ Some additional requirements: – maximal values to define the distance factor f_L are $\tan\varphi = 0.6 \cap N > 1.3 \Rightarrow f_L = 13$; – if the structures are designed for N = 1.2, then the distance factor must not exceed the respective curve; – if the perpendicular height of the structures H_K is ≥ 4.5 m, the highest values of f_L must not exceed the respective curve in Figure 61; – the margin in the determination of distances should be used to adapt the supporting structures to site specific requirements; – climatic conditions have to be considered in the determination of distances in order to guarantee the reliability of structures. On north-faced slopes and in regions with high annual precipitation (e.g. the Swiss pre-Alps) values for tan φ may be even smaller than 0.50; – if the slope angle between two rows of structures changes, one should use the slope angle ψ between the two root points of the rows of structures to the distance L.

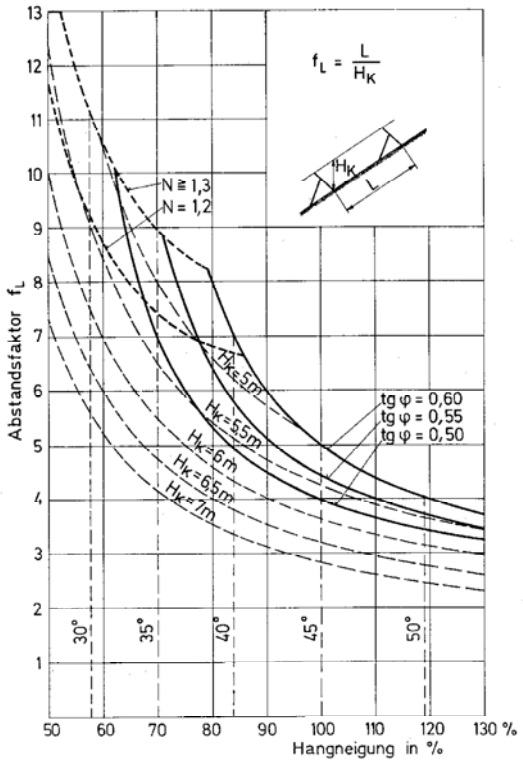

***Figure 61**: Distance factors.*

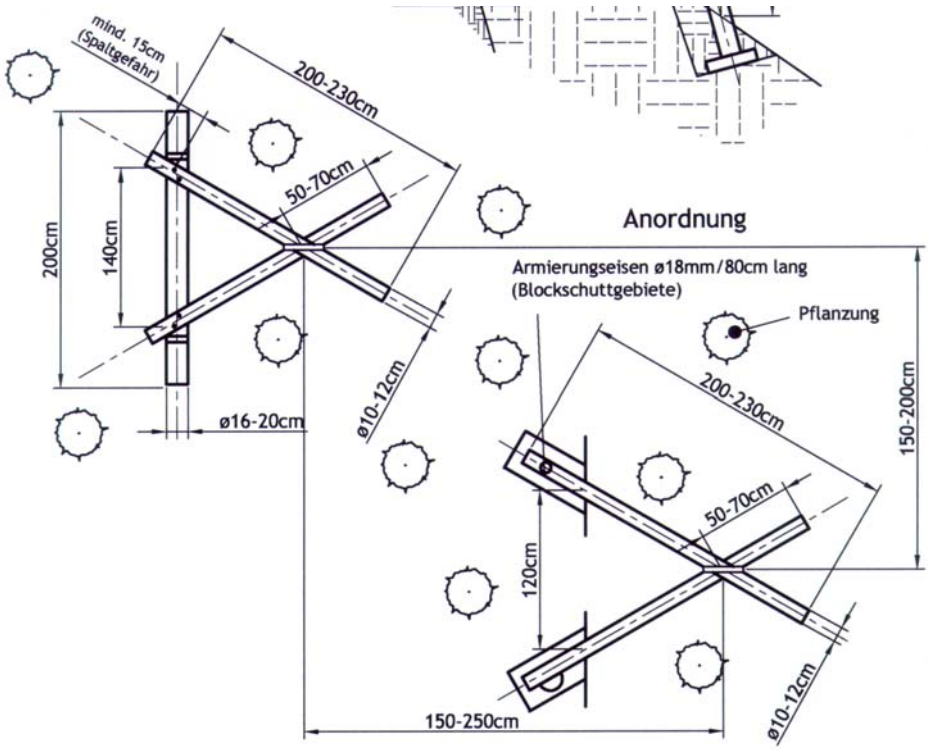

***Figure 62:** Distance between single tripods and the arrangement of the seedlings around the tripods.*

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<table>
<tr>
<td>Costs analysis:</td>
<td>Benchmark values for temporary supporting structures:

<table>
<tr><td></td><td>Costs D_K 2.6 [€/running meter]</td><td>Costs D_K 3.4 [€/running meter]</td></tr>
<tr><td>Timber rakes</td><td>330</td><td>470</td></tr>
</table>

<table>
<tr><td></td><td>Costs [€/unit]</td><td>Costs [€/ha]</td></tr>
<tr><td>Tripods</td><td>120-180</td><td>150’000</td></tr>
</table>

***Table 18**: Benchmarks for overall costs of temporary supporting structures.*</td>
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Design-specific References:	Leuenberger, F. (2003) Bauanleitung Gleitschneeschutz und temporärer Stützverbau. [Construction guidelines for protection measures against gliding snow and temporary supporting structures] Davos: Eidg. Institut für Schnee- und Lawinenforschung. Margreth, S. (2007) Lawinenverbau im Anbruchgebiet. Technische Richtlinie als Vollzugshilfe [Technical guidelines for avalanche control in the starting zone] Bundesamt für Umwelt Bern, WSL Eidg. Institut für Schnee- und Lawinenforschung Davos (eds.). SIA (2003) SIA Norm 260: Grundlagen der Projektierung von Tragwerken. [Projecting of structures] Schweizerischer Ingenieur- und Architektenverein, Zurich. SIA (2003) SIA Norm 261: Einwirkungen auf Tragwerke. [Loading of structures] Schweizerischer Ingenieur- und Architektenverein, Zurich. SIA (2003) SIA Norm 265: Holzbau. [Timber structures] Schweizerischer Ingenieur- und Architektenverein, Zurich.

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	4.2.5 Permanent supporting structures

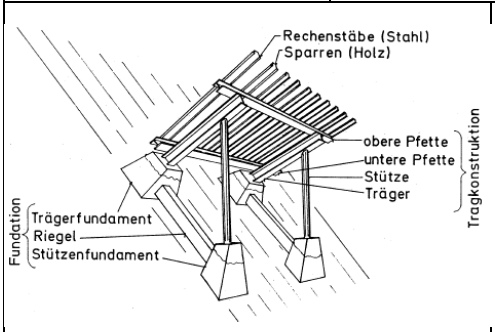

***Figure 63**: Snow rake.*

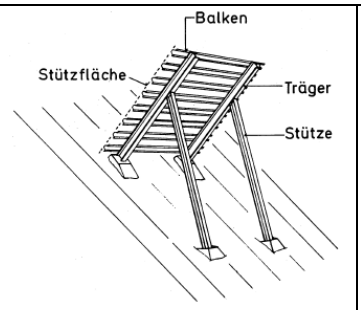

***Figure 64**: Snow bridge.*

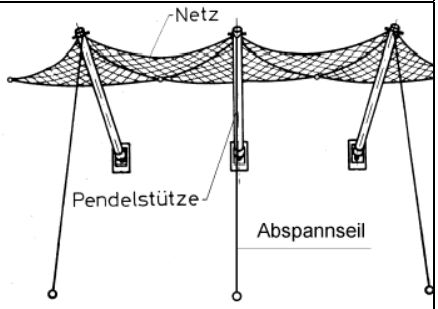

***Figure 65**: Snow net.*

Description:	Supporting structures comprise two major types of design: (1) rigid structures that sustain only small elastic deformation such as snow rakes (Figure 63, upright crossbeams, so-called rafters), and snow bridges (Figure 64, horizontal crossbeams, so-called bars); (2) rocking structures that are capable to support a limited deflection such as snow nets (Figure 65). Functionally, there are no differences between both designs. Both are erected more or less perpendicular to the slope and are well anchored in the ground. They act as barriers against creeping and gliding motions. A back-pressure zone extends uphill from the structure for a slope distance of at least three times the vertically measured snow height. This means that motions are diminished down-slope toward the obstacle. Within the back-pressure zone, additional pressure stresses are produced in the snowpack. These pressure stresses are parallel to the slope and depend primarily upon gliding. The supporting structure is able to withstand all such stresses. In this way the shear and tensile stresses that produced slab avalanches before the structures were built are reduced in the back-pressure zone of the structure. (photo gallery) (example)
Purpose:	Structures intended to support, sustain, or retain the snow cover in place and to prevent it from sliding downhill. Supporting structures have to bear mainly static loading resulting from the creeping, gliding, and settling snowpack. They are built in the starting zone of the avalanche (release area). Snow masses already in motion cannot be stopped by supporting structures as they are to weak to withstand large dynamic forces. However, they are designed to withstand small slides and sluffs. (advantages) (drawbacks) (scopes)
Main design criteria:	Type of structure: the choice between rigid and rocking structures

	depends on the site specific conditions of snow, terrain, and foundation. Generally, snow nets are less damageable by creeping and gliding motions and rock fall. However, their anchoring on loose ground is more difficult than that of rigid structures. Height of structures: depends on the expected maximal snow depth at the site of the structure. Foundation of structures: in general, two separate foundations are used for this type of structure. One is an upper or uphill foundation (so-called beam/ground foundation), the other is a lower or downhill foundation (so-called support foundation). Arrangement of structures: depends on the release area, the slope angle, and the distance between rows of structures (which again depend on the average and the maximal snow cover). Static/Dynamic design criteria: Static design criteria of supporting structures have to warrant the withstand against snow loading. Supporting structures are not designed to withstand large dynamic forces. Thus, only slides and sluffs have to be considered for dynamic design criteria.
Materials:	Snow fences and snow bridges are constructed in steel. The anchoring of the structures requires concrete fillings. For the foundation of the structures on regular soil anchors, micro piles, base plates, and concrete fundaments are in use. Brittle fracture: One major requirement is the reliability of the chosen steel quality against brittle fracture. Special attention has to be given to welding sensitive devices, large thickness of sheeting plates, cold forming, residual stress, etc.
Costs analysis:	This evaluation of the costs is based on the prices list of the own country.
References:	

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Example:	**Defining optimal countermeasures:** 1. Hazard analysis: – estimation of the expected maximum snow depth H_{ext} for hazard scenarios H_{30}, H_{100}, and H_{300} with recurrence periods of 30, 100, and 300 years; – estimation of the avalanche run-out area for each hazard scenario using a numerical avalanche modeling program such as AVAL-1D (Christen et al. 2002). 2. Risk analysis: – statistical calculation of the recurrence of the expected extreme snow depth H_{ext}; – estimation of potential damage (identification of endangered values; vulnerability analysis; temporal and spatial coincidence of avalanche event and presence of mobile values); – calculation of the initial risk R_0; – analysis of potential protection scenarios and ex-ante calculation of the residual risk R_1. 3. Risk evaluation: – determination of a cost-risk function for each protection scenario; – determination of a benefit-risk function for each protection scenario; – calculation of the net benefit for each scenario; – selection of the maximum net benefit scenario. **Implementation of the selected protection scenario:** 4. Identification of the main design criteria – Determination of the effective height of the structure D_K; – determination of the number and arrangement of supporting structures; – determination of the distance between the rows of supporting structures; – determination of the anchoring type. 5. Detailed cost and time calculation. 6. Detailed construction plan: – detailed pegging out of supporting structures at the site; – commencement of work at building site. 7. Construction phase: – site engineering; – control of achievements.

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Advantages:	– Large experience with design criteria of avalanche supporting structures (see Margreth, 2007); – large field experience with implementation under different conditions; – low maintenance and no operating costs.

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Drawbacks:	– Limited appropriateness for very large release areas -> uncertain effectiveness; – expensive when applied to large release areas; – negative impact on the beauty of scenery (external costs of supporting structures).

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Scopes:	– As shown by Margreth & Romang (2006), one major scope in the field of permanent supporting structure is the determination of their effectiveness and the implications on hazard mapping deriving thereof; – namely discussed is the determinability of the effectiveness, which so far has been assumed by using a default value. This might not be justified as site and project specific conditions greatly impact the effectiveness of PSS; – also controversially is the question, if the construction of PSS should be reflected in the hazard map. Conditionally, that risk mitigation is included into hazard mapping, there arise a bench of questions concerning the financing of mitigation (e.g., PSS might lead to a downgrade of the hazard zone attributed to a specific land. This land would yield higher prices, and thus the private benefitor should contribute proportionally to his gain to the financing of PSS.

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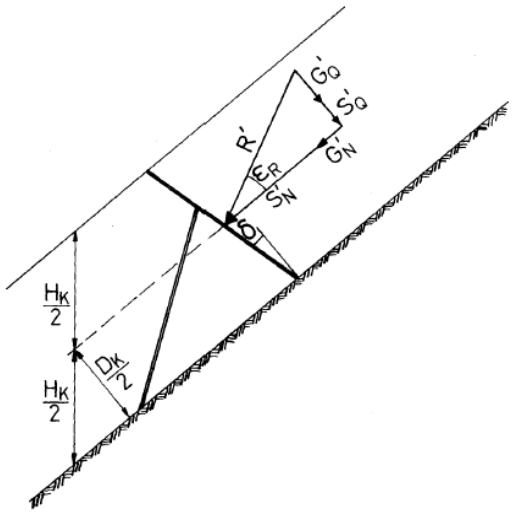
Specific design criteria:	**Snow loading on the supporting structure:** Component of snow pressure parallel to the slope: $S'_N = H_K^2 N f_c \ [kN/m']$ with H_K = vertical height of the structure; N = glide factor depending on the roughness of the ground surface (for determination see Margreth 2006); f_C = altitude factor expressing the general increase in snow density with altitude (for determination see Margreth 2006). Component of snow pressure perpendicular to the slope: $S'_Q = S'_N \cdot (a / N \tan\psi) \ [kN/m']$ with ψ = slope angle; choose α = 0.35 or α = 0.5 whichever gives the most unfavorable load for the particular part being considered. Component of the weight of the snow prism for a rigid supporting plane, allowing for a higher snow density near the supporting plane: $G' = 1.50 D_K^2 \tan\delta \ [kN/m']$ with D_K = effective height of the structure [m], whereby $D_K = H_K \cos\psi$; δ = angle between the supporting plane and the slope distance. End-effect forces (additional forces per unit length of the line of structures operating parallel to the slope): $S'_R = f_R S'_N$ [kN/m'] with $f_R = (0.92 + 0.65 \cdot N) \cdot A/2 \leq (1.00 + 1.25 \cdot N)$ being the end-effect factor. Resultant of all snow pressures are derived by vectorial addition of the above components (Figure 66): $R' = \sqrt{(S'_N + G'_N + S'_R)^2 / (S'_Q + G'_Q)^2}$ [kN/m'] which is applied in direction $\tan\varepsilon R = R'_Q / R'_N$ whereby ε_R = angle between the resultant and the parallel to the slope. Overall design of the structure has to acknowledge the resultant and should be completed in accordance to SIA norms 261, 262, 263 (SIA 261, 262, 263: 2003).  ***Figure 66**: The resultant of the snow pressures.*

Transverse loading on posts due to accumulation of snow:

This pressure is relatively small and may be expressed by means of a regularly distributed linear force:

$$q_S = \eta\, H_K{}^2 N\, f_C \frac{\text{Diameter of post}}{\text{Length of post}} \text{ [kN/m']}$$

with η = efficiency in dependence on the gliding factor N (normally taken as 1.0); H_K = vertical height of the structure [m]; Diameter and length of post [m]. The direction of q_S is perpendicular to the axis of the post, and directed downhill. Posts with an asymmetric cross section should not be free to rotate about their long axis. The line of application of q_S goes through the center line of the post. Permanent weight of the structures is normally not considered.

Snow loading parallel to the supporting plane:

As the state of nature of the distribution of pressures among the supporting plane is often unstable, sterner assumptions for specific loading on the elements of the plan have to be applied. The Figure 67 shows the basic load as $P' = R'\cos(\delta - \varepsilon_R)$ [kN/m'] with P´ = component of R´ vertical to the plane; ε_R = angle between the resultant and the parallel to the slope. Consequently, the specific transverse loading parallel to the supporting plane is $p_h = P'\cos\delta/(0.77 D_K) = P'/(0.77 B_K)$ [kN/m'] and the linear transverse loading acting upon a bar is $p'_B = p'_h \cdot b$ [kN/m'] with b = width of loading (width of bar plus share of the neighboring clearance).

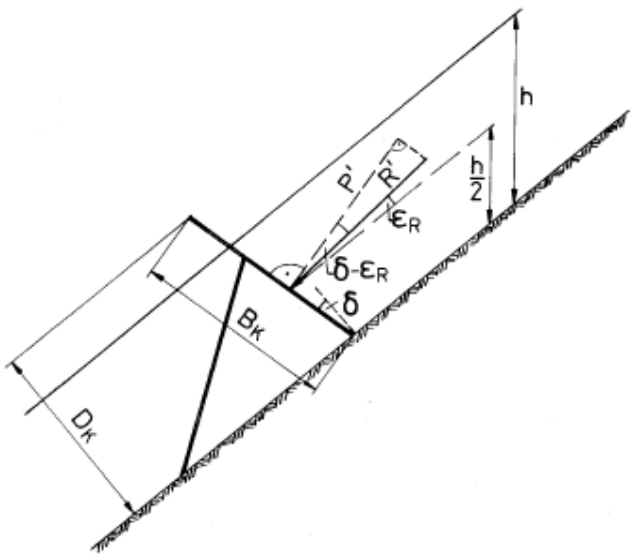


***Figure 67**: Loading parallel to the supporting plane.*

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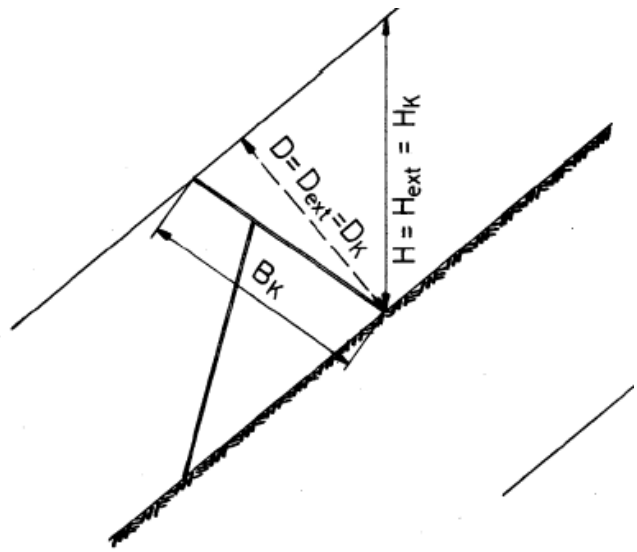
Height of structures:	The vertical height of the structure H_K is defined as the mean vertical distance between the upper border of the supporting plane and the ground. The vertical height of the structures H_K must correspond to the expected extreme snow depth H_{ext} at the location of the structure: $H_K \geq H_{ext}$ [m]. This fundamental requisite must be recognized for effective protection against avalanches during catastrophic events and for the design of the structures. It should be noted that the structures, depending on their features and on site-specific wind conditions, may themselves considerably influence snow depositions. The slant height of the structure B_K is defined as the mean height of the supporting plane measured parallel to the supporting plane of the structure. The lower limit is formed by the ground (Figure 68). The effective height of the structure D_K is defined as the mean distance between the upper border of the supporting plane and the ground measured perpendicular to the slope, which is analogous to the thickness of the snow (Figure 68).  ***Figure 68:*** *Slant height of the supporting plane.*

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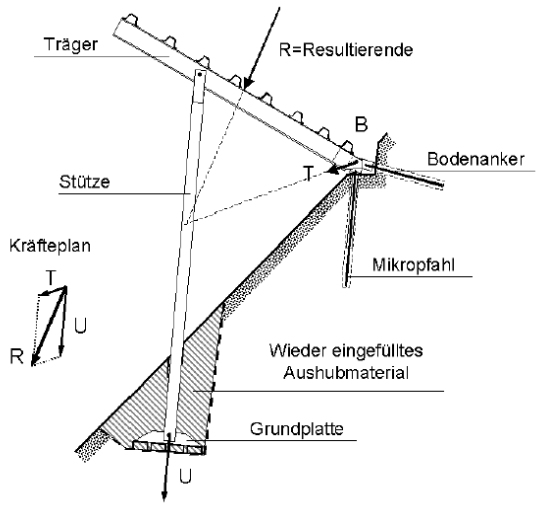
Foundation of structures:	One task of the project manager is to thoroughly analyze the ground conditions for foundations (SIA 260: 2003). Thereby, he has to consider: – geological ground conditions (thickness of ground, nature and joint density of the ground, nature of overlap, humidity and frost conditions, solifluction, soil chemistry); – determination of the ground resistance (e.g. by anchor pulling test); – choice of the structure type: as they have different requirements for the foundation, foundation conditions have to be analyzed before this choice (e.g. by tentative anchor); – type of foundation. There are two types of foundation normally used on solid soil (Figure 69 & Figure 70). Depending on the type of loadings described above, ground anchors, micro piles, base plates, and/or concrete fundaments are used to fix the structures on the soil. In general, two separate foundations are used for this type of structure. One is an upper or uphill foundation (so-called beam/ground foundation), the other is a lower or downhill foundation (so-called support foundation). In case of unstable or phreatic ground, it is advisable to connect the upper and lower foundations by a member called a pressure bar, which accepts tensile and compression stress. In these special cases SIA norm 267 should be consulted (SIA 267: 2003).  ***Figure 69**: Structure with separated foundations. The forces impacting the structures are represented assuming a post jointed on both sides and girder pivotable in B. The supporting foundation consists of a base plate, while the bearing foundation is based on a combination of a horizontal soil anchor and a vertical micro pile.*

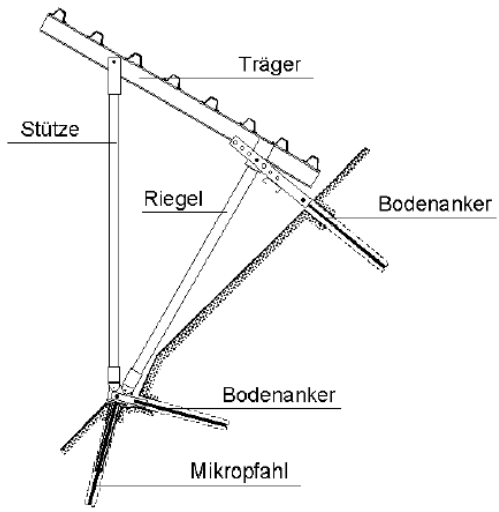


***Figure 70**: Structure with waling, where the down-slope foundation bases on a combination of a horizontal soil anchor and a vertical micro pile, while the up-slope foundation is based only on a soil anchor perpendicular to the slope.*

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Arrangement of structures:	Additionally to the overall requirements of supporting structures, the slope distance between structures and lines of structures has to be designed fulfilling three conditions: (1) the structures must not be damaged by maximal snow load; (2) dynamic stress by snow creeping and drift has to be accepted; (3) the velocity of snow creeping and drift in between the structures must not exceed a defined threshold. That way, the momentum responsible for damages below the structures is limited to moderate scale. The distances between the rows of structures parallel to the slope is given by $L = f_L \cdot H_K$ [m] with the distance factor f_L depending on the friction angle φ between soil and snow, the glide factor N, and the height of the structures H_K. Figure ***71*** shows f_L (φ, H_K) and allows to derive the standard measure for f_L. to acknowledge for soil surfaces and special safety demands, the following benchmarks are valid: $N > 2 \Rightarrow \tan\varphi = 0.50 - 0.55; \quad N < 2 \Rightarrow \tan\varphi = 0.60$. Some additional requirements: – maximal values to define the distance factor f_L are $\tan\varphi = 0.6 \cap N > 1.3 \Rightarrow f_L = 13$; – if the structures are designed for N = 1.2, then the distance factor must not exceed the respective curve in – Figure ***71***; – if the perpendicular height of the structures H_K is ≥ 4.5 m, the highest values of f_L must not exceed the respective curve in – Figure 71; – the margin in the determination of distances should be used to adapt the supporting structures to site specific requirements; – climatic conditions have to be considered in the determination of distances in order to guarantee the reliability of structures. On north-faced slopes and in regions with high annual precipitation (e.g. the Swiss pre-Alps) values for tan φ may be even smaller than 0.50; – if the slope angle between two rows of structures changes, one should use the slope angle ψ between the two root points of the rows of structures to the distance L.

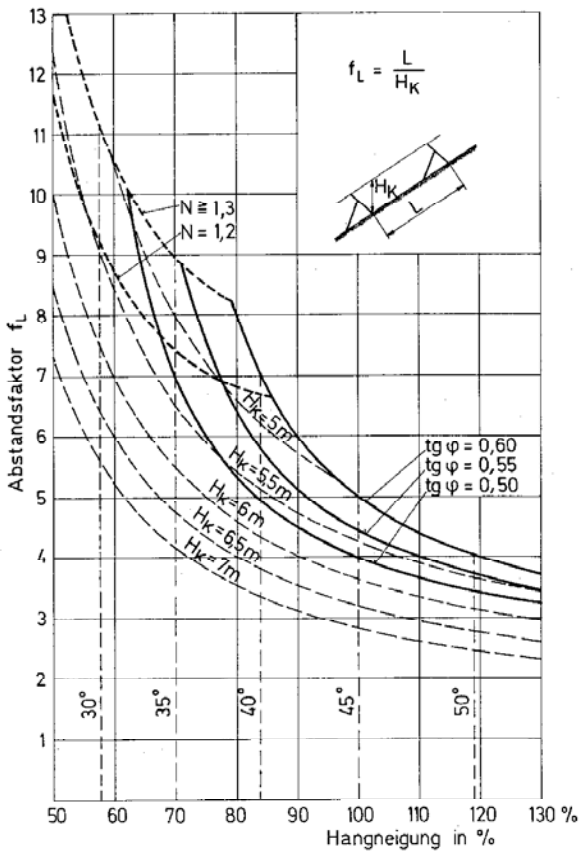

***Figure 71**: Distance factors.*

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Costs analysis:

Investment costs	Operating costs	Maintenance costs	Costs of repair	Life expectancy
IC [€/m']	OC [% of IC]	MC [%of IC]	RC [%of IC]	n [years]
1'000	0	0.25	0.25	70

* Risk Reduction used for the calculation of the residual risks

***Table 19**: Benchmarks for overall costs of permanent supporting structures.*

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Design-specific References:	Christen, M., Bartlet, P., and Gruber, U. (2002) AVAL-1D: An avalanche dynamics program for the practice. Proceedings of the International Congress Interpraevent 2002 in the Pacific Rim, Matsumoto, Japan, 715-725. Margreth, S. (2007) Lawinenverbau im Anbruchgebiet. Technische Richtlinie als Vollzugshilfe [Technical guidelines for avalanche control in the starting zone] Bundesamt für Umwelt Bern, WSL Eidg. Institut für Schnee- und Lawinenforschung Davos (eds.). Margreth, S. & Romang, H. (2006, forthcoming) Consideration of avalanche defense measures in hazard maps: a great challenge for risk management. Proceedings IDRC, Davos. Margreth, S. (1996) Experiences on the use and the effectiveness of permanent supporting structures in Switzerland. Proceedings ISSW, Banff, 233-238. SIA (2003) SIA Norm 260: Grundlagen der Projektierung von Tragwerken. [Projecting of structures] Zürich: Schweizerischer Ingenieur- und Architektenverein. SIA (2003) SIA Norm 261: Einwirkungen auf Tragwerke. [Loading of structures] Zürich: Schweizerischer Ingenieur- und Architektenverein. SIA (2003) SIA Norm 262: Betonbau. [Concrete structures] Zürich: Schweizerischer Ingenieur- und Architektenverein. SIA (2003) SIA Norm 263: Stahlbau. [Steel structures] Zürich: Schweizerischer Ingenieur- und Architektenverein. SIA (2003) SIA Norm 267: Geotechnik. [Geotechnics] Zürich: Schweizerischer Ingenieur- und Architektenverein. Wilhelm, C. (1999). Kosten-Wirksamkeit von Lawinenschutz-Massnahmen an Verkehrsachsen. Vorgehen, Beispiele und Grundlagen der Projektevaluation. [Cost-effectiveness of avalanche mitigation on roads. Methodology, examples and fundaments of project evaluation] BUWAL, Bern.

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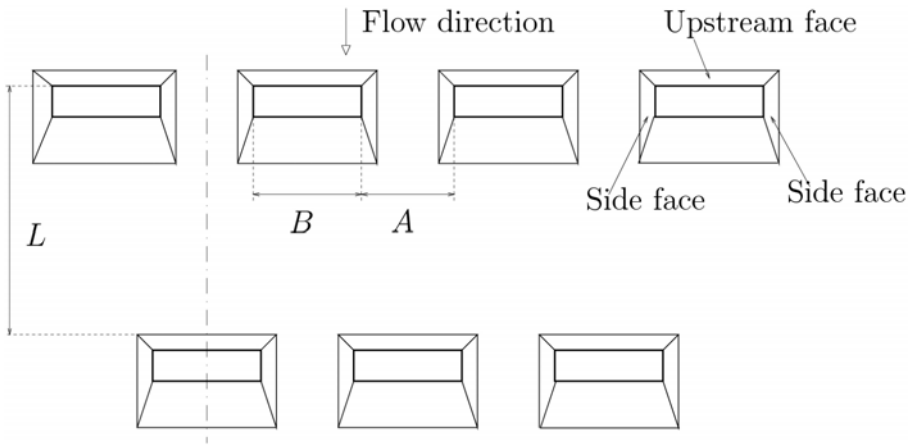
	4.2.6 Catching and deflecting dams, braking structures
	
Specific design criteria	Design height depends on the velocity and flowing height of the design avalanche, the deflection angle of the structure to the flow direction and the steepness of the wall facing the avalanche (see Annex A2). There must be sufficient space above a dam to store the volume of snow corresponding to the tongue of the design avalanche successfully stopped by the dam. For earth dams stability against avalanche impact is usually not an issue. However, for special construction build of concrete or steel impact force need to be regarded.
Layout breaking structures	Recommendations regarding the geometry and layout of the mounds  ***Figure 72**: Plan view of two staggered rows of braking mounds.*

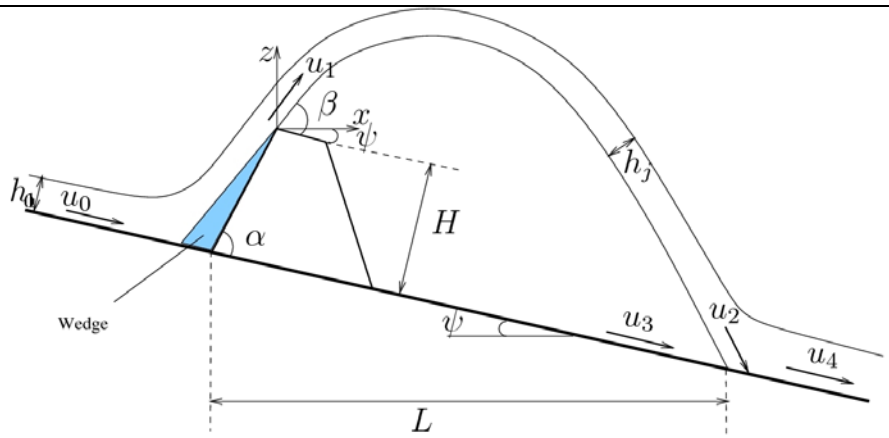


***Figure 73**: Schematic diagram of a jet of length L with upstream flow thickness h and jet thickness h_j.*

Symbols used in the following:

upstream flow height	h_0
upstream velocity	u_0
throw speed	u_1
jet thickness	h_j
upstream slope angle	ψ
mound slope	α
throw angle	β
design mound height	H
mound length	B
lateral separation distance	A
longitudinal separation / throw length	L
depth of snow deposit	h_s

Chute experiments with granular materials lead to the following recommendations for the geometry of avalanche braking mounds.

1. The **height of the mounds**, ($H–h_s$), above the snow cover should be 2–3 times the thickness of the dense core of the avalanche, h_0. Increasing the height of the mounds beyond this, for a fixed width of the mounds, does not significantly reduce the run-out according to the experiments.
2. The upstream face of the mounds should be steep. For the chute experiments with glass beads (ballotini), $\alpha \approx 60°$ was sufficient since a steeper upstream face only marginally improved the energy dissipation. This result may not be appropriate for natural snow avalanches because of the different physical properties of the

materials.

3. The **aspect ratio** of the mounds above snow cover, $(H–h_s)/B$, should be chosen close to 1.
4. The **lateral separation distance** A should be similar to or shorter than B. ‘
5. The mounds should be placed close together with steep side faces, so that jets launched sideways from adjacent mounds will interact. Many short mounds were found to be more effective than fewer and wider mounds for the same area of the flow path covered by mounds.
6. If there is sufficient space in the terrain for a second row, it should be staggered with respect to the first row.
7. The spacing, L, between the rows must be chosen sufficiently long so that the material launched from the mounds does not jump over structures farther down the slope.

The throw length L can be estimate by solving the system of differential equations:

$$\ddot{x} = -\frac{f}{h_j}\dot{x}\sqrt{\dot{x}^2 + \dot{z}^2}$$

$$\ddot{z} = -g - \frac{f}{h_j}\dot{z}\sqrt{\dot{x}^2 + \dot{z}^2}$$

where

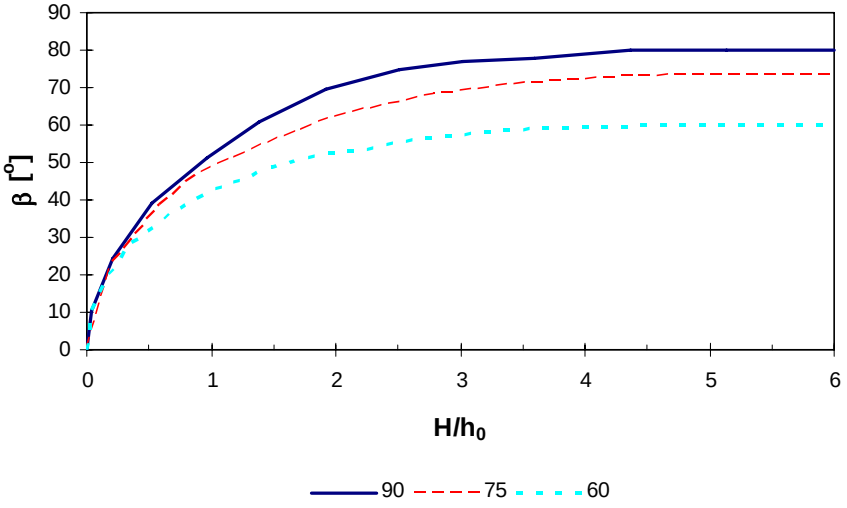
$$\dot{x}(0) = u_1 \cos(\beta - \psi)$$

$$\dot{z}(0) = u_1 \sin(\beta - \psi)$$

and

$$u_1 = k\sqrt{u_0^2 - 2gH\cos\psi}$$

For natural snow avalanches, it is recommended that the throw length is computed for the three values k = 0.7, 0.8 and 0.9 and that the result for k = 0.8 be used to calculate the minimum distance between a row of mounds and the next retarding or retaining structures below. Recommended value for f/h_j is $0.004m^{-1}$ in computations of the throw length. The recommended choice for the throw angle β is given in the figure for three slope angles α.

	
Geotechnical design	Ground investigations must be accomplished to ensure that the soils are usable for the construction and that the stability of the underlying ground is sufficient. All kinds of loose materials, from clay, silt, sand, gravel and rocks may in principle be used for the construction of a dam. In fine-grained, cohesive materials as clay and silt, the drainage of water is a very slow process. Pore pressures might build up during the construction phase or later during heavy rainfall and reduce the stability of the dam. A rule of thumb says that if more than 10 % of the dam fill consists of fine-grained material one has to make extra precautions in the construction to ensure that the water drainage from the body of the dam is sufficient, to obtain satisfactory stability of the dam. This induces extra costs for the construction. It is therefore a clear advantage to use coarse grained, frictional materials as gravel and rocks for dams made of loose deposits.
Dam geometry design	Fine-grained cohesive materials will not be stable with inclinations steeper than 1: 2. For friction materials as sand and gravel, the maximum steepness of the dam sides should not exceed 1: 1.5 (34°) to obtain satisfactory stability. For coarser frictional materials one can obtain a stable inclination of the dam sides up to 1: 1.25 (39°). The available space for the construction will often play an important role on the design. If enough space is available, earth fills are usually preferred. Otherwise, steeper dams must be used. In steeper dams one should use dry walls, reinforced earth or concrete. The steeper inclination is a clear advantage for the stopping and deflecting effect.
Drainage design	If water occurs in the excavation, precautions must be taken to keep the construction masses as dry as possible as water soaked masses are difficult to handle. Because of the high water content in such masses, the angle of repose is lower and it will be difficult to obtain the designed inclinations of the fill during the construction. Water built into the dam will reduce the stability of the dam also. Usually, the water content in the

	dam is at the highest during the construction, especially if much fine grained soils are used. The construction period is therefore often a critical phase for the stability of the dam. Surface streams and brooks must be diverted from the dam area. If possible, the flowing water should be directed around the dam along the base of the upper fill, or kept completely away from the dam area. If necessary, one could lead the water under the dam in culverts, but there is always a possibility that such culverts may be blocked, either by avalanche snow or earth materials. If a possibility exists for water build up behind a dam, the dam should be designed for hydrostatic pressures. Some countries have special regulations concerning this problem. The weight of the dam itself may block natural drainage channels in the ground and force groundwater upwards into the dam itself and by this reduce the stability of the dam itself. A high ground water table can be avoided by making ditches under the base of the dam to ensure sufficient drainage. In addition, the bottom layer of the dam should always be built by self-draining materials. Both the dam sides and the sides of the cut should be protected against water erosion. This could be done by use of different kinds of vegetation or geotextiles to stabilise the surface. Water courses in the dam area must be protected against erosion by (stones, boulders, etc.) unless the water flows on the bedrock itself.
Advantages earth fills	The advantages of using natural loose deposits for the construction are mainly: • materials are often at hand • natural loose deposits are cheaper than other materials • maintenance costs are low the appearance of nature like constructions are more easily accepted by the public at the visual impact is less than for an artificial structure such as a concrete dam.
Disadvantages earth fills	Disadvantages of dams made purely of loose deposits are many: • dams require much space. A 15 m high dam with inclination 1:1,5 on horizontal ground is 45 m wide at the base, plus the width of the dam crown (2-4 m). When the excavation area is included one needs at least about 100 m for the construction. As dams are built in the runout zone, the terrain is often sloping, and with increasing terrain inclination the lower fill will rapidly increase in width and volume. • the volume of a dam is roughly proportional to: $h^2 \cdot \cot g\alpha$, per unit length, where h is the vertical dam height and α the inclination of the dam sides. As can be seen, the volume increases rapidly with the dam height. Although unit prices per m^3 will decrease with the volume of the dam, high dams with natural inclination of the dam sides will be costly.

	• by using earth materials it is difficult to obtain steep enough dam sides, and they are therefore less effective than dams made of concrete or reinforced earth • for deflecting dams in steep terrain, the effective inclination of the dam sides (measured perpendicular to the dam axis) will decrease with increasing terrain inclination. The angle of repose will in such cases be found along a plane between the direction of the cross section and the longitudinal dam axis.
Effectiveness	Large avalanches flowing at high speed can hardly be stopped by dams and there are many examples of avalanches overtopping such dams. The effectiveness of catching dams is therefore dependent upon a location near the end of the run-out zone of the avalanches. Usually deflecting dams have higher efficiency than catching dams. The effectiveness will to a large degree also be dependent on the steepness of the wall facing the avalanche. Usually, walls steeper than approximately 45° are preferable. There is not much observation evidence for the effectiveness of braking mounds from natural avalanches, but laboratory experiments with granular materials indicate that they can reduce the speed and run-out distance of avalanches.
Cost	The cost will mainly depend on: • Volume of fill • Geotechnical quality of the loose deposits in the construction area • Transport distance of fill material The lowest cost will be when the quality of the loose deposits in the construction area is good. In such cases in place material can be used as fill, and by lowering the terrain upslope from the dam, the fill height can be reduced. Typical cost range for earth walls will be € 5-15 per cubic meter fill. If fill material has to be transported over long distances the unit cost can be substantially higher. Steeper walls will be more costly, and dry walls roughly will cost € 100 per square meter, concrete € 200 per square meter (typical costs in Norway). The maintenance cost for earth fills are usually small, unless damage to the fill.

Design-specific References:	SATSIE (2006) Deliverable D14: The design of avalanche protection dams. Recent practical and theoretical developments. Draft version. Edited by T. Jóhannesson (IMOR). Contributions by U. Domaas, P. Gauer, C. B. Harbitz, K. Lied (NGI), K. M. Hákonardóttir, T. Jóhannesson (IMOR), C. J. Keylock (SGUL), L. Rammer (AIATR), A. Bouchet, T. Faug, F. Naaim-Bouvet, M. Naaim (Cemagref-ETNA), M. Barbolini, and M. Pagliardi (UP-DIIA).

	Margreth, S. 2004. Avalanche control structures, SLF, Pôle Grenoblois d'Etudes et de Recherche pour la Prévention des Risques Naturels. UEE session 2004: Avalanches: Risque, zonage et protections.

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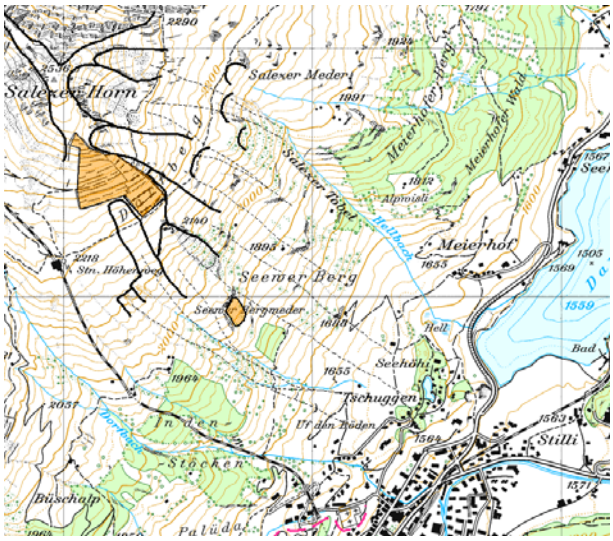
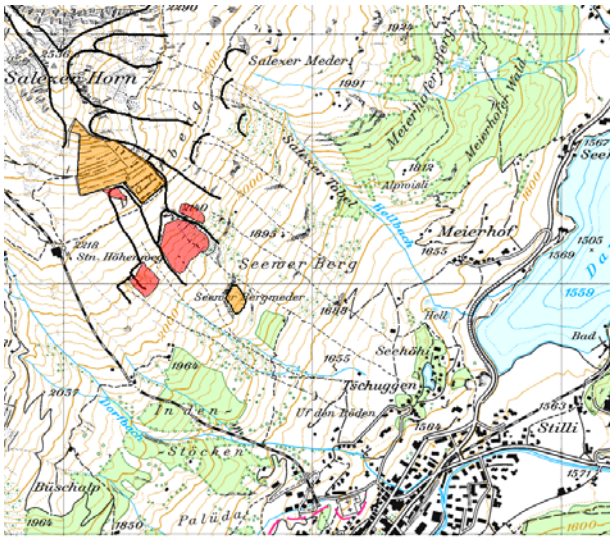
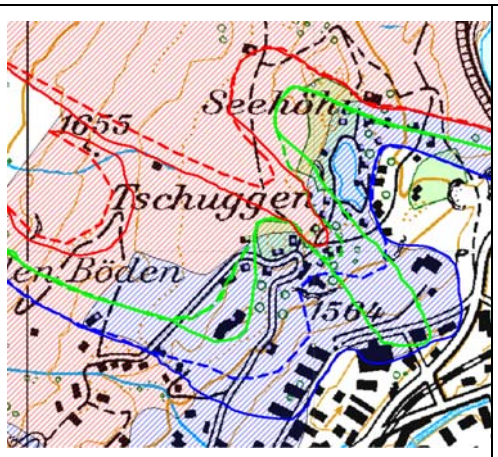
	4.3 Case Study Snow Avalanches	
System under analysis	The system analyzed in this case study is the “Dorfberg”, an avalanche prone residential area in Davos, Switzerland (Fig. 4.6). This area is endangered by a large avalanche track having a recurrence period of approximately 40 years (Bischof et al. 2006). In order to protect people and infrastructure beneath the avalanche track, about 11 M€ (in 2005€) have been invested within the last 50 years in Permanent Supporting Structures (Fuchs & McAlpin 2005). However, there remains a residual risk to people living in the endangered area as only a part of the release area has been sheeted. This residual risk is managed by evacuating the area as soon as snow heights become critically.	
Figure 4.6		The “Dorfberg” avalanche site in Davos, Switzerland: Orange areas are protected by snow rakes.
Hazard process	To estimate the hazard potential of the “Dorfberg” avalanche, the existing hazard map of the municipality of Davos, the local avalanche cadastre and the SLF avalanche database were used. According to the Swiss practice (Wilhelm 1997), three scenarios representing recurrence periods of 30, 100, and 300 years were separated (Fig. 4.7).	

Figure 4.7		Run-out area of the "Dorfberg" avalanche. The Red line marks the 30yr. event; the green line marks the 100 yr. Event; the blue line marks the 300 yr. event.
Values at risk	In order to calculate the damage potential in the area at risk, data about the building structure, the insured values, the number of persons living in endangered buildings, and traffic routes were used. 48 buildings are located within the hazard zone; Buildings were assigned to 5 classes of vulnerability to avalanches; The overall insured value of the buildings was assessed at 40.8 M€; On average, approximately 80 persons reside inside the run-out area; Approximately 180m of the main road and 850m of side roads are inside the run-out area.	
Calculation of the initial risk R_0	Calculation of the initial risk R_0 using the approach by Borter (1999): (4.2) $r_{ij} = P(o_j) \cdot P(e_i) \cdot f(I_j, v_i) \cdot C_i$ [1/year] where: r_{ij}: risk of object i to avalanche scenario j $P(o_j)$: occurrence probability of avalanche scenario j $P(e_i)$: exposure probability of object i $f(I_j,v_i)$: function of loss dependent on the intensity of j and the vulnerability of i C_i: consequences of a loss of i (measured either in natural or in monetary units) Equation (1) yields the risk for one risk category (humans, private property, infrastructure) and one event scenario. The total risk for the run-out area can be obtained by summing up different risk categories (persons	

	outdoor, indoor, in cars, buildings) and different event scenarios (30, 100, 300y.) as expressed by equation (2). (4.3) $R = \sum_{i=1}^{n}\sum_{j=1}^{m} r_{ij}$ where n = risk categories and m = event scenarios Using Equation (1) and (2) we obtain: initial risk to buildings: 0.222 destroyed buildings/year or 55'850 €/year (2006 €); initial risk to persons (∑ outdoor, in cars, in buildings): 0.0529 Fatalities/year.
	4.2.2 Impacts of mitigation measures
Choice of measures	In order to mitigate the avalanche risk at the "Dorfberg"-avalanche run-out area, installation of additional 1300 m' of PSS is proposed (Fig. 4.8). Alternatively, one could build check dams protecting the endangered buildings. Rough estimations of cost and risk reduction showed that PSS outperform check dams (Bischof et al. 2006).
Figure 4.8	 The "Dorfberg" avalanche site in Davos, Switzerland: Orange areas are protected by snow rakes; red areas mark the projected PSS.
Mode of operation	PSS are intended to support, sustain, or retain the snow cover in place and to prevent it from sliding downhill. They are erected more or less perpendicular to the slope and well anchored in the ground in order to act as barriers against creeping and gliding motions. A back-pressure zone extends uphill from the structure for a slope distance of at least three times the vertically measured snow height. This means that motions are diminished down-slope toward the obstacle. Within the back-pressure zone, additional pressure stresses are produced in the snowpack. These pressure stresses are parallel to the slope and depend primarily upon

	gliding. The supporting structure is able to withstand all such stresses. In this way the shear and tensile stresses that produced slab avalanches before the structures were built are reduced in the back-pressure zone of the structure.
Design criteria	Main design criteria of PSS are: Type of structure: the choice between rigid and rocking structures depends on the site-specific conditions of snow, terrain, and foundation. Generally, snow nets are less damageable by creeping and gliding motions and rock fall. However, their anchoring on loose ground is more difficult than that of rigid structures; Height of structures: depends on the expected maximal snow depth at the site of the structure; Foundation of structures: in general, two separate foundations are used for this type of structure. One is an upper or uphill foundation (so-called beam/ground foundation), the other is a lower or downhill foundation (so-called support foundation); Arrangement of structures: depends on the release area, the slope angle, and the distance between rows of structures (which again depend on the average and the maximal snow cover); Static/dynamic design criteria: Static design criteria of supporting structures have to warrant withstand against snow loading. In Switzerland, supporting structures are designed for the maximum snow load within a return period of 100 years. However, they are not designed to withstand large dynamic forces. Thus, only slides and sluffs have to be considered for dynamic design criteria; Design criteria for PSS are briefly summarized later. For a detailed technical discussion see Margreth (2006).
Discounted costs	According to Wilhelm (1999), the following proxy costs can be used: Investment costs [IC]: 1300 m' of PSS with DK 4.0m = 1'300€/m' or 1.6 M€; Maintenance and repair costs [MRC]: *n* *(1% of [IC]); Discount factor [*r*]: 2% as used for mitigation projects in Switzerland; Expected life time [*n*]: 80 years

	Total discounted costs per year: $(\mathrm{IC} + n\mathrm{MRC}) \cdot \frac{r \cdot (1+r)^n}{(1+r)^n - 1} = 72.5$ K€
	4.2.3 Determination of the risk reduction
Hazard mitigation impact	The impact analysis of PSS, considers three scenarios (Margreth & Romang 2006): Avalanche release outside of the sheeted area: nearby PSS impact non-sheeted areas as the fracture width and the avalanche volume are significantly reduced leading to shorter avalanche run outs. Avalanche release in the sheeted area: Given that the structure height is chosen correctly and most of the release area is sheeted, these releases are small as the avalanche volume and the flow rate are much reduced by PSS. Avalanche release over the filled up structures: Over-snowing depends on the structure height (designed on return periods of 100 years) and the expected extreme snow depth. In rare situations (return period > 100 years), releases over the filled up structures are possible, but the fracture depth and consequently the avalanche volume is much reduced compared to non-sheeted areas. Using avalanche dynamics models, the reduced run out distances of the avalanche can be calculated based on these scenarios. However, uncertainties of modeling (snow height, secondary starting zones, friction coefficients) have to be considered when calculating the mitigation impact of PSS. The impact analysis of the projected PSS (Fig. 4.9) reveals that the greatest loss reduction can be expected for the 300y. scenario, in which 11 buildings are not any more within the hazard perimeter. For the 30y. and 100y. scenarios, the impact of the project is far less.

Figure 4.9		The potential run-out reduction due to the projected PSS: the dotted lines mark the new scenarios for the, 30, 100, and 300 yr. event, while the solid lines mark the initial scenarios.
Calculation of the residual risk R_1:	Calculation of the residual risk R_1 according to Equation (4.2) and (4.3): Residual risk to buildings: 0.132 destroyed buildings/year or 22'932 €/year; Residual risk to persons (∑ outdoor, in cars, in buildings): 0.0355 fatalities/year.	
Derivation of the effective risk reduction:	If the reliability of PSS would be equal to 1 (P_f (PSS)= 0), the risk reduction of the projected PSS could be easily obtained by: (4.4) $RR_{general} = R_0 - R_1$ However, every structure bears the risk of failure due to inappropriate design, material defect, or excessive load. As PSS function on the basis of parallel systems where more than on element must fail to cause the system's overall failure, failure of an individual structure does generally not lead to an avalanche. If, however, the design is inappropriate or the snow event is extreme, the height of structure might be smaller than the maximal snow height and PSS can be over-snowed leading to increased risk. Further, Barbolini et al. (2002) showed that the modeling of avalanches imply run-out estimation errors of +/- 30 m, giving rise to the assumption that the effectiveness of PSS is lower than 100%; in other words PSS do not perform 100%. To adjust for these reasons, equation (4.4) is multiplied by a reliability factor Z (see equation 4.5). (4.5) $RR_{real} = (R_0 - R_1) \times Z$ where $Z = 1 - P_f(\text{PSS})$ In Schneider (1997), theoretical derivation of the failure probability P_f is described. Practical experience shows that under best-practice design and implementation, P_f (PSS) is between 5-10% (Margreth, personal communication). For the cost/effectiveness analysis, we thus used $RR_{real} = 0.9 \times RR_{general}$.	

	4.2.4 Cost/Effectiveness ratio
Cost/Effective-ness ratio	The cost analysis has shown that the annual cost of the projected PSS amounts to 72.5 K€. The residual risk analysis has shown that the projected PSS is expected to avert on average 0.021 fatalities, 0.081 house damages, and 29.6 K€ damages per year. Thus, this project has a: C/E ratio of 3.45 M€/life saved; C/E ratio of 0.9 M€/house saved.
Transformation into Cost/benefit ratio	In this case study, we additionally assessed the insurance value of each building in the hazard zone to calculate a B/C ratio of 0.41 for the protection of buildings (dividing the yearly-expected prevention of damages by the annual costs of the projected PSS). Alternatively, one can transform C/E ratios into C/B ratios, when market prices to valuate the averted losses are available. Therefore, we can also obtain the B/C ratio for the averted damages to buildings by 1) estimating the mean market price of affected buildings, 2) insert this estimation as default for the value of a saved house into the C/E, and 3) invert this to obtain the B/C ratio. For the purpose of demonstration, we took the median value of the project's building portfolio to calculate a B/C ratio of 0.53, which is in the same order of magnitude as the detailed B/C ratio.
	4.2.5 Concluding remarks
General remarks	Conclusions drawn upon the presented case might be not trivialized as they are site specific. However, C/E ratios found at the "Dorfberg site" are in the range of other PSS projects in Switzerland (Wilhelm 1997).
Scope of further research	Further research on PSS has to be focused on (1) the determination of reliability and effectiveness of PSS; (2) the reduction in uncertainties, which impact the design of PSS.

References:	ASTRA (1994) Richtlinie Einwirkungen auf Lawinenschutzgalerien. EMDZ, Berne. Barbolini, M., Natale, L. & Savi, F. (2002) Effects of release conditions uncertainty on avalanche hazard mapping. Natural Hazards 25, 225-244. Bischof, N., Bruendl, M., Guler, A. & Stoffel, L. (2006) Integral Avalanche Risk Management – a case study from Davos, Switzerland. Proc. CSCE 1st Speciality Conference on Disaster Mitigation, Calgary, Canada. Borter, P. (1999). Risikoanalyse bei gravitativen Naturgefahren, [Risk analysis of mass movements] Bundesamt für Umwelt, Wald und Landschaft, Bern. Fuchs, S. & McAlpin, M.C. (2005) The net benefit of public expenditures on avalanche defence structures in the municipality of Davos, Switzerland. Natural Hazards and Earth System Sciences 5, 319-330. Margreth, S. (2007) Technische Richtlinie für den Lawinenverbau im Anbruchgebiet. [Technical guidelines for avalanche control in the starting zone] Bundesamt für Umwelt, Bern & Eidg. Institut für Schnee- und Lawinenforschung, Davos. Margreth, S. & Romang, H. (2006) Consideration of avalanche defense measures in hazard maps: a great challenge for risk management. Proc. IDRC, Davos, Switzerland. Schneider, J. (1997) Introduction to safety and reliability of structures, IABSE, Zürich. SIA (2003) Basis of Structural Design. Code 260, Swiss Society of Engineers and Architects SIA, Zurich. Wilhelm, C. (1997) Wirtschaftlichkeit im Lawinenschutz. Methodik und Erhebungen zur Beurteilung von Schutzmassnahmen mittels quantitativer Risikoanalyse und ökonomischer Bewertung, [Economics of avalanche protection. Methods and analyses to evaluate protection measures using quantitative risk analysis and economic evaluation techniques] Eidg. Institut für Schnee- und Lawinenforschung, Davos.

Chapter 5

NON-STRUCTURAL COUNTERMEASURES FORECASTING/OPERATIONAL METHODS

	5.1 Definitions
Monitoring:	Continuous observation of a system or measurement of characteristic parameters of a system over days, month and years.
Early warning:	Warning against dangerous natural hazard events 72 to 36 hours in advance. An early warning is targeted to safety services and professionals and is distributed via specific, mostly protected communication channels.
Warning:	Warning against dangerous natural hazard events more than 36 to 6 hours in advance. A warning is targeted to safety services and professionals and is distributed via specific, mostly protected communication channels.
Forecasting:	A prediction several hours or days in advance which describes the characteristic phenomena of an expected weather situation or weather related situation (e.g. avalanche situation). A forecasting is generally broadcasted to the public via many different communication channels.
Alert/Alarm:	An acoustic, optical or mechanical signal which informs about a dangerous event shortly before its occurrence. An alarm is targeted to the public.

	5.2 Debris Flow
General remarks	
Introduction:	Debris flows are generally developing very rapidly due to some triggering factors (see explanations in WP1). Since debris flows are generated in small catchments often as a consequence of local thunderstorms and local geomorphologic conditions, the possibilities for forecasting and warning are limited, because of the following reasons: – the time period for warning is very short, and the – process knowledge is low and the prediction of the local weather pattern is highly uncertain. Because of these limitations there is no general, national or regional forecasting for debris flows like for other hydrological processes, e.g. for large scale floods. Local predictions for debris flows are only possible with good local expert knowledge. These predictions can only provide a rough estimation whether debris flow could occur.
Data and models	
	Data which should be known to make a debris flow prediction are: – amount and location of erodable material; – water content of erodable material; – discharge; – slope angle and morphology of the terrain; – local precipitation intensity; – air temperature (altitude of 0°C). These data vary heavily from site to site and therefore difficult to measure. At the moment there is no national or regional measurement and observer network for forecasting of debris flows. However, there are some data available: – Network of the Federal Meteorological Office MeteoSwiss. – IMIS-Network, which has been established for national, regional and local avalanche forecasting. Some of these stations are equipped with precipitation sensors; – water depth gauges of the Federal Office for the Environment FOEN; – network for the measurement of debris of the working group for operational Hydrology (GHO); – several cantonal and local network for the measurement of precipitation. In Switzerland there is no operational model which is able to predict

	debris flow. There are some models available in the research community.
Operational forecasting and warning	
	Comparing to avalanche forecasting, there is no forecasting of debris flows. The most important prerequisite for debris flows are intensive precipitations, sufficient erodable material and a steep, channeled terrain (> 25 %). Rapid increases of air temperatures, a high zero degree line with the consequence of an accelerated snow melt are favorable factors. A very important basis for warning is the weather forecast and the Meteo-warning or Meteo-early warning of the Federal Meteorological Office MeteoSwiss. If the meteorological factors and the amount of erodable material in the starting zone is very well known, the probability for debris flows in a certain area can be roughly estimated by local experts. But this estimation is much more uncertain than it is e.g. for snow avalanches. In order to develop the basis for a forecasting system for debris flows and hydrological processes from small catchments, a research project has been started at WSL-SLF (IFKIS-Hydro). The goal of this project is to build up a similar organization, as it has been build up in the field of avalanche forecasting. First systems has been installed at three test sites in Switzerland. The main goal of this project can be summarized as follows: – establishment of a observer and measure network in several catchments; – definition of standards for measurements and observations; – coordination of existing networks for an optimal information of the safety services and fire brigades; – unified data storage and communication; – unified education courses; – additional research activities in the field of debris flow release. Because most of the processes which causes debris flows are not fully understood yet and the necessary data are not available, an operational forecasting system is not to be implemented within the next five to ten years.
Alarm systems	
Description and examples of alarm stations:	Since warning against debris flows is very difficult and uncertain, the timescale for safety measures is in the range „minutes to seconds“ which allows an alarm only. There are a number of alarm stations in use which release an alarm as soon as a debris flow has been triggered. These alarm stations are often combined with avalanche detection stations. For debris flow detection there are sensors available which combine an acoustic, a seismic and a pressure sensor. This sensor is typically integrated in an alarm station and allows the detection of a debris flow event or in winter of an avalanche. The sensor is most often installed at the edge of a debris flow or avalanche gully. Like alarm stations used for avalanches, these systems are foremost used

	to protect traffic lines. An example for such an alarm station in Switzerland is the "Ritigraben" station, which protects the cantonal road from the village Visp up to the Matter valley in the canton of Valais. The station consists of two seismic sensors in the starting zone, hard- and software for controlling and data analysis as well as for alarm triggering. When an alarm is triggered, an electronic signal switches two traffic lights to red and a telephone alarm is transmitted to the police station in Visp. After 15 minutes the traffic lights switches to a blinking, yellow signal. During these 15 minutes the police checks whether the alarm was correct and conducts necessary measures. The station exists since summer 1995 and proved to be effective.

	5.3 Rock Avalanches
General remarks	
Introduction	Rock avalanches differ from debris flows and snow avalanches by that destabilization develops over very long time, and there is no evident meteorological trigger for the release. As there are very few examples of major rock avalanches in the world, the understanding of the release processes are limited. Since structural measures in most cases are not suitable, early warning systems are the only risk reduction measures available. Such systems must be based on monitoring of wide range of possible processes leading to destabilization of the rock mass.
Data and models	
	Data which should be included in rock avalanche prediction are: - Measurement of movement both on the surface and along possible gliding surfaces - Measurement detecting noise - Measurement of precipitation and temperature - Measurement of water pressure along failure surfaces - In case of tsunami hazard, sensors detecting sudden wave generation There are several models available on animation of rock mass movement and stability evaluation (e.g. UDEC). However, the reliability of these models is strongly dependent on detailed information of the rock joint geometry and rock strength properties. As the rock joint system may be complex and go deep into the ground, the models must be operated with great caution.
Operational forecasting	
	At the moment there is no operational national network for forecasting of rock avalanches in the world. However, in Norway regional forecasting center is planned in the northwest fiord district of Southern Norway. Several possible rock avalanche sites have been identified where systems for monitoring, in the first place surface movement, are under preparation. One site is already heavily instrumented (Åkneset) and will serve as basis for gaining experience in building up of early warning systems of rock avalanches.
Alarm systems	
	Since rock avalanches involve large volumes and may move with great speed over huge areas with possible catastrophic consequences, alarm systems should be based on alarm triggering in good time, typically hours or days, before the release. Because of insufficient empirical data, high frequency of false alarms must be expected.

Åknes alarm system	At the Åkneset site alarm will be triggered if the rate of movement accelerates beyond a certain level. The exact rate of movement that may lead to a catastrophic event is not known. In Åknes the early warning is based on the rate of movements. Five alert levels are used:

Alert level	Rate of movement	Activity
Green	• Small variations in movement	• Situation can be handled by staff at the emergency control centre • Technical maintenance
Blue	• Seasonal differences • Threshold-value 1	• Higher frequency of recordings • Geological expert group is informed
Yellow	• General increased movement • Threshold-value 2	• 24 hours continuous observations • Geological expert group is involved • Police and municipalities are informed
Orange	• Acceleration in movement • Threshold-value 3	• Emergency control centre continuously manned • All relevant personnel in emergency management are involved
Red	• Increased acceleration in movement • Threshold-value 4	• Alarm (siren signal) • Evacuation is initiated

The alert levels are based on the velocity (mm/day) of the movement of joint opening.

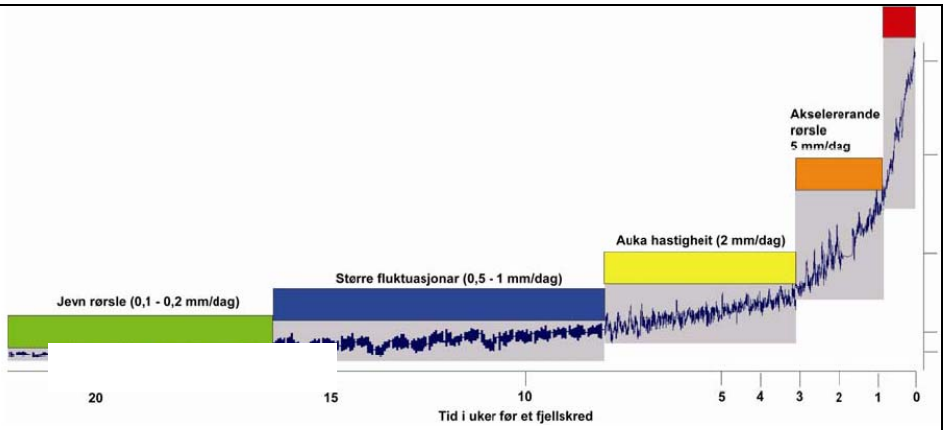


Figure: The alert levels depend on the rate of movement. Y-axis: Velocity of movement (mm/day). X-axis: Time (indicated by weeks before a possible rock avalanche)

	5.4 Snow Avalanches
General remarks	
	Operational forecasting and warning has quite a long tradition in Switzerland and the other alpine countries. In Switzerland avalanche forecasting was established at the Swiss Federal Institute for Snow and Avalanche Research in Davos in 1942 as a support for the Swiss army during the Second World War. After the Second World War it was extended for civil purposes. Especially since the severe avalanche winter 1951 avalanche warning was continuously improved. In the following section avalanche forecasting is generally presented with a focus on the situation in Switzerland. The general basis and the concept is comparable with handling in other alpine countries.
Data and models	
Data:	In avalanche forecasting there are several measurement and observer network in use: – ENET-Network: Network of 11 automatic stations above 2000 m a. s. l. commonly used and maintained by SLF and the Federal Meteorological Office MeteoSwiss. The data are available in hourly resolution and are stored in the data base at SLF. These stations consist of a wind station and a snow station. At the wind station air temperature (ventilated), air humidity, wind direction and wind velocity (heated) is measured and at the snow station air temperature, snow depth, snow surface temperature (infrared), and the temperature profile soil – snow; – IMIS-Network: Network of 75 stations consisting of wind station and snow stations. At the wind station air temperature, air humidity, wind direction and wind velocity is measured and at the snow station additionally air temperature, air humidity, snow depth, snow surface temperature (infrared), reflected short-wave radiation, and snow temperature at different levels; – SLF-Observer network: About 160 SLF-observers have a specific measurement and observation program depending on the location of the station and the educational level of the observer. Measurements and observations of these observers are transmitted once or twice a day via Internet forms to the SLF and are stored in the SLF database. Additional observations of the terrain are available by mountain guides. Essential information is provided by observation of the snowpack stratigraphy, i.e. snow profiles, which are transmitted every two weeks by SLF-observers.

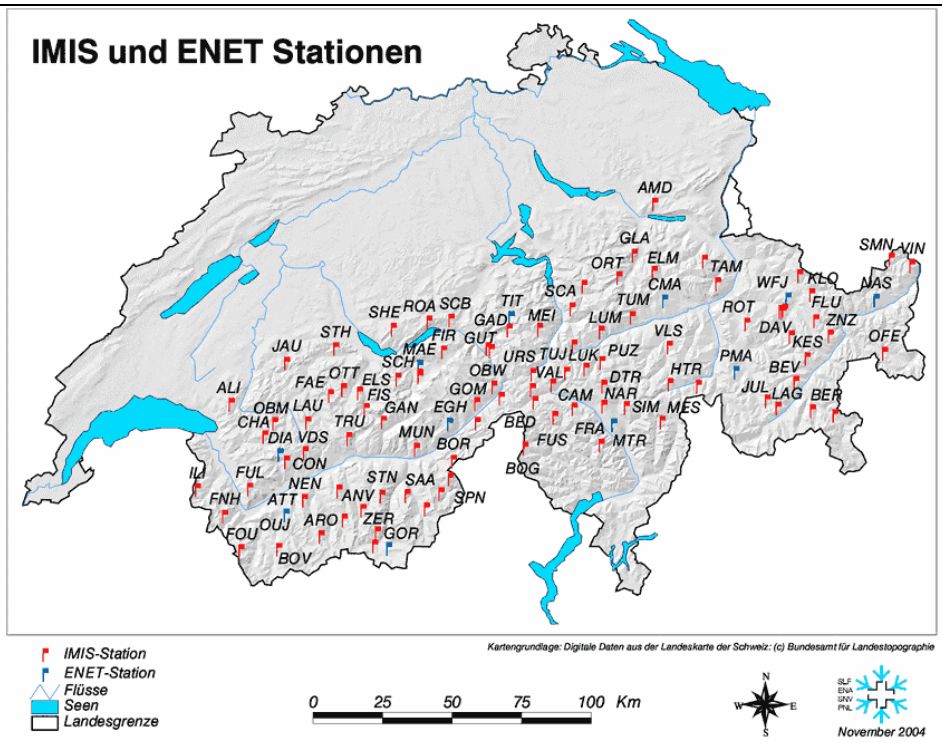


Figure 74: *Network of automatic weather stations for avalanche forecasting in Switzerland. Source: SLF.*

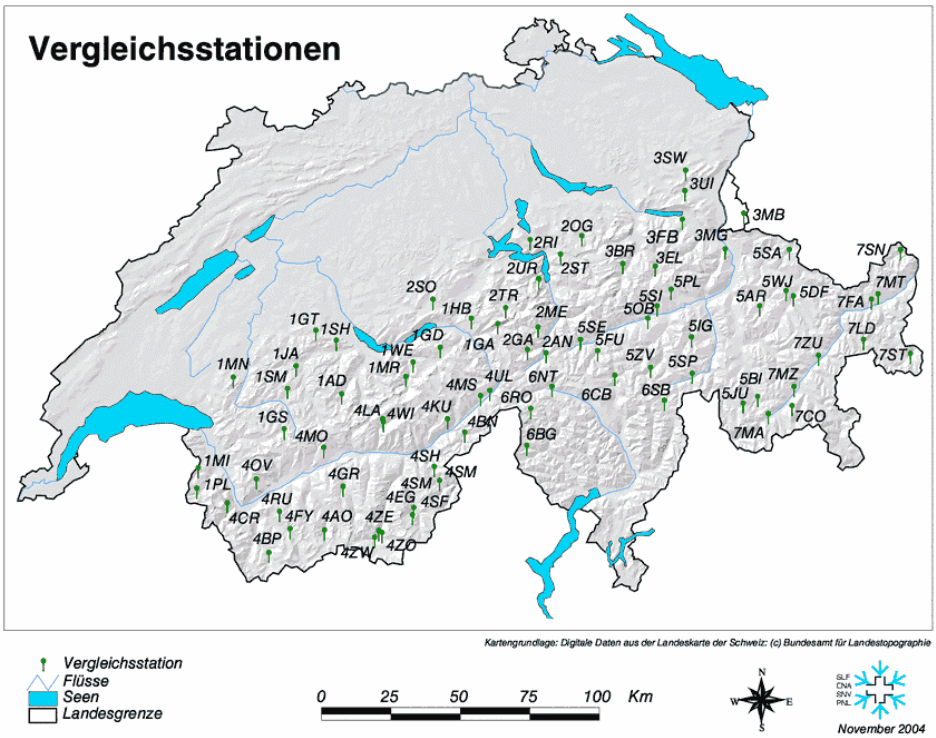


Figure 75: *Network of observer stations for avalanche forecasting in Switzerland. Source: SLF.*

Meteorological models	The meteorological basis for avalanche forecasting is provided by weather forecast and models of the Federal Meteorological Office MeteoSwiss. The used models are: – ECMWF: Model of the European Centre for Medium Range Forecasting; – GME: Model of the German Meteorological Office; – GFS: Model of the American Meteorological Office.
Statistically based models and physically based models	In addition to the measurements and observations a numerical model for modeling the physical processes of the snowpack is in use. At SLF the model SNOWPACK is used to calculate parameters which are not directly measured by the sensors at the automatic weather stations. The basis of these simulations are the data of the automatic weather stations. The calculated parameters are: depth of new snow, index of snow transport by wind, dew point temperature. The model is also able to calculate grain size, grain type, density and water content of the snowpack.
Visualisation of data	Data of measurement and observer stations as well as the results of the model SNOWPACK are assessable for safety services via the password-protected Internet platform IFKIS-InfoManager. This tool includes the national and the regional avalanche forecasting bulletins and all data, observations and model results which are relevant for safety decisions. The information given in the InfoManager is the fundamental basis the safety services take into account for decisions like closure of traffic routes, evacuation of buildings, etc.. The module IFKIS-MIS which is integrated in InfoManager offers also the possibility for an information exchange between safety services. IFKIS-MIS is an Internet application in which safety decisions can be documented and which automatically distributes new messages to the registered members of this service. Thus, IFKIS-MIS helps that all safety services in a region can be kept on an equal information level and supports the classical information exchange by phone.

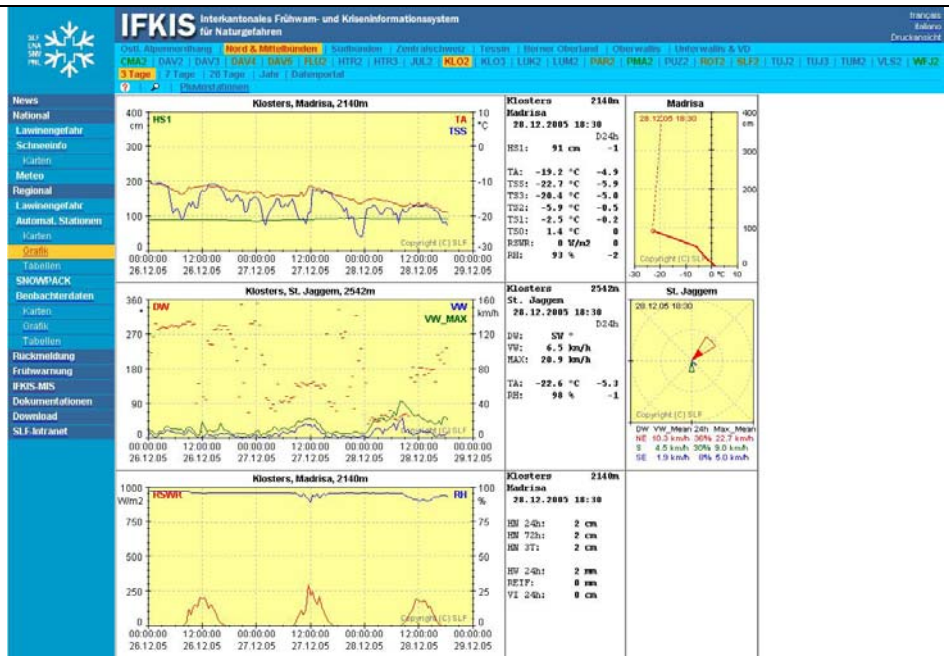


Figure 76: *Screen shot of IFKIS InfoManager, the platform for visualization of measurement data and model results for avalanche safety services in Switzerland.*

As mentioned previously the successful information system for avalanches is adapted to hydrological processes from small catchments in a research project at WSL-SLF. The information system is based on IFKIS InfoManager for avalanches and has a similar layout.

Operational forecasting and warning

National bulletin:

In Switzerland there are several products for operational forecasting and warning.

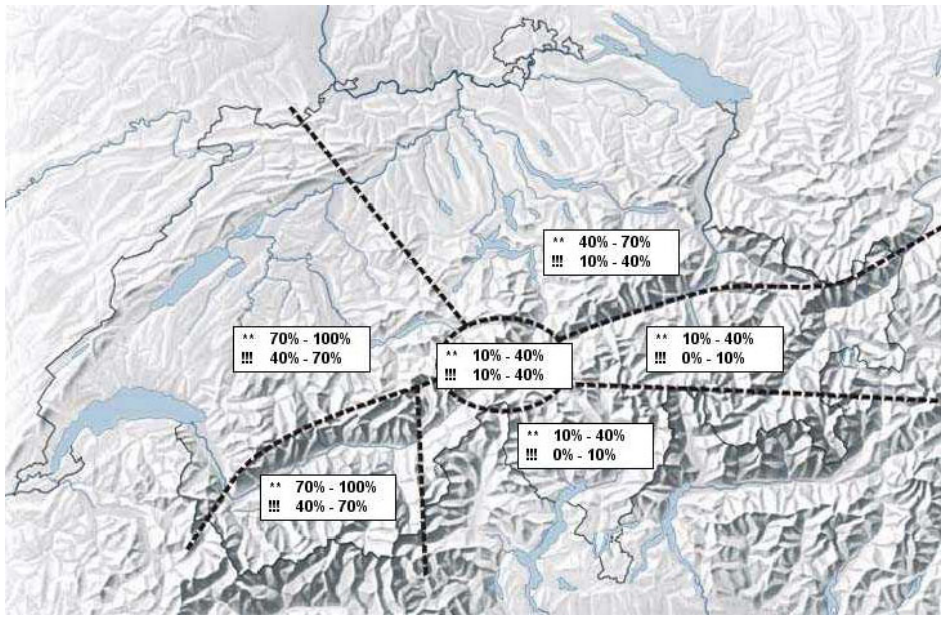
National avalanche bulletin (evening bulletin) Switzerland

The national bulletin is published daily at 5 p.m. It contains information about the stability of the snowpack and the probability of an avalanche release in a certain altitude and a certain aspect of a region. It is broadcasted via telephone (No. 187), fax, radio, TV, newspapers, Internet and Wap.

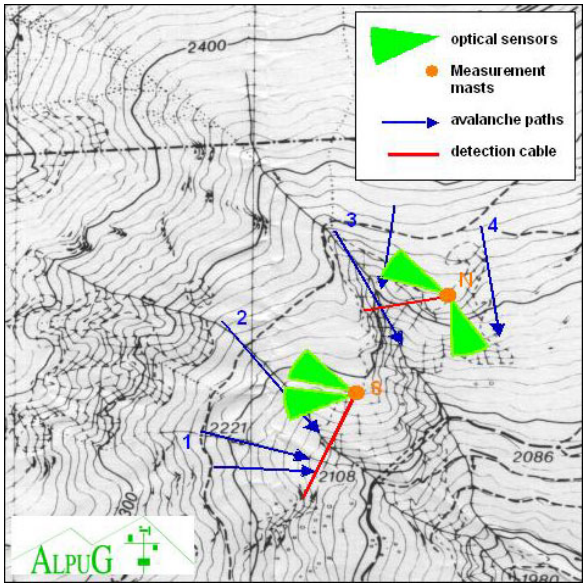
The used terms are explained in a separate publication, the interpretation help for the national bulletin.

In addition to the national bulletin there are several maps:

- snow depth (data basis: measurement stations, NOAA-AVHRR satellite data);
- snow depth (data basis: automatic measurement stations, 2000 or 2500 m a. s. l. level;
- snow depth in comparison to long year average;
- new snow depth (24 hours);
- new snow depth (3 days);
- snowpack stability.

Regional bulletin:	**Regionalized avalanche bulletin (morning bulletin) Switzerland** The regionalized avalanche bulletins are issued for the regions eastern part of the northern slope of the Alps, North- and Central Grisons, South Grisons, Swiss Plateau, Bernese Oberland, Eastern part of Valais, Western part of Valais and Vaud (in French). It is published every day at 8 a.m. On top of the bulletin there is a flash with a comprehensive information on the situation followed by a graphic showing the danger level at a certain altitude and aspect. At the bottom there are meteorological information and measurement station data. They are distributed via Internet, fax and they are available at many ski resorts.
Early warning:	**Early warning „Heavy snow fall“ or early warning „Snow and avalanche danger“ Switzerland** When the amount of new snow within the next 36 hours exceeds 100 cm with a probability of 70 to 100 % Meteo Swiss send out information to SLF, which considers also the developing, avalanche situation. When the danger level „five“ (very high) is not reached within the next 36 hours, then an information „Heavy snowfall“ is issued via InfoBox or IFKIS InfoManager. Also when the avalanche situation is getting critical (danger level five is reached within the next 36 hours with a probability of 40 to 70 %) an early warning „Snow and avalanche danger“ is issued. The safety services are directly informed by SMS, Pager and/or E-mail and are requested to consult the InfoBox. The early warning service „Heavy snowfall“ and „Snow and avalanche danger“ is a collaboration between MeteoSwiss and SLF and differs from the Meteo-early warning and Meteo-warning issued by MeteoSwiss.  ***Figure 77:*** *Example of the map early warning „Snow and avalanche danger“ from 3rd March 2006. Source: SLF.*

	Legend: ** probability of new snow > 100 cm next 36 hours 100% !!! probability of avalanche danger level 5 (40 to 70%) 0-10%: not probable 10% to 40%: less probable 40% to 70%: probable 70% to 100%: very probable
Alarm stations	
	In addition to forecasting and warning there are alarm stations in use. In most of the cases these systems are used to protect traffic lines. They all work according to the same principle. When an avalanche is released, a detection system registers the process and a signal is transmitted by cable or radio to the alarm basis station at the endangered zone. A red light or a tocsin is put into operation and via a relay station an alert is transmitted to the alarm center. The flow time from the detection site along the track to the endangered zone has to be larger than the evacuation time. The typical technical solutions, which are in use are optical detection, mechanical detection, and seismic and acoustic detection.
Radar detection:	**Radar detection** Based on the principle of Doppler radar there are pulsed Doppler radar and continuous Doppler radar in use. The pulsed Doppler radar send out short pulses in a certain time range. A pulsed Doppler radar system is e.g. used in Austria to detect and to measure artificially released avalanches but can be used as an avalanche alarm system as well. In a continuous pulsed radar the signal is sent out continuously with multiple channels. It is e.g. used by the SLF at the international experimental test site „Vallée de la Sionne“ in the canton of Valais. At Vallée de la Sionne Doppler radar is applied for calculation of the avalanche velocity.
Mechanical detection:	**Mechanical detection** Most of the mechanical sensors are working with weighing cables, which are mounted on a cable crossover the avalanche gully. When an avalanche is moving downwards the gully, the snow touches the weighed cables causing an electrical signal, which is transmitted to the endangered area. There traffic lights are set to stop traffic on the road and/or rail and an acoustic signal is released. A further system works with gates, which are mounted on one side of the gully. The moving avalanche snow pushes the gates to the lateral side of the gully, which causes an interruption of an electrical circuit. The signal switches the traffic light to red and provides an acoustic signal.
Seismic and acoustic detection:	**Seismic and acoustic detection** The method is currently used both for research projects and for operational use. For seismic detection three-component geophones are used to record the avalanche activity in an area of several kilometers in diameter.

	The acoustic system called „ARFANG“ which uses infra-sound microphones are either mounted on pylons in a horizontal cross shape or they are buried in the ground. They are also used to record the avalanche activity in an area.
Examples:	**Examples of avalanche detection and alarm systems** There are several avalanche detection and alarm systems in use in Switzerland. As an example the alarm station Embd in the canton Valais is presented. At the alarm station Embd avalanches from four starting zones are detected. The station is located at 2250 m a. s. l. and measures snow movements, forces on the detection cables (mechanical sensor) and concussions of the detection cables. When an avalanche is detected, alarm is released via radio and the traffic lights are switched on. In addition a message is transmitted to the head office in Sierre.  ***Figure 78:*** *Alarm station Embd in the canton of Valais. Source: AlpuG.*

Chapter 6

Concluding Remarks

The defense against all the massive flow risk (snow avalanches, debris flow and rock avalanches) here considered can be done with structural or non-structural countermeasures.

The decision on the choice between the two typologies of countermeasures depends mostly on local constraints, on budget availability and on possible efficiency of the type of intervention. The cost benefit analysis is the instrument to take a rational decision in this respect.

The guiding role of this deliverable is that in any case it is nowadays necessary to put a great effort in the strategy design, in order to convince the public authorities, the private stakeholder and the population that any intervention is necessary and efficient.

With this target, it is necessary to employ all the most recent findings and the most advanced methods. This is not always possible, but science and technology have recently made a lot of new knowledge available, such as calculation methods and mathematical models that have transformed the old restoration criteria, mostly based only on the personal experience of the designer, in a sophisticated science.

This deliverable has collected and organized in a clear design system these recent methods. The design criteria in this deliverable are organized

	in different hierarchy of knowledge. The first chapter describes the decisional process. It includes the cost benefit analysis and the impact analysis. The appendix contains some detailed examples of the design of the different typologies of countermeasures. This stage is necessary to develop the proper strategy for the realization of an efficient countermeasures system, satisfying both the safety demand and cost optimization, and environmental impact reduction. This step has to be considered a necessary process in the future design of the interventions for risk reduction. The deliverable offers an example of this kind of analysis. The part relevant to the efficiency and costs depends strongly on good and convincing design criteria that use all the most recent designing methods. In chapters 2, 3 and 4 these modern and conscious design criteria have been described in detail and some detailed applications are also presented. This part is one of the most valuable aspects of the deliverable, in which the long experience of some of the IRASMOS partners have been condensed. In chapter 5 the non structural countermeasures are described. This is also a part of the most recent strategies for risk reduction: they are mainly based on operational forecasting and warning and on the development of alarm systems. The decision between one of the two strategies cannot be taken in advance, but it requires a preliminary analysis on the local condition. The cost-benefit analysis will help the decision makers in the final choice. The deliverable, together with the companion deliverable D2.1 is aimed at this purpose.

Annex A1

DESIGN EXAMPLES OF COUNTERMEASURES AGAINST DEBRIS FLOW

A1.1 – Closed check dams

Example of weir design :

We want to design the weir of a check-dam for a given peak discharge of $30\ m^3\ s^{-1}$. The weir has a trapezium cross section with the width of the base equal to 6 m and a side slope 1:1.
We analyze both the two modes of outflow above the weir: the first, transitory, before the complete filling of the volume upstream the dam; the second, when the dam is filled.

1) Before back-filling:

In this case the outflow above the weir is that of a crested weir:

$$Q = \frac{C_c}{15}\left(11 b_g + 4 b_1\right) h_0 \sqrt{2 g h_0}$$

with b_1 base of the weir. From this formula it is possible to deduce the depth h_0 upstream of the dam:

$$30 = \frac{0.385}{15}\left(15 \cdot 6 + 8 h_0\right) h_0 \sqrt{2 \cdot 9.81 \cdot h_0}$$
$$h_0 = 1.85\ m$$

In the weir the depth of the flow h_g is:

$$h_g = h_c = \sqrt[3]{\frac{Q^2}{gb_g^2}} = \sqrt[3]{\frac{30^2}{9.81 \cdot 6_g^2}} = 1.37 \quad m$$

Therefore the height of the weir Y_g has to be at least equal to:

$$Y_g = \frac{3}{2} h_c = 2.06 \quad m$$

2) After back-filling:

In this case also the characteristics of the stream must be known: the Strickler friction coefficient of the stream is assumed to be $K_S = 35\, m^{1/3}\, s^{-1}$ and the width of the base of the stream cross section, in backfilled condition, is equal to 12 m.
When the dam is filled, we impose the conservation of the total energy between the section upstream the dam and the section of the weir:

$$Q = AK_s R_h^{\frac{2}{3}} \sqrt{i_\theta}$$

$$30 = 35 \cdot \frac{\left(12 \cdot h_m + h_m^2\right)^{\frac{5}{3}}}{\left(12 + 2\sqrt{2} \cdot h_m\right)^{\frac{2}{3}}} \cdot \sqrt{0.026}$$

$$h_m = 0.62 \ m$$

$$h_c = \sqrt[3]{\frac{Q^2}{gB^2}} = \sqrt[3]{\frac{30^2}{9.81 \cdot 12^2}} = 0.86 \ m$$

upstream the dam flow is supercritical, being $h_m < h_c$.
Therefore from the conservation of the energy we deduce the depth of the flow in the weir:

$$h_g + \frac{Q^2}{2gh_g^2\left(B_g + nh_g\right)^2} = h_m + \frac{Q^2}{2g\left(h_m B_m\right)^2}$$

$$h_g + \frac{30^2}{2 \cdot 9.81 \cdot \left(6 \cdot h_g + h_g^2\right)^2} = 0.62 + \frac{30^2}{2 \cdot 9.81 \cdot \left(0.62 \cdot 12\right)^2}$$

$$h_g = 1.33 \ m$$

Return back.
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Example of design of countermeasures against local erosion:	In a torrent with rectangular section with base B equal to 10 m, slope 2%, d_{90} = 50 mm and Q = 10 m^{3}s^{-1}, there is a dam with altitude Y_b of 2 m and width B_g of 6m. 1) Calculate the erosion down the dam. 2) Determinate the altitude of counterdam suitable to reduce the erosion of 50%. We assume that the slope of the reach downstream the dam is steep.

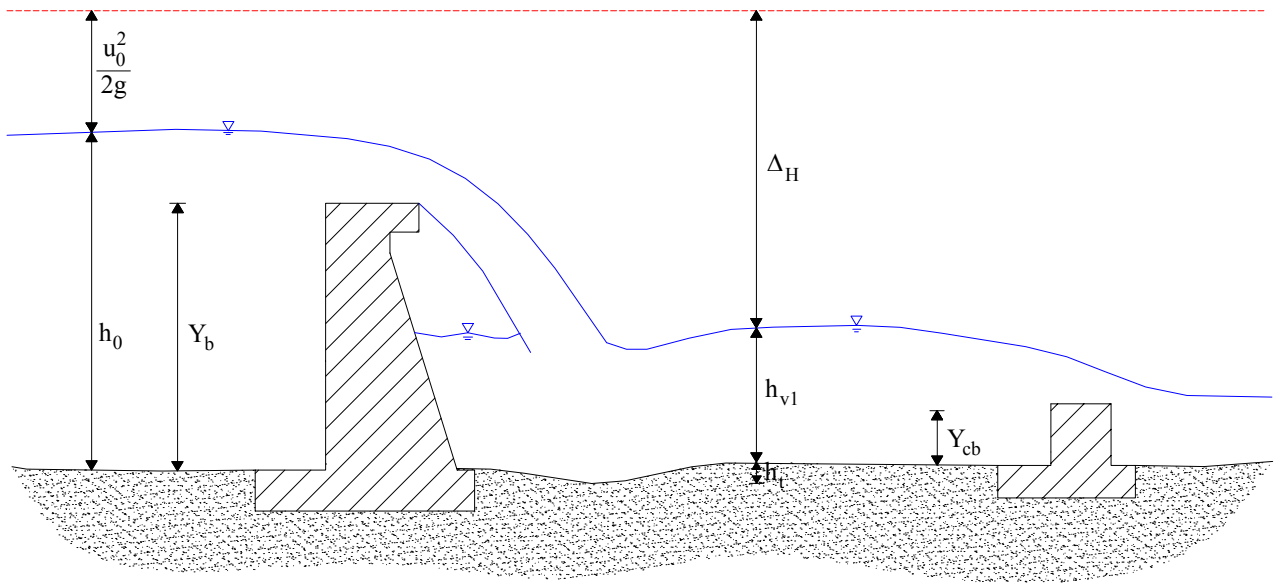

***Figure 79**: Scheme of dam and counterdam.*

1) To calculate h_{v1} we proceed in such a way:

- We calculate a first tentative value h'_{v1}, under the hypothesis of energy conservation between the section of the weir and the section downstream:

$$Y_b + \frac{3}{2}\sqrt[3]{\frac{Q^2}{gB_g^2}} = h'_{v_1} + \frac{Q^2}{2g\left(Bh'_{v1}\right)^2}$$

$$2 + \frac{3}{2}\sqrt[3]{\frac{10^2}{9.81\cdot 6^2}} = h'_{v1} + \frac{10^2}{2\cdot 9.81\cdot\left(10\cdot h'_{v1}\right)^2}$$

We obtain two values for h'_{v1}:

$$\left(h'_{v1}\right)_{1,2} = 0.13\ m \quad and \quad 2.98\ m$$

Due to the fact that the slope of the reach is steep, we reject the higher value and we keep the small one:

$$h'_{v_1} = 0.13 \ m$$

The critical depth is:

$$h_c = \sqrt[3]{\frac{Q^2}{gb_g^2}} = \sqrt[3]{\frac{10^2}{9.81 \cdot 10_g^2}} = 0.47 \quad m .$$

- We now calculate a second tentative value for h''_{v1}, under the hypothesis in the downstream reach (rectangular wide bed):

$$Q = Bh''_{v1} k_s \left(h''_{v1}\right)^{\frac{2}{3}} \sqrt{i_f}$$

$$10 = 10 \cdot h''_{v1} \cdot 43 \cdot \left(h''_{v1}\right)^{\frac{2}{3}} \sqrt{0.02}$$

we obtain:

$$h''_{v1} = 0.34 \ m .$$

Now we can calculate the global momentum corresponding to h'_{v1} and h''_{v1}:

$$S'_{v1} = \frac{1}{2}\gamma\left(h'_{v1}\right)^2 B + \rho\frac{Q^2}{Bh'_{v1}} = 77768.08 \ N$$

$$S''_{v1} = \frac{1}{2}\gamma\left(h''_{v1}\right)^2 B + \rho\frac{Q^2}{Bh''_{v1}} = 35191.76 \ N$$

Because we have:

$$S'_{v1} > S''_{v1}$$

we can assume as h_{v1} this value:

$$h_{v1} = h'_{v1} = 0.13 \ m .$$

To calculate the value of Δ_H, we apply the balance of the energy between the section of the dam and the section of downstream:

$$Y_b + \frac{3}{2}\sqrt[3]{\frac{Q^2}{gB_g^2}} = h_{v1} + \Delta_H$$

$$\Delta_H = 2 + \frac{3}{2}\sqrt[3]{\frac{10^2}{9.81 \cdot (6)^2}} - 0.13 = 2.85 \ m$$

Then we calculate the entity of the maximum erosion:

$$h_t = 4.75\frac{\Delta_H^{0.2} q^{0.57}}{d_{90}^{0.32}} - h_{v1}$$

$$h_t = 4.75\frac{2.85^{0.2}\left(\frac{10}{6}\right)^{0.57}}{(50)^{0.32}} - 0.13 = 2.11\ m$$

2) Before answering the second question, we must calculate the value of h_{v1} that allows a reduction of 50% of the original erosion, resolving the next system of equations:

$$\frac{2.11}{2} = 4.75\frac{\Delta_H^{0.2}\cdot\left(\frac{10}{6}\right)^{0.57}}{(50)^{0.32}} - h_{v1}$$

$$\Delta_H = 2 + \frac{3}{2}\sqrt[3]{\frac{10^2}{9.81\cdot 6^2}} - h_{v1}$$

Then we obtain:

$$h_{v1} = 1.02\ m$$

Finally, by imposing the conservation of the energy between the section of the counterdam and the section in correspondence of the value of h_{v1} just calculated, we deduce the height of the counterdam that reduces of 50% the original erosion:

$$h_{v1} + \frac{Q^2}{2g\left(h_{v1}B\right)^2} = Y_{cb} + \frac{3}{2}\sqrt[3]{\frac{Q^2}{gB^2}}$$

$$1.02 + \frac{10^2}{2\cdot 9.81\cdot(1.02\cdot 10)^2} = Y_{cb} + \frac{3}{2}\sqrt[3]{\frac{10^2}{9.81\cdot 10^2}}$$

$$Y_{cb} = 0.37\ m$$

Return back.
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Example of static design:	
Data:	B1 = 2.5 m B2 = 5 m b = 1 m h = 1 m h_v = 4 m H = 6 m P = 2 m γ_t = 17680 N/m3 γ_{sat} = 20819 N/m3 γ_c = 23500 N/m3 K_0 = 0.5 K_a = 0.295 φ = 35° N_γ = 48.03 N_q = 33.30 To execute all the verification required by regulations.

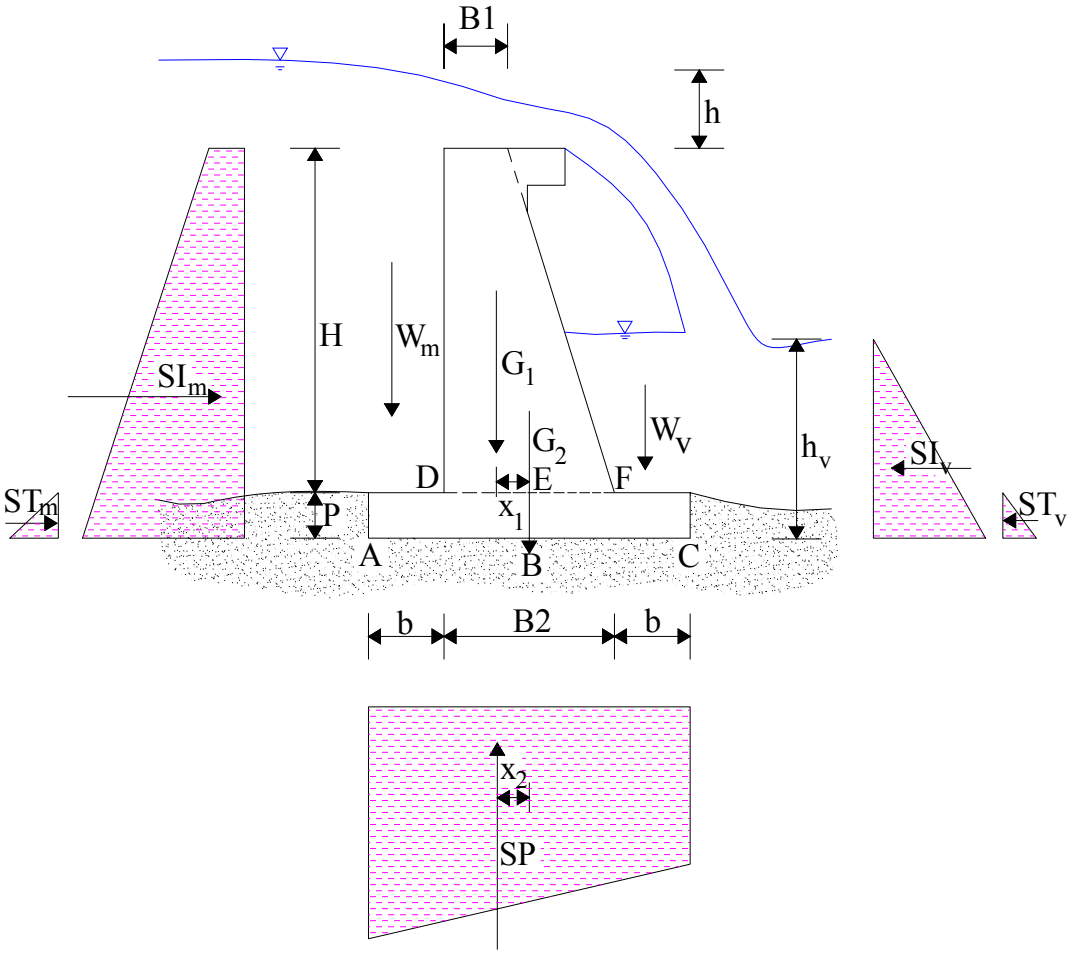

***Figure 80**: Scheme of the agent forces on a dam.*

Strength resistance of material:	$S\,Im = \frac{1}{2}\gamma\left(H+P+2h\right)\left(H+P\right) = \frac{1}{2}9810\left(8+2\cdot 1\right)\cdot 8 = 392400.00\ N$ $b_{SI_m} = \left(H+P\right)\left(3h+H+P\right)/\left[3\left(2h+H+P\right)\right] = 8\cdot\left(3\cdot 1+8\right)/\left[3\cdot\left(2\cdot 1+8\right)\right]$ $= 2.93\ m$ $ST_m = \frac{1}{2}\left(\gamma_{sat}-\gamma\right)P^2K_a = \frac{1}{2}\left(20819-9810\right)\cdot 2^2\cdot 0.295 = 6495.43\ N$ $b_{ST_m} = P/3 = 2/3 = 0.67\ m$ $SI_v = \frac{1}{2}\gamma h_v^2 = \frac{1}{2}\cdot 9810\cdot 4^2 = 78480.00\ N$ $b_{SI_v} = h_v/3 = 4/3 = 1.33\ m$ $ST_v = \frac{1}{2}\left(\gamma_{sat}-\gamma\right)P^2K_0 = \frac{1}{2}\left(20819-9810\right)\cdot 2^2\cdot 0.5 = 11009.20\ N$ $b_{ST_v} = P/3 = 2/3 = 0.67\ m$ $G_1 = \gamma_c\left(B_1+B_2\right)\cdot H/2 = 23500\left(5+2.5\right)\cdot 6/2 = 528750.00\ N$ $x_1 = \left(B_2-B_1\right)\left(B_2+2B_1\right)/\left[6\left(B_1+B_2\right)\right] = \left(5-2.5\right)\left(5+2\cdot 2.5\right)/\left[6\left(5+2.5\right)\right]$ $= 0.56\ m$ $G_2 = \gamma_c\left(2b+B_2\right)P = 23500\cdot 2\cdot 7 = 329000.00\ N$ $W_m = \gamma bH = 9810\cdot 1\cdot 6 = 58860.00\ N$ $W_v = \gamma b\left(h_v-P\right) = 9810\cdot 1\cdot 2 = 19620.00\ N$ $SP = \frac{1}{2}\gamma\left(H+P+h_v+h\right)\left(2b+B_2\right) = \frac{1}{2}\cdot 9810\cdot\left(6+2+4+1\right)\left(2\cdot 1+5\right)$ $= 446355.00\ N$ $x_2 = \left(B_2+2b\right)\left(H+P+h-h_v\right)/\left[6\left(H+P+h+h_v\right)\right]$ $= 7\left(6+2+1-4\right)/\left[6\left(6+2+1+4\right)\right] = 0.45\ m$ It executes the verification at the level $H+P$. The moments acting like clockwise are assumed positive. The normal force N and the inflected moment M_B acting on foundation are: $N = 446355.00-\left(528750.00+329000.00+58860.00+19620.00\right)$ $= -489875.00\ N$ $M_B = 6495.43\cdot 0.67+392400\cdot 2.93+446355.00\cdot 0.45+19620.00\cdot 3-$ $-58860.00\cdot 3-528750.00\cdot 0.56-11009.20\cdot 0.67-78480.00\cdot 1.33$ $= 832439.42\ Nm$ Since $A = 7\cdot 1 = 7\ m^2$ and $W = 7^2\cdot 1/6 = 8.16\ m^3$, the tensions σ_1 and σ_2

at the borders, assumed positive the traction, are:

$$\sigma_{1,2} = -\frac{489875.00}{7} \pm \frac{832439.42}{8.17} = 31945.05,\ -171909.34\ \ Nm^{-2}$$

i.e. a modest traction above and a modest compression down.

Repeating the calculations of the forces and of the moments act at the depth H:

$$SI_m = \frac{1}{2}\gamma\left(H+2h\right)H = \frac{1}{2}\cdot 9810\cdot\left(6+2\cdot 1\right)\cdot 6 = 235440.00\ \ N$$
$$b_{SI_m} = H\left(3h+H\right)/\left[3\left(2h+H\right)\right] = 6\left(3\cdot 1+6\right)/\left[3\left(2\cdot 1+6\right)\right] = 2.25\ \ m$$
$$SI_v = \frac{1}{2}\gamma\left(h_v - P\right)^2 = \frac{1}{2}\cdot 9810\cdot\left(4-2\right)^2 = 19620.00\ \ N$$
$$b_{SI_v} = \left(h_v - P\right)/3 = \left(4-2\right)/3 = 0.67\ \ m$$
$$SP = \frac{1}{2}\gamma\left(H+h+h_v-P\right)B_2 = \frac{1}{2}\cdot 9810\cdot\left(6+1+4-2\right)\cdot 5 = 220725.00\ \ Nm$$
$$x_2 = B_2\cdot\left(H+h-h_v\right)/\left(6\cdot\left(H+h+h_v\right)\right) = 5\cdot\left(6+1-4\right)/\left(6\cdot\left(6+1+4\right)\right)$$
$$= 0.23\ \ m$$

$$N = 220725.00 - 528750.00 = -308025.00\ \ N$$
$$M_E = 235440.00\cdot 2.25 + 220725.00\cdot 0.23 - 528750.00\cdot 0.56 -$$
$$-19620.00\cdot 0.67 = 271261.35\ \ Nm$$

The tensions at the border are:

$$\sigma_{1,2} = -\frac{308025.00}{5} \pm \frac{271261.35}{4.17} = 3445.68,\ -126655.68\ \ Nm^{-2}$$

been $A = 5\cdot 1 = 5\ \ m^2$ and $W = 5^2\cdot 1/6 = 4.17\ \ m^3$. The tensions of traction and of compression are much more modest.

Sliding stability analysis:	$F_o = SI_m + ST_m - SI_v - ST_v = 392400.00 + 6495.43 -$ $-78480.00 - 11009.20 = 309406.23\ N$ $N = G_1 + G_2 + W_m + W_v - SP = 528750.00 +$ $+329000.00 + 58860.00 + 19620.00 - 446355.00$ $= 489875.00\ N$ $f = tg\varphi = 0.7$ We obtain: $\eta_s = \frac{N \cdot f}{F_o} = 1.10 < 1.2$ NO! Therefore, the stability against sliding is not guaranteed, and the foundation needs to be extended as shown in Figure 81:

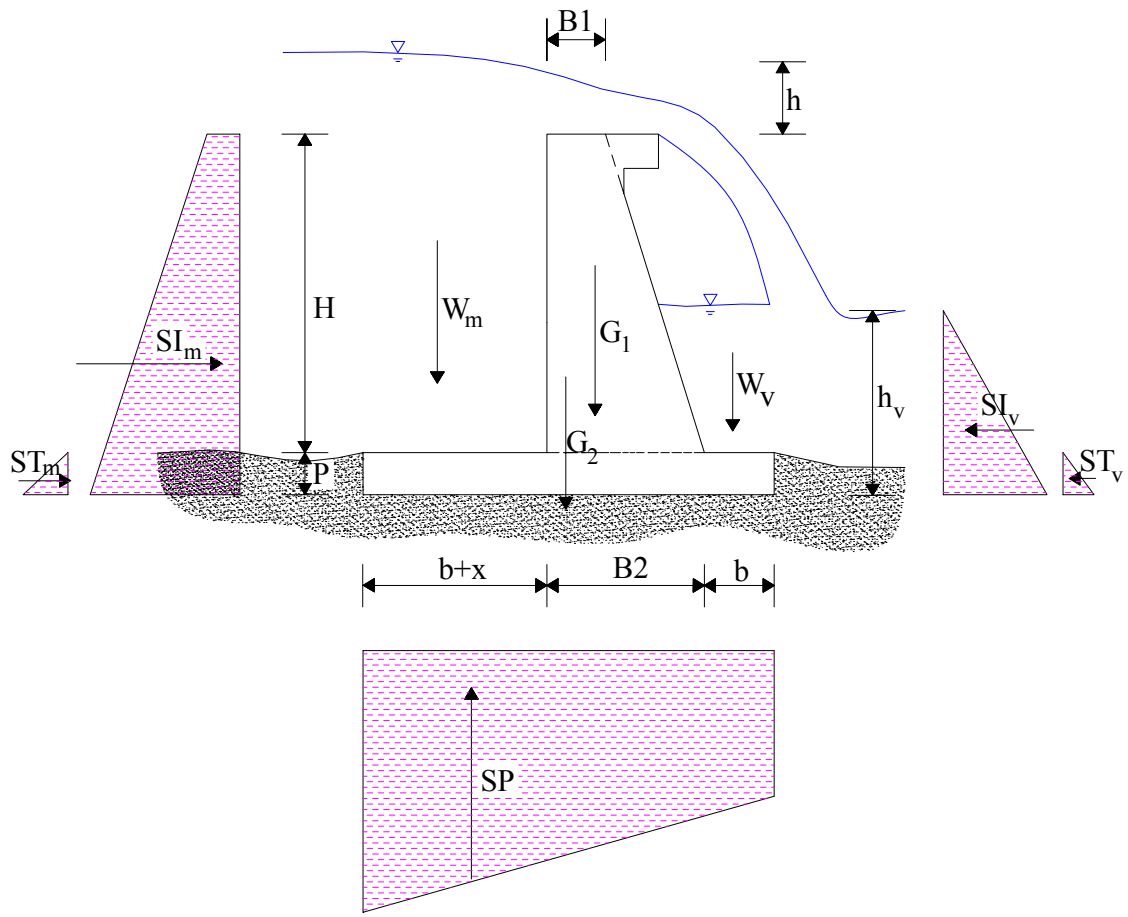

***Figure 81**: New scheme of the agent forces on a dam.*

Due to this new geometry the forces become:

$$W_m = \gamma(x+b)H = 9810\cdot(x+1)\cdot 6 = 58860\cdot(x+1)$$
$$G_2 = \gamma_c(2b+B_2+x)P = 23500\cdot(2\cdot 1+5+x)\cdot 2 = 47000\cdot(7+x)$$
$$SP = \frac{1}{2}\gamma(2b+B_2+x)(H+P+h+h_v) =$$
$$= \frac{1}{2}\cdot 9810\cdot(2\cdot 1+5+x)(6+2+1+4) = 63765\cdot(7+x)$$

$$N = G_1 + G_2 + W_m + W_v - SP = 528750.00 + 47000\cdot(7+x) + 58860\cdot(x+1) +$$
$$+19620 - 63765\cdot(7+x) \;\; N$$
$$F_o = 309406.23 \;\; N$$

Then x assumes a value that satisfies the sliding stability analysis:

$$N\cdot f = 1.2\cdot F_o$$
$$x = 0.96 \;\; m$$

It is assumed that:

$$x = 1 \;\; m$$

Therefore the definitive structure assumes this dimensions:

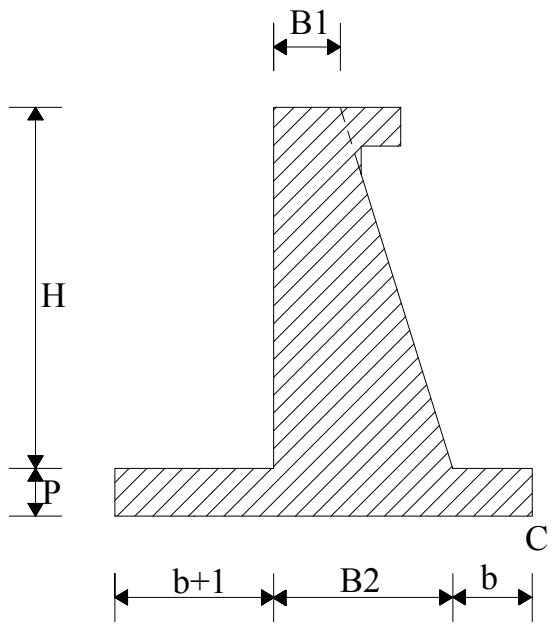

***Figure 82**: The dimension of the definitive dam.*

We must execute also the next verifications.

Rotation stability analysis:	Recalculating the forces that change their value because of the new geometry of the dam: $W_m = \gamma(b+1)H = 9810 \cdot 2 \cdot 6 = 117720.00\ N$ $G_2 = \gamma_c(2b+1+B_2)P = 23500 \cdot (2 \cdot 1+1+5) \cdot 2 = 376000.00\ N$ $SP = \frac{1}{2}\gamma(H+P+h+h_v)(2b+1+B_2) = \frac{1}{2} \cdot 9810 \cdot (6+2+1+4)(2 \cdot 1+1+5) = 510120.00\ N$ We must calculate the stabilizer moment M_s and the active momentum force M_r with respect to C: $M_s = 78480.00 \cdot 1.33 + 11009.20 \cdot 0.67 + 19620.00 \cdot 0.50 + 528750.00 \cdot 4.56 +$ $+376000.00 \cdot 4 + 117720.00 \cdot 7 = 4860704.56\ Nm$ $M_r = 392400.00 \cdot 2.93 + 6495.43 \cdot 0.67 + 510120.00 \cdot 4.51 = 3454860.44\ Nm$ Then η_r results: $\eta_r = \frac{M_s}{M_r} = 1.4 > 1.2$ OK !

Verification of round bearing analysis:	It is supposed to act in drained conditions in the presence of a soil with cohesion c' equal to zero. – Calculating the eccentricity of the loading: $\sum M = 392400.00 \cdot 2.93 + 6495.43 \cdot 0.67 - 528750.00 \cdot 0.06 - 117720.00 \cdot 3 + 510120.00 \cdot 0.45 + 16620.00 \cdot 3.5 - 78480.00 \cdot 1.33 - 11009.20 \cdot 0.67 = 955668.37 \ Nm$ $\sum N = 528750.00 + 376000.00 + 117720.00 + 19620.00 - 510120.00 = 531970.00 \ N$ $ecc = \frac{\sum M}{\sum N} = 1.80 \ m$ The reduced base is equal to: $B' = B - 2 \cdot ecc = 4.4 \ m$. – Calculating the effective vertical stress at the base of foundation: $q' = (\gamma_{sat} - \gamma) \cdot P = 22018 \ Nm^{-2}$ – Calculating the limit loading: $q'_{lim} = \frac{1}{2} B' N_\gamma (\gamma_{sat} - \gamma) + q' N_q = \frac{1}{2} \cdot 4.4 \cdot 48.03 \cdot (20819 - 9810) + 22018 \cdot 33.30 = 1896476.39 \ Nm^{-2}$ – Calculating the admissible loading of the foundation (to assume unitary depth of the foundation): $Q'_{amm} = \frac{Q'_{lim}}{3} = \frac{1896476.39 \cdot 4.4}{3} = 2781498.71 \ N$ – Then, making the verification: $Q'_{amm} + SP > 3 \cdot (G_1 + G_2 + W_m + W_v)$ $\frac{3291618.71}{1042090.00} \approx 3.2 > 3$

Piping verification:	Apply the two methods: – Lane: $\dfrac{\frac{1}{3}L_o + L_v}{\Delta h} \geq F^*$ $\qquad \dfrac{\frac{1}{3}8+2+2}{(7-2)} = 1.33$ it is smaller than all the tabulated values of F^* and so it isn't verified. – Terzaghi: $\dfrac{\frac{\gamma_{sat}-\gamma}{\gamma}}{\frac{\Delta h}{L}} > 4$ $\qquad \dfrac{\frac{20819-9810}{9810}}{\frac{5}{12}} = 2.69 < 4$ NO! The verification can be satisfied acting in these two modes: – increasing the weight of the soil down the foundation of the dam with the addition of a loading q for the unity of surface: $F = \dfrac{\frac{\gamma_{sat}-\gamma}{\gamma}}{\frac{\Delta h}{L}} + \dfrac{q}{\gamma \Delta h} = 2.69 + \dfrac{q}{9810 \cdot 5} = 4$ $q = 64256 \ Nm^{-2}$ – or inserting a gangplank anchored to the conglomerate down the foundation: $3.5 = \dfrac{\frac{1}{3}\cdot 8 + 4 + x}{(7-2)}$ $x = 10.8 \ m$ So the length of the gangplank results equal to $\dfrac{x}{2} = 5.5 \ m$ approximately.

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Example of structural design:	Example of measurement of the dam in pre-burying conditions in according to the elastic calculation (serviceability state limit):

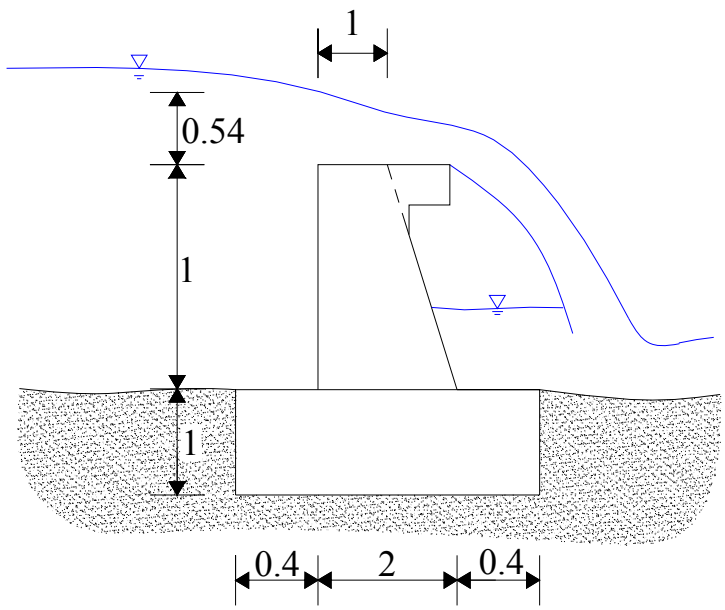

***Figure 83**: Scheme of the dimensions of the dam*

<u>Measurement of the elevation wall:</u>

The actions that act on the foundation of the elevation wall, in the pre-burying conditions, result:

$$N = -51.81 \ KN$$
$$M = -10.08 \ KNm$$
$$V = 12.01 \ KN$$

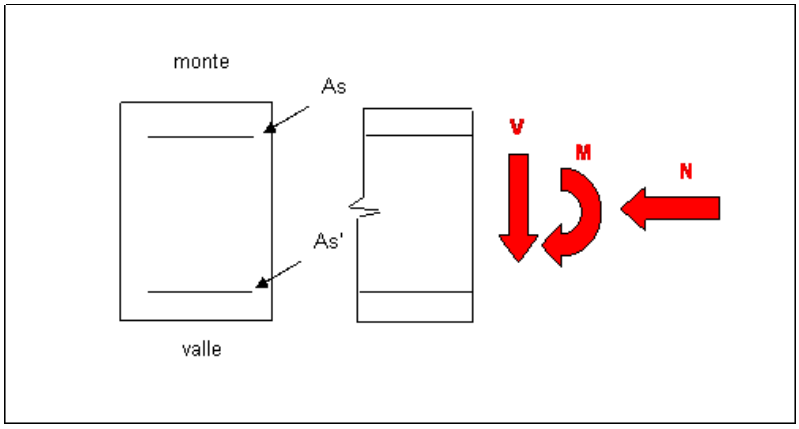

***Figure 84**: Scheme of the actions that act on the foundation of the elevation wall of the dam (in the* pre-burying *conditions).*

Consider the next materials:

- concrete $R_{ck}30$ with maximum admissible tension equal to $\overline{\sigma_c} = 9.75\ N/mm^2$;
- steel FeB44K with maximum admissible tension equal to $\overline{\sigma_s} = 255\ N/mm^2$.

The section of the base is marked by:

- base $b = 1000\ mm$ (depth of 1 m);
- altitude of sham beam $h = 2000m$;
- bar cover $c = 40\ mm$;
- useful altitude of the section $d = 1960\ mm$;
- reinforcement ratio $\beta = 0.5$;
- ratio between steel and concrete modular ratio stressed $n = 15$.

Calculation of the eccentricity of the loading:

$$e = \frac{M}{N} = \frac{10.08}{51.81} = 0.19\ m$$

$$e_{lim} = \frac{h}{6} = \frac{2}{6} = 0.33\ m$$

Been $e < e_{lim}$ the section is all compressed, so it is necessary to verify that there aren't excessive compressions.

<u>Elastic calculation of the section:</u>

At first to calculate minimum area of the armatures:

$$A_{s,min} = 0.0015 \cdot A_c = 0.0015 \cdot 1000 \cdot 2000 = 3000\ mm^2$$
$$A'_{s,min} = \beta A_{s,min} = 0.5 \cdot 3000 = 1500\ mm^2$$

The choice:

$$4\phi 16 \rightarrow A_s = 803.84\ mm^2$$
$$2\phi 16 \rightarrow A'_s = 401.92\ mm^2$$

Equalizing to zero the static moment, calculated respect to neuter axis, of the reagent homogenous section, we obtain the position of the neuter axis (x):

$$S_{nn} = 0$$

$$\frac{bx^2}{2} + nx\left(A_s + A_s^{'}\right) - nA_s c - nA_s^{'} d = 0$$

$$\frac{1000x^2}{2} + 15x\left(803.84 + 401.92\right) - 15 \cdot 40 \cdot 803.84 - 15 \cdot 1960 \cdot 401.92 = 0$$

$$x = 140 \ \ mm$$

Calculate the inertial moment of the reagent homogenous section respect to the neuter axis:

$$I_{nn} = \frac{bx^3}{3} + nA_s\left(x - c\right)^2 + nA_s^{'}\left(d - x\right)^2$$

$$I_{nn} = \frac{1000 \cdot 140^3}{3} + 15 \cdot 803.84\left(140 - 40\right)^2 + 15 \cdot 401.92\left(1960 - 140\right)^2 = 2.1 \cdot 10^{10} \ \ mm^4$$

The maximum tension of compression of the concrete is equal to:

$$\sigma_c = \frac{Mx}{I_{nn}} = \frac{10.08 \cdot 10^6 \cdot 140}{2.1 \cdot 10^{10}} = 0.0672 \ \ N / mm^2$$

The verification of the concrete results satisfied:

$$\sigma_c = 0.0672 \ \ N / mm^2 < \overline{\sigma_c} = 9.75 \ \ N / mm^2$$

but also that of the steel:

$$\sigma_s = \frac{n\sigma_c\left(d - x\right)}{x} = \frac{15 \cdot 0.0672 \cdot \left(1960 - 140\right)}{140} = 13.104 \ \ N / mm^2 < \overline{\sigma_s} = 255 \ \ N / mm^2$$

$$\sigma_s^{'} = \frac{n\sigma_c\left(x - c\right)}{x} = \frac{15 \cdot 0.0672\left(140 - 40\right)}{140} = 0.72 \ \ N / mm^2 < \overline{\sigma_s^{'}} = 255 \ \ N / mm^2$$

<u>*Shear stress:*</u>

Calculation of the tangential tensions in the concrete:

$$\tau_{c0} = 4 + \frac{R_{ck} - 15}{35} = 4.43 \ \ N / mm^2$$

$$\tau_{v \ max} = \frac{V \cdot 1000}{0.9 \cdot b \cdot d} = \frac{12.01 \cdot 1000}{0.9 \cdot 1000 \cdot 1960} = 0.0068 \ \ N / mm^2$$

Calculation of the minimum armature as per normative:

$p_{max} = min\left(0.8 \cdot d, 333\right) = min\left(1568, 333\right) = 333$ we choose p=300 mm.

$$A_{st,min} = \frac{V \cdot 1000}{0.9 \cdot d \cdot \tau_{c0}} = \frac{12.01 \cdot 1000}{0.9 \cdot 1960 \cdot 4.43} = 1.54 \ mm^2 / m$$ we choose 1 stirrup.

Calculation of the brackets:

$$\phi_{st,min} = \sqrt{\frac{4 \cdot A_{st,min} \cdot p}{1000 \cdot \pi \cdot n°_{st}}} = \sqrt{\frac{4 \cdot 1.54 \cdot 300}{1000 \cdot \pi \cdot 1}} = 0.77 \ mm \rightarrow we\ choose\ 1\phi 8 \rightarrow A_{st} = 50.24 \ mm^2$$

The verification for the steel results satisfied:

$$\sigma_s = \frac{\tau_{v,max} \cdot b \cdot p}{n°_{st} \cdot \pi \cdot \phi^2 / 4} = \frac{0.0068 \cdot 1000 \cdot 300}{1 \cdot \pi \cdot 8^2 / 4} = 40 \ N / mm^2 < \overline{\sigma_s} = 255 \ N / mm^2$$

<u>Measurement of the foundation:</u>

The actions that act on the section of the foundation, in the pre-burying conditions, result:

$$N = -128.30 \ KN$$
$$M = -2.138 \ KNm$$
$$V = 10.85 \ KN$$

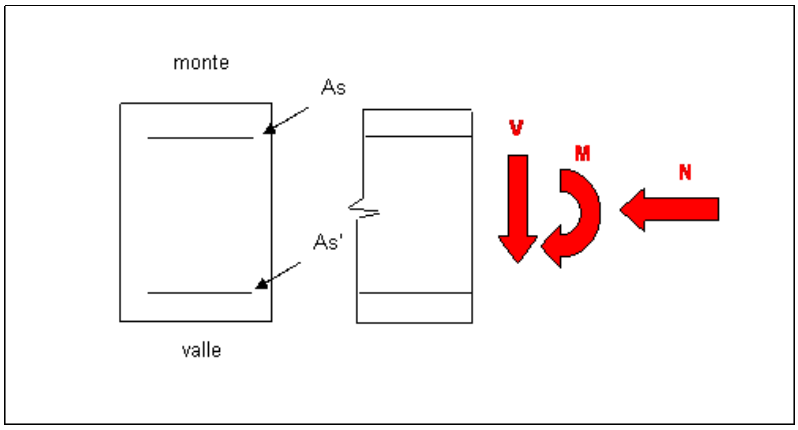

***Figure 85**: Scheme of the actions that act on the section of the foundation of the dam (in the pre-burying conditions).*

Considering the next materials:

- concrete $R_{ck}30$ with maximum admissible tension equal to $\overline{\sigma_c} = 9.75 \ N / mm^2$;
- steel FeB44K with maximum admissible tension equal to $\overline{\sigma_s} = 255 \ N / mm^2$.

The section of the base is marked by:

- base $b = 1000\ mm$ (depth of 1 m);
- altitude of sham beam $h = 2800m$;
- bar cover $c = 40\ mm$;
- useful altitude of the section $d = 2760\ mm$;
- reinforcement ratio $\beta = 0.5$;
- ratio between steel and concrete modular ratio stressed $n = 15$.

Calculation of the eccentricity of the loading:

$$e = \frac{M}{N} = \frac{2.138}{128.30} = 0.017\ m$$

$$e_{lim} = \frac{h}{6} = \frac{2.8}{6} = 0.47\ m$$

Being $e < e_{lim}$ the section is all compressed, so we must verify that there aren't excessive compressions.

<u>Elastic calculation of the section:</u>

At first calculate the minimum area of the armatures:

$$A_{s,min} = 0.0015 \cdot A_c = 0.0015 \cdot 1000 \cdot 2800 = 4200\ mm^2$$
$$A'_{s,min} = \beta A_{s,min} = 0.5 \cdot 4200 = 2100\ mm^2$$

The choice:

$$2\phi 16 \rightarrow A_s = 401.92\ mm^2$$
$$2\phi 16 \rightarrow A'_s = 401.92\ mm^2$$

Equalizing to zero the static moment, calculated respect to neuter axis, of the reagent homogenous section, we obtain the position of the neuter axis (x):

$$S_{nn} = 0$$
$$\frac{bx^2}{2} + nx\left(A_s + A'_s\right) - nA_s c - nA'_s d = 0$$
$$\frac{1000x^2}{2} + 15x\left(401.92 \cdot 2\right) - 15 \cdot 40 \cdot 401.92 - 15 \cdot 2760 \cdot 401.92 = 0$$
$$x = 172\ mm$$

Calculate the inertial moment of the reagent homogenous section with respect to the neuter axis:

$$I_{nn} = \frac{bx^3}{3} + nA_s(x-c)^2 + nA_s'(d-x)^2$$

$$I_{nn} = \frac{1000 \cdot 172^3}{3} + 15 \cdot 401.92 \cdot (172-40)^2 + 15 \cdot 401.92(2760-172)^2 = 4.2 \cdot 10^{10}\ mm^4$$

The maximum compressional stress of the concrete is equal to:

$$\sigma_c = \frac{Mx}{I_{nn}} = \frac{2.138 \cdot 10^6 \cdot 172}{4.2 \cdot 10^{10}} = 0.0088\ N/mm^2$$

The verification of the concrete is satisfied:

$$\sigma_c = 0.0088\ N/mm^2 < \overline{\sigma_c} = 9.75\ N/mm^2$$

but also that of the steel:

$$\sigma_s = \frac{n\sigma_c(d-x)}{x} = \frac{15 \cdot 0.0088 \cdot (2760-172)}{172} = 1.99\ N/mm^2 < \overline{\sigma_s} = 255\ N/mm^2$$

$$\sigma_s' = \frac{n\sigma_c(x-c)}{x} = \frac{15 \cdot 0.0088(172-40)}{172} = 0.101\ N/mm^2 < \overline{\sigma_s'} = 255\ N/mm^2$$

<u>Shear stress:</u>

Calculating the shear stress in the concrete:

$$\tau_{c0} = 4 + \frac{R_{ck} - 15}{35} = 4.43\ N/mm^2$$

$$\tau_{v\ max} = \frac{V \cdot 1000}{0.9 \cdot b \cdot d} = \frac{10.85 \cdot 1000}{0.9 \cdot 1000 \cdot 2760} = 0.0044\ N/mm^2$$

Calculation of the minimum armature as per normative:

$p_{max} = min(0.8 \cdot d, 333) = min(2208, 333) = 333$ we choose p=300 mm.

$A_{st,min} = \frac{V \cdot 1000}{0.9 \cdot d \cdot \tau_{c0}} = \frac{10.85 \cdot 1000}{0.9 \cdot 2760 \cdot 4.43} = 0.99\ mm^2/m$ we choose 1 stirrup.

Calculation of the brackets:

$$\phi_{st,min} = \sqrt{\frac{4 \cdot A_{st,min} \cdot p}{1000 \cdot \pi \cdot n°_{st}}} = \sqrt{\frac{4 \cdot 0.99 \cdot 300}{1000 \cdot \pi \cdot 1}} = 0.62\ mm$$

$$\rightarrow \quad we\ choose\ 1\phi 8 \quad \rightarrow \quad A_{st} = 50.24\ mm^2$$

	The verification for the steel is satisfied: $$\sigma_s = \frac{\tau_{v,max} \cdot b \cdot p}{n°_{st} \cdot \pi \cdot \phi^2 / 4} = \frac{0.0044 \cdot 1000 \cdot 300}{1 \cdot \pi \cdot 8^2 / 4} = $$ $$= 26.26 \ N/mm^2 < \overline{\sigma_s} = 255 \ N/mm^2$$

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	A1.2 - Open check dam
Example of design of a slit check dam:	**Capacity of an open-check dam.**

We consider a torrent with rectangular section and:

- width $B = 8$ m;
- slope $i_f = 40\%$;
- length $L = 1000$ m;
- liquid discharge $Q_w = 100$ m^{3}s^{-1};
- solid discharge $Q_s = 220$ m^{3}s^{-1};
- debris flow discharge $Q_{df} = 320$ m^{3}s^{-1};
- characteristic diameter $d_{50} = 0.05$ m;
- concentration $C^* = 0.5$;
- linear concentration $\lambda_d = 10$;
- density of solid material $\rho_s = 2600$ kgm^{-3};
- density of water $\rho = 1000$ kgm^{-3};
- tan $\varphi = 0.6$;
- dynamic friction angle $\varphi_d = 30°$;
- constant $a = 0.042$;

and with open check dams having the characteristics:

- width of the weir b=5 m;
- height of the weir Y_b=7 m.

We calculate the capacity of an open check dam.

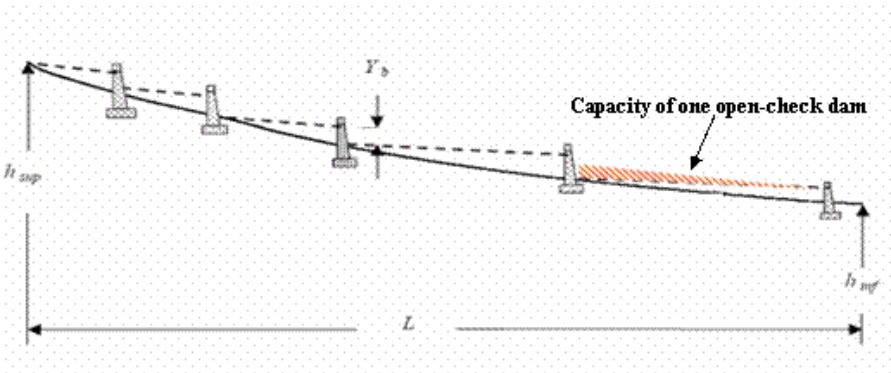

Figure 86: *Capacity of an open check dam.*

1. The volumetric concentration of the debris flow is:

$$C = \frac{Q_s}{Q_w + Q_s} = \frac{220}{100 + 220} = 0.69$$

2. We calculate the slope that causes the debris flow inverting the formula:

$$C = \frac{\tan\alpha}{\Delta(\tan\varphi - \tan\alpha)}$$

so we obtain:

α=17°

3. We calculate the depth of the flow above the debris flow and the coefficient χ_{df} resolving this system of equations:

$$u_{df} = \chi_{df}\sqrt{h\sin\alpha}$$
$$u_{df} = \frac{Q_{df}}{Bh}$$
$$\chi_{df} = \frac{2}{5}\frac{h}{\lambda_d}\sqrt{g\frac{\rho}{\rho_s}\frac{1+C\Delta}{a\sin\varphi_d}}$$

so we have that:

h=6.1m and χ_{df}=5 $m^{1/2}s^{-1}$.

4. The deposit of the solid material upstream the open check dam results equal to (in the simplified case):

$$\frac{\Delta z}{h} = R - 1 \quad \rightarrow \quad \Delta z = 6.1\cdot(1.6-1) = 3.7m.$$

5. The number of open check dams in the reach of the torrent and the distance between one of these and another one results:

$$n = \frac{i_f L - L\tan\alpha}{Y_b} = \frac{(0.40-0.31)\cdot 1000}{7} = 12$$
$$\Delta x = \frac{L}{n} = 82m$$

6. The capacity of one open check dam results:

$$V = \frac{(i_f - \tan\alpha)\cdot\Delta x\cdot B\cdot\Delta x}{2} = \frac{(0.40-0.31)\cdot 82\cdot 8\cdot 82}{2} = 2287m^3.$$

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	A1.3 – Side Protection Walls
Example of design of side protection walls:	This example describes how to do the geotechnical verifications required for the side protection wall represented in Figure 87.

1° case:

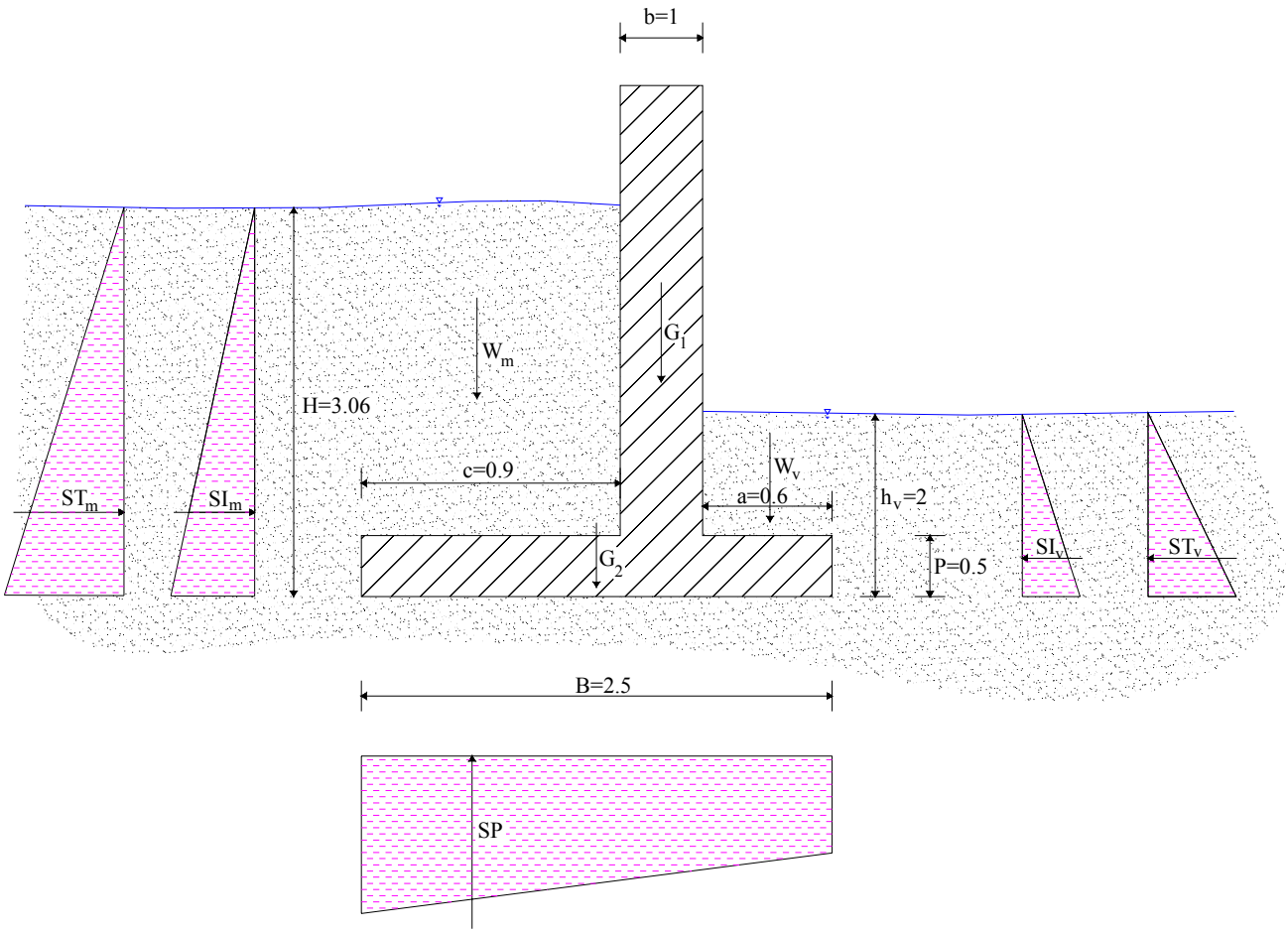

***Figure 87**: Scheme of the agent forces on a side protection wall (1° case).*

	$SI_m = \frac{1}{2}\gamma H^2 = \frac{1}{2}\cdot 10\cdot 3.06^2 = 46.82\ KN$ $b_{SI_m} = \frac{H}{3} = 1.02\ m$ $ST_m = \frac{1}{2}K_a\left(\gamma_{sat}-\gamma\right)H^2 = \frac{1}{2}\cdot 0.24\cdot 9\cdot 3.06^2 = 10.11\ KN$ $b_{ST_m} = \frac{H}{3} = 1.02\ m$ $SI_v = \frac{1}{2}\gamma h_v^2 = \frac{1}{2}\cdot 10\cdot 2^2 = 20\ KN$ $b_{SI_v} = \frac{h_v}{3} = 0.67\ m$ $ST_v = \frac{1}{2}K_0\left(\gamma_{sat}-\gamma\right)h_v^2 = \frac{1}{2}\cdot 0.38\cdot 9\cdot 2^2 = 6.84\ KN$ $b_{ST_v} = \frac{h_v}{3} = 0.67\ m$ $SP = \frac{1}{2}\gamma\left(H+h_v\right)B = \frac{1}{2}\cdot 10\cdot\left(3.06+2\right)\cdot 2.5 = 63.25\ KN$ $b_{SP} = \frac{B}{3}\frac{\left(2\gamma h_v+\gamma H\right)}{\left(\gamma h_v+\gamma H\right)} = \frac{2.5}{3}\cdot\frac{\left(2\cdot 2+3.06\right)}{\left(2+3.06\right)} = 1.16\ m$ $G_1 = \gamma_c b\left(H-P+h\right) = 25\cdot 1\cdot\left(3.06-0.5+1.5\right) = 101.5\ KN$ $b_{G_1} = \frac{b}{2}+a = \frac{1}{2}+0.6 = 1.1\ m$ $G_2 = \gamma_c PB = 25\cdot 0.5\cdot 2.5 = 31.25\ KN$ $b_{G_2} = \frac{B}{2} = \frac{2.5}{2} = 1.25\ m$ $W_v = \gamma_{sat}a\left(h_v-P\right) = 19\cdot 0.6\cdot\left(2-0.5\right) = 17.1\ KN$ $b_{W_v} = \frac{a}{2} = \frac{0.6}{2} = 0.3\ m$ $W_m = \gamma_{sat}c\left(H-P\right) = 19\cdot 0.9\cdot\left(H-P\right) = 43.78\ KN$ $b_{W_m} = \frac{c}{2}+b+a = \frac{0.9}{2}+1+0.6 = 2.05\ m$
Sliding stability analysis:	$N = G_1+G_2+W_m+W_v-SP = 101.5+31.25+17.1+43.78-63.25$ $= 130.38\ KN$ $F_o = SI_m+ST_m-SI_v-ST_v = 46.82+10.11-20-6.84 = 30.09\ KN$ $\eta = \frac{Ntg\varphi}{F_o} = \frac{130.38\cdot tg38}{30.09} = 3.39 > 1.3$

Rotation stability analysis:	$M_{stab} = SI_v b_{SI_v} + ST_v b_{ST_v} + W_v b_{W_v} + G_1 b_{G_1} + G_2 b_{G_2} + W_m b_{W_m} =$ $= 20 \cdot 0.67 + 6.84 \cdot 0.67 + 17.1 \cdot 0.3 + 101.5 \cdot 1.1 + 31.25 \cdot 1.25 + 43.78 \cdot 2.05$ $= 263.57 \ KN \cdot m$ $M_{rib} = SI_m b_{SI_m} + ST_m b_{ST_m} + SPb_{SP} = 46.82 \cdot 1.02 + 10.11 \cdot 1.02 + 63.25 \cdot (2.5 - 1.16)$ $= 142.82 \ KN \cdot m$ $\eta = \frac{M_{stab}}{M_{rib}} = \frac{263.57}{142.82} = 1.85 > 1.5$
Piping verification:	Lane: $F = \frac{\frac{1}{3} L_o + L_v}{\Delta h} = \frac{\frac{1}{3} \cdot 2.5 + 3.06 + 2}{(3.06 - 2)} = 5.5 > 3.5$ Terzaghi: $F = \frac{\frac{\gamma_{sat} - \gamma}{\gamma}}{\frac{\Delta h}{L}} = \frac{\frac{19 - 10}{10}}{\frac{1.06}{7.56}} = 6.4 > 4$
Verification of round bearing analysis:	Suppose to work in drained conditions. Calculate the eccentricity of the loading: $\sum M = 46.82 \cdot 1.02 + 10.11 \cdot 1.02 + 63.25 \cdot 0.09 + 101.5 \cdot 0.15 + 17.1 \cdot 0.95 -$ $-6.84 \cdot 0.67 - 20 \cdot 0.67 - 43.78 \cdot 0.8 = 42.22 \ KN \cdot m$ $\sum N = 101.5 + 31.25 + 17.1 + 43.78 - 63.25 = 130.38 \ N$ $ecc = \frac{\sum M}{\sum N} = 0.32 \ m$ The reduced base is equal to: $B' = B - 2 \cdot ecc = 1.86 \ m$ Calculate the effective vertical stress at the base of the foundation: $q' = (\gamma_{sat} - \gamma) \cdot P = 18 \ KNm^{-2}$ Calculate the borderline loading: $q'_{lim} = \frac{1}{2} B' N_\gamma (\gamma_{sat} - \gamma) + q' N_q = \frac{1}{2} \cdot 1.86 \cdot 78.03 \cdot (19 - 10) + 18 \cdot 48.93$ $= 1533.85 \ KN \cdot m^{-2}$ Calculate the admissible loading of the foundation (unitary depth of the

foundation):

$$Q'_{amm} = \frac{Q'_{lim}}{3} = \frac{q'_{lim}B'}{3} = \frac{1533.85 \cdot 1.86}{3} = 950.99 \ KN \cdot m^{-1}$$

The check is:

$$Q'_{amm} + SP > 3 \cdot \left(G_1 + G_2 + W_m + W_v\right)$$

$$\frac{1014.24}{193.63} \approx 5 > 3$$

2° case:

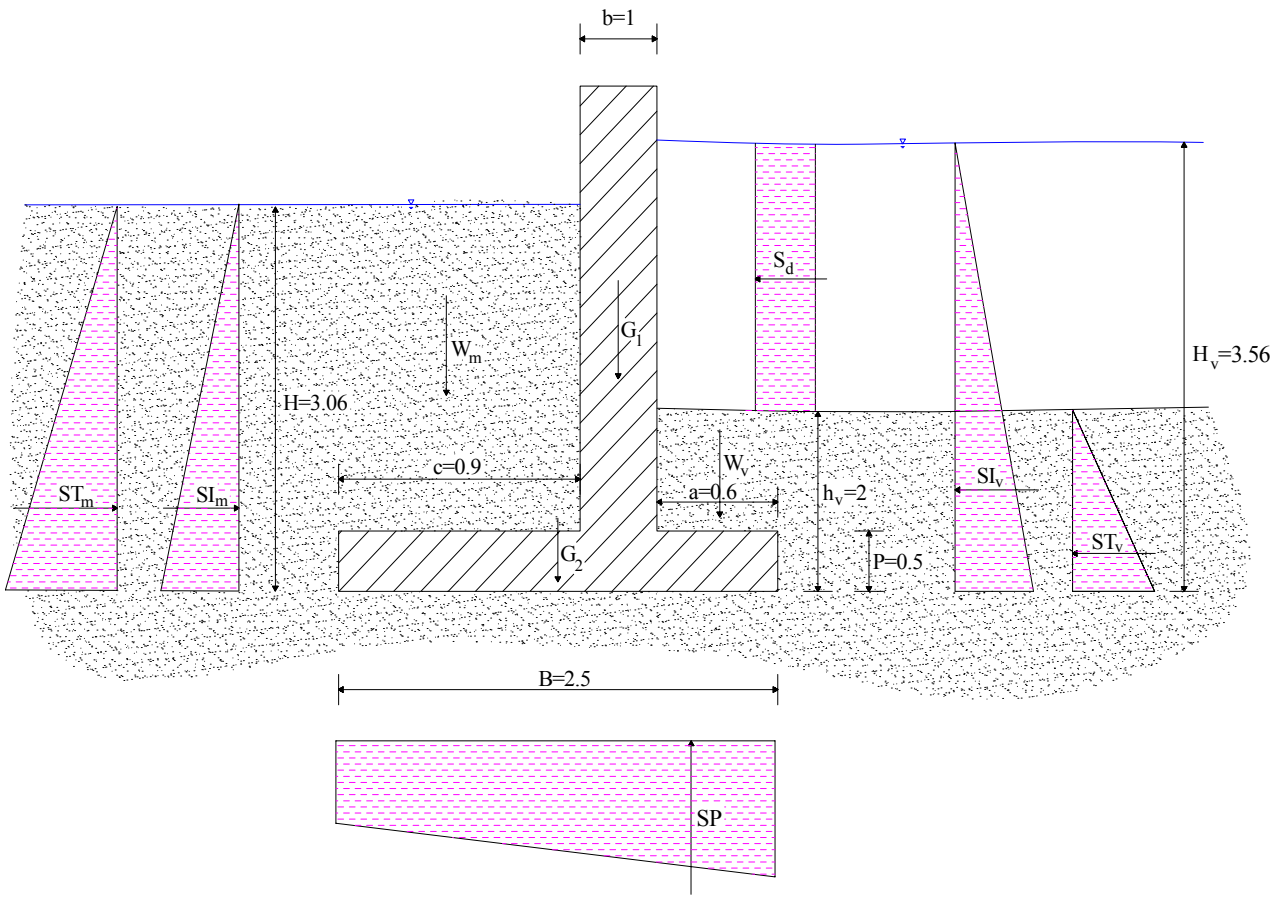

Figure 88: *Scheme of the agent forces on a side protection wall.*

In this case consider the contribution of the dynamic driving force of the debris flow that bangs the wall with an angle of 30°.

	$SI_m = \frac{1}{2}\gamma H^2 = \frac{1}{2}\cdot 10\cdot 3.06^2 = 46.82\ KN$ $b_{SI_m} = \frac{H}{3} = 1.02\ m$ $ST_m = \frac{1}{2}K_a\left(\gamma_{sat}-\gamma\right)H^2 = \frac{1}{2}\cdot 0.24\cdot 9\cdot 3.06^2 = 10.11\ KN$ $b_{ST_m} = \frac{H}{3} = 1.02\ m$ $SI_v = \frac{1}{2}\gamma H_v^2 = \frac{1}{2}\cdot 10\cdot 3.56^2 = 63.37\ KN$ $b_{SI_v} = \frac{H_v}{3} = 1.19\ m$ $ST_v = \frac{1}{2}K_0\left(\gamma_{sat}-\gamma\right)h_v^2 = \frac{1}{2}\cdot 0.38\cdot 9\cdot 2^2 = 6.84\ KN$ $b_{ST_v} = \frac{h_v}{3} = 0.67\ m$ $SP = \frac{1}{2}\gamma\left(H+H_v\right)B = \frac{1}{2}\cdot 10\cdot\left(3.06+3.56\right)\cdot 2.5 = 82.75\ KN$ $b_{SP} = \frac{B}{3}\frac{\left(2\gamma H\ +\gamma H_v\right)}{\left(\gamma H\ +\gamma H_v\right)} = \frac{2.5}{3}\cdot\frac{\left(2\cdot 3.06+3.56\right)}{\left(3.06+3.56\right)} = 1.22\ m$ $G_1 = \gamma_c b\left(H-P+h\right) = 25\cdot 1\cdot\left(3.06-0.5+1.5\right) = 101.5\ KN$ $b_{G_1} = \frac{b}{2}+a = \frac{1}{2}+0.6 = 1.1\ m$ $G_2 = \gamma_c PB = 25\cdot 0.5\cdot 2.5 = 31.25\ KN$ $b_{G_2} = \frac{B}{2} = \frac{2.5}{2} = 1.25\ m$ $W_v = \gamma_{sat}a\left(h_v-P\right) = 19\cdot 0.6\cdot\left(2-0.5\right) = 17.1\ KN$ $b_{W_v} = \frac{a}{2} = \frac{0.6}{2} = 0.3\ m$ $W_m = \gamma_{sat}c\left(H-P\right) = 19\cdot 0.9\cdot\left(H-P\right) = 43.78\ KN$ $b_{W_m} = \frac{c}{2}+b+a = \frac{0.9}{2}+1+0.6 = 2.05\ m$ $S_d = \alpha\rho_{df}u^2(H_v-h_v)^2\sin\vartheta = 0.5\cdot 1900\cdot 1.5^2\cdot 1.56^2\cdot\sin 30 = 2.6\ KN$ $b_{S_d} = \frac{(H_v-h_v)}{2}+h_v = 2.78\ m$

Sliding stability analysis:	$N = G_1 + G_2 + W_m + W_v - SP = 101.5 + 31.25 + 43.78 + 17.1 - 82.75$ $= 110.88\ KN$ $F_o = SI_v + ST_v + S_d - SI_m - ST_m = 63.37 + 6.84 + 2.6 - 46.82 - 10.11$ $= 15.88\ KN$ $\eta = \frac{Ntg\varphi}{F_o} = \frac{110.88 \cdot tg38}{15.88} = 5.46 > 1.3$
Rotation stability analysis:	$M_{stab} = SI_v b_{SI_v} + ST_v b_{ST_v} + S_d b_{S_d} + W_v b_{W_v} + G_1 b_{G_1} + G_2 b_{G_2} + W_m b_{W_m} =$ $= 63.37 \cdot 1.19 + 6.84 \cdot 0.67 + 2.6 \cdot 2.78 + 17.1 \cdot 0.3 + 101.5 \cdot 1.1 +$ $+31.25 \cdot 1.25 + 43.78 \cdot 2.05 = 332.81\ KN \cdot m$ $M_{rib} = SI_m b_{SI_m} + ST_m b_{ST_m} + SPb_{SP} = 46.82 \cdot 1.02 + 10.11 \cdot 1.02 + 82.75 \cdot 1.22$ $= 159.02\ KN \cdot m$ $\eta = \frac{M_{stab}}{M_{rib}} = \frac{332.81}{159.02} = 2.09 > 1.5$
Piping verification:	Lane: $F = \frac{\frac{1}{3}L_o + L_v}{\Delta h} = \frac{\frac{1}{3} \cdot 2.5 + 3.06 + 2}{0.5} = 11.79 > 3.5$ Terzaghi: $F = \frac{\frac{\gamma_{sat} - \gamma}{\gamma}}{\frac{\Delta h}{L}} = \frac{\frac{19-10}{10}}{\frac{0.5}{7.56}} = 13.64 > 4$
Verification of round bearing analysis:	Suppose to work in drained conditions. Calculate the eccentricity of the loading: $\sum M = 6.84 \cdot 0.67 + 63.37 \cdot 1.19 + 2.6 \cdot 2.78 + 43.78 \cdot 0.8 + 82.75 \cdot 0.03 -$ $-46.82 \cdot 1.02 - 10.11 \cdot 1.02 - 101.5 \cdot 0.15 - 17.1 \cdot 0.95 = 35.19\ KN \cdot m$ $\sum N = 110.88\ N$ $ecc = \frac{\sum M}{\sum N} = 0.32\ m$ The reduced base is equal to: $B' = B - 2 \cdot ecc = 1.86\ m$ Calculate the effective vertical stress at the base of the foundation:

	$$q' = (\gamma_{sat} - \gamma) \cdot H = 27.54 \ KNm^{-2}$$ Calculate the borderline loading: $$q'_{lim} = \frac{1}{2} B' N_\gamma (\gamma_{sat} - \gamma) + q' N_q = \frac{1}{2} \cdot 1.86 \cdot 78.03 \cdot (19 - 10) + 27.54 \cdot 48.93$$ $$= 2000.64 \ KN \cdot m^{-2}$$ Calculate the admissible loading of the foundation (unitary depth of the foundation): $$Q'_{amm} = \frac{Q'_{lim}}{3} = \frac{q'_{lim} B'}{3} = \frac{2000.64 \cdot 1.86}{3} = 1240.40 \ KN \cdot m^{-1}$$ The check is: $$Q'_{amm} + SP > 3 \cdot (G_1 + G_2 + W_m + W_v)$$ $$\frac{1232.15}{193.63} \approx 6 > 3$$

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A1.4 – Transport Channel

Example of transport channel design:

A torrent is characterized by a trapezium section with aside slope of 2:1, base B of 5 m and slope i_f of 5%. The d_{90} is of 35 cm and the flow is equal to $Q = 200$ m^{3}s^{-1}. To define the stable diameter on the bed and on the banks of the transport channel.

Adopting $d = 0.35$ m one obtains:

$$K_s = \frac{26}{(d)^{\frac{1}{6}}} = \frac{26}{(0.35)^{\frac{1}{6}}} = 31\ m^{\frac{1}{3}}s^{-1};$$

From the formula of uniform motion it deduces the depth of the water h:

$$Q = K_s\sqrt{i_f}\,\frac{\left(Bh + sh^2\right)^{\frac{5}{3}}}{\left(B + 2\sqrt{5}h\right)^{\frac{2}{3}}}$$

$$200 = 31\sqrt{0.05}\,\frac{\left(5h + 2h^2\right)^{\frac{5}{3}}}{\left(5 + 2\sqrt{5}h\right)^{\frac{2}{3}}}$$

$$h = 2.33\ m$$

And so:

$$A = 5\cdot 2.33 + 2\cdot(2.33)^2 = 22.51\ m^2$$

$$C = 5 + 2\cdot\sqrt{5}\cdot 2.33 = 15.42\ m$$

$$R_h = \frac{A}{C} = 1.46\ m$$

with $B = (5 + 2.33) = 7.33\ m$ the base of the rectangular equivalent section and $h = 2.33\ m$, determining $B/h = 3.15$.

The friction velocity for a trapezoidal section can be evaluated assuming $\xi = 0.94$ in the following expression:

$$u_* = \sqrt{\xi g R_h i_f} = \sqrt{0.94\cdot 9.81\cdot 1.46\cdot 0.05} = 0.82 ms^{-1}$$

The Reynolds's grain number is:

$$Re_* = \frac{u_* d}{\nu} = \frac{0.82\cdot 0.35}{1.31\cdot 10^{-6}} = 2.2\cdot 10^5$$

so $\theta_0 = 0.057$.

The Shields's critical parameter for the bed material of the transport

channel is:

$$\theta_c = \theta_0 \left(\cos\alpha - \frac{\rho_s}{\rho_s - \rho} \frac{\sin\alpha}{\tan\varphi} \right) = 0.057 \left(\cos(2.86) - \frac{2650}{1650} \frac{\sin(2.86)}{\tan(38)} \right) = 0.051$$

The new value of the diameter is equal to:

$$d = \frac{u_*^2}{g \Delta\theta_c} = \frac{0.82^2}{9.81 \cdot 1.65 \cdot 0.051} = 0.81 \ m$$

which is noticeably greater than the assigned value; therefore iterations are needed until the calculated value of *d* equals that of the initial one. After two iterations one finds that the stable diameter on the bed of the transport channel is equal to 85 cm.
For the determination of the stable diameter on the bank one follows the identical procedure.
Assuming as starting value *d* = 0.5 m, one obtains:

$$K_s = 29 \ m^{\frac{1}{3}} s^{-1}$$

and assuming uniform flow conditions:

$$h = 2.40 \ m .$$

$A = 23.52 \ m^2$, $C = 15.73 \ m$, $R_h = 1.50 \ m$ and $u_* = 0.753 \ m$ (assuming $\zeta = 0.77$ for the material on the bank of the trapezoid section 2:1).
The value of Shields's parameter for the material on the bank of the transport channel is equal to:

$$\theta_c = \theta_0 \left(\cos\alpha - \frac{\rho_s}{\rho_s - \rho} \frac{\sin\alpha}{\tan\varphi} \right) \sqrt{1 - \frac{\sin^2\alpha}{\sin^2\varphi}}$$
$$= 0.057 \cdot \left(\cos(2.86) - \frac{2650}{1650} \frac{\sin(2.86)}{\tan(38)} \right) \sqrt{1 - \frac{\sin^2(26.57)}{\sin^2(38)}} = 0.0351$$

The new value of the diameter is equal to:

$$d = \frac{u_*^2}{g \Delta\theta_c} = \frac{0.753^2}{9.81 \cdot 1.65 \cdot 0.0351} = 0.99 \ m$$

After three iterations one reaches at convergence and the stable value of the diameter on the bank of the transport channel is equal to 1 m.

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Annex A2

DESIGN EXAMPLE OF COUNTERMEASURE AGAINST SNOW AVALANCHE

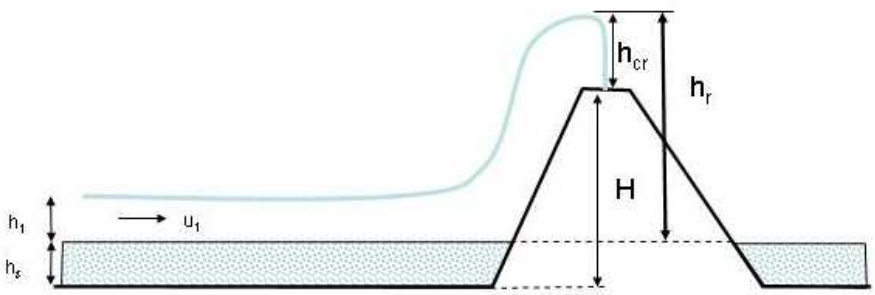
A2.1 – Catching and deflecting dam in the run-out zone

Design dam height

Catching dam

It is assumed that the upstream velocity u_1 and the flow height, h_1, is known (employing e.g. a numerical avalanche models). Also the design snow depth, h_s, (avalanche deposit or snow cover) in front is assumed to be known. Then, two dynamic requirements for determination of the minimum dam height above snow cover needs to be met:

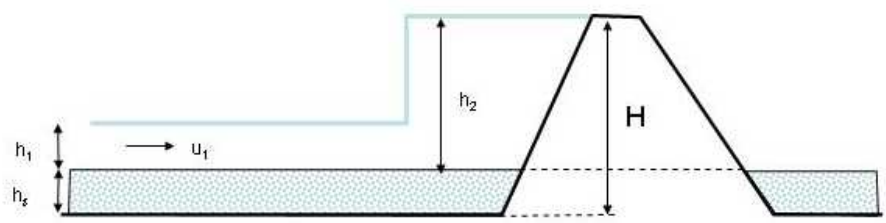
1) Criterion for supercritical overflow (catching dam)

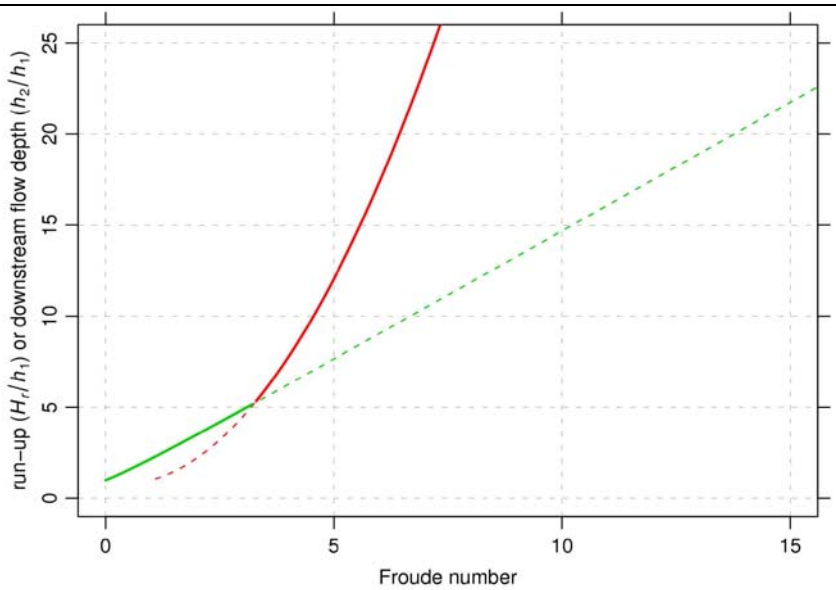


$$\frac{h_r}{h_1} = \frac{H_{cr} + h_{cr}}{h_1} = \frac{1}{k} + \frac{1}{2}(kFr)^2 - \frac{1}{2}(Fr)^{2/3}$$

where:

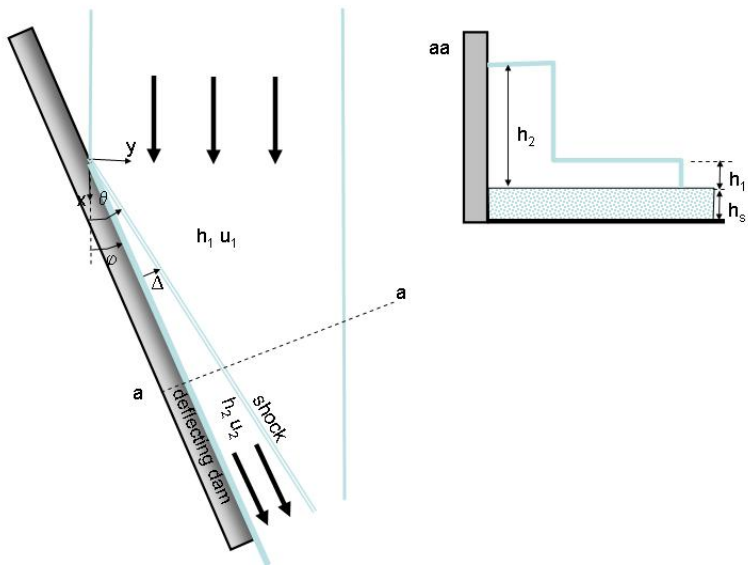
Froude number $$Fr = \frac{u_1}{\sqrt{g \cos\psi h_1}}$$

	upstream flow height h_1 upstream velocity u_1 upstream slope angle ψ critical dam height $H_{cr} = H - h_s$ critical flow depth $\frac{h_{cr}}{h_1} = Fr^{2/3}$ momentum loss coefficient k (= 1 no momentum loss) (proposed momentum loss coefficient = 0.75 + 0.1 (60 - α)/30 for 30 ≤α ≤.60, where alpha is the slope angle of the dam) 2) Upstream shock depth (catching dam)  $$\left(\frac{h_2}{h_1}\right)^3 - \left(\frac{h_2}{h_1}\right)^2 - \left(1 + 2Fr^2\right)\left(\frac{h_2}{h_1}\right) + 1 = 0$$ This equation gives an implicit relation for h_2. Now, the required design dam height, H_{Des}, is given by $$H_{Des} = \max(h_2 + h_s, h_r + h_s)$$ The combined requirements derived from supercritical overflow and flow depth downstream of a shock is expressed graphically in the following figure.



Supercritical run-up, h_r/h_1 = $(H_{cr}+h_{cr})/h_1$ (red curve), and flow depth downstream of a normal shock, h_2/h_1 (green curve) as functions of Froude number, Fr, for a catching dam. The curves are drawn for horizontal terrain (ψ = 0), assuming no momentum loss in the impact (k = 1). The part of each curve corresponding to larger dam height is drawn as a solid thick curve.

Deflecting dam



Schematic figure of an oblique shock above a deflecting dam showing the deflecting angle, φ, the shock angle, θ, their difference Δ = θ−φ.

1) Criterion for supercritical overflow (deflecting dam)

The criterion for supercritical overflow for deflecting dams is similar to that

for catching dams except the Froude number normal to the dam axis is used

$$\frac{h_r}{h_1} = \frac{H_{cr} + h_{cr}}{h_1} = \frac{1}{k} + \frac{1}{2}(kFr_\perp)^2 - \frac{1}{2}(Fr_\perp)^{2/3}$$

where:

Froude number normal to dam axis $\quad Fr_\perp = \dfrac{u_1 \sin\varphi}{\sqrt{g\cos\psi h_1}}$

2) Upstream shock depth (deflecting dam)

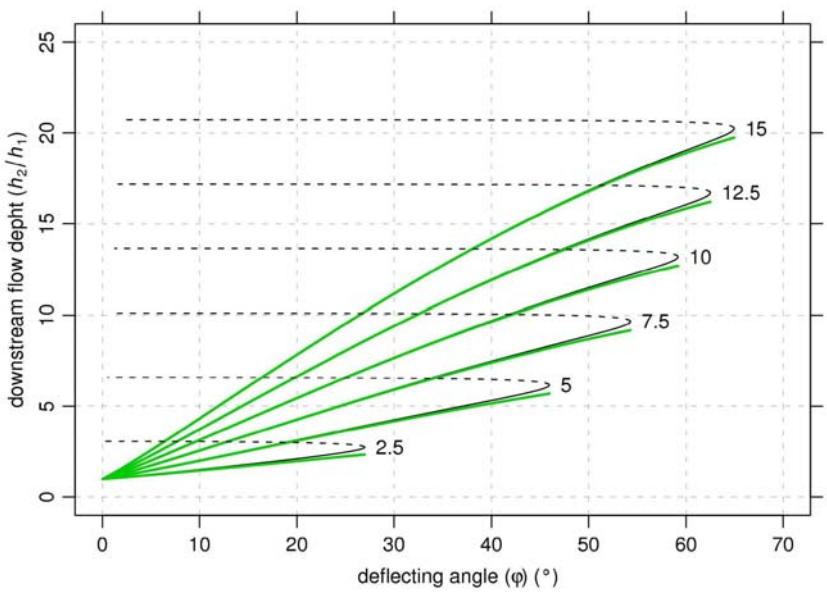
$$\frac{h_2}{h_1} = \frac{2\sqrt{6Fr_\perp^2 + 4}\cos\delta + 1}{3},$$

where

$$\delta = \frac{1}{3}\left(\frac{\pi}{2} - a\tan\left(\frac{9Fr_\perp^2}{Fr_\perp\sqrt{27\left(16 + 13Fr_\perp^2 + 8Fr_\perp^4\right)}} - 8\right)\right),$$

The widening of the oblique shock is approximately

$$\Delta = \frac{\cos\varphi\sin\varphi}{\cos^2\varphi(h_2/h_1) - 1}$$

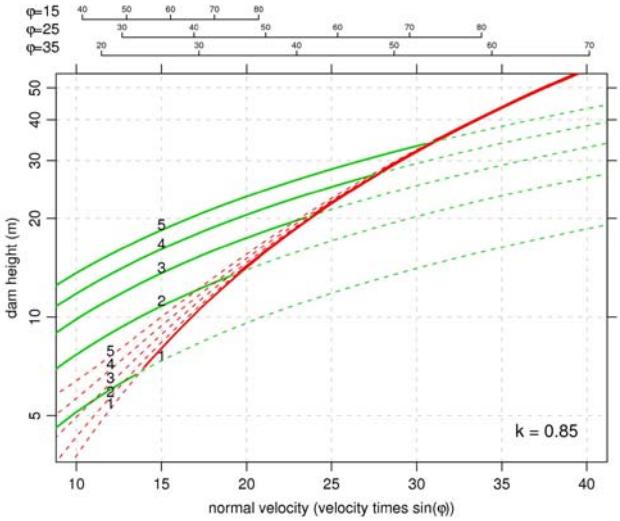


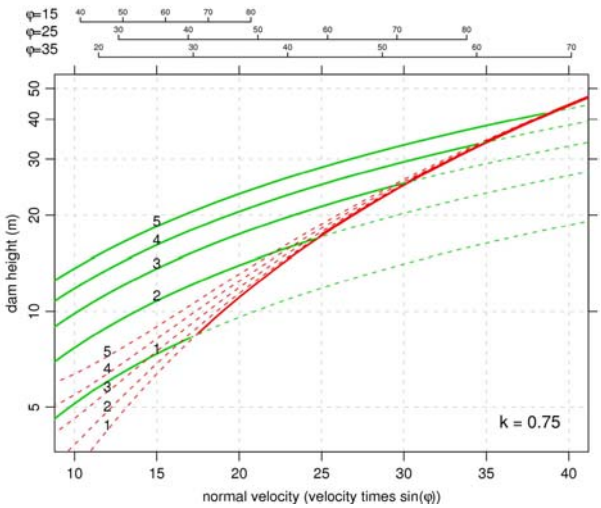
Flow depth downstream of an oblique shock as a function of deflecting angle φ. Thin solid (weak shock) and dashed (strong shock) curves show the solutions given by the oblique shock relations. Thick green curves show the results given by the approximate solution defined by the equation for the upstream shock depth. The curves are labeled with the Froude number Fr = $u_1/(gh_1 \cos\psi)^{0.5}$.

Again, the required design dam height, H_{Des}, is given by

$$H_{Des} = \max(h_2 + h_s, h_r + h_s)$$

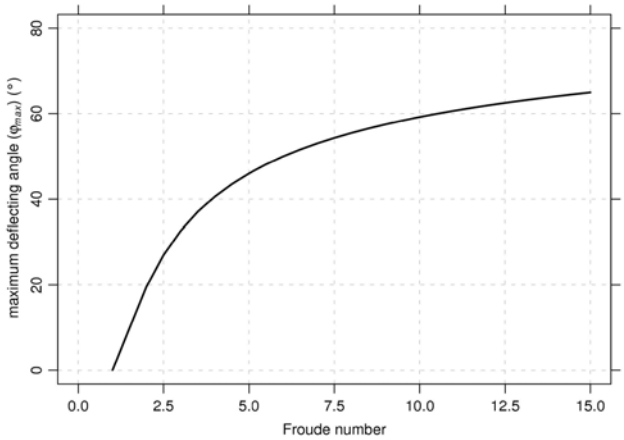
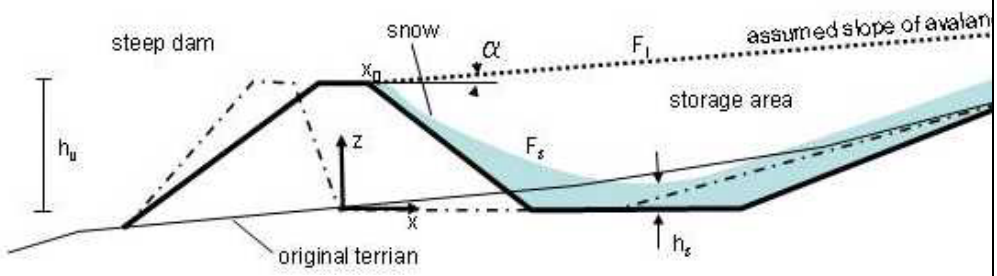
The following figures provide a design help for various parameter combinations.

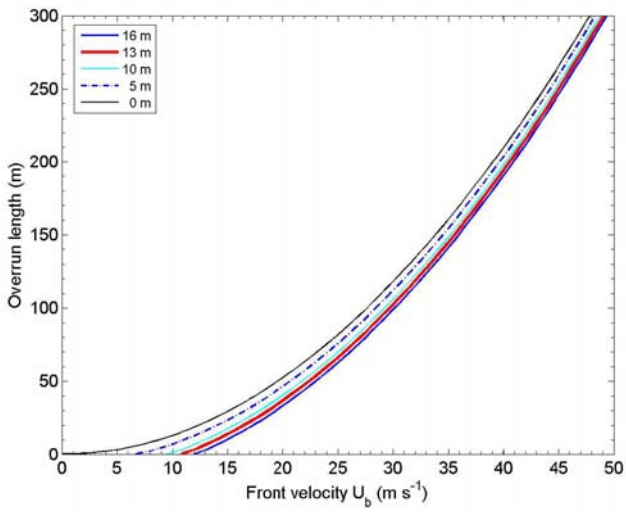




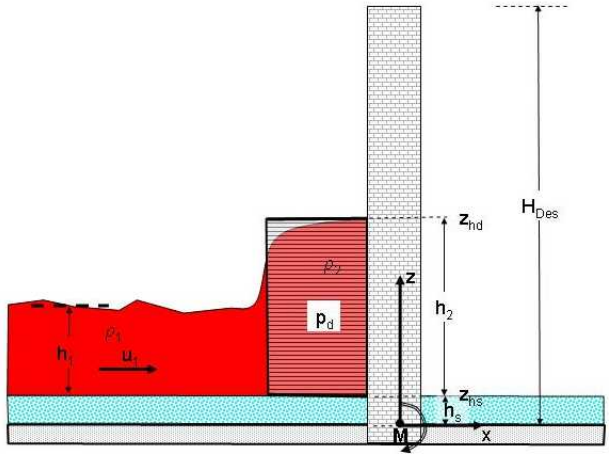
Design dam height above the snow cover $H-h_s$ as a function of the component of the velocity normal to the dam axis, $u_\perp = u_1 \sin\varphi$, for several different values for the depth of the oncoming flow h_1. Momentum loss in the impact with the dam is assumed with k = 0.85 (left, corresponding to dams built from loose materials) and k = 0.75 (right, corresponding to steep dams). The figures show curves derived from both supercritical overflow (red curves) and shock dynamics (green curves). The design dam height should be picked from the higher of the two curves corresponding to the estimated design flow depth. The part of each family of curves corresponding to the higher dam is drawn with solid, thick curves. The labeled axes at the top of the figures show velocity corresponding to the deflecting angles φ = 15, 25 and 35°. Dam height normal to the terrain determined from the figures must be transformed to vertical dam height.

To keep a deflecting dam most efficient, the deflecting angle, φ, need to be kept small. The maximum deflecting angle is a function of the $Fr = u_1/(gh_1$

	$\cos\psi)^{0.5}$ as shown in the figure below. 
Storage capacity	 The storage volume may then be found from the equation $$V_s = w\int_{F_s}^{F_l}(F_l(\mathbf{x}) - F_s(\mathbf{x}))dx$$ where $F_l(\mathbf{x})$ is the elevation of a straight line from the top of the dam towards the mountain with a chosen slope in the range α = 0–10°, $F_s(\mathbf{x})$ is the elevation of the top of the snowcover before the avalanche falls, and x_0 and x_1 are the locations of the dam and the point where the line intersects the snow covered mountainside, respectively. For dams where dry-snow avalanches are expected, deposit slopes close to α = 0° should be used, but for locations where wetter avalanches are typical slopes up to α = 10° can be chosen.
Overrun length of catching dams	The overrun length of an avalanche that surpasses a catching dam can be approximated by $$\frac{l_{ovr}}{h_{fb}} = b_1 \frac{u_b^{\ 2}}{2gh_{fb}} + b_0$$ where: overrun length l_{ovr} front velocity at the dam base u_b height of the free board $h_{fb} = H_{Des} - h_s$

	design dam height H_{Des} snow depth in front of dam h_s empirical coefficients $b_1 \approx 2.56$ $b_0 \approx -1.41$  Overrun length vs. front velocity U_b calculated for the catching dam
Static/ dynamic design criteria	 The construction of a building or wall-like structure in an avalanche prone area requires an assessment of reasonable design loads, i.e., estimates of the total maximum force, F_m, and moment, M, due to an avalanche. In general, three flow regimes should be distinguished for the determination of the impact force on a wall-like structure. • dense flow • fluidized flow (also referred to as saltation layer) • suspension flow (powder part)

However, here we consider only the dense flow part:

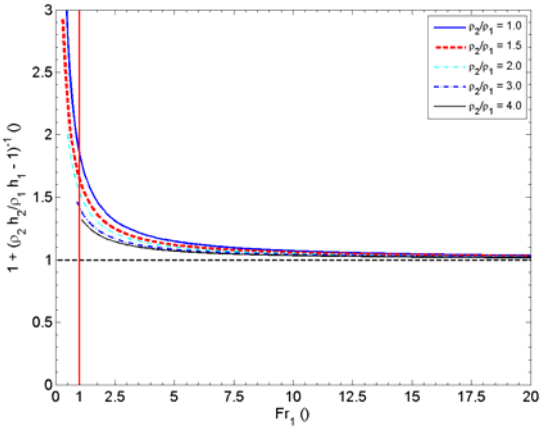


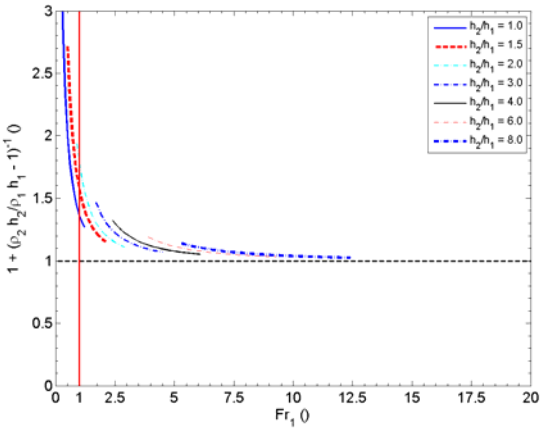
The force per meter width, F_{dx}, normal to the dam can be approximated by (disregarding the slight reduction by inclined walls)

$$F_{dx} = \int_{z_{hs}}^{z_{hd}} p_d dz \approx \rho_1 (u_1)^2 \left[f(Fr) + \frac{1}{2Fr^2} \right] \frac{h_1}{h_2} (z_{hd} - z_{hs}) \text{ [N m}^{-1}\text{]}$$

where:

Froude number	$Fr = \frac{u_1}{\sqrt{g \cos\psi h_1}}$
upstream flow height	h_1
upstream velocity height	u_1
upstream density	ρ_1
height downstream of shock	h_2
down stream density	ρ_2
snow depth	h_s
upstream slope angle	ψ
intensity factor	f(Fr)
z-coordinates	z_{hd}, z_{hs}





Intensity factor f (Fr) =1+($\rho_2 h_2/\rho_1 h_1$−1)−1 versus Fr. Left panel, ρ_2/ρ_1 taken as parameter; $h_2/h_1 \geq 1$. The density ratios between 1 and 4 are chosen to correspond to possible density ratios in snow avalanches. Right panel, h_2/h_1 taken as parameter; densification ρ_2/ρ_1 ranges between 1 and 5.

Maximum shear stress on the face may be approximated by

$$\tau \approx \mu\rho_1(u_1)^2\left[f(Fr)+\frac{1}{2Fr^2}\right] \text{ [Pa]}$$

where the friction coefficient, μ, is in the range of 0.3 to 0.5.

The moment about the x-axis is

$$M_{sy} = \frac{z_{hs} + z_{hd}}{2} F_{dx}$$

Geotechnical design

Dams made of loose deposits (earth materials)

Ground investigation

Ground investigations must be accomplished to ensure that the soils are usable for the construction and that the stability of the underlying ground is sufficient.

By the ground investigations one must:

- check the depth to the underlying bedrock and the amount of loose deposits
- collect soil samples for geotechnical testing in the laboratory

Core drillings or excavation to be used to obtain soil samples.

- The soil samples must be analysed in a geotechnical laboratory.
- A sieve analysis is important and should always be performed.
- Construction of grain-distribution curves which clarifies the relative amounts of the different types of materials in the sample (clay, silt, sand, gravel).
- In special cases, triaxial tests will be performed to calculate the angle of repose of the masses.

Based on the soil samples, **geotechnical experts** must calculate

- the global stability of the ground,
- the stability of the dam itself
- and make a detailed plan for the construction. (Slope angles of the fill, build up of the dam, erosion protection, drainage of the dam area etc).

Dam construction

A dam is most commonly constructed of natural soils found at the dam site or in the vicinity of the dam. A dam built in mass balance is a clear advantage for the economy.

Mass balance means that the excavation is done in situ, just above the dam, and that all the excavated masses are used in the dam fill. By such a procedure the fill volume may be reduced also, as the effective dam height is the sum of the fill itself and the depth of the excavated masses.

When dealing with earth fill dams, and especially with dams where fine grained materials are used, the following points must be assessed:

- quality of the earth materials
- treatment of organic material in the ground
- design of the dam
- design of the excavation area
- water, drainage and erosion protection

Quality of the earth materials

All kinds of loose materials, from clay, silt, sand, gravel and rocks may in principle be used for the construction of a dam. In fine-grained, cohesive

materials as clay and silt, the drainage of water is a very slow process. Pore pressures might build up during the construction phase or later during heavy rainfall and reduce the stability of the dam.

A rule of thumb says that if more than 10 % of the dam fill consists of fine-grained material one has to make extra precautions in the construction to ensure that the water drainage from the body of the dam is sufficient, to obtain satisfactory stability of the dam. This induces extra costs for the construction. It is therefore a clear advantage to use coarse grained, frictional materials as gravel and rocks for dams made of loose deposits.

Organic materials

If organic masses are present they must be removed before the construction, both under the dam itself and from the excavation area. If the organic materials are not removed, they will be compressed and settle by the weight of the dam. Bog material will settle up to 90 %. Such organic layers may exhibit weak layers and act as failure planes below the dam, especially in sloping terrain.

Design of the dam

Fine-grained cohesive materials will not be stable with inclinations steeper than 1: 2. For friction materials as sand and gravel, the maximum steepness of the dam sides should not exceed 1: 1.5 (34°) to obtain satisfactory stability. For coarser frictional materials one can obtain a stable inclination of the dam sides up to 1: 1.25 (39°).

In steeper dams one should use dry walls, reinforced earth or concrete, see section below. The steeper inclination is a clear advantage for the stopping and deflecting effect.

In conclusion, the slope of loose materials should not be steeper than the figures given below:

- *Fine grained materials max 1:2*
- *Sand, gravel max 1:1,5*
- *Loose layered rocks 1:1,25*
- *Dry walls with rocks 3,5:1-4:1*
- *Reinforced earth, geotextiles 4:1*

Fine grained masses must be sorted out from the excavation. If one decides to use fine grained material in the dam, the fines must be built into the dam in succession with coarser material to ensure sufficient drainage. A common practise is to make a layered construction with horizontal thin layers, coarse grained alternating with fine grained layers. The layers should not exceed a thickness of 0.5 m, and be levelled out and compacted by heavy machinery.

Design of the excavation area

	The excavation area above the dam must be made broad enough to prohibit the avalanche masses to jump over the dam from the natural terrain surface above the excavation. A minimum width of the excavation area should be about 50 m. The width depends on the avalanche velocity and avalanche volume and must be calculated in each single case. The layout of the excavation must ensure that the effective height of the dam is retained; the excavation must not be so deep and narrow that the dam ends up in a "ditch". The sides of the cut must be gentle enough to ensure stability of the earth masses along the cut, and should normally not be steeper than 1:1.5. Coarser deposits (gravel, boulders) will be stable up to 1:1.25, and if clay and silt makes up for most of the cut, the inclination should not be steeper than 1:2. **Dams with steeper sides** It is possible to increase the inclination of the dam sides by the use of materials as: • dry walls • reinforced earth • concrete, steel As earlier mentioned, a steep dam at the avalanche side of the dam will increase the effect of the dam. Dry walls consist of a "masonry" of boulders with a back fill of other earth materials. The boulders should not be smaller than abut 0,5 m^3 and be built up in bonded layers. Experience has shown that dry walls with inclinations up to 4 : 1 (76°) are stable, provided that the foundation is adequate and the dry wall itself is designed to withstand the earth pressure from the backfill. To withstand the earth pressure the thickness of the dry wall must increase with the height of the dam. The ratio of the thickness of the dry wall to the dam height should not be less than 1:5, i. e., a 10 m high dam will need a 2 m thick dry wall. To increase the stability, it is advantageous to tilt the boulders a little into the wall. The foundation of the dry wall must consist of materials not subjected to frost heave, (sand, gravel, boulders) and both the foundation and back fill must be drained. Calculations to ensure sufficient global stability and stability of the dam itself is necessary As for the use of reinforced earth, many solutions are possible, and different commercial products are available on the market. The reinforcement may consist of nets or gabion boxes of galvanised steel, or net constructions of polymers, as polyethylene, polypropylene, polyester etc. The reinforcement is applied as the outer cover of the dam, and makes it possible to build dams with inclinations at 4:1. The earth materials (usually gravel or stones) are embedded into the cases or into the nets and kept in place by additional

	anchors built into the dam. All such constructions must be designed by geotechnical experts to ensure a safe and optimal layout. A combined defence structure system consisting of 10 m high braking mounds and a catching dam is built by using steel cases and steel rods as reinforcement. Concrete constructions are well known as deflectors and for catching purposes. The advantages in preference to constructions made of earth materials are, firstly, that it is possible to obtain a vertical wall construction, which is more effective concerning kinetic energy dissipation from the moving avalanche. Secondly, the concrete walls are much slender than dams made of natural deposits only, and consequently needs much less space. The major drawbacks are high costs and unpleasant visual impacts. For such slender constructions the avalanche impact pressure must be carefully calculated as the walls must withstand the pressure without tilting or being displaced. For these reasons, the foundations are especially important. In bedrock, the foundations are usually made by steel tension anchors (ribbed bars) in boreholes with cement grout. In loose deposits one must ensure that the ground is able to withstand the weight of the concrete construction plus the loads from the avalanche impact. A foundation platform of concrete is normally used as a base for the wall. The base must be frost proof and the area around it well drained. Steel constructions may also be used for deflecting and retaining purposes. In special cases steel has been used for this purpose, but such constructions are not common, as they tend to be more costly than other materials.

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