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"Partnerships for a Safer World in the 21st Century"

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Building a Culture of Prevention

**VAW**
Laboratory of Hydraulics, Hydrology and Glaciology of the Swiss Federal Institute of Technology, Zurich

Coping study on

# DISASTER RESILIENT INFRASTRUCTURE

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"Partnerships for a safer world in the 21st century"

UNITED NATIONS

# PREFACE

In December 1998 an agreement was signed to provide support for the organization of IDNDR Program Forum to be held in July 1999 and its preparatory process through undertaking a coping study on the theme *Disaster Resilient Infrastructure* by Versuchsanstalt für Wasserbau, Hydrologie und Glaziologie **(VAW)** of ETH Zurich within the project "Coping Studies on Research Needs for Future Disaster Reduction". These coping studies are implemented and coordinated by the Graduate Institute of International Studies, Geneva, the Programme for the Study of International Organizations (HEI-PSIO).

**VAW** is doing research only in some fields of natural hazards i.e. floods, debris flow, impulse waves and ice avalanches. Therefore, it was necessary to find partners to contribute to this report. Fortunately it was possible to find experts in each field of natura! hazard that were willing to write a chapter of this report. I take this opportunity to thank all authors for their valuable conmbutions. **A** detailed list of all authors **is** provided.

To contribute to the coping study was a chalienge. It is not easy to summarize the essentials on such limited space. And if the report gets too voluminous it would be too difficult tc read. 1 hope that the right equilibrium was found and this report introduces the reader on the main problems, **risks,** but also research needs and necessary activities to be taken in relation to natural hazards.

I want to thank Dr. Warner, Director of PIIO, the project coordinator for the excellent cooperation and Dr. Hager for having coordinated as a project head.

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# GENERAL REMARK AND SUMMARY

H.-E. Minor

Economic losses attributable to natural hazards rise steadily as the figures of Munich Reensurance demonstrate. And eacn year a notable number of persons are killed or displaced at least for some time from their home areas. There are several reasons for the increase of impact by natural hazards:

- *Extension of settlements* including the corresponding infrastructure and productive plants is continuing. Not only the growing number of people is a reason but also the steady improvement of the built environment. Globalization and the pronounced division of labor in world economy will add even more in the future.

- There is and will be *more infrastructure* in the future that can be damaged; its construction cost is steadily increasing.

- Human activities with its settlements and infrastructure spread into endangered areas sometimes brcause no other space is available. This is convenient just on a short sight. *Construction costs* at flood plains for example are lower than on hilly ground. Since floods occur not every year, larger floods more than five years ago are normaly forgotten.

- Sports-activities and *tourism* also push into more extreme areas and add to the necessary infrastructure.

All these structures are exposed to a high **risk** but at the same time they are expected to withstand disastrous impacts during natural hazards. This is not always possible. Man must realize that *100% safety does not exist,* especially not if structures are exposed consciously to natural hazard. They cannot be made safe against all possible impacts of natural hazards. In some cases it is simply not possible because of lack of technical means while it would be much too expensive in other situations.

Another approach is to define *hazard zones*. In the most critical zones with a high hazard potential construction could be prohibited, in the second zone with a moderate potential hazard, prescriptions should be made to armour structure against the natural hazard, and in a third zone owners have to be informed about existing hazard. Additionally it is essential to build up a *second line of defence* in case the first defense line fails. Needless to state that a warning system as well as rescue measures have to be installed. The warning system is then effective provided real-time-prediction is possible and the *rescue measures* are effective if extensive training has been carried out for specific hazards.

The various natural hazards have different character because they are governed by different physical processes. Accordingly, the methods of *hazard intervention* also differ. Table 0.1 attempts to demonstrate these differences and at the same time intends to show the possibilities of intervention. Three zones have been distinguished:

- Origin or source of hazard,
- Propagation or spreading area, and
- Zone of impact.

For extreme natural hazards, structures are essentially not able to resist, while other can be dealt with by a correct design. For many natural hazards it is nearly impossible to intervene at the source, for some, however, this approach is feasible such as landslides. Then, of course, this should be the first line of activity. As can be seen from Table 0.1 intervening in the propagation/spreading area is effective for many natural hazards.

In addition to the possible actions to be taken as listed in Table 0.1, consequent *regional planning* with definition of hazard zones would reduce considerably the impact of natural hazards to infrastructure *Hazard zoning* should be defined not only for one natural hazard

scenario, but all natural hazards of a site should be investigated at the same time define the combined **risk** of endangered areas.

In this context it must be mentioned that different hazards are treated separately by the corresponding specialist. However, two or more natural hazards may ***interact*** and the experts have to come up with a common definition of solution. **Future** research has to take this aspect also into account.

The different chapters of this report aim to present the specific research needs in more detail or define the necessary activities *to* be carried out to make infrastructure more disaster resilient, as regarded by the authors.

Table 1 Possibilities of hazard intervention

| Hazard / Lone | ① wind | ② snow avalanches | ③ ice avalanches | ④ rockfall | ⑤a landslides | ⑤b debris flow | ⑥ impulse wave: | ⑦ earthquake | ⑧ forest fires | ⑨ floods |
|---|---|---|---|---|---|---|---|---|---|---|
| Origin or source | none | supporting structures, artificial release of avalanches, silviculture, reforestation | none | limited | slope geometry, retaining structures, slope reinforcement, drainage. Large, steep slopes cannot be stabilized | check dams in torrents and scour measures, as for landslide | stabilize slides, draw down reservoir, control slide velocity | none | | none |
| Propagation spreading area | little, consider vortex shedding | deviation dams, retaining dams, retarding structures | structures to divert avalanche away from vulnearable structures | various structures to hold brick rockfall | none | protection dams to keep debris flow from vaiunerable structures<br><br>Japanese trap | drawdown of reservoir | none | | Retention basins, flood plain diversion structures, reduce sediment and drift wood supply |
| Zone of impact | apply state of the art design | shed structures | very limited | rock sheds | structures arc csscntially not able to resist thrust by landslide | see landslides | vcry limited | adequate design of structures, but also of infrastructure | | increase river capacity, flood dykes, more space for river, increase erosion resistance |

# 1 WIND LOADS

B. Zimmerli

## 1.1 INTRODUCTION

The best chance to reduce disaster caused by storms is to build the structures conforming to the state-of-the-art in wind engineering or at least for normal structures to the wind codes (Münchner Rückversicherungs-Gesellschaft 1989). Structures and their components should be built with sufficient resistance. Very exposed terrains like the top of mountains and exposed coastal regions need special regulations imposed by the government.

**An** important influence on damage control have *insurance companies.* They have the possibility to steer to a certain point the public, the industry and the authorities, and may force the policy holders to prevent damage.

## 1.2 STATIC AND DYNAMIC WIND LOADS

The dynamic pressure acting on the surface of the building is proportional to the square of the wind velocity. **At** the edges of the roofs and the walls in the wind direction the pressure and suction forces may reach the multiple of the usual dynamic wind pressure. Generally the distribution is more erratic if the shape of the building is irregular and sharply-edged. The damage on roofs and facades shows clearly, where the extreme pressures have an influence. Often designers are suprised by the locations of the damage, which means that the *engineering education* is incoinplete.

In the design of buildings and components the indications and prescriptions of the codes should be used. In certain cases the static approach to wind loads does not show the true interactions between the structure and the wind flow. The unsteady and turbulent flow can cause resonance of the structure. Fig.1.1 shows a design mistake in the concept of cooling tower construction. Damages may occur even at moderate wind velocities. The parameters of

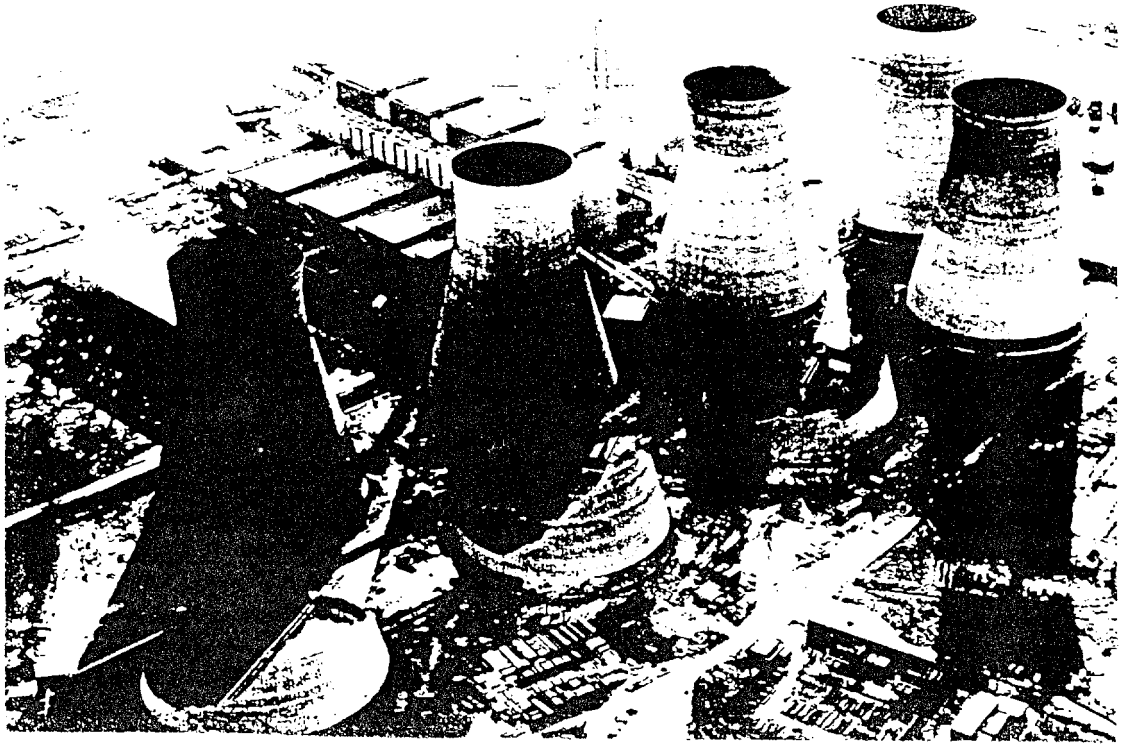


Fig. 1.1 Cooling towers damaged by exceptional wind load.

vortex shedding used in the codes are known. The modem tendency to higher and at the same time lighter structures make them more susceptible to vibration.

Wind engineering deals with *wind-induced vibrations*. Measurements in situ and in wind tunnels together with computer simulations of the wind flow allow specialists to predict the wind loads on unusal and new structures.

## 1.3 CODES

Most of the countries have detailed design regulations concerning wind action. Some are not compelling and sometimes even contradictory. Therefore, engineers tend to save money with measures against wind load. Economically this is certainly the wrong way since even in extremely exposed coastal regions the savings are less than 4 percent, compared to perhaps the total loss of the structure. The basic values of wind velocities are often derived from a wind velocity (as a mean over 10 minutes) measured in the average every 50 years. These values are converted to peak pressures.

Some of the problems pertaining to wind codes will be shortly discussed. First not all countries can provide the necessary *meteorological data* over a sufficiently long period, especially in topographically exposed terrains. Changes of the climate cannot be predicted with the necessary reliability.

Secondly the wind codes are no more readable for ordinary engineers or architects. Only with typical examples for wind actions in courses for professionals will this problem be resolved. The regulations often show buildings of simple shape whereas very complex structures must be built in daily application. This problem becomes even more acute in the context of roofs and facades. For many structural shapes the pressure and suction distributions are not known. The investor often is not willing to pay wind tunnel testing for better load prediction. Experience of damage is not transferred to the knowledge of engineers. Only a few years after major disasters engineers are forced to reduce expenses against wind action. The influence of neighbouring buildings is not sufficiently taken into account. The danger of debris transported by wind in agglomerations is also not considered. The many details which have to be accounted for in design against wind action ask for a long experience in the profession of engineers, architects and especially workmen. All the preceding aspects taken into account will allow us to build structures which resist storms.

## 1.4 POSSIBILITIES TO PREVENT DISASTERS

Every year insurance companies all over the world pay damages of a huge extent. They would be able to collect data of all the damages and to transmit the *experiences* to the interested engineers and architects. This happens very seldom, but it could be the decisive step to a disaster-resilient strategy. Since there is still no move in this direction, some typical arguments for damages and possible preventions will be discussed. The whole spectrum of essential causes, even such beyond wind storms, will be discussed.

### 1.4.1 Roofs

Why are roofs damaged most of all? Causes include larger wind velocity with increasing height over ground, vortex shedding at sharp comers and cantilevering roofs, small roof weight, insufficient anchoring of the roof to the building, poor fixing of the skin to the roof structure and inexistent maintenance of roof elements such as chimneys or antennas. Roof damage is especially annoying because the rain destroys the insulation and increases the total damage considerably. The parts loosened from the roof are transported by the wind. They hit other buildings. Sometimes the stability of the whole building is threatened if the roof is blown off. Measures against these kinds of damage can be derived from what has been said

**Fig.1.2** Roof damaged **by falling tree during** a storm.

The trees must have a reasonable distance from the building or they should only reach the eaves (Fig.l.2). Due to a limited overhang suctions and pressures are reduced. Reliable connections between the roof and the underlying structure are important. Nails are normally not the ideal solution. The skin of the roof can be fixed to the beams by screws.

### 1.4.2 Exterior walls and facades

Exterior walls are damaged by extreme storms only. Since more and more expensive and at the same time sensitive materials are used, the wind forces will impair curtain walls and panels more frequently in the future. Debris and hailstones increase the damage to such buildings dramatically. Glass areas are even more sensitive to wind actions (Fig.l.3). Resonacce vibrations are an additional cause of failure. The same is valid for gates and sliding doors of hangars and halls. Because of the demand for small weights the gates are produced of light metals. Under high wind pressure this material may deform extremely. Often the gates leave the frame and are therefore destroyed. Possible preventive measures are given as examples.

The elements of the facade and of the insulation need *sufficient anchoring* to the masonry and to the steel profiles. The material should be resistent against hail. Big glass elements should have flexible suspensions and subdivided panels. Light metal gates should be stiffened to such **an** extent, that the deformations are limited according to the dimensions of the frame. The determining elements must be maintained and checked on corrosion. It is important to take into account the internal pressure correctly.

### 1.4.3 Airdomes and tents

Airdomes are a modem development in civil engineering whereas tents are rediscovered today. Both are very susceptible to wind action, especially if certain important safety measures are overlooked (Fig.l.4). The material is normally PVC with a thickness of 0.7 - 1.2 mm. In the interior a small overpressure is needed. The wind forces on the domes are modest. Damage starts if the interior pressure is not increased early enough or if the anchors cannot withstand the lift forces. Therefore, the following rules should be respected for airdomes.

At least two bellows must be installed, which can quickly increase the interior pressure with the upcoming storm and which are able to maintain the pressure if one below fails. Since the electricity is often interrupted during storms, an emergency power generator is needed. The steering of the bellows should be connected with an anemometer, which allows to increase the pressure if the critical wind velocity is reached.

**Fig.1.3** Damages on building facade **due** to storm.

The bearing structure of a tent has to be designed to extremely high wind actions. The safety regulations of tents normally consider this problem for severe hazard situations. Important is the anchoring of the cables into the ground. If the material is tearing, the situation is modified completely. The wind entering the tent may cause a so-called 'flying' building.

### 1.4.4 Scaffolding and cranes

The sufficient anchoring of temporary structures is often neglected (Fig.1.5). This leads to heavy damages not only for the scaffolding and the cranes but also for the environment. Cars, buildings and sometimes even persons may be harmed. Some simple rules have to be respected to prevent damage.

**Fig. 1.4** Roof damage of air dome.

The scaffolding needs a safe connection with the building under construction or under repair. Weather forecasts have to be followed to stop working early enough. The maintenance of scaffolding and cranes is important. The cranes should be fixed to the rails with bolts and Joints. The crane-jib should be free to adapt itself to the wind direction, without threatening neighbour buildings. A further possibility is to take apart the crane or the scaffolding shortly before the storm arrives.

### 1.4.5 Tanks, vessels and cooling towers

During construction tanks are often threatened by storms because the cover providing the necessary stiffness is missing. The overalling of the open section caused by vortex shedding in resonance with the structure can lead to complete structural failure (Fig.l.l).

The prevention of damage is possible if the tanks are connected sufficiently to the foundation and if overalling is prevented by stiffeners and tendons. The roof should be mounted during an early stage of erection.

### 1.4.6 Towers, masts and stacks

Because of rhe usually slender form, towers, masts (Fig.l.6) and stacks are highly susceptible to vibration. If certain measures are taken the risk of damage is small. Tendons can reduce long period vibrations, but tendons have to be checked against vibration and the anchorage in the ground must be guaranteed. The vortex shedding can be reduced by aerodynamic devices. Passive and active damping measures reduce wind induced vibrations.

### 1.4.7 Bridges

Similar vibration problems are observed with long span suspension bridges. Torsional vibratioric and galloping effects may cause the collapse of the bridge (Fig.1.7). Wind tunnel tests and detailed calculations allow to reduce the risk of failure. The section must be adapted to

**Fig.1.5** Damage of temporary construction by **wind.**

aerodynamic needs. Horizontal and vertical stiffeners as well as active and passive dampers connected to the hangers reduce vibrations.

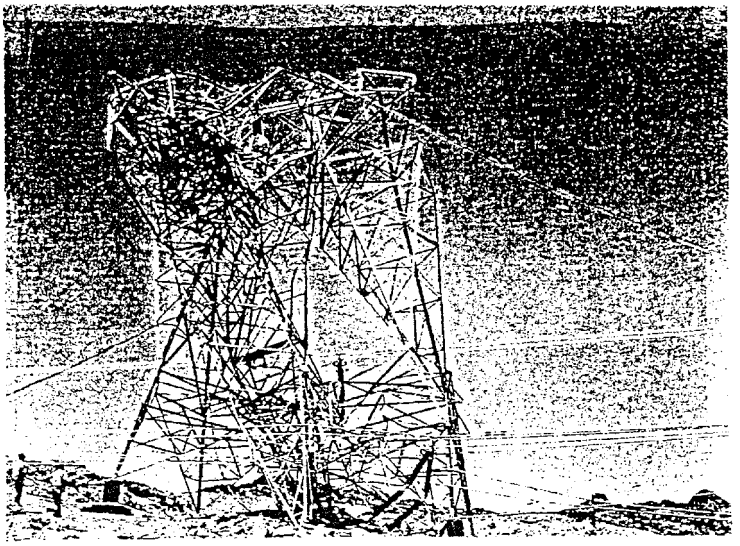


**Fig.1.6** Damage of mast due to **wind** storm.

### 1.4.8 **Erection** stages

During construction the storm risks are usually higher than for the final stage. The construction stages need exact hazard szenarios like the completed structure. Sheathing often gets similar wind forces as roofs and wails. Risks can be avoided by simple organizational measures.

Struts, bracings and tendons must be used to stabilize the construction stages, sometimes they may be mounted only shortly before the storm arrives. The material has to be secured (bundled, fixed) on the camp especially if light masses and big surfaces are involved.

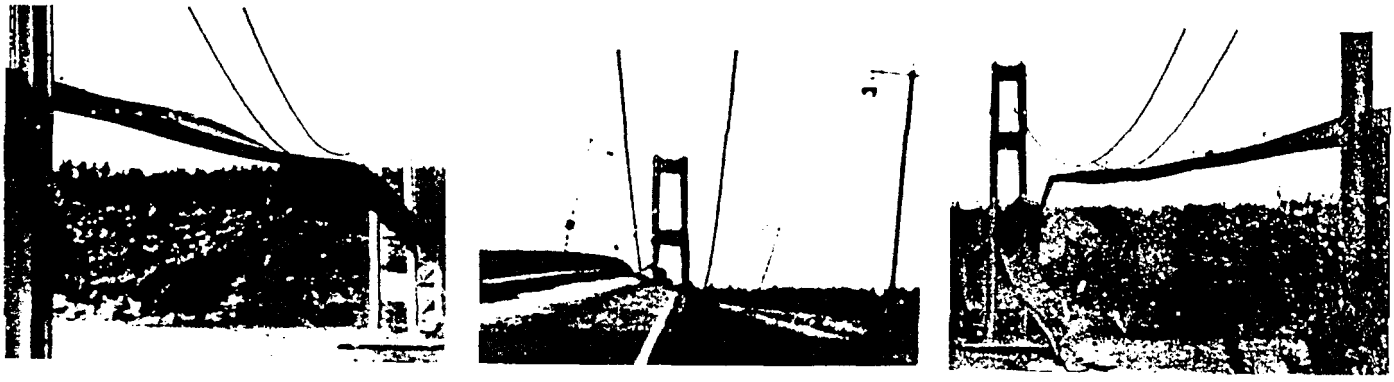


**Fig.1.7** Stages of damage of suspension bridge.

## REFERENCE

Münchener Rückversicherungs-Gesellschaft (1989). Storm.

# 2 SNOW AVALANCHES

W. J. Ammann, P.M.B. Fohn

**SUMMARY**

For alpine countries, avalanches represent one of the major hazards, threatening people in villages as well as on highways and railways. Measures have to be taken to reduce avalanche risk: Avalanche hazard mapping as a basis for land use planning is one of the most cost-effective measures to reduce or even avoid avalanche exposure. In many situations, technical measures such as supporting structures and deflecting dams, or short-term measures such as avalanche forecasting, artificial avalanche release or evacuation might be the action of choice to reduce the avalanche risk to an acceptable level. It is important to evaluate the different measures within the frame of an overall risk management procedure.

## 2.1 INTRODUCTION

In the European Alps expanding settlements and increasing mobility due to tourism lead to a growing number of constructions in terrain threatened by avalanches. Eleven million people live in the Alpine region from France over Switzerland, Italy, Austria to Slovenia. Due to winter tourism this number is temporary tripled. A number of important highways and railways cross the Alps. For example over 19'000 vehicles daily cross the Gotthard pass, a very important transit route between Italy and Germany (CIPRA, 1998). Since the catastrophic avalanche winter in 1950/51 the mobility of people in terms of vehicle-kilometres might have increased by a factor of 100. It is therefore no surprise that avalanche mitigation has continued to play an important role in the life of the people living in the Alps. In Switzerland alone, over the past 50 years, about 1.5 billion Swiss francs have been invested in engineering construction work for avalanche protection such as snow supporting structures, deflectors or snow sheds. Together with the daily avalanche forecasting, the avalanche hazard zoning and sustainable silviculture of the protection forests this has led to a high degree of safety (compared to other hazards) in denseiy populated mountainous areas and on roads with high traffic volume.

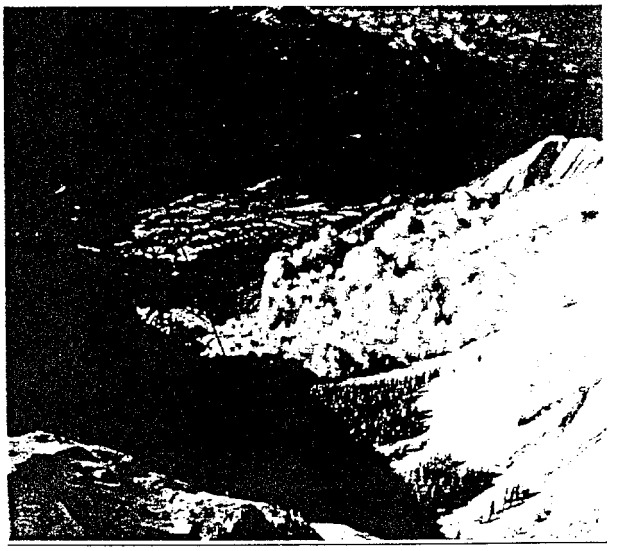


**Fig.2.1** Devastating dense flow avalanches at Evolène/VS, Switzerland, causing 12 fatalities on 21st February 1999.

**Fig.2.2** Huge powder snow avalanche, released for research purposes at the SLF test-site Vallée de la Sionne/VS, Switzerland on 10th February 1999.

Although avalanche research and avalanche hazard mitigation have made major progress in the last decades, there are still deficiencies and not enough knowledge and suitable tools to sufficiently *protect* life and property. The catastrophic winter 1998/99 with many hundreds of devastating avalanches all over the Alps clearly showed this. In Switzerland alone, 17 people have been killed during January/ February 1999, half of them in buildings, half of them on roads and in the backcountry. The total amount of *damages* is estimated at 1 billion CHF composed of 250 Mio CHF direct damage cost and 750 Mio. CHF indirect damage cost. The neighbouring alpine countries suffered from similar experiences during this devastating period. A total of 75 fatalities in the European Alps had to be counted in January/ February 1999.

## *2.2* AVALANCHE HAZARD AND DAMAGE SCENARIOS

Instabilities in the snow cover and external impacts can cause avalanches at slopes with an angle of 25°-45°. *Extreme weather situations* with heavy snowfall during several days may lead to catastrophic avalanches threatening villages, access roads and railways. Different kinds of avalanches occur depending on the characteristics of the snow pack, the snow volume involved, the slope angle or additional loading. *Slab avalanches* are most frequent and typically are of moderate size and involve snow masses in the order of a few 1000m$^3$ up to some 10'000m$^3$. On a long year average, most fatalities are due to accidental snow slab avalanche releases, initiated locally by off-piste skiers, ski mountaineers, or similar leisure activities (in Switzerland 24 out of 26 fatalities). Especially in the last decade only very few people have been killed on open roads or in settlements (Tschirky, 1998).

**Fig.2.3** Situation at Goppenstein/VS in February 1999. Several dense flow avalanches from both valley sides threatened the Loetschberg railway, the access road and a temporary construction site for the tunnelling work.

This annual statistic may drastically change during a winter period with exceptional meteorological and nivological conditions as experienced in January/ February 1999. Situations with return periods of several decades may threaten a whole country severely endangering people and their settlements, vehicles on roads and railways, forest and agricultural landscape.

The devastating avalanches of January/ February 1999 were mostly big *powder-snow* and/or *dense flow avalanches* consisting mainly of dry, loose snow, which started to rupture spontaneously under their own weight. The rupture plane was often situated at the base of several snow layers, accumulating a 2 - 4 m thick snow pack. Huge snow masses up to more then 1 Mio $m^3$ were sometimes involved and the resulting avalanches advancing down to vailey level, endangering settlements, roads and railways (Fig.2.1 and 2.2).

In addition to direct damages in terms of fatalities, destroyed buildings, devastated forests and agricultural landscape, even much more important *indirect damage* costs result. These latter are costs due to interrupted roads and railways, failures in the electricity distribution and in communication, reduced accessibility of tourist resorts, and decrease in hotel reservations (Fig.2.3).

## 2.3 AVALANCHE PROTECTION MEASURES

### 2.3.1 General Overview

Several classification possibilities exist for the large variety of avalanche risk reducing measures. Most used is a sub-division into short- and long-term protection measures (Salm et al. 1990):

*Short-term protection measures*
- avalanche forecasting, warning
- artificial release of avalanches
- closure of roads and railways
- evacuation of people and cattle

*Long-term protection measures*
- hazard mapping, land use planning
- construction measures
- supporting structures (starting zone of avalanches)
- deviation dams (avalanche path)
- snow sheds (roads, railways crossing avalanche path)
- retaining dams (deposition zone of avalanches)
- retarding constructions (deposition zone of avalanches)
- silvicultural measures
- silviculture
- reforestation, combined with technical measures

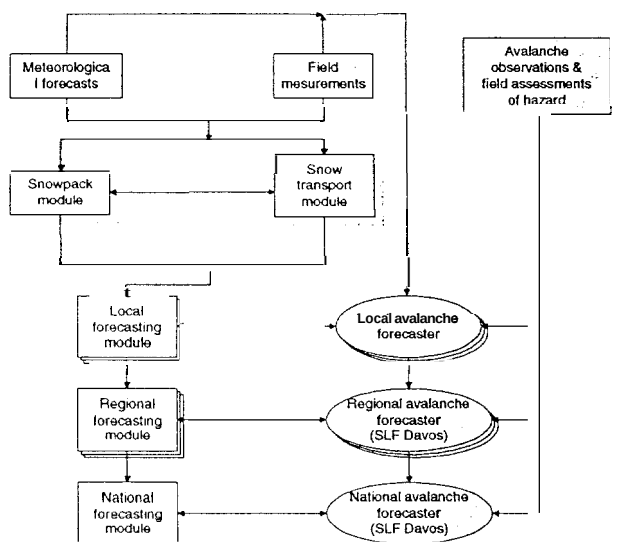
### 2.3.2 Avalanche forecasting

The avalanche hazard forecasting and the subsequent measures such as evacuation of people in exposed settlements, the shut-down of roads and railway lines and the artificial release of avalanches under controlled conditions are called organisational or short term measures. Efficient use of these measures need a close interaction and co-operation between all national, regional and local security commission staff-members and nation-wide public avalanche awareness programmes. All Alpine countries operate national and/or regional *avalanche warning centres*, which forecast the avalanches on a daily basis. With the introduction of the European Avalanche Hazard Scale in 1993 a common language to describe the snow cover stability and the probability of an avalanche release has been found which is now being used in all European countries (Meister 1994).

Avalanche warning has been a key task of the Swiss Federal Institute for Snow and Avalanche Research in Davos (SLF) since it was established over half a century ago. In the past, the predominant methods used for avalanche forecasting at SLF have been conventional, i.e. snow stability and avalanche hazards were predicted without analytical techniques such as formal numerical and symbolic algorithms. Until the early 1990s *avalanche forecasting* at SLF was mainly based on intuition, experience and local knowledge of the forecaster.

A paradigm shift is currently taking place: Information systems and computer programs are becoming more and more important, assisting the forecaster in collecting and analysing large amounts of field data (Russi et al., 1998). Mathematical analysis of measurements, numerical simulations of weather and snow-pack (Lehning et al. 1998) and symbolic and statistical computations of the avalanche hazard are the key elements of modem avalanche forecasting, which can be described as *computer-aided avalancheforecasting* **(CAF)** A similar approach is followed in France (Durand et al. 1998). However, *theforecaster* with his intuition, experience and local knowledge still plays a decisive role in the forecasting process. While the computer helps to assimilate information, to assess the hazard risks, to support the forecaster in his decision and to distribute forecasts via modem communication channels, it is still the forecasters ultimate responsibility to check and modify the computer's prediction.

A three level concept (national, regional, local level) for the avalanche forecasting is being realised within the Swiss concept *Avalanche Warning Switzerland CH2000*. The SLF provides the first two levels, the daily national bulletin and the regional bulletins, local security commissions are responsible for the local level (regional means an area of 1000 - *5000* km2, local an area of 100 $km^2$). The overall aim of this concept is to modernise avalanche warning in Switzerland and to improve the temporal and spatial resolution of avalanche forecasting on a national, regional and local level, thereby helping to prevent avalanche accidents. Fig.2.4 shows the general architecture containing all major modules and information paths. Shaded boxes denote input sources and white boxes indicate computer models.



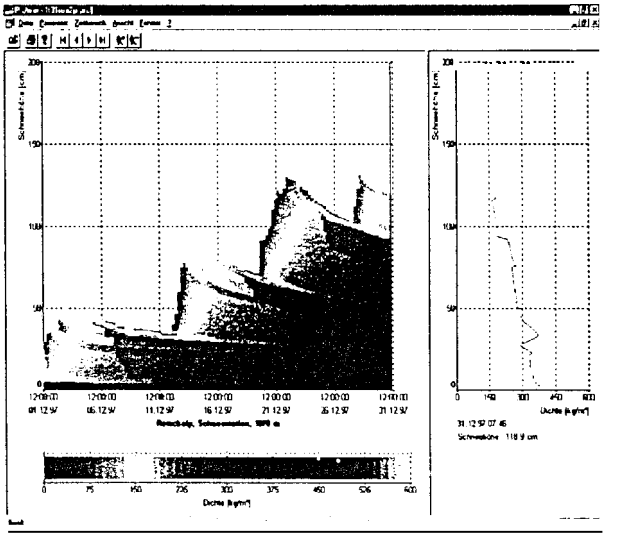



**Fig.2.4** General architecture for the avalanche forecast in Switzerland (Russi et al., 1998)

**Fig.2.5** Snow cover layering (seasonal density evolution), calculated with *Snowpack*, a computer model developed at the SLF (Lehning et al. 1999).

The *bulletins* represent an important tool for all local and regional security commissions in their risk management processes, e.g. to close a road, to evacuate people or to order the artificial release of potential avalanches. Basic information for the bulletins are gathered by a network of 75 snow and avalanche observers and 60 automatic measuring stations throughout the Swiss Alps (Russi et al. 1998). The forecaster's expert knowledge completed with a continuuusly operated *numerical snow pack model* (Fig.2.5, Lehning et al. 1999) analyse the extensive set of data. The numerical model evaluates the internal state of the snow cover involving temperature, density and grain type profile, moisture content, layering, surface or depth hoar. Additional software like e.g. NXD2000 (Russi et al. 1998) for the local level

provides a fast and efficient decision support (Fig.2.6). Even so these models made good progress in the last years, they cannot fulfil all wishes of precision in space and time (Buser 1989, Fohn 1998).

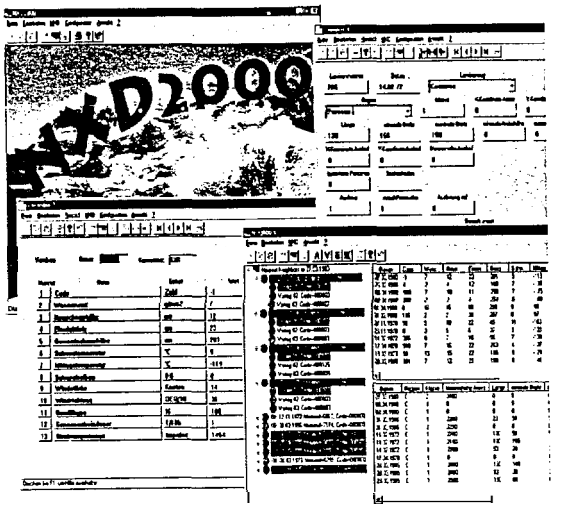



**Fig.2.6** Input and output window of NXD2000, the Swiss local avalanche forecasting software tool.

While the web is the main information channel for the general public, a service called *InfoBOX* was set up by the SLF three years ago. This service links together national, regional and local avalanche specialists in Switzerland. So far, about 140 specialists, i.e. people who are in charge of avalanche safety in villages or towns, in ski areas or on highways, are using the SLF InfoBOX service. Via this service, snow and weather data from automatic stations or weather forecasts can be accessed 24h a day (Fig.2.7).

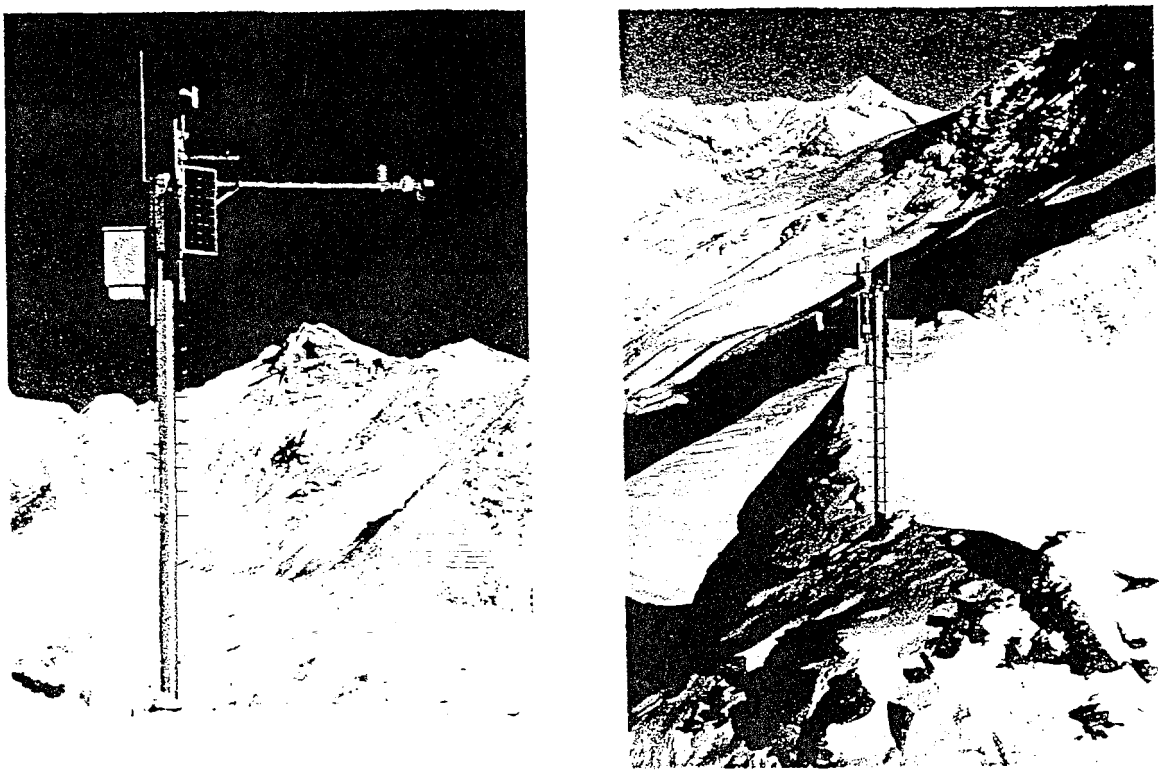


**Fig.2.7** IMIS network. Automatic snow (left) and weather **(right)** station at Simplon Pass/VS Switzerland.

All other short-term, *temporary measures* like artificial release, traffic closure, evacuation of people and cattle are subsequent measures in critical periods. Stoffel (1996) discussed the different techniques for artificial avalanche release. Evacuation has to be based on well defined evacuation procedures to prevent additional hazardous situations for the people involved. In such critical situations the local government supported by avalanche experts has to take responsibilities.

### 2.3.3 Avalanche hazard mapping

Hazard maps serve as basic document for avalanche risk evaluation, especially with respect to land-use planning (for a more detailed overview see Margreth and Gruber 1998). In Switzerland hazard mapping begun after two catastrophic avalanche periods in January and February 1951. The first avalanche hazard maps in Switzerland were elaborated for Gadmen and Wengen in the Canton of Bern, 1954 and 1960, respectively. Dangerous zones were designated according to occurred disastrous events in a more qualitative way without taking into account climatic factors or quantitative avalanche calculations. In course of time the methods were improved and avalanche models introduced to calculate the dynamic behaviour. This development led for example to the *Swiss guidelines for avalanche zoning* (BFF 1984) and the *Guidelines for the calculation of dense flow avalanches* (Salm et al. 1990). The two publications are today the most important tools for the elaboration of avalanche hazard maps in Switzerland. In recent years numerical simulation, GIS and DTM tools led to substantial improvements (Gruber et al. 1998a,b). Two parameters were chosen to quantify the potential hazard for a given site:

- Expected *frequency of* an avalanche reaching a given site (frequency is normally expressed by the return period),
- *Intensity* of an avalanche (intensity is expressed by the avalanche pressure exerted on a wall of a building. As this pressure is assumed to increase with the square of speed and propcrtional to density, the kinetic energy of snow masses is also included).

To be able to distinguish variable hazard intensities and run-out scenarios, several *hazard zones* are defined:

*Red zone:* Pressures of more than 30 $kN/m^2$ for avalanches with a return period of up to 300 years, and/ or avalanches with a return period up to 30 years independent of pressure,

*Blue* zone: Pressures of less than 30 $kN/m^2$ for avalanches with return periods between 30 and 300 years,

*Yellow zone:* For powder-snow avalanches with pressure less than 3 $kN/m^3$, and return periods more than 30 years. For dry-snow avalanches with pressure unknown, and return periods more than 300 years,

*White* zone: No avalanche impacts to be expected,

*Gliding Snow:* Area of pronounced danger for gliding snow at locations without avalanches or with impacts larger than by avalanche effects.

The elaboration of hazard maps must strictiy follow scientific criteria and methods including expert knowledge. The goal is to determine the *extreme avalanche* on a reliable basis. Field visits to assess the avalanche terrain, the examination of the avalanche cadastre as a map with all known avalanches in history, including their extent and date, additional information from competent local people or from old chronicles, the check of local climatic conditions and dynamic avalanche calculations are important tools. The *dynamic calculations* are used for:

- Predicting an extreme event, probably not registered in a cadastre,
- Delimiting the hazard zones for the different return periods,
- Calculating run-out distances and pressures as a function of avalanche frequency.

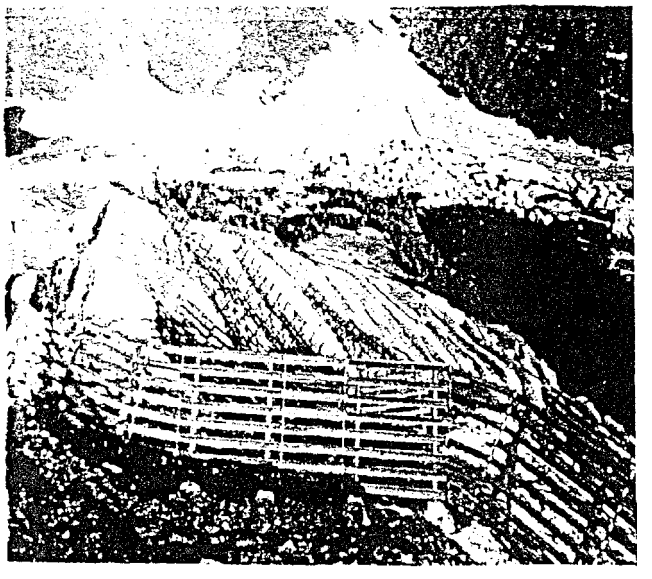
In Switzerland the *Voellmy-Salm model* is used since more than 20 years for estimating avalanche speeds, flow heights and run-out distances of dense flow avalanches (Salm et al. 1990). The use of the Voellmy-Salm model requires a careful estimation of its input parameters as fracture depth, friction parameters *or* avalanche size (Margreth et al. 1998). To check the sensitivity the calculations have to be made with different input parameters. Critical assessment of the results is important. It has to be pointed out that dynamic calculations are just one part of hazard assessment. In recent years, many such dynamic calculation methods have been proposed, some of which are routinely and effectively used by practitioners (Salm et al. 1990). *Numerical methods* using FE- or FD- techniques have set new standards in the use of avalanche dynamics models (McClung et al. 1995, Bartelt et al. 1997). User-friendly GIS- and DTM-tools are additional assets to complete and facilitate avalanche hazard mapping (Gruber et al. 1998a, 1998b).

### 2.3.4 Technical measures

Technical, long-tenn avalanche defense measures are used in the starting zone to prevent the release of avalanches (supporting structures) and in the avalanche track and run-out zones (avalanche sheds, deflecting and catching dams) to reduce the damaging effect of descending avalanches.

*Supporting structures*

The wide application of supporting structures had its beginning after the severe avalanche winter 1950/51. Since then the technology has reached an advanced stage. More than 500 km supporting steel bridges and snow nets have been built over the last *50* years. **All** experience gathered through these decades is summarised in the Swiss Guidelines (1990). The aim of supporting structures is to prevent the start of large avalanches or at least to *limit* snow motions to an harmless extent. Fully developed avalanches can not be stopped and retained by supporting structures (Margreth 1996).



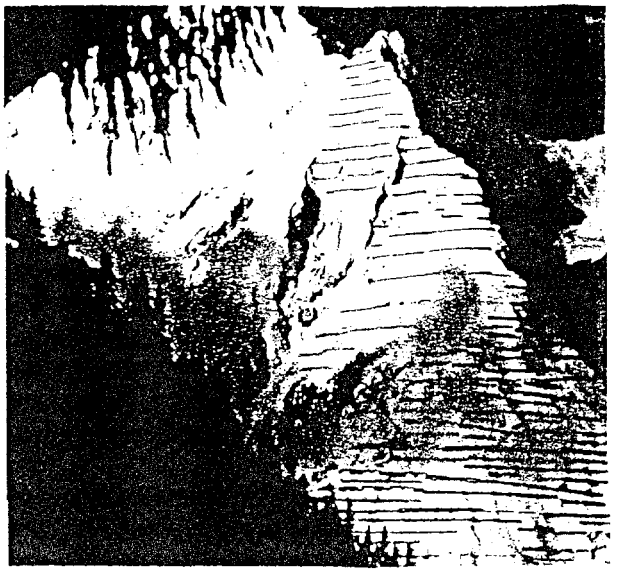


**Fig.2.8** Steel supporting structure above Davos/Switzerland (Schiahorn).

The first effect of supporting structures is to introduce an overall increase in the *stability* of the inclined snow. The acting snow-pack forces are redistributed, compressive reaction forces are increased, shear forces, which dominate stability, are decreased. The second effect consists in limiting the *mass* of *snow* put in motion and in retarding and catching it. The vertical height must correspond to the extreme snow depth with a return period of at least 100 years. The adopted snow height is a crucial point for the design to guarantee the effectiveness of supporting structures. In February 1999, some lines of structures were overfilled with snow, more than 550 cm of snow were measured at 2500 m a.s.l. Technically feasible are constructions for up to 7 m of snow. Typical structure heights in Switzerland vary between 3m and 5m.

Today *steel* **bridges** and *flexible snow nets* are predominantly used. The costs for supporting structures are about 1.0 - 1.5 Mio CHF. Due to these high costs, supporting structures are mostly used for the protection of settlements. The constructions are designed for a period of 100 years. *Maintenance* of older supporting structures is therefore becoming more and more important.

### *Deflecting and catching dams*

Deflecting and catching dams are relatively cheep compared to the supporting structures but need enough space and volume to be effective. Deflecting and catching dams are normally *earth darns,* sometimes combined with *stone masonry* to increase the slope-inclination at the impact side. The height of catching dams may reach 15 - 20 m, depending on the avalanche velocity and the snow volume to be retained (Fig.2.9). An overflow of the dam crest has to be avoided. Catching dams may also be used to retain mudflows.

**Fig.2.9** Deflecting dam near Disentis/GR Switzerland.

**Fig.2.10** Snow deposit of an avalanche near Elm/GL Switzerland (end of February 1999).

### *Avalanche sheds*

Avalanche sheds to protect roads and railway lines are effective measures if the avalanche track is narrow and the shed construction sufficiently long (Fig.2.3). In situations where the avalanche deposition zone is widely spread, a shed construction would become too long. In such situations, and in view of an integral **risk** management, *road closures* are often the only cost-effective measures (Fig.2.10). Since a few years Swiss guidelines exist on the design of avalanche sheds (ASB/SBB 1994). One meter of snow shed costs, as an average, about 25'000 CHF.

### 2.3.5 Mountain forest

The mountain forest corresponds to the most effective and the cheapest protection for villages, roads and railways. The trees retain the snow; stabilise the snow-pack and prevent avalanches to start. The mechanical resistance of the trees is not sufficient to stop avalanches. Therefore, the protection function of the mountain forest against avalanches is only valid for *starting zones* below the timber-line. In Switzerland, about 1000 $km^2$ of forest area serves primarily as avalanche and rock-fall protection. If this effect would have to be replaced by technical means, a yearly investment of 2 Billion CHF would be necessary.

## 2.4 AVALANCHE RISK AND MANAGEMENT

Risk management is an integral approach of human thinking and acting covering the anticipation and the assessment of risk, the systematic approach to limit the risk to an accepted level and to undertake the necessary measures. Avalanche risk is the result of the temporal and the spatial overlapping of the two independent domains *potential avalanche danger* and *spatial area in* use. The avalanche danger is described by the avalanche probability and the extent of the avalanche. The spatial area in use corresponds to the probability of presence of any objects and the value of these objects (or the number of people present).

To avoid the disastrous effects of avalanches, different kinds of *prevention measures* are used to reduce the avalanche risk to an acceptable level. These measures have to be seen as an integral set of possible protection measures. In most cases a combination of the different measures is used. The optimal combination can be found by maximising the cost-effectiveness and cost-benefit of all possible avalanche control measures. The identification of the avalanche danger in terms of probability of occurrence, the estimation of the risk potential based on the vulnerability of the corresponding values exposed to risk, the assessment of protection goals and the cost-estimation for control measures are basic principles to be applied in an integral risk management approach (Wilhelm 1997 and 1998, Heinimann et al. 1998).

***Cost-effectiveness*** can be expressed in terms of amount of money spent per saved life (Wilhelm 1997). For avalanche control measures, it varies to a large extent, depending on the actual situation (1 to 20 Mio CHF). The whole risk management process is iterative with several assessment and control loops. For preliminary design purposes Wilhelm (1998) established simplified cost-effectiveness evaluation charts which will be published as a BUWAL-Guideline for the **risk** assessment of roads and railways.

## 2.5 RESEARCH NEEDS

### 2.5.1 Physics and mechanics of snow

Snow as material for avalanches is a complex mixture of air, water and ice, which is in our environment always close to its melting point and henceforth changes its physical properties continuously in time and space. This metamorphic process which changes the shape of the snow particles from fine dendrites to rounded grains or other shaped particles depending on temperatures, density, solar insulation and wind has to be known in detail if the formation of the various types of avalanches should be predictable for detailed avalanche forecasting. Unfortunately, a massive lack of knowledge still exists to quantitatively describe shrinking, settling and re-cristallisation processes combined with the corresponding changes in mechanical properties such as shear resistance and cohesion within the snow pack.

### 2.5.2 Avalanche forecasting

To increase the acciiracy of avalanche forecasting in time and space, research has to concentrate on *questions* such as:

- How can the stability of a slope be quantitatively assessed and introduced in operational avalanche forecasting service? What are the triggering mechanisms for the release of avalanches?,
- How can the known local and temporal variability of the snow cover on slopes and of its stability be taken into account?,
- How can snow drift be quantitatively described on a local to regional scale and how can this description be used to improve avalanche forecasting? and
- How can the information available on a local, regional and national scale be combined and used as input to avalanche warning models (statistical methods, expert systems, neuronal networks) which support the decision process?

### 2.5.3 Avalanche hazard mapping

Avalanche hazard mapping is closely linked to avalanche dynamics. Various dynamic avalanche models have been developed in the last 20- 40 years based on different flow types (hydraulic, aerosol, mixed, granular). Also statistical models, based on a few topographical factors and observed run-out distances compete with the various flow type models as far as run-out distances are concerned (Lied 1998).

Significant improvements in the avalanche dynamics calculations which serve as a basis for hazard mapping could be obtained by:

- Improved knowledge of initial conditions (fracture area and depth of sliding snow layers, quality of sliding snow, e.g. friction coefficients) all dependent on the return period,
- Development of adequate physical models to describe the flow regime of dense-flow avalanches (Bartelt and Gruber 1997), the snow entrainment in powder-snow avalanches (Issler 1998) and the impact mechanisms on structures.

Validation of physical models and numerical modelling with field and laboratory data. Real progress will only be possible when field and laboratory data will be available covering all major parameters influencing avalanche dynamics. Since 1997, the SLF operates therefore a test-site in the Vallée de la Sionne/VS, Switzerland (Fig.2.2 and 2.10, Ammann 1998). There it is possible to study the overall dynamic behaviour of dense-flow and powder-snow avalanches and to measure avalanche impact forces along their path.

### 2.5.4 Technical measures

Avalanche defence structures and dams still need *improvements:*

- Design of the load bearing capacity of the foundations (anchors),
- Design of defence structures in permafrost sub-soil (Stoffel 1995),
- Implementation of maintenance strategies for existant structures,
- Design of deflecting and retaining dams (McClung and Mears 1995),
- Design of reinforced structures in the blue avalanche hazard zone.

### *2.5.5* Risk management

Major improvements in risk reduction may be achieved by a consequent risk management. Research efforts are needed in the following domains:

- The devastating events in January/ February 1999 demonstrated the importance of indirect damage costs. Damage patterns have changed, the increased mobility and missing awareness of the public are major reasons. To develop strategies to take care of this changed damage pattern will be an important task.
- What is the acceptable risk level? Has aversion to be taken into account?
- Development of tools to assess the cost-effectiveness of different defence strategies for settlements, roads, railways.

- Implementation of a strategy for the continuous education of local and regional avalanche safety responsibles.

Fig.2.11 SLF avalanche test-site Vallée de la Sionne. View on avalanche track with the location of different obstacles.

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# 3 ICE AVALANCHES

M. Funk, S. Margreth

## 3.1. INTRODUCTION

Ice avalanches form when a large mass of ice breaks off from a glacier, drops down slope driven by gravity and bursts into smaller pieces of ice. In most cases, ice avalanches are the normal process of mass wastage of high altitude, alpine glaciers on steep slopes, so called hanging glaciers. A typical feature of hanging glaciers is the absence of an ablation area where ice can melt and the process by which ice mass is being removed is the release of chunks of ice (typically $10^3$-$10^5$ $m^3$) at the glacier front. Such events can happen at any time of the year and have often been observed in the Alps and in other high mountain regions of the world.

The detachement of larger ice masses from a steep glacier is an extremely rare event and characteristically occurs during an active phase with enhanced sliding motion, which usually lasts for a few weeks during the melt season. The slip off of a large ice mass can also occur if the stability of a steep glacier reaches a critical state. The failure of one of the retaining forces can then lead to an ice fall. The typical mass is $10^5$-$10^6$ $m^3$ and has sometimes the potential to impinge on settlements and other fixed installations in high mountain areas.

The effect of ice avalanches is comparable to that of snow avalanches, the main difference being that they can occur at any time of the year. The most destructive ice avalanches happen in winter when they release or entrain additional snow masses. Such combined snow/ice avalanches can cover very long run-out distances.

## 3.2. HISTORICAL ICE AVALANCHES

The largest known ice fall in the Alps happened on 11 September 1895 in the Bernese Alps in Switzerland, where the major part of a glacier (five million m3) slipped off from the summit area of Altels and fell onto an alpine pasture killing six people and 158 cows (Fig.3.1) (Heim 1896, Rothlisberger 1981). Both authors have attempted to analyse the possible causes of this detachment. The prominent curve-shaped fracture line at the head of the niche left by the avalanche indicates failure under tensile stresses. Basal and shear adhesion and shear resistance at lateral abutments were the stabilizing forces. The gravitational pull and the sum of the retaining forces have reached a critical equilibrium from which the spontaneous release of the avalanche occured when one of the retaining forces failed. Probably this occured by warming of a frontal zone frozen to the bedrock. As a consequence, the aerial extent of this zone became reduced and the loading on the still frozen parts increased up to the shear strength of ice. Very warm summers in the years preceding the ice fall support this hypothesis.

More recently, on 30 August 1965 a major portion of the terminus of Allalingletscher (Valais, Switzerland) detached unexpectedly, slid down a rock slope of some 27° over a vertical distance of 400 m and continued for further 400 m across the flat bottom of the valley, claiming 88 victims at the Mattmark construction site (Fig.3.2). The volume of the ice avalanche was estimated at 0.5-1 million $m^3$. Subsequent glaciological investigations showed that the ice avalanche had occured during a phase of enhanced motion as a result of intensive bed-slip of an even larger mass than that which broke off. It could be inferred that enhanced sliding on the rock substratum must have occured already some 2-3 weeks prior to the sudden slump of the avalanche. It could be observed that such active phases with enhanced sliding motion took place quite regularly at intervals of one to three years, starting usually in late summer or in fall, but always ending at the beginning of winter without any significant ice fall occurence. Similar movement irregularities can be obseived on many other steep glacier tongues. This active phase was a necessary bur riot a sufficient condition for the lower part of the Allalingletscher tongue to slide off. The convex bed topography and an unfortunate mass distribution, combined with the active phase, played a major role in that catastrophe (Rothlisberger 1981, Rothlisberger and Kasser 1978).

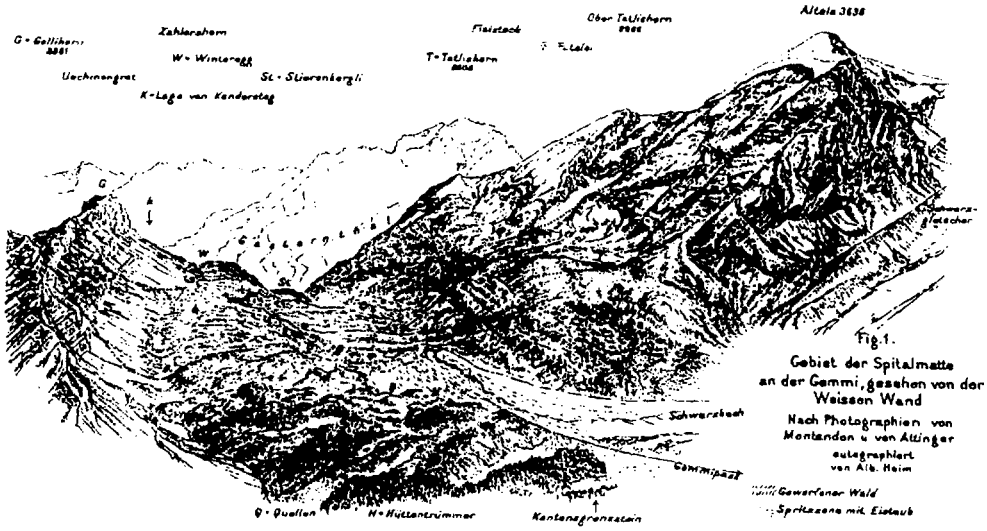


**Fig.3.1** The ice avalanche from Altels. The curve-shaped line of rupture is visible immediately under the Altels summit (after Heim 1896).

On 5 September 1996 large ice masses (0.25 million $m^3$) broke off from Gutzgletscher (above Grindelwald, Bernese Alps, Switzerland) and dropped down as two major powder and dense flow avalanches in the direction of the road that connects Grindelwald to Grosse Scheidegg (Fig.3.3).


**Fig.3.2** The ice avalanche from Allalingletscher.

The road was blocked over a distance of 20 m and the air pressure injured three persons and knocked down some hikers (Margreth and Funk 1998). The Gutzgletscher, a so-called hanging glacier, is situated on a moderately inclined bed in the very steep north-west face of Wetterhorn mountain and ends in a frontal ice cliff. Small ice avalanches occur there every year, but they never endangered the road during the summer season.

**Fig.3.3** The ice avalanche from Gutzgletscher **(Foto** U. **Schiebener).**

## 3.3 STARTING ZONES AND RUN-OUT DISTANCES OF ICE AVALANCHES

**A** number of ice avalanche situations have been recorded and analysed by Alean (1985) in a comprehensive study, in which he discusses some of the morphological features of the starting zone and investigates the relation between ice volume, run-out distance, velocity and travel time. It is appropriate to distinguish between two main types of idealized **bedrock** morphologies **with** respect to potential starting zones affecting the failure process. According to Haefeli (1965) two different kinds of failure can occur: wedge failure (type I) and slab failure (type 11) (Fig.3.4):

- For glaciers with *a type* 1 starting zone (break-type), the underlying rock topography shows in general a sharp break in angle, so that the glacier (called hanging glacier) flows over a moderately inclined (< 10") bed and ends in a frontal ice cliff (e.g. Gutzgletscher). The temperatures at the bed of a type I starting zone can be either temperate (at pressure melting point) or cold (below pressure melting point). When the ice cliff becomes too steep or even overhanging, an ice lamella breaks off (Iken 1977). Typical ice volumes breaking off in this process are $10^3$-$10^5$ $m^3$.

- For glaciers with a temperate bed and a *type II* starting zone (ramp-type) very large ice volumes (typically $10^5$-$10^6$ m3) can be released. The complex mechanisms leading to the failure of these large ice masses resting on ramp-type starting zone were discussed above for the cases of Allalingletscher and Altels. In case of glaciers frozen to the bed, observations showed that ice masses (typically $10^3$-$10^5$ m3) become detached by progressive fracture at englacial interfaces.

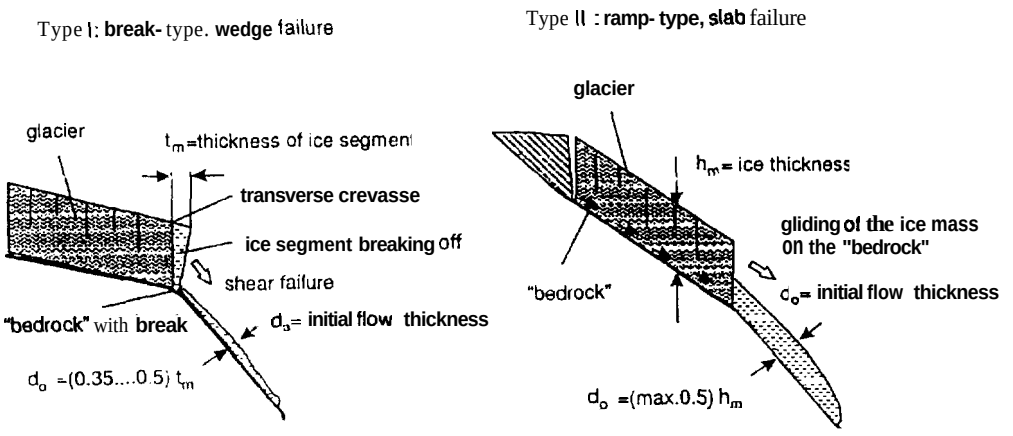


**Fig.3.4** Types of starting zones (after Margreth and Funk 1998).

Major ice avalanches from cold ramps or hanging glaciers can occur during the whole year, whereas their occurence seems to be limited to the late melt season in the case of temperate ramps.

To obtain a rough estimate of the run-out distance of ice avalanches, Alean (1985) proposed to relate the average slope of ice avalanche trajectories to their volumes and characteristic terrain parameters (Fig.3.5). With a classification of terrain parameters, a grouping of the points in Fig.3.5 was possible (lines A1, B1, C1 and D1; Alean, 1985). The model proposed is not complete enough for detailed hazard mapping. This method is only justifiable for short reaches or for overview studies. In addition, the powder part is neglected in this model.

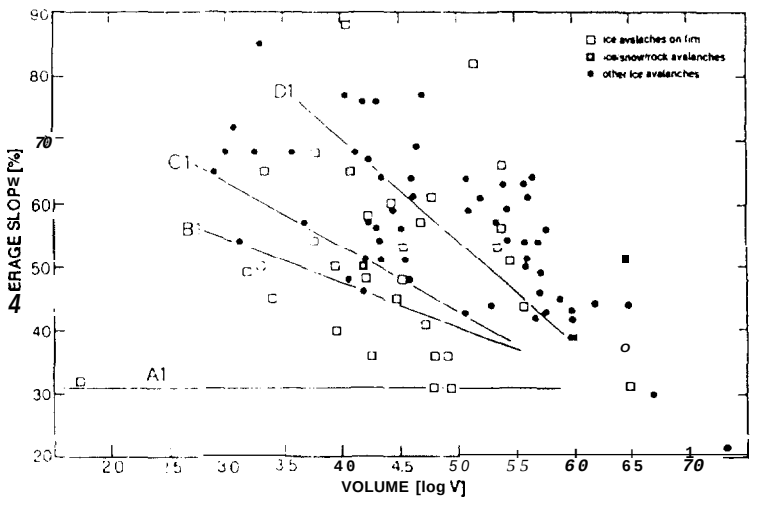


**Fig.3.5** Average slopes (%) and volumes (V in $m^3$) of ice avalanches (after Alean 1985)

## 3.4 ICE AVALANCHE HAZARD MAPPING

Classical hazard maps are used for municipal landuse planning. For this type of hazard map the scenario of the extreme winter event is in general decisivo. The hazard zones are defined in terms of recurrence interval and potential impact pressure. In Switzerland the red zone means a high potential hazard with impact pressures of 30 kPa or more for recurrence intervals up to 300 years. In red zones building activities are prohibited. In the blue zone the potential hazard is considered to be moderate and there is a limited possibility to build reinforced buildings. Sometimes a yellow zone is added which accounts for the powder part of avalanches. In comparison to snow avalanches it is much more difficult to assign a realistic recurrence interval to extreme ice avalanche events.

Another type of hazard map is used as a tool for *avalanche warning* and evacuation during periods of imminent glacier fall. These hazard maps are prepared for specific ice fall scenarios with varying break off volumes. Closure of roads or evacuations are imposed according to the prevailing hazard situation. For the *hazard assessment* of ice avalanches similar procedures as for snow avalanches are used. The mapping requires as far as possible application of quantitative and objective criteria, including:

- *Avalanche history:* information from former ice fall events is very valuable especially for the calibration of avalanche dynamic models.

- *Analysis of topography and terrain parameters:* characteristic features in the terrain must be recognised. Therefore the starting zone, track and runout have to be examined. Often different flow directions are possible. Steep slopes below a hanging glacier can be starting zones for secondary snow avalanches. If there are cliffs in the track powder avalanches can form.

- *Glaciological analysis:* first, the: potential for ice falls of a dangerous glacier has to be identified. Unfavourable developments of a glacier and possible break off volumes can be detected by regular monitoring using aerial photographs and hotogrammetry. Typical scenarios with variable ice masses for winter and summer conditions are established. The most reliable method to predict the time of breaking off is based on glacier motion measurement.

- *Avalanche dynamics calculations:* the results from avalanche dynamics calculations can be used to quantify avalanche impact pressures and runout distances for different ice masses. As the physical processes of ice avalanches are largely unknown, no advanced numerical models exist. Therefore simple models developed for snow avalanches are also applied for ice avalanches. The model calculations are useful if the input parameters can be calibrated from well documented events. The initial conditions depend mainly on the starting zone type (Fig.3.4). Current avalanche dynamic models often fail for complex situations.

- *Expert knowledge and judgement:* in parallel to scientific reasoning, expert knowledge and judgement is fundamental. An important point is that the expert explains what is factual and firmly known and what are the consequences of the main uncertainties.

## 3.5 PREDICTION OF THE BREAK-OFF TIME

A warning by an expert is difficult to use for hazard mitigation unless it includes a forecast of the time of final detachment. The possibilities to forecast a major ice avalanche depend on the mechanisms leading to the detachment of a large ice mass. In this respect, the thermal conditions at the glacier bed are important. Measurements of the movement of ice masses on ramp-type starting zones (type 11) frozen to the bed or ending in a frontal ice cliff (type I starting zone) during the destabilization phase have revealed a regularity by which they accelerate for a long time prior to the actual breaking off (Rothlisberger 1981).Using a finite element computational model for the analysis of stress and flow of an ice mass breaking off from a cliff,

Iken (1977) has shown that a stepwise crack extension alternating with viscous flow leads to the observed form of the velocity-time relationship. In the case of a lamella breaking off at the frontal ice cliff of hanging glaciers (type I starting zone), which are not always frozen to the bed, the volume of blocks or lamellae breaking simultaneously off from the cliff is limited. It seems that also in this case a particular movement versus ame relation is valid like that for cold ice.

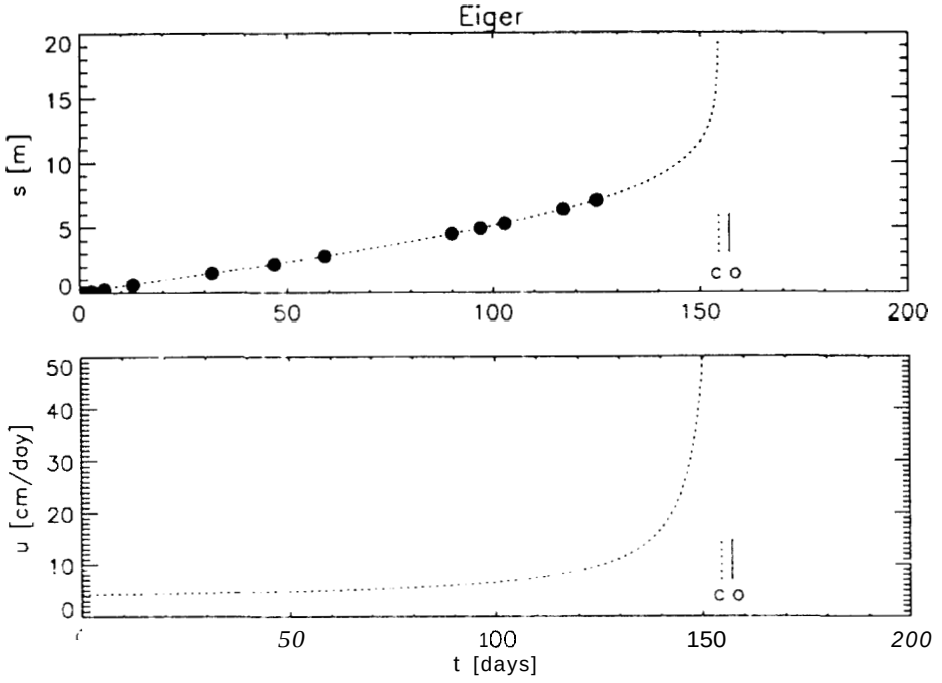


**Fig.3.6** Upper panel: measured positions $s_i$ (circles) and resulting hyperbolic function. Lower panel: velocity-time function. Day 0 corresponds to March 16, 1990.o: observed event (August 20, 1990) and c: calculated $t_\infty$ (August 17, 1990).

The case of a type II starting zone with a temperate glacier bed is more complicated. When a large area of the bed is inclined at a critical angle, large amounts of ice may break off (Altels 1895 and Allalingletscher 1965). The processes leading to the slip off of the tongue of Allalingletscher during a particular sliding situation do not allow any predictions, except that a high rate of sliding is one of the observable factors. This active phase was only one of the necessary condition for the ice fall. While the gradual development of the active phase permits to forecast the growing avalanche risk, it remains impossible to predict the time of breaking off. No such experiences exist for Altels-type ice falls. The only reported observation was a progressive increase of the ice thickness in the frontal zone during the year prior to the catastrophe (Heim 1896).

The experience with hanging glaciers or steep glaciers frozen to the bed is much better. In the case of a cold ice mass on a 45° slope of Weisshorn above Randa (Valais, Switzerland), Röthlisberger (1981) and Flotron (1977) found that the curve representing the progressive increase in velocity with time can be closely approximated by the following hyperbolic function:

$$u = u_0 + \frac{a}{(t-t_\infty)^n}, \qquad (3.1)$$

where $u$ is the velocity at time $t$ and the other parameters are constant. To determine the assumed time of breaking off $t_\infty$, a least square procedure was applied with the integrated form of equation (1) and the displacement measurements $s_i$ at different times $t_i$ (Wegmann et al., 1998). Although theoretically breaking off occurs at $t=t_\infty$ experience has shown that rupture takes place

for $t < t_\infty$ when high acceleration rates are reached. Recently, a large icefall from a hanging glacier situated in west facing slope of Eiger (Bernese Alps, Switzerland) could be forecast within three days based on 13 position measurements $s_i$, the last measurement $s_n$ being performed one month prior to break off (Fig.3.6) (Funk, 1995; Lüthi and Funk, 1997). Chances of forecasting this type of ice avalanche are therefore fairly good.

## 3.6 OUTLOOK

The investigation of steep glaciers is of practical importance when they are the origin of hazardous ice avalanches. Expenence shows that very large ice avalanches are extremely rare events. Related to this scarcity, human activities tend to spread into endangered zones. A basic rule valid for large ice falls is, that if a particular one has occured once, it will happen again in a similar way. Unfortunately, our records do not extend sufficiently far back to enable an early warning of a dangerous situation. As a consequence, forecasts of imminent danger become necessary.

Chances of forecasting ice avalanches originating from cold glaciers or from glaciers ending on type I starting zones (break-iype, **Fig.3.4)** are fairly good if adequate displacement measurements are performed during the destabilization phase.Additional experience, especially immediately before final breaking off, is necessary to give more insight into the critical acceleration phase.

Temperate glaciers ending on type II starting zones (ramp-type, Fig.3.4) can release large ice masses. Expenence has shown that the risk is small during most of the year (quiescent phase). Only during a period of a few weeks with enhanced sliding during the late melt season a detachment can be expected.In view of the uncertainties concerning the mechanisms of the fast sliding phase, a successful prediction of individual avalanche events is very difficult. Further efforts to improve the understanding of the fast sliding process and the release mechanism of large ice avalanches are needed. As the physical processes of ice avalanches are largely unknown, no advanced numerical model exist. To improve the accuracy of ice avalanche hazard maps further progress in avalanche modeling is necessary.

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# 4 ROCKFALLS

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

## 4.1 INTRODUCTION

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

**fall topple slide flow spread.**

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

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

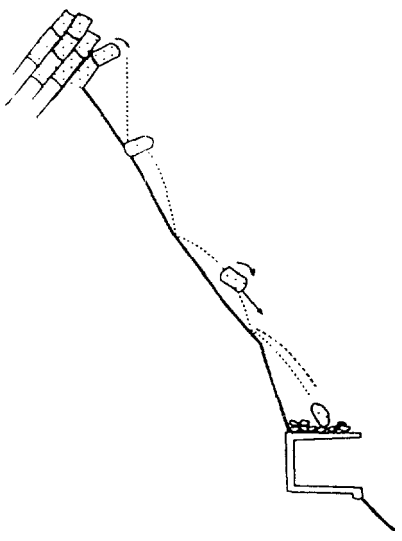


**Fig.4.1** Different particle positions during a rockfall.

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

Rock block *fall* analysis *methods* are used in order to predict the block path and the block energy during movement (Giani 1992). A block detached from a rock face may have the following types of movement during flight: fret: falling, bouncing, rolling, or sliding. Analytical

procedures for the mathematical description of the rockfall phenomenon, that consider the geometrical and mechanical characteristics, have been set up by several researchers in the last twenty years. The analytical formulations can be divided into two categories: rigorous methods and lumped mass methods. In the ***rigorous method analysis,*** the size and shape of the blocks are assumed to be known "a priori" and ail the block movements, including those involving the block rotation, are considered. In the ***lumped mass method,*** however, the single block is considered to be a simple point of mass m and velocity v. Therefore, the rotational moments are not taken into account.

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

## 4.2 ROCKFALL RESILIENT INFRASTRUCTURE

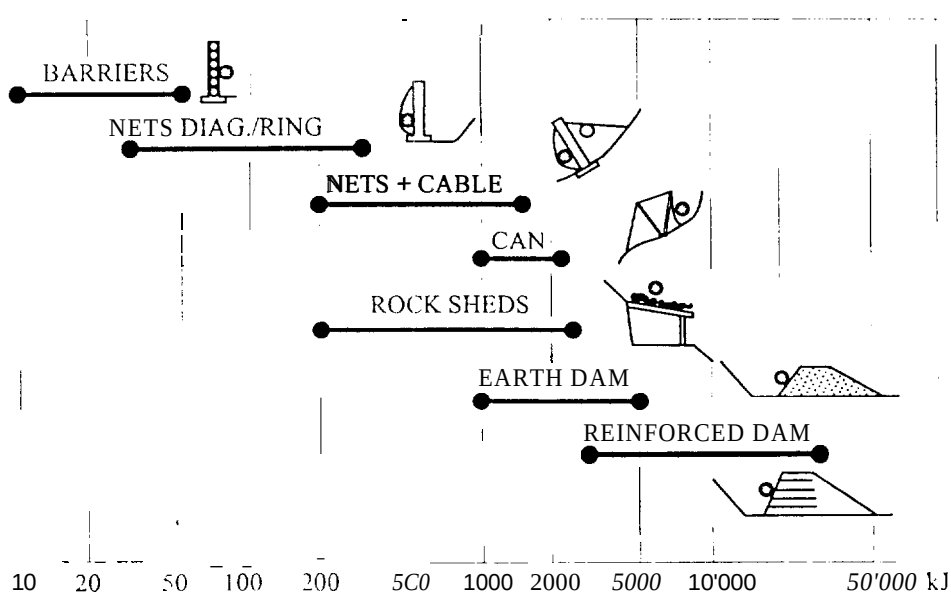
### 4.2.1 Stabilisation methods

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

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

### 4.2.2 Protecting measures (Fig. 4.2)

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




**Fir. 4.2** Energy dissipated by different protection structures

**Modification of slope geometry**

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

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

**Barriers and wire net systems**

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

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

**Rock Sheds**

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

**Reinforced earth retaining structures**

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

## 4.3 WIRE NET ROCKFALL BARRIERS

### 4.3.1 Introduction

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

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

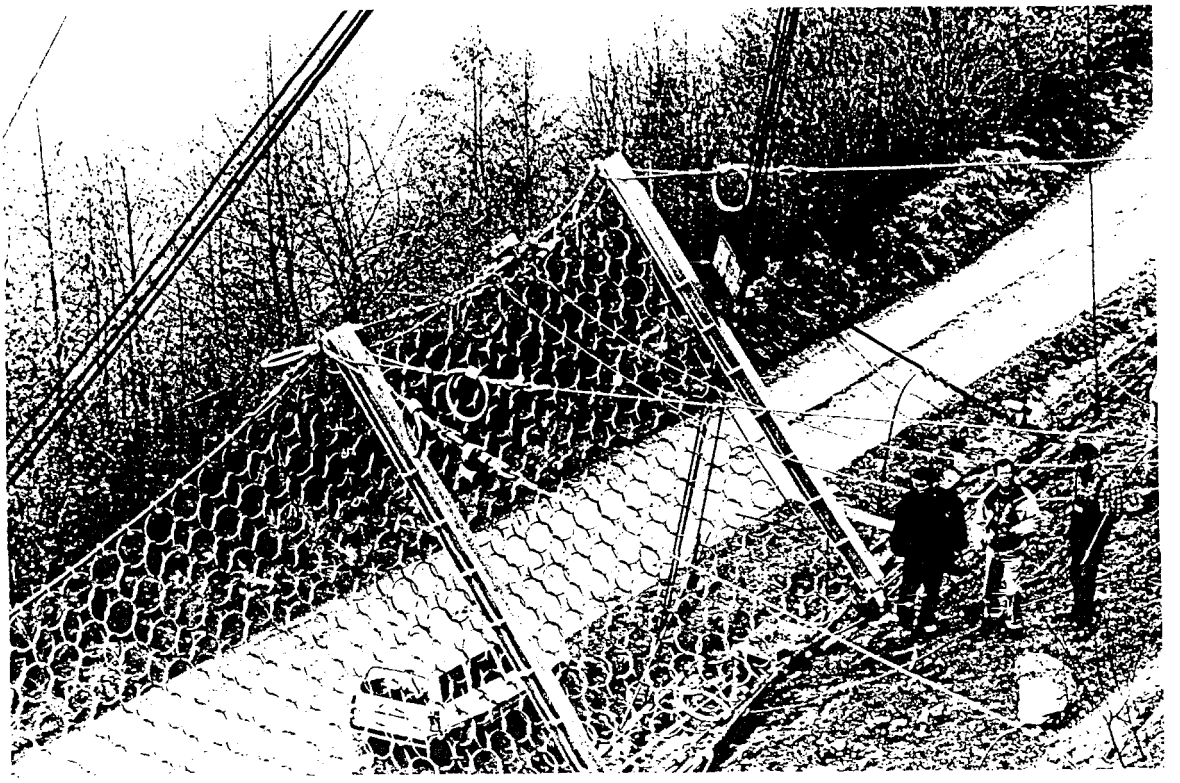


**Fig.4.3** Rockfall Barrier with Brake Rings and Strain Gauges.

### 4.3.2 Full-Scale Testing *of* Rockfall Barriers

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

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

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

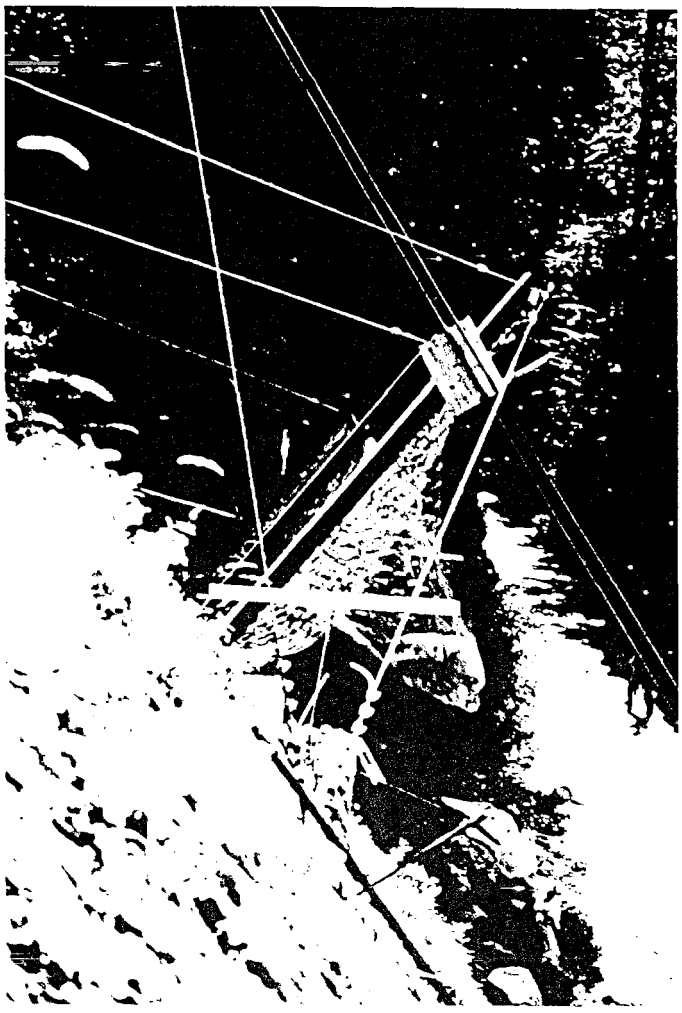


**Fig. 4.4** Rockfall Barrier at impact.

### 4.3.3 Forces arid Design Criteria

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

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

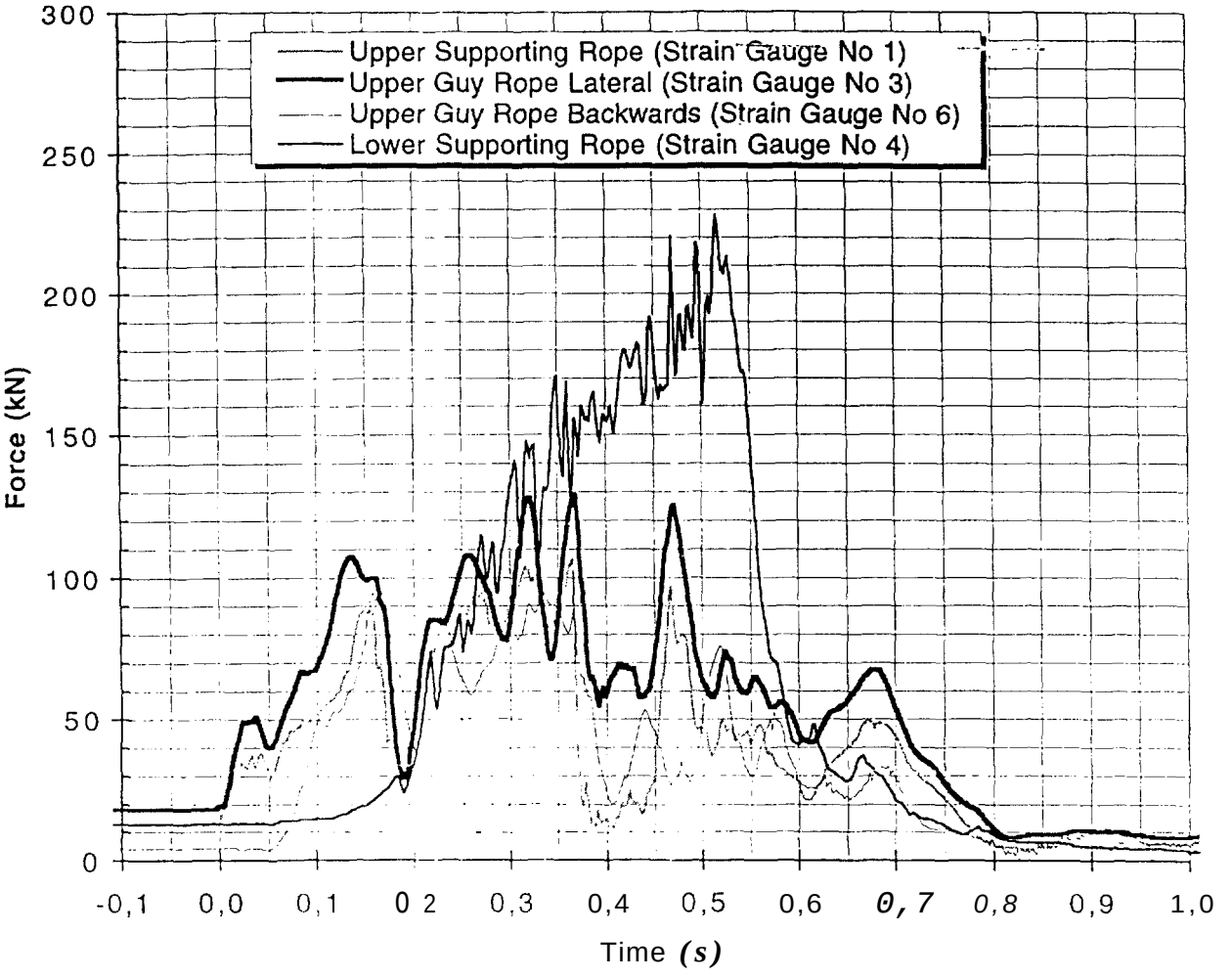



**Fig. 4.5** Strain Gauge Readrngs (1500 kJ – Test; 500 **Samples per** Second)

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

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

### 4.3.4 Summary and Outlook

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

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

## 4.4 ROCK SHEDS

### 4.4.1 Introduction

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

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

### 4.4.2. Description of problem

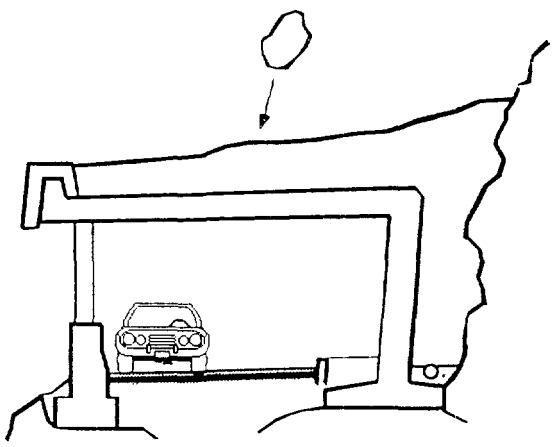


**Fig. 4.6 Rockshed.**

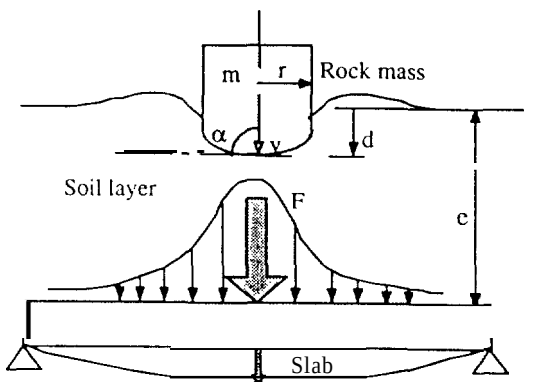



**Fig. 4.7** Definition of **problem.**

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

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

### 4.4.3 Test device

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

**Table 4.1 Range of test parameters**

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

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

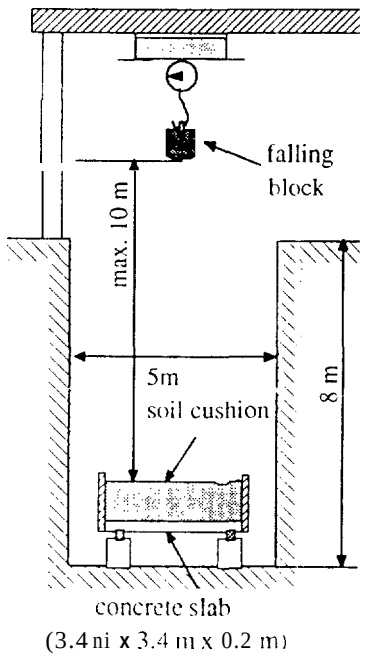



Fig. 3.8 Elevation of test shaft.

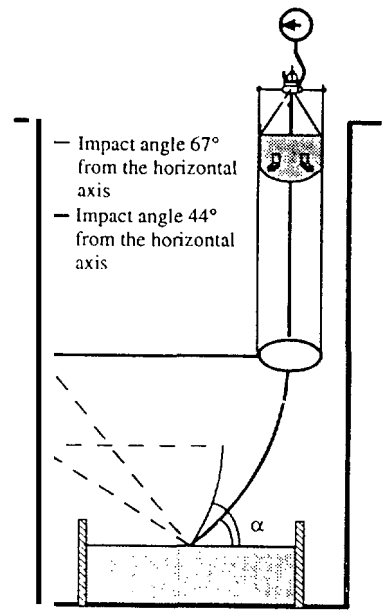



Fig. 4.9 Elevation of testing shaft and for inclined impacts.

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

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

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

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

### 4.4.4 Quantitative evaluation of forces

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

### 4.4.5 Inclined impacts

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

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

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

### 4.4.6 Conclusions

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

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

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

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# 5 LANDSLIDES

C. Bonnard and L. Vulliet

## 5.1 INTRODUCTION

Landslides, permanent movements or catastrophic events can be defined according to the types of phenomena as well as by their intensity and probability of occurrence. According to a generally admitted classification the generic term *Landslides* includes all gravity-induced movements of **a** soil or rock mass along a slope (WP/WLI 1990), which may be composed **of** five major mechanisms (1) *fall,* (2) *topple,* **(3)** *slide,* (4), *flow,* or *(5)spread.* Both first quoted mechanisms are already dealt with in the preceding chapter on rockfall, whereas spread is a peculiar phenomenon hardly related with slopes, but more to large scale settlement of underlying soft layers, for which no global resilient infrastructure can be imagined, except rigid structures for buildings. 'Therefore this chapter mainly deals with *slides* (translational, rotational, complex phenomena of various sizes — from 1 ha to several $km^2$) and with mud and debris *flows* which often extend over smaller areas than slides, but are more dangerous in terms of damage.

The first major characteristic to mention for these two types of phenomena is their non-repetitivity : unlike snow avalanches they induce either a slow permanent movement or a sudden violent displacement of soil masses with a variable percentage of water, but the conditions of such events will never be reproducible, so that, for their analysis, statistics is not an adequate tool and experiencs of past events is not always significant. However, a detailed observation of their characteristics and behaviour often allows a proper determination of the hazard they imply, by the adequate identification of warning signs and analysis of preparatory and triggering causes.

When focusing on the relation between the dangerous naturai phenomenon and the man-made structure to be protected (buildings, roads, lifelines), it is worth noting that in the case of debris flows the disnstcr resilient infrastructures have to be designed mainly in the *transition zone*, as no relevant action can be taken in the source area, except reforestation, and as the cost of structures resisting by themselves to the pressure of the flow is too high. In the case of slides, the notions of zone of origin, transition and impact are irrelevant, as most of the man-made structures are located on the moving mass itself; therefore either the planned infrastructures tend to block or slow down the sliding mass, which is the most frequent case (eg. by drainage, fill or anc hors), or the structures themselves are designed so as to tolerate or impede movements of their basement (DUTI 1985). The foundations of bridge piles laid on bedrock can also be protected from the sliding mass above by a concrete shaft surrounding them and ensuring a sufficient gap to allow an independent movement.

## 5.2 MAIN FACTORS FOR LANDSLIDE RESILIENT INFRASTRUCTURE

**As** far as *slides* are concerned, four main factors condition the feasibility of stabilization projects. First the *depth* of the active sliding mass imposes limits to any type of resilient infrastructure arid is one of the major criteria to select the appropriate system (for instance length of anchors, depth of drainage boreholes or trenches, etc.). Thus many landslide zones in Switzerland as well as in the world cannot be stabilized due to the large depth of the sliding niass, exceeding *100* m.

Then the *velocity* of the slide, considering mainly its average permanent movement, determines the possibility of application of some construction techniques, specially when the major displacements are concentrated at a unique slip surface. For example, vertical drainage boreholes are only applicable when the movement is slow enough (approx. 1 cm/year) to allow a stabilizing effect to occur before they are sheared.

**A** third important point is the occurrence of *differential movements* in some lateral zones of a slide or at the limits of a secondary slip surface which induce unhomogeneous conditions in terms of depth, velocity, movement pattern, so that the buildings and lifelines in these zones can be severely affected. These first three factors are somehow related to the general notion *of* intensity in a hazard analysis.

The last factor rather deals with the concept of probability which has to be also considered in a hazard and risk analysis. It includes the *potential* for progressive or *sudden accelerations* which often cause distress to supposedly resilient infrastructure. Such accelerations depend mainly on the variations of climatic conditions, either at a short-term scale (high intensity rainfall during some days to some months) or at a long-term scale (periods of several wet years, global climate change). The effects of an increase of precipitations can be either direct, raising the groundwater level and inducing higher driving forces in the sliding mass, or indirect, for example through more significant erosion rate at the toe of the slide. Such a relation is quite often complex and needs long-term monitoring to assess this parameter.

**As** far as *debris flows* are concerned, the last factor mentioned above is certainly prevailing, as high intensity *storms* are the main cause of disaster. In this case the major difficulty lies in the determination of local precipitation distribution as intense rainfall occurring on a limited drainage area may not be recorded at the nearby raingauge station.

**A** second factor for assessing the intensity of the event is the *slope* of the stream in which it occurs, as it will directly condition the velocity of the debris flow, and thus the potential for increasing the sediment mass by erosion.

A final factor is the availability of *loose materials* in the upper part of the drainage area either at the surface and liable to direct erosion, or along the torrent channel which may be mobilized by local lateral slides, inducing the phenomenon of retention and consecutive violent outflow.

## 5.3 SLIDE RESILIENT INFRASTRUCTURE

In order to reduce or to stop permanently the movements of slides and thus limit their disasterous impact, foiir main classes of remedial measures can be used. Indeed they also apply to the stabilisation of rockfall source zones. This presentation corresponds to an internationally approved list (WP/WLI 1990, Popescu 1996).

### 5.3.1 Modification of slope geometry

**As** the driving and resisting forces within a sliding mass are mainly related to the geometrical Characteristics of the slope, the basic way to reach a definitive and sure stabilisation of a slide, provided it is of fairly limited dimensions, can include the following earthworks :

- Removal of material from the upper area, with a possible substitution by lightweight fill.
- Construction of a buttress berm or fill at the toe (Sève and Pouget 1998).
- Reduction of the general slope angle and trimming of loose surface material.

However these earthworks often require a lot of surrounding space to organize construction activities and affect a large part if not the whole surface of the unstable zone, so that in many cases it is not econoinically and socially applicable.

### 5.3.2 Retaining structures

A similar action, but inducing a localized increase of resisting forces by the application of structural means, at the surface or at shallow depth, can include the following types of retaining structures :

- Gravity retaining walls and reinforced concrete walls.
- Crib block walls, which are more flexible than retaining walls.
- Gabion walls, also useful against toe erosion (Federico 1985).
- Passive piles and caissons, sheet piles.
- Reinforced earth retaining structures (with strip/sheet polymer/metallic elements).
- Buttressed counterforts of coarse-grained material, providing an increase of shear resistance (Sève and Pouget 1998).

Several quoted solutions combine also the advantages provided by drainage action and the reliability offered by mass movements. However their action at shallow depth may impede providing a reliable protection against deeper instability phenomena that can be induced by long-term erosion at the toe of the slide.

### 5.3.3 Internal slope reinforcement

A series of improvements in drilling and grouting techniques as well as in the design of soil/rock inclusions have allowed an impressive development of stabilization means by internal reinforcement, leading to applications even at large depth and insuring long-term stability. The main advantage of most of the following techniques consists in applying effective resisting forces at the level of the slip surface :

- Rock bolts and soil nailing.
- Anchors (prestressed or not).
- Micro-piles and anchored piles (see Fig.5.1) (Wichter and Meiniger 1985).
- Grouting and jetting.

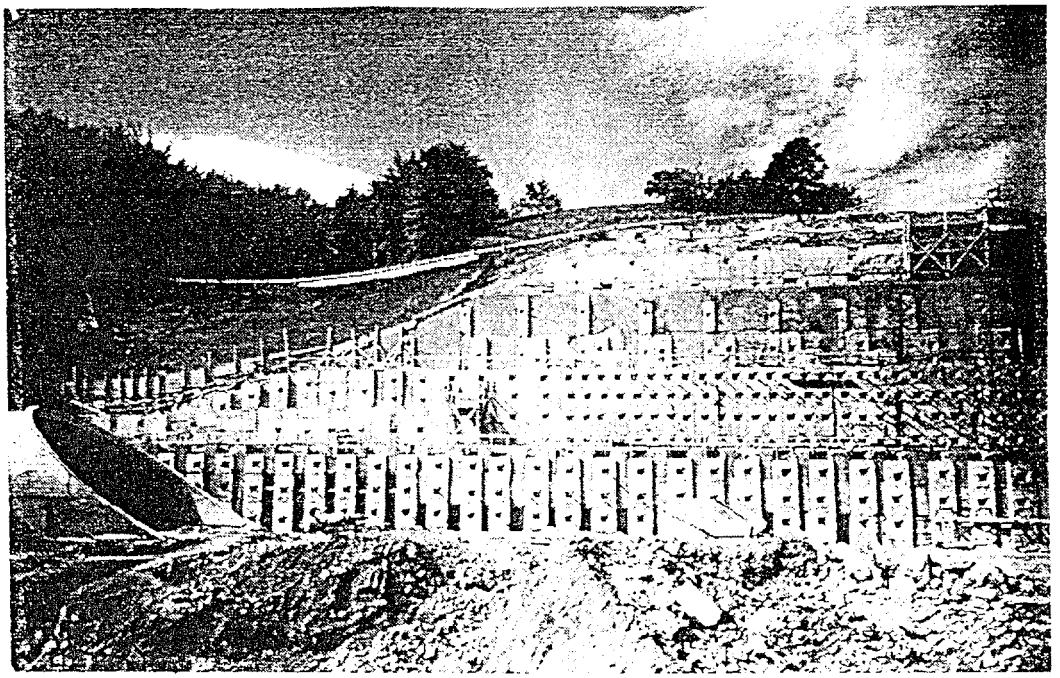

Fig.5.1 La Criblette landslide stabilized by prestressed anchors along A9 motorway near Lausanne.

In the same category of stabilization means it is possible to add heat treatment and elecírosmosis as well as freezing, although their applicability is really reduced to very specific situations. Finally much has been said about biotechnical slope stabilization, which induces an internal reinforcement at very shallow depth through man-made wooden structures and the roots of some specific plants. But although it appears a sustainable means, its long-term effect is often not guaranteed as the vegetation may decay due to drought and as deeper slip surfaces may cause the destruction of such structures.

### 5.3.4 Drainage

Finally, giving due consideration to the fact that *groundwater* is the main destabilizing factor of unstable slopes, the different types of drainage, superficial or underground, constitute one of the most efficient means to control or at least reduce slide movements, especially for large landslides. Several systems have been developed and applied in various sites of Switzerland (as well as in the world), in a whole range of situations, implying sliding volumes from some thousands of $m^3$ to 1 billion $m^3$. The main drainage systems include :

- *Surface drains* to divert run-off water from flowing onto the slide area, by collecting ditches or by wooden, mortar or steel channels.
- Shallow or deep *trench drains* (max. depth 15 m) with pipes, filled with free-draining geomateriaís, i.e. coarse granular fills protected by geosynthetics (Cancelli 1985).
- *Buttress counterforts*, localized trenches, masks or gabion structures providing a draining and a mechanical effect.
- Vertical small diameter *boreholes* with pumping or vacuum dewatering, siphoning or self draining into a gallery or an underlying pervious rock layer (Noverraz and Bonnard 1993).
- Vertical large diameter *wells* filled with coarse material, with gravity draining at the toe by a horizontal borehole or a gallery.
- *Subhorizontal boreholes* from the surface or from a shaft.
- *Drainage tunnels*, galleries or adits.

Additionally vegetative planting which induces a higher evapotranspiration can also be considered as a drainage means.

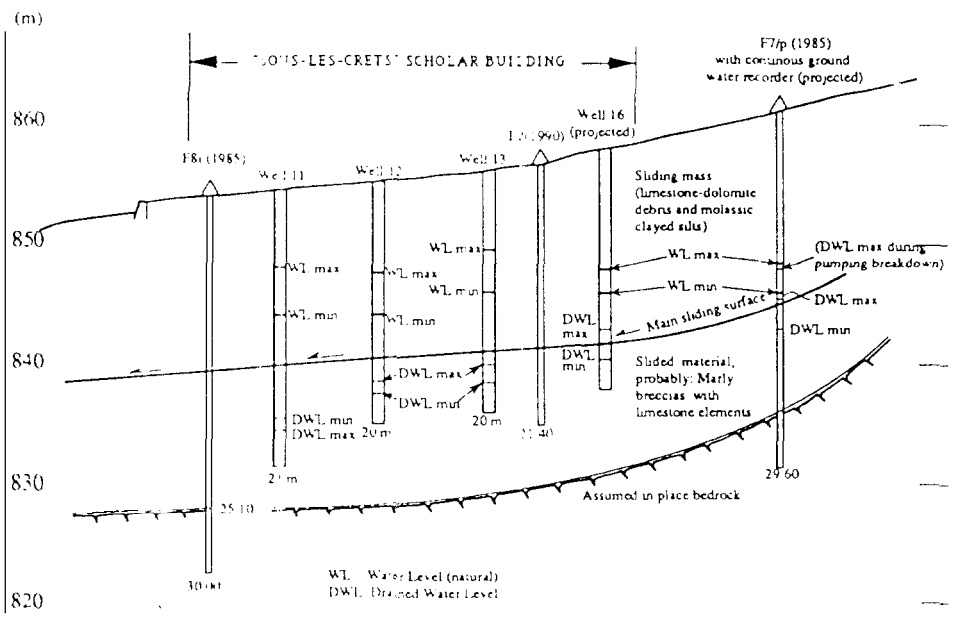


Fig.5.2 Cross-section of St-Imier landslide with location of investigation boreholes and drainage wells.

Two major problems arise however in the use of drainage. First the future *efficiency* of a drainage system of any kind is difficult to predict, as the localized effect of drainage works will not fully extend to the whole slip surface and thus only increase locally the resisting forces (Sève & Pouget, 1998). Then the actual groundwater conditions are difficult to assess and can be modified substantially by extreme climatic conditions. Drainage works need also regular monitoring and maintenance works in order to check that their long-term operational capability corresponds to the original design. Despite of these limitations inducing often a partial stabilization, ***drainage works*** often appear as the most adequate **and** economical counter-measure to insure the safety of structures and lifelines located on ***a*** slide (Fig.5.2) (Noverraz and Bonnard 1993, Gabus et al. 1988).

Some major drainage works have been carried out these last 20 years in Switzerland in order to provide a complete or improved stabilization of large landslides. Although being certainly not exhaustive, a list of some interesting cases (including only drainage works) may be given :

- Arveyes Landslide (canton of Vaud) where 16 deep boreholes equipped with pumps reduced to 1 mm/year the velocity of a $25 \cdot 10^6$ $m^3$ slide (1983-1986) (Gabus et al. 1988).
- La Frasse Landslide (canton of Vaud) where 28 boreholes equipped with pumps tried to reduce the velocity of the lower part of a $60 \cdot 10^6$ $m^3$ slide; but several were rapidly sheared (1995). Surface drainage channels had also been carried out (Noverraz et al. 1998).
- Ballaigues Landslide (canton of Vaud) where hundreds of vertical boreholes (spacing at 2 m) discharge drained water in the underlying pervious rock 40 m below the surface and significantly reduced the movements of a slide (1983-84) (Noverraz et al. 1998).
- St-Imier Landslide (canton of Berne) where 16 boreholes equipped with pumps limited the velocity (*to* 5 mm/year) of a slide on which **a** college was built (1981), despite of some very wet years (Noverraz and Bonnard 1993).
- Braunwald Landslide (canton of Glarus) where a drainage trench carried out by a diaphragm wall equipment was built with a jacked tunnei beiow it, in order to protect **a** hotel at the edge of a very large slide zone (1983).
- Campo Vallemaggia slide (canton of Ticino) where a 2 km long gallery with radial boreholes below a huge landslide ($1 \cdot 10^9$ $m^3$) was built in order to control the movements which showed sometimes high accelerations (1996) (Noverraz et al. 1998).

## 5.4 DEBRIS FLOW RESILIENT INFRASTRUCTURE

The high energy developed by debris and mud flows (showing velocities in excess of 10 m/s) and the Iarge volume of transported material (reaching sometimes 50000 m3) impedes most of the time the safe design o!' containment structures with concrete walls or earthfill dams (an interesting exception may be quoted at Les Crétaux Landslide (canton of Wallis) where two reservoirs were operated alternately and then emptied to store frequent small debris flows). Therefore four main classes of remedial measures can be used to control these events before they hit structures or lifelines which are essentially not able to resist to such thrust.

### 5.4.1 Escalonated river protection scheme

The first possible action is in build a series of concrete or wooden **dams** of limited height, in the stream beds with a debris flow potential, so as *to* reduce the flow velocity, impede regressive erosion and retain a part of the debris flow mass before it reaches the lower alluvial **fan where** structures have to be protected. *Check dams* have been carried out quite often in Alpine regions for more than a century, providing even a partial stabilization of a whole slide slope at the Swiss largest landslide at Heinzenberg/GR (Noverraz et al., 1998).

### 5.4.2 Lateral protection dams and dikes

The second possible action irequently carried out consists in the construction of lateral dams or ***dikes*** along the stream bed in the expansion zone (alluvial fan), so as to avoid that the debris flow may overtop the natural banks and destroy nearby structures. Debris flow mass control is then often shifted downward, for instance into the riverbed where the stream merges with the main river, but such situations may be solved by an adequate junction angle allowing the river flow to erode the deposited material. This type of work is frequent in the Swiss Alps.

### 5.4.3 Emergency spillway structure

When the capacity of the stream bed or artificial channel near the impact zone may be exceeded in case of very important debris flow, it is possible to foresee a concrete emergency ***spillway*** structure that will divert a part of the debris flow towards a safe zone where no major damage is liable to occur. Such work has been constructed at the Pissot stream bed (canton of Vaud), downstream of a first retaining structure of limited capacity and before the channelized stream passes above **A9** motorway.

### 5.4.4 Structure separating bed load from water (Japanese trap)

The last control system of debris flows consists in a large open reinforced concrete structure built below the riverbed and covered by a *steel ruck* with large spacings between the bars. This type of sieve allows the draining of the debris flow mass, as the water will fall into the structure and be evacuated downstream in the stream bed, whereas the large size bed load rolling on the subhorizontal gate will loose their transport means, i.e. the muddy water, and thus stop on the gate or just downstream. Such type of work developed in Japan has been built on the Dorfbach near Randa (canton of Wallis) and has proved quite successful.

## 5.5 GUIDELINES TO IMPROVE THE SAFETY OF DISASTER RESILIENT INFRASTRUCTURE

Despite of the development of new stabilization techniques and the improvement of landslide modelling, the long-term reliability of disaster resilient infrastructure tends to decrease with time, due to maintenance problems, whereas the safety requirement and induced risks increase, following the construction of more and more buildings and lifelines in exposed zones. Therefore any type of slope stability improvement works has to be completed by a comprehensive *monitoring system* allowing for early detection of a critical behaviour, based on adequate warning signals. But until the alarm criteria corresponding to such systems are duly established and tested, which may take time, it is necessary to complement resilient infrastructure with *passive management measures* relying on limited use of endangered land, whatever are the economic pressures towards its development.

The major *research needs* deal with the relation between drainage efficiency and the hydrogeological conditions, especially their evolution with time during crisis events, for which continuous pore water pressure monitoring is one of the most important information. The role of unsaturated layers in the slope stabilization represents also a major research challenge *to* master the long term behaviour of landslides. **All** the monitoring data should provide a basis for an adequate risk analysis in which direct and indirect economical factors as well as *safety criteria* can be duly included, which will certainly lead to the necessity of more landslide resilient infrastructure.

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# 6 IMPULSE WAVES

W.H. Hager

## 6.1 INTRODUCTION

Waves induced by large masses moving into water are referred to as impulse waves. Given the typically large momentum exerted by such masses, i.e. the product of mass times velocity, the resulting phenomena produce often shallow waves. Water waves can indeed be subdivided into deep water and shallow water waves, the first having a small wave height compared to the still water depth, and the second having a large relative amplitude. Deep water waves are essentially based on a linear theory and are typically wind exerted. They are not further considered here.

*Shallow water waves* are highly nonlinear and a mathematical approach is complicate. Because of large wave amplitudes, the hydrostatic pressure dismbution does not apply, and streamline curvature effects have to be accounted for. **A** basic wave type is referred to as the *solitary wave* characterized with a single positive wave peak propagating over an otherwise plane surface. An intermediate type between solitary and sinusoidal wave types is referred to as the cnoidal wave, that typically results froín large masses plunging into a fluid.

Currently, some information on impulse waves is available, mainly in terms of compact masses plunging into a reservoir. Actual slides involve granulate, mud or snow, and cannot be simulated with a compound mass such as a single block of rock. Impulse waves thus involve a *three-phase flow* containing the slide material, water as fluid and air due to entrainment effects. This complication is certainly a major reason for limited knowledge on this highly interesting but dangerous phenomenon.

**Fig. 6.1** Vajont arch dam after impulse wave **and** overtopping in 1963.

Impulse wave into surface waters may originate from earth, rock, snow or glacier movements. All these slides are related to water, such as thunderstorms, avalanches, extreme weather conditions or rythmic water level changes. Slides moving into water bodies damage the region of origin by mechanical fracturing, result in wave action that can destroy shore regions or can even damage infrastructure such as roads, buildings or other near-shore structures. Of particular relevance are destructions in man-made reservoirs because a slide can cause runup or overtopping of a dam that retains a large volume of water. As a result, dambreak may occur with significant damage to downstream regions. Such a scenario occurred at various locations worldwide, but the *Vajont slide* in Northern Italy on October 9, 1953, is certainly the most widely known. Conditions were not all together critical for this incidence although the immense rock avalanche of some 300 Mio $m^3$ discharged into the reservoir and created an impulse wave of about 100m height. Fortunately, Vajont dam resisted this shock, but a large water volume overtopped it and killed 3000 inhabitants of the downstream located village Longarone (Fig.6.1). The following introduces the main features of impulse waves with regard to disaster resilient infrastructure. This topic is closely related to slides, rock falls and flood waves, and the corresponding contributions should also be consulted.

## 6.2 FEATURES OF IMPULSE WAVES

Waves generated in water bodies due to momentum release involve three features (Fig.S.2):

① Wave generation due to mass impulse on water body,
② Wave propagation over the water surface, and
③ Wave impact on water boiindaries, such as shores or man-made structures, including possible overtopping.

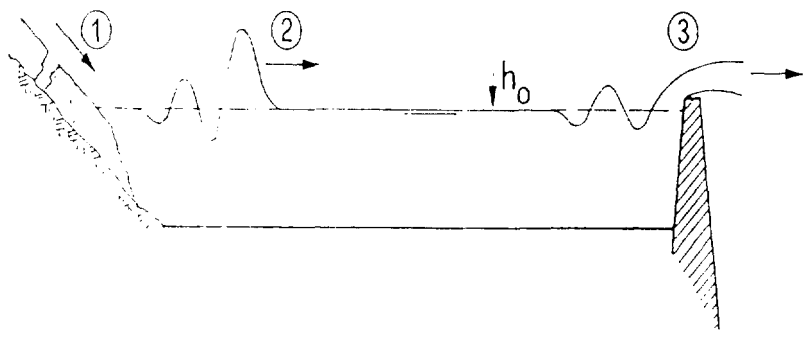


Fig.6.2 Impulse wave mechanisms on reservoir, schematic.

Impulse waves can be characterized as follows:

- The first wave has normally the maximum wave amplitude, and contains also the maximum energy,
- Waves develop from a highly complex phenomena close to impact location into a gradual surface phenomenon that is amenable to computational analysis,
- Waves decay in height as they travel over a water body of nearly constant depth,
- Typical developed impulse waves are either of solitary or cnoidal wave type, and
- Wave runup and particularly wave overtopping depend on shore conditions and are governed by difficult physical processes.

Given the complications due to various phases and abrupt temporal changes, both the impact an the runup conditions are currently not amenable to prediction, except for hydraulic modeling provided the proper similarity laws are accounted for. Two cases have received particular attention in the past, namely the plane impulse wave and the spatial impulse wave. These are described below.

The *plane impulse wave* was considered under the following conditions **(Fig.6.3):**

**(1)** Relative wave smaller than wave breaking limit, i.e. a relative amplitude smaller than **78%** of the still water depth,
(2) Relative propagation domains larger than 5 and smaller than around 100 times the still water depth,
**(3)** Slide velocity larger than about half of wave celerity,
**(4)** Slide angle larger than about 30° and smaller than 60°, and
(5) Dense slide such as for a rock avalanche but not for exploded material containing much dust.

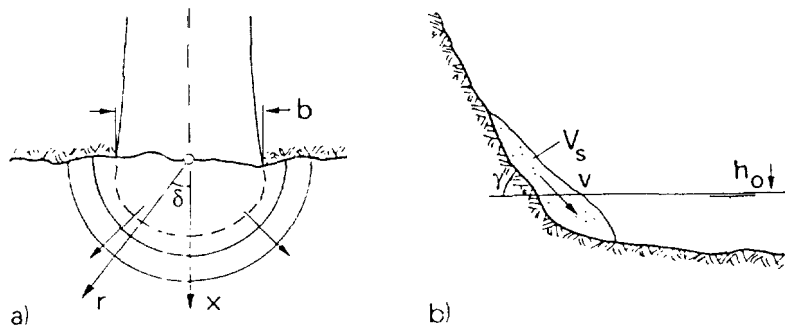


**Fig.6.3** Wave generation into reservoir a) plan, **b)** section.

The maximum wave amplitude relative to the still water depth $a_M/h_o$ depends on four dimensionless quantities, namely the slide angle y with a large effect, the relative slide volume $V_s/(bh_o^2)$ with an intermediate effect, and the prodiict of density ratio $(\rho_s/\rho_w)$ times the relative distance $(h_o/x)$ with a relatively small effect. The notation is explained in Fig.6.3, and $\rho_s$ and $\rho_w$ are densities of slide and water, respectively. For any given potential slide location, the slide density $\rho_s$, the slide angle y and the still water depth $h_o$ can hardiy be influenced. The only parameter with a certain degree of variability is the slide volume per unit width $V_s/b$. The still water depth can of course be reduced for slides that do not immediately occur, provided the water elevation can be drawn down by operating a *bottom outlet.*

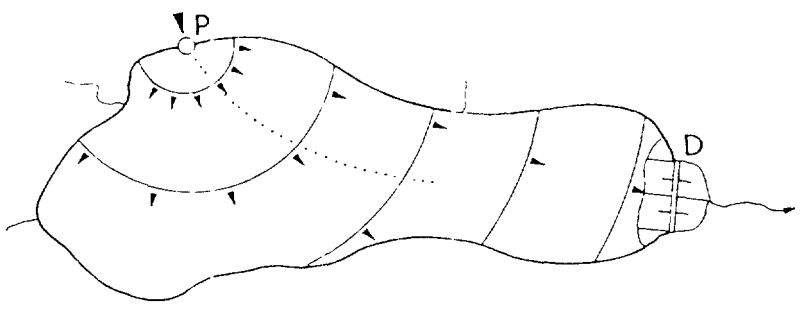


**Fig.6.4** Impulse wave generated at point P **and** propagating towards reservoir shores and dam D.

For a slide running into a reservoir resulting in ***spatial impulse wave,*** the relative wave amplitude $a_M/h_o$ follows previous characteristics, except of lateral wave propagation (Fig.6.4), and waves propagate radially from the impact location into the water body. The highest wave occurs in the direction of the slide and waves decay .laterally. Because **of** energy radiation, slides into a 3D water body are much smaller than into the plane reservoir.

## 6.3 IMPULSE WAVE RUNUP AND OVERTOPPJNG

Consider a water body of still water depth $h_o$ containing an impulse wave **of** amplitude $a_M$ and propagation velocity $c_w$. To estimate the potential of damage, the runup characteristics on a shore must be known. With L, as the wave length (Fig.6.5) between $0.5 \le h_o/L_w \le 2$, the runup height $R=r/h_o$ depends essentially on the relative wave height $h_M/h_o$ and slightly on the runup angle $\beta$ and the relative wave length $L_w/h_M$. The *runup **height*** may be somewhat reduced when increasing the runup angle to, say, 90°. Wave breaking occurs if the index $\tan\beta/(h_M/L_w)^{1/2}<3$.

An analysis of VAW data summarized by Vischer and Hager (1998) shows that

$$\frac{r}{h_M} \cong 1.25 \left(\frac{\pi}{2\beta} \cdot \frac{L_w}{h_o}\right)^{0.20} . \tag{6.1}$$

This indicates that the runup height increases significantly with the maximum wave height, and slightly with the product $[L_w/(\beta \cdot h_o)]$. To control runup, one may practically adjust only the wave height $h_M$, therefore.

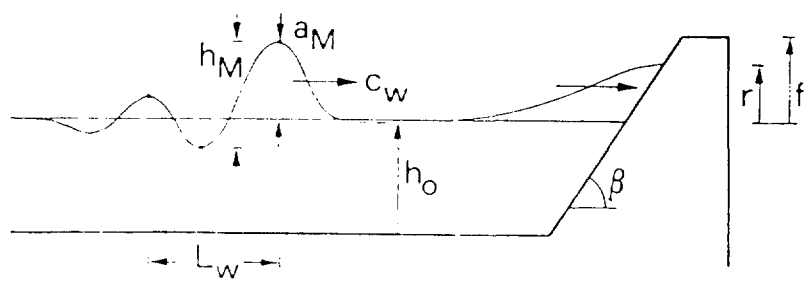


**Fig.6.5** Wave runup on dam or shore, definition of **variables.**

Reservoirs with a dam downstream are particularly endangered because ***dam overtopping*** can occur in addition to wave runup. Fig.6.6 shows a definition sketch with $L_d$ as crest width, r as runup height on a hypothetical shore of angle $\beta$, and f the freeboard under still water conditions. The *effect **of** freeboard* can be described as

$$V_d/V_o = (1-f/r)^2 , \tag{6.2}$$

with $V_d$=overtopping volume, and the reference volume

$$V_o = C_1(g h_M^6 h_o^2 t_w^2)^{2/9} \tag{6.3}$$

where $C_1$= a constant of the order 0.6, depending on the crest geometry. The overtopping period varies mainly with the wave period $t_w=L_w/c_w$ where $c_w=[g(h_o+a_M)]^{1/2}$ is the propagation velocity. Both wave runup and wave overtopping are based on a plane approach flow. Roughening a shore has practically no effect on both wave runup and overtopping characteristics.

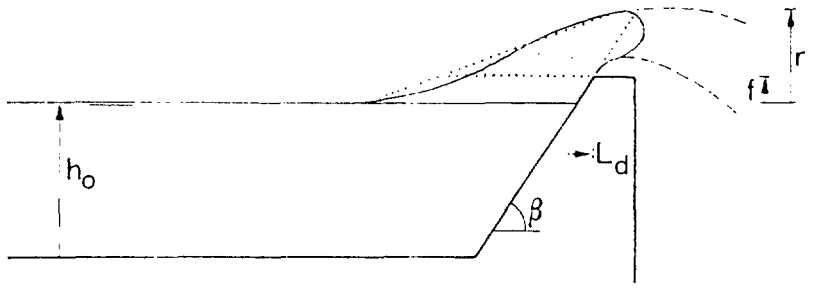


**Fig.6.6** Overtopping of **dam** with **(–) overtopping volume.**

Assuming an average value $h_M=1.5a_M$ allows further elaboration f Eq.(6.3) for $L_w \cong h_o$ s

$$V_o = C_2 \left(a_M^2 h_o\right)^{2/3} \left(1 - \frac{a_M}{2h_o}\right) \tag{6.4}$$

The relative reference volume $V_o/h_o^2$ depends thus exclusively on the relative wave amplitude $a_M/h_o$. Note that $V_o$ corresponds asymptotically to the overtopping volume for $(f/r) \rightarrow 0$.

## 6.4 CONSEQUENCES FOR INFRASTRUCTURE

### 6.4.1 Reservoir overtopping

The infrastructure endangered by impulse waves is located along a reservoir shore for wave runup, and downstream of the reservoir for wave overtopping. The latter scenario can be compared with a *dambreak* and may not directly be countered. The only measure against a dambreak is defining hazard zones, such as *zone 1* for immediate danger where there is no chance for evacuation, because of the darn proximity. Accordingly, no important infrastructure may be erected in this zone In *zone* 2, evacuation within a short time is possible though not simple. This zone should riot be accessible for population and infrastructure for strategic purposes. In *zone 3* only slight damages due to dambreak floods are expected, and evacuation is strictly possible. Procedures of evacuation should be trained with the population involved, and future infrastructure should be so designed that damages do not lead to significant conflicts, such as for access roads to villages endangered, supply lines or buildings of higher priority. The general recommendations given for protection against flood waves should be consulted, because the dambreak wave is a kind of extreme flood wave. Certainly a key mode of protection against dambreaks is the *dam safety technique,* involving regular control of all dam facilities, including monitoring of slides and settlements, geotechnical displacements, seepage flow and adopting hydrologic changes

### 6.4.2 Wave runup

Impulse waves may damage infrastructure mainly by wave runup, except for extremely large slides into a practically full reservoir resulting in overtopping. According to Eq.(6.1) the runup height r is mainly influenced by the wave amplitude $h_M$, which depends significantly on the slide angle $\gamma$, the slide volume per unit width and the relative distance of slide impact to runup location $x/h_o$. For large slides only the *reservoir depth* $h_o$ can be influenced if a bottom outlet is available. It will be hardly possible to reduce the relative slide volume by widening the slide, as also the slide angle. In order to reduce the reservoir level from an originally full reservoir that

corresponds to the design condition (worst case scenario) the *temporal slide prediction* is of outmost importance. Again, one may distinguish between three cases:

(1) The slide occurs completely unexpectedly. This may be due to poor management or surveyance, or due to extreme thunderstorms. Then reservoir drawdown is impossible, and the slide may cause large scale damage. For cases with a proper management, the latter incident cannot happen because responsibility would restrict the reservoir filling elevation.
(2) The slide is predicted but within such a short time of occurrence that no significant reservoir drawdown can be introduced. Then all people and perhaps material of general value have to be safed.
**(3)** The slide occurs within days or weeks, depending on meteorological conditions mainly. Then, reservoir drawdown should be initiated to a degree that the downstream community is not endangered, but losses in values may be accepted. The latter scenario **is** closely related to modern dam safety, and should be the usual case. **A** slide has to be monitored and controlled by adequate engineering methods. Cases (1) and (2) have to be excluded.

Case **3** is, to be sure, more or less the only strategy to counter impulse waves. For the other two cases, there is practically no method of defense, given the enormous potential of energy set free by impulse waves. **A** way to protect strategically significant access roads are tunnels, rather than roads along shores. This concept is often dictated in Alpine countries because of extremely large slopes. If an access is strategically determining, this concept can largely contribute to reducing damages. For example, a *bottom outlet gate* has to be operated under ail other conditions, because it will ultimately reduce damages of impulse waves. if such a gate gets squeezed, or does not move because of power supply shutdown, an complete emergency concept can fail.

## *6.5* RESERVOIR DRAWDOWN

A relevant question related to impulse waves is the *drawdown capacity* of a reservoir. If a general reservoir shape is considered that is controlled by a bottom outlet, the time t required for drawing down an initially full reservoir of depth $H_o$ to depth H(t) can be given as

$$t/t_* = \frac{1}{60.5}\left[1-(4/H_o)^{\delta-0.5}\right] . \qquad (6.5)$$

Here $t_*=V_o/[C_d A(2gH_o)^{1/2}]$ is a reference time with $\delta$=reservoir shape factor, $V_o$=full reservoir volume, $C_d$=discharge coefficient, approximately equal to 0.60, and A=bottom outlet area. Typically a value $\delta$=2.5 may be used, with extremes $\delta$=1.5 for bucket-shaped, and $\delta$=3.5 for V-shaped reservoirs. Eq.(6 5) thus simplifies *to*

$$t/t_* = \frac{1}{2}\left[1-(H/H_o)^2\right] . \qquad (6.6)$$

To reduce the reservoir elevation by 10% and 50%, say, one would use $t/t_*$=0.10 and $t/t_*$=0.38, respectively. **A** full reservoir drawdown requires $t/t_*$=0.50.

Consider a reservoir volume $V_o=10^7 m^3$ with a reservoir depth $H_o=10^2$m. For **a** cross-sectional area of the bottom outlet $A=10m^2$, one has $t^*=2.5\cdot10^7/[0.60\cdot10\cdot(2\cdot10\cdot10^2)^{1/2}]$ =93'200s=1.08d. Such a small reservoir might thus be drawn down to 90% and 50% elevation heights within t=2·5h, and 10h, respectively, and might be completely emptied within one day. If the reservoir had a smaller deptti but the same volume $t_*$ increases accordingly. The time scale is thus dictated by $t_*$, that is drawdown of a reservoir is fast for:

- Smali reservoir volume $V_o$
- Large bottom outlet section **A,**
- Large reservoir depth II,.

For a given reservoir, one may only influence parameter **A,** and a *large bottom outlet* is

## 6.6 RECOMMENDATIONS

Impulse waves can have a desasterous potential of damage combined with a significant degree of uncertainty. There are practically only two procedures to protect infrastructure: (1) evaluation of a emergency scheme by assuming the most critical combination of parameters, and (2) analysis of reservoir drawdown in terms of reservoir and downstream flow characteristics. In addition the following items may be added:

(1) Reservoirs with a potential to impulse waves should be controlled with a *bottom outlet*. If a bottom outlet is missing, its addition must be seriously considered, not only to counter impulse waves but also in terms of general dam safety, reservoir sedimentation and a more flexible reservoir management. If a bottom outlet is available, the drawdown features should be evaluated in terms of drawdown time and tailwater floods.

(2) Strategic *access roads* should be protected from floods and impulse waves by road tunnels. Also, electric power supply lines have to be so arranged that power is available even during large floods. A bottom outlet whose outlet gates cannot be moved is strictly of no value. The addition of an emergency power system based on fuel can be considered as an alternative. Given the extreme pressure forces exerted on outlet gates, its operation should be periodically tested.

(3) *Research activirics* in impulse waves are currently small, given the complex interactions of geology, rock and ice mechanics, soil mechanics and hydraulics. The latter item has received particularly scarce attention, and questions relating to the effect of approach slide velocity, slide mixture characteristics, effect of air entrainment or momentum transfer during impact on the water body are not yet treated at all. To the author's knowledge, VAW has currently one of the few research activities worldwide on impulse waves, and it is hoped that the current experiniental approach will be extended by a numerical research study to predict far-field effects of impulse waves on reservoirs.

Impulse waves are one of the spectacular but also one of the very dangerous natural hazards, therefore. Its knowledge should definitely be improved by an appropriate funding. This report aims to outline the possible procedures and to indicate directions which should be taken to reduce damages of lives and values.

## REFERENCE


- Vischer, D.L., Wager, W.H. (1998). *Dam hydraulics*. John Wiley & Sons: Chichester, New York.

# 7 EARTHQUAKES

J. A. Studer

## 7.1 C IARCTERISTICS AND DAMAGES

An earthquake is a sudden ground motion produced by abrupt displacement of rock masses, usually within the upper 5 to 30 km of the earth's crust. Most earthquakes result form the movement of one rock mass over another in response to tectonic forces. The rupture of the rock masses causes the ground to vibrate at frequencies ranging from about 0.1 to 50 Hz (cycles per seconds). As a generalisation the *shaking severity* increases as the magnitudes of the earthquake increases, and decreases as the distance form a site to the fracture plane increases. Experience shows that surface geological materials and the topography may influence the level and nature of the ground shaking strongly, an effect often underestimated in current hazard assesssments.

> There are no means to prevent earthquakes and currently no possibility exists to predict short term occurence with accuracy in terms of location and size of earthquake and time of occurence. The only possibilities to reduce the earthquake damages are appropriate planning and construction measures.

In terms of human and economic loss seismic shaking is the most significant factor contributing to the overall earthquake hazard. Shaking conmbutes to losses not only through *direct vibratory damages* to man-made structures but also indirectly through triggering of *secondary effects* such as landslides or rockfalls or even other forms of ground failures (soil liquefaction, slumping or settleinents). Thus, an important element in seismic hazard zoning on a regional basis is the geographical assessment of potential ground shaking.

In addition to strong ground motion, a variety of associated phenomena can cause serious damage and loss of lives:

*Surface faulting*. The offset or tearing of the earth surface by differential movements across a fault is an obvious hazard to structures built across active faults. A variety of structures have been damaged by surface faulting, including buildings, railways, roads, tunnels, bridges, canals, water wells and water mains, electricity lines and sewers. Surface faulting can be particularly severe to structures partly embedded in the ground and for underground pipelines or tunnels. Surface faulting generally affects a long and narrow zone ranging from few meters to more than 100 m. Subsidiary branch faults have extended as much as 10 km from the main fault and secondary faulting has been observed more than 25 km away from the main fault. The lengths of the rupture have ranged from few 100 m up to about 400 km. Their size is important for zoning purposes around active faults.

*Tectonic subsidence and uplift* are usually accompanied by surface faulting. The deformation may be local, affecting a narrow zone near the fault break, or may involve major differential vertical and horizontal movements over broad parts of the earth crust. This local deformation can distort or tilt structures. Regional tectonic deformations constitute a hazard to shore-line facilities and extensive hydraulic systems where broad scale changes in land elevation occur relative to the water level. Such changes can affect hundreds of square km of the earth surface and can damage harbour facilities, canals and other structures. During the 1964 Alaska earthquake for example piers, docks, breakwater structures, roads, railways, airstrips, and buildings were tectonically lowered relatively to the sea level resulting in permanent or intermittent inundation. In oilier areas tectonic uplift caused shallowing of harbours and waterways and thus restricted their use.

*Landslides, rockfalls, avalanches*. Earthquake shaking can dislodge rock and debris on steep slopes, triggering rockfall, snow and ice avalanches. Ground shaking can initiate shallow debris slides on steep and less often rock slumps and rock fall on moderate slopes. Under

certain geological conditions shaking can reactivate dormant slumps or block slides. Avalanches can be triggered in weakly cemented fine graded material, such as loess, that form steep stable slopes under ncn-seismic conditions. Even small water-saturated sand lenses can mgger major landslides in nearly horizontal clay deposits, as occured in the 1964 Alaska earthquake in Anchorage.

*Liquefaction.* Areas having layers of water-saturated loose fine sand or silt typically deposited in the past 10'000 years can temporally loose their strength and behave as a viscous fluid due *to* severe ground shaking. Structures founded on such deposits settle, tilt or rip apart (Japan) as the soil spreads laterally. Buried structures may float up. Ground shaking can cause lateral movements on the top of liquefied surface layers. Such large ***subsoil*** *deformations* usually interrupt service lines (water supply, sewer, gas or electricity). Due to soil liquefaction the port facilities of Kobe for instance were out of service for several months due to the February 1995 earthquake. In the harbour area the entire watersystem went out of service due to liquefaction, induced excessive subsoil deformations, and hindering also fire fighting activities.

*Tsunamis.* A tsunami (Japanese word meaning harbour wave) consists of a series of long waves caused by a sudden vertical displacement of a large area of the sea floor during an undersea earthquake. In deep water the waves may not be observed. Upon reaching shallow water around islands or the continental shelf the height of the wave increases greatly reaching 30 m and a speed of over *SO* km per hour. The devastating wave front of a tsunami crashes inland, sweeping all away sometimes beaching ships hundreds of meters inland. Successive wave crests, typically arriving 10 to 45 minutes later, may continue to pond the coast for several hours. Several days may pass before the sea returns to normal state.

*Seiches.* In lakes tectonic movements can induce long waves also, with a period of several minutes to few ten minutes. These can spill low level shores or dams causing erosion damage.

## 7.2 IMPORTANCE OF INFRASTRUCTURE FOR DISASTER RESPONSE AND REHABILITATION

As already mentioned, earthquakes cannot be prevented and there are no possibilities to predict short-term occurence with any degree of accuracy. Earthquakes affect large areas by various effects and may produce enormous hiitriati and economic losses. They have therefore a significant effect on development of a country. *Earthquake* ***resilient*** *infrastructure* becomes a prerequisite for an effective disaster response and fast reconstruction activities after an event as for fast economical recovery.

In *developing countries* governmental organisations and industries are usually concentrated to few heavily populated areas. An event in such an area has an enormous effect on such a country. The development of the whole country can be *set* back for years leading also to further social and political problems. Disaster resilient, particularly earthquake resilient infrastructure is an important issue of the overall sustainable development process of such a country, therefore.

## 7.3 VULNERABILITY OF INFRASTRUCTURE

Vulnerability is defined as the degree of loss to a given element at risk resulting from a given hazard at a given severity level (e.g. vulnerability of a 4 stoned office building of masonry type in an earthquake intensity MSK XI). For an infrastructural system one has to distinguish between the system Vulnerability and the vulnerability of each component (service lines, structures, or control systems). Conventional vulnerability assessment concentrates often only on structural vulnerability (damage to the structural system), but the functional vulnerability is at least as important. *Functional vulnerability* usually is higher than structural vulnerability, such tliat functional failure precedes the structural failure. Functional vulnerability often can be reduced with highly cost effective means.

### 7.3.1 Characteristics of infrastructural systems

Every infrastructure-system consists of structures (individual and interconnected structures), equipment, power-supply, control systems, etc. One distinguishes between ***object-oriented*** systems (OS) as hospitals, police- and fire-stations, central food-storage **and** network-oriented systems (NS) as electricity-, gas-, water-, sewer-systems.

The following types of infrastructural systems are particular important **during** disasters:

| | |
|---|---|
| Public Services: | Hospitals (OS)<br>Police-stations (OS)<br>Fire-stations (OS)<br>Central food distribution centres (OS) |
| Water: | Water supply (NS)<br>Sewers (NS) |
| Transportation: | Roads, highways (NS)<br>Railways (NS)<br>Airport (OS)<br>Harbours (NS) |
| Telecommunication: | Surface based telecommunication (NS)<br>Modular telecommunication (OS) |
| Energy supply: | Electricity (NS)<br>Gas (NS)<br>Petrol, Gasoline (OS or NS) |

The characteristics and the individual importance of those systems and its components vary in every country from site to site.

### 7.3.2 Vulnerability of infrastructural systems

In a infrastructural system not every structure, or subsystem has the same importance to maintain the functionality of the system. During a disaster not every public service has to function to the same extent as in normal times, e.g. to maintain the public health system in emergency periods. Not every hospital has the same importance, and equal emergency capacities. The responsible *authorities* of every infrastructural system have do define services to be provided during each type of disaster. This so-called *reduced mode* will vary with the disaster type and its intensity. The system vulnerability has to be evaluated to maintain such reduced modes. To carefully define reduced modes of services is a delicate political problem with ecnomic consequences.

Line-based systems as water and power lines cross wide areas with different geological and topographical conditions involving local and temporal interruptions. Such systems can most effectively be strengthened by introducing some network redundancy. Redundancy also improves the *operational availabilities* in normal times.

The behaviour or object-based systems depends heavily on local site conditions. **A** careful *site selection* with respect to earthquake hazard is important. Avoiding unfavourable site conditions such as loose soil deposits, or high water table reduces the hazard damages. When strengthening is foreseen, the influence on the shaking level and the overall underground behaviour has to h e evaluated carefully. Practical experience demonstrates often lacks in this regard.

### 7.3.3 Vulnerability of infrastructural components

*Structures*

Methods to asses the vulnerability of structures are well established. Experience from past earthquakes demonstrates that structures built according to modem codes, typically later than 1980, face limited damage. Methods also exists to assess the behaviour of underground due to strong shaking (e.g. liquefaction potential). At least some parts of an infrastructure have also to fulfill serviceability criteria.

Mechanical *and electrical components*

Experience for the assessment of vulnerability for ordinary mechanical and electrical equipment under earthquake excitation is not really available.

## 7.4 RISK MITIGATION MEASURES

Mitigation means taking actions to reduce the effects of a hazard before it occurs. The term mitigation applies to a wide range of activities aiming to better assess the hazard and to reduce the vulnerability of systems. These can range from the physical protection, like constructing stronger buildings and strengthening existing structures, introducing redundancy in a system to procedural *improvements* like introducing standard techniques for incorporating hazard assessment in land-use planning, or preparation of disaster response and reconstruction plans.

Building disaster-protection takes time. In urban areas most of the infrastructure is not built to modem codes and quality assurance methods. In rural areas most buildings and part of the infrastructure are non-engineered. To achieve an earthquake resilient infrastructural system, therefore, takes time and requires a continuous effort in improving the system resistance, maintaining or improving the system safety level and providing the necessary funding. This task is difficult for disasters with longer return periods like earthquakes. An entire earthquake resilient infrastructural system is economically nor feasible as also not reasonable. Therefore, a *priority* of real needs is based on:

(1) Careful definition of the required reduced mode of the system,
(2) Evaluation of the importance of all system components to maintain the reduced mode,
(3) Hazard assessment taking into account local conditions and
(4) Vulnerability of the important components.

Mitigration planing should aim to develop a *safety* culture in which all members of the society, from regional to local governmental orgariisations, leaders of industries and services as well the general public are aware of the hazard they face and will support mitigation efforts.

The main principles to achieve an earthquake resilient infrastructure system are:

- Design of *service networks systems* (transportation, water supply and sewer, energy supply or telecommunication) needs careful planning to reduce the systems *failure*. Long supply lines cut at any point. Interconnected networks are less vulnerable to local failures provided that individual sections can be isolated when necessary. Systems with centralised control facility again involve a higher risk than decentraiised systems with several interconnected control centres. In such systems redundancies are a must.

- Careful location of *new facilities*, in particular infrastructural systems as also community facilities like hospitals or schools play an important role in reducing settlement vulnerability in urban areas. Deconcentration of risk elements is important. Microzoning of earthquake hazard is a basic tool for risk mitigation.

- Link between different sectors of economy may be vulnerable to disruption by disaster. *Diversification of the economy* is a way to reduce the **risk** of economy breakdown in the aftermath of an event and thus reducing the capability of a fast recovery. **A** strong economy seems to be the best defence against any type of disaster. Within a strong economy, governments are able to maintain a resistant infrastructure and provide economic incentives to encourage institutions and individuals to take disaster mitigation measures.

The main steps of *earthquake risk mitigation* can be summarised as follows:

- Assess carefully regional and local settlements, economy and social vulnerability. **Assess** also the effects of large events on the economical and social conditions. Set priorities where and to which extent mitigation measures are mandatory.

- Define infrastructural systems important for disaster response and reconstruction.

- Define socially and economically acceptable functionality for every infrastructural system.

- Assess carefully hazard by taking into account regional and local geological and topographical effects as well as secondary hazards mggered by earthquakes.

- Assess infrastructural system vulnerability taking into account structural and functional aspects as well as redundancy and fast repair possibilities.

# 8 FOREST FIRES

M. Conedera

## 8.1 INTRODUCTION

Forests vary significantly with the location. The following refers particularly to Alpine forests with a typical elevation of 500 to 2000 m a.s.l. in the moderate climate zone. In Switzerland most forest fires occur in the Southern **part,** a small **region** of 4'000 km$^2$ (10% **of** the total national area) with a forest cover of **44%** (176'000 **ha).** Other minor fire-sensitive regions **are** the Northern part of the canton of Grison and the canton of Wallis.

## 8.2 FOREST FIRE DATA BASES

In 1992 a forest fire research project was started within the Swiss National Research Program 31 **(NRP** 31) *Climate Change arid Natural Disasters* by the. branch station south of the Alps of the Swiss Federal Institute €or Forest, Snow and Landscape Research (FNP Sottostazione Sud delle Alpi). The NRP 31 project enabled to reconstruct fire data concerning date, time, duration, cause of ignition, area burnt, fire type, forest habitat, and other variables from more than 5500 fire events since 1900 (Conedera et al. 1993). This information has been organised in **a** relational database. **A** similar data base is now in progress for the canton of Wallis (Bochatay and Moulin 1998). The spatial and temporal analysis of wildfire Occurrence has been studied for the canton of Grison through a case study (Langhart et al. 1998).

## 8.3 FIRE HISTROY

Based on the corresponding fire data base the fire history for southern Switzerland in this century has been recreated (Conedera et al. 1996). The significance of these factors was then verified by comparing the results with charcoal concentrations in recent sediments from the lake of Origlio (Tinner et al. 1998). The most notable aspect of fire regime development in this century is the general increase in the occurrence of fires since the sixties with a **marked** rise of summer fires since the seventies (Conedera et al. 1996, Fig. 8.1).

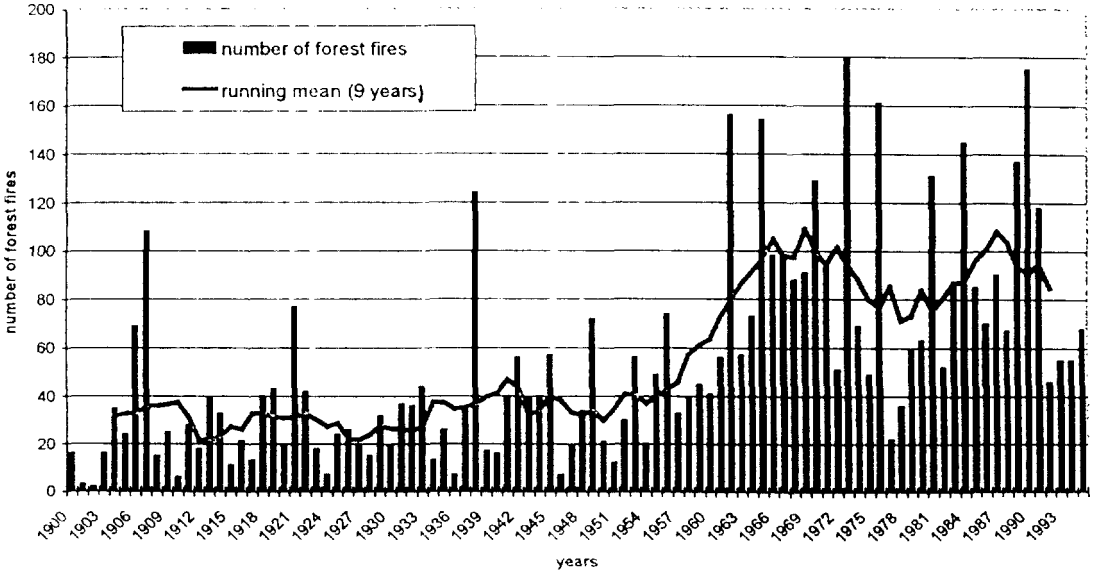


Fig.8.1 Development of number of fires and running mean (9 Years) in Southern Switzerland.

Paleoecological methods were used in order to reconstruct prehistoric forest fires and their possible effects on vegetation (Tinner and Conedera **1995,** Tinner et al. **1998,** Tinner et al. **1999,** Berli et al. **1994).** In southern Switzerland the highest fire frequency occurred in the Holocene during the Bronze and Iron ages due to anthropogenically induced fire (slash and bum practices, Tinner et al. **1999). All** marked peaks in the charcoal curve since the Neolithic correlate with decreases of tree pollen (Tinner and Conedera **1995,** Tinner et al. **1999).**

## 8.4 EFFECTS OF FOREST FIRES

Different fire ecology studies on the effects of forest fires are being carried out. The main issues are:

- Post-fire vegetation reaction (tree, shrub and grass layer),
- Effects on invertebrate diversity,
- Post-fire runoff and soil erosion (splash and sheet erosion),
- Effects on soil water content,
- Effects on soil microorganisms.

**Tab. 8.1:**

| **Institute** | Unit | **Group** leader(s) | **Field of activity** | **Participa tion EU-Projects** |
|---|---|---|---|---|
| **Swiss Federal** Institute for Forest, Snow and Landscape Research | FNP Sottostazion c Sud delle Alpi | Marco Conedera Peter Marxer Marco Moretti | Fire ecology, **Fire** management | Minerve **II;** Promethe us s.v. |
| | ecological processes | Peter Blaser | Effects on soil | |
| | Biodiversity | Peter Duelli | Effects on invertebrates | |
| | landscape dynamics and inanagemen t | F. Schweingruber | Dendroecolo gy | |
| | avalanche dynamics | Perry Bartelt | Modelling | Inflame |
| ETH Zurich | D-WAHO | Daniel Mandallaz | **Risk** prediction | Minerve II |
| University of Berne | Geobotanic Institute | Brigitta Ammann Willy Tinner | Paiaeohistory | |
| | Department of Geography | Helmut Elsenbeer | Effects on soil | |
| University of **Zurich** | Department of Gcoyraptiy | Britta Allgöwer Andreas Bachmann | Modelling **Fire** Management | Minerve **II;** Inflame |
| University of Lausanne | Institute of Botany | Pierre Hainard | Effects on vegetation | |
| University of **Basel** | Department of Geography | Helmut Leser C. Wüthrich | Effects on soil | |

Although these studies are going on, first results for the chestnut forests in southern Switzerland are available:

- Repetition of fires leads to an impoverishment of the vegetation towards fire-tolerant species (Delarze et al. 1992, Hofmann et al. 1998, Berli 1996),
- This development is not only dependent on the original floristic composition, but also on the survival strategies of the different species (Hofmann et al. 1998),
- Long-term repeated fires lead to a reduction of the nutrient level (Marxer et **al.** 1998, Delarze et al. 1992, Hofmann et al. 1998),
- Forest fires increase soil erosion, runoff and **risk** of debris flow. The magnitude of this effect seems to be a function of fire severity (Marxer et al. 1998),
- In burned areas the richness of species of many faunistic groups (spiders, carabids, ants) is higher than in unburned areas (Moretti et al. 1998),
- Distribution of the species' abundance of the burned areas reflect typically an unstable (disturbed) and dynamic ecosystem (Moretti et al. 1998),
- Different fire regimes (fire frequency and time elapsed since the last fire) have clear effects on faunistic diversity (Moretti et al. 1998).

## *8.5* FIRE RISK PREDICTION

The general increase in the occurrence of forest fires since the sixties (Conedera et al. *1996)* makes it increasingly necessary to improve the *forest fire risk prediction methods.* Different fire **risk** prediction approaches are operational or in development in Switzerland:

- Statistical model based on the Poisson distribution (Mandallaz and Ye 1997),
- Hybrid expert system for the spatial prediction of wildfire danger (Bolognesi 1996),
- GIS-based framework for wildfire risk asiessment (Schoning et al. 1997).

Due to the anthropogenic origin of most fires, factors describing human activities (i. e. weekends or holidays) had to be integrated in fire **risk** forecasts (Mandallaz and Ye 1997).

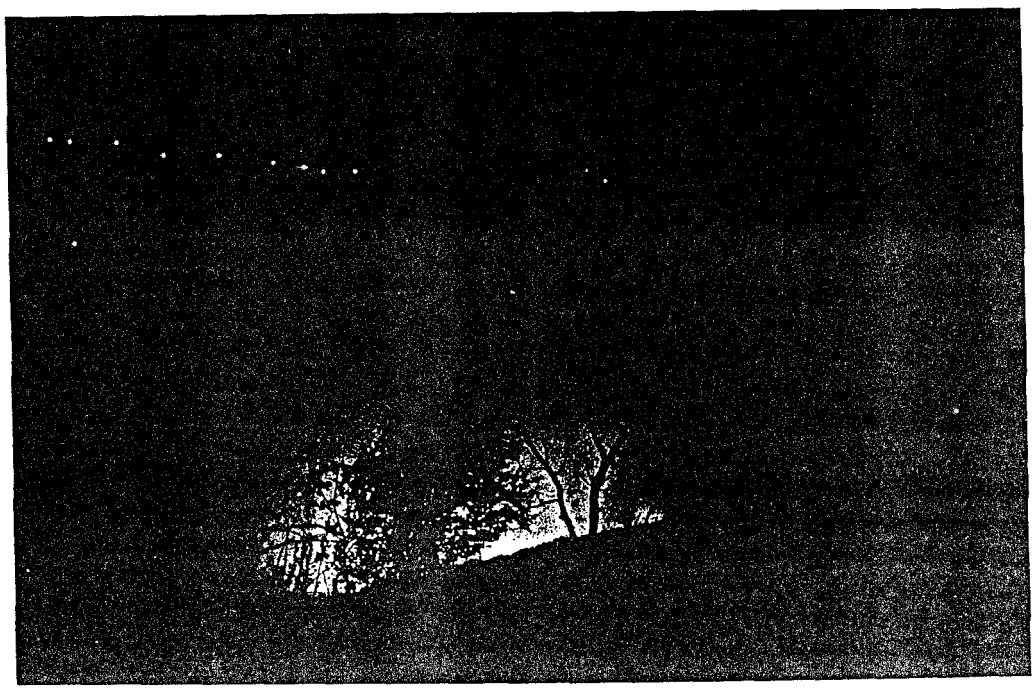

Fig.8.2 Forest lire in Sta. Maria (Misox) in **April** 1997.

## 8.6 FIRE BEHAVIOUR MODELLING

Fire behaviour modelling may be approached with two different scales: Landscape scale (Schoning et al. 1997) and Fuel bed scale (burn table, Bartelt 1997). The basis for the fire behaviour modelling on the *landscape level* is the Rothermel model for the behaviour of surface fires. For any given point it calculates local intensity and spread parameters for the head of a surface fire. The fire spread model is implemented in **SPARKS,** a prototype fire behaviour modelling application. It is fully integrated in a commercial Geographical Information System (ARC/INFO), built on its raster modelling and applications development functionalities (Schoning et al. 1997). In order to provide input data for the fire behaviour modelling, fuel models were built for different forest types in Switzerland (Allgower et al. 1998).

The *small scale approach* proposes to investigate the thermodynamical properties of the forest fuel bed as well as the mechanics of forest fire spread by field observations, laboratory experiments and numerical modelling in combination. Fire intensity and fire spread velocity are studied in laboratory experiments (burn table) and numerical modelling (Bartelt 1997).

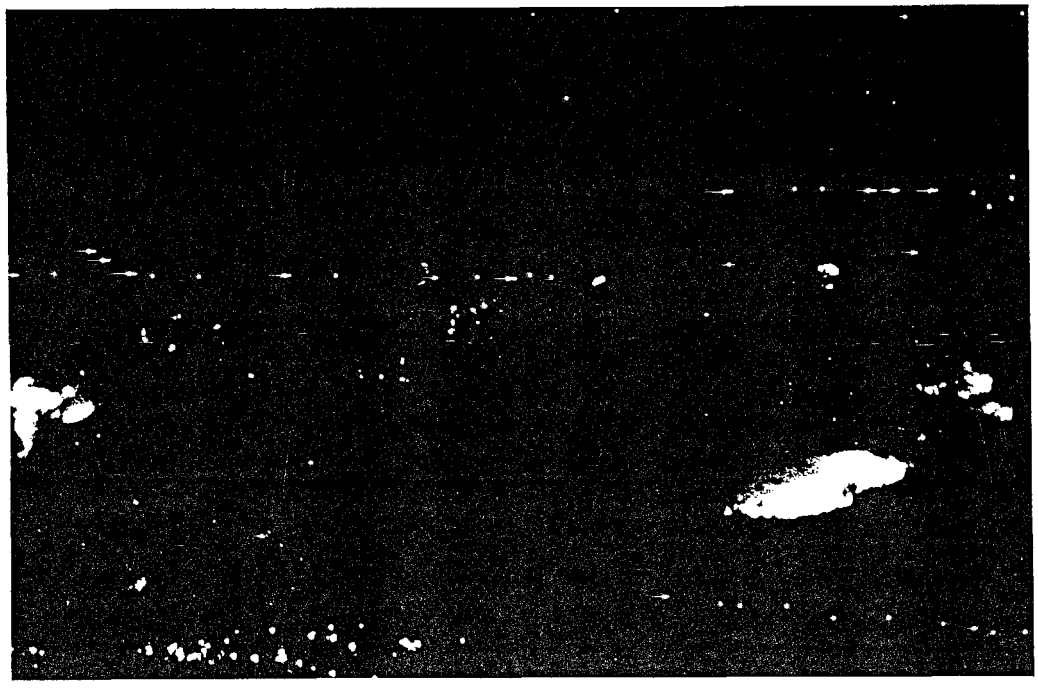

Fig.8.3 Night view of the burning slope above Mezzivico (Tessin) in April 1997.

## 8.7 FIRE MANAGEMENT

Although forest fires in the Alps seldom became a threat to life and property of local residents and tourists, some problems originate from forest fires in connection with the protection function of the forest, soil conservation or economical aspects of the timber industry. Therefore it is important to implement the acquired knowledge on forest fires in *decision support systems* and computer based management tools.

Different studies were already carried out on this topic: GIS-analysis for wildfire management planning in the Swiss National Park, Internet applications in the context of wildfire management, a GIS-based framework for wildfire risk assessment (Bachmann et al.

1997, Bärtsch et al. 1998) and a study on the integration of fire effects on vegetation in fire management strategies (Fürst and Conedera 1996).

Fig.8.4 Fire line of the 1998 organized fire experiment by the FNP Sottostazione **Sud** delle Alpi in St. Antonino (Ticino).

## **8.8** CONCLUSIONS

With these activities the Swiss research groups participated in European projects like MINERVE 2, INFLAME and PROMETHEUS s.v. since 1994 (Tab.8.1). The ongoing studies on prediction, modelling, ecology and effects of forest fires allow to obtain decisive instruments for supporting the responsible fire management authorities and fire brigades in order to aim at a more differentiated fire management strategy for Switzerland.

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# 9 FLOOD PROTECTION AND INFRASTRUCTURAL DEFENSE

D.L. Vischer

Three domains of infrastructure defense against floods may be distinguished: (1) Dams and nuclear power plants, with a federal protection level, (2) Plants of water and energy supply with directions and recommendations involving authorities and private associations, and **(3)** Remaining infrastructure for which no guidelines are actually available, and for which infrastructure defense is a private concern. Currently, this state is particularly discussed though not realized.

## 9.1 DAMS AND NUCLEAR POWER PLANTS

For dams and nuclear power plants, the federal authorities are in. charge of security aspects. This is due to the national and even international significance of such infrastructure. Therefore, their damage is not the prime concern but the prevention of particularly long reaching consequences.

**Up** till today, Switzerland has not been *hit* by dam failures. The fact, *that such disasters* have occured internationally rind that roughly one third may be demonstrated to result from overtopping is noteworthy. As long as dams are operational, floods can be significantly stored or at least dampened. If full dams fail, however, the resulting flood is much larger than when there would have been no storage, and has an enormous potential of destruction. The Swiss regulations aim to exclude any dem failure, therefore. This concept is based on three bases, including (1) structural safety, (2) control and (3) emergency concept. in the following, these three items are shortly reviewed.

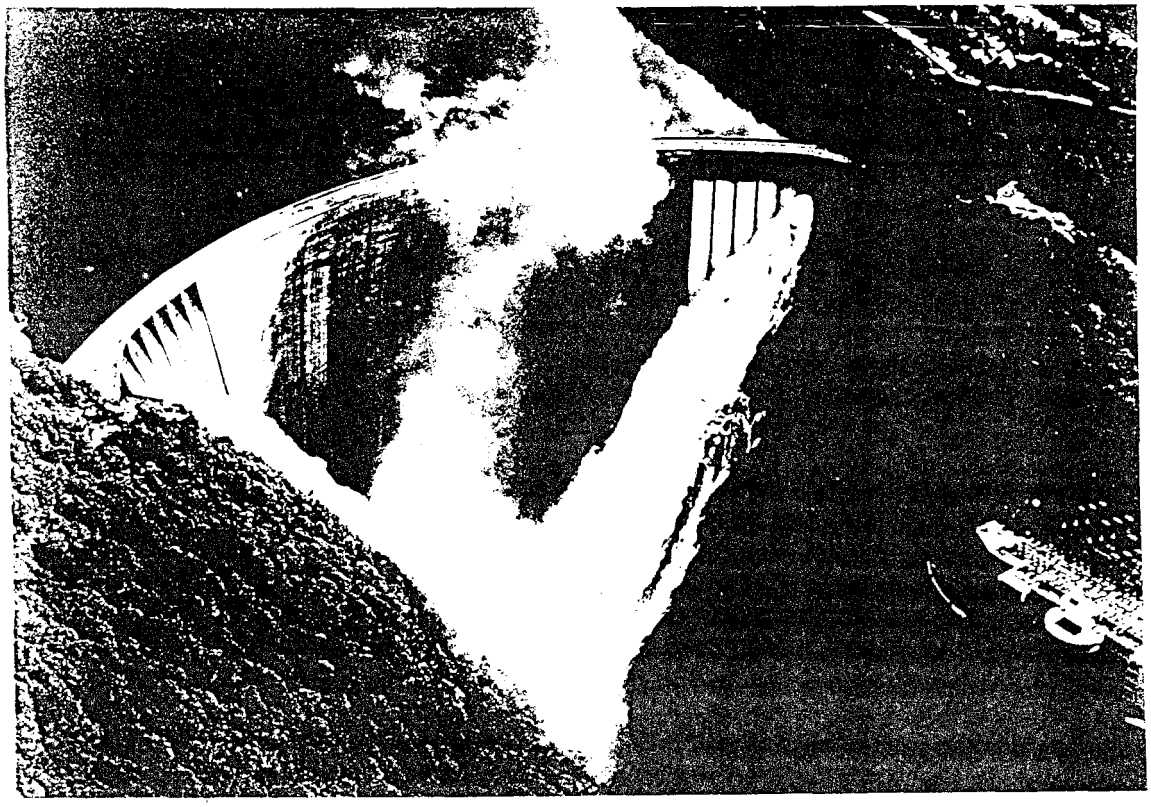

Fig.9.1 Spillway overflow at Contra arch dam close to Locarno, Switzerland, 221m height, into service since 1966.

*Structural safety* is considered the most important feature of dams including their design and execution. Dams have to resist of course the enormous water pressure, particularly during floods. In addition they include a spillway to exclude overtopping (Fig.9.1). The hydraulic design of spillways includes thus two design discharges, namely the one thousand year flood, and the catastrophic discharge.

The *one thousand year flood* should be diverted without any damage to the dam. The following assumptions are thereby considered: (1) This flood enters a full reservoir, with ail intakes closed (particularly important for Switzerland with an general use of water power), (2) The reservoir freeboard relative to wave action and other imponderables are guaranteed. For dams equipped with gates, the (n-1) condition has also to be satisfied, invoking that the gate with the largest discharge capacity is closed during the flood event.

The *catastrophic discharge* is defined as being equal to the one thousand years discharge times a factor of 1.5 applied to both flood peak and basis, and corresponds roughly to the ten thousand years discharge. This flood should pass the spillway without initiating dam failure but causing some minor damages. The freeboard can partly be used and the (n–1) condition applies only to embankment dams.

*Control* as the second basis of the security concept guarantees that a dam with itsconsiderable longevity does not temporally change. This may mean that gated spillways remain always fully operational, or that temporal changes in hydrology are accounted for by a corresponding design flood. The latter may cause an increase of the spillway facilities.

The *emergency concept* as the third basis includes emergency drawdown of the reservoir level to inhibit dam failure or warning of downstream zones if this is impossible. In regions where the flood would arrive within 1.5 hours after a dam break, the warning is made with specially installed water emergency sirens.

For nuclear power plants, flood protection is basically required though not detailed up till today. There are currently no standards available as for dams, and a local procedure is accounted for. Switzerland with only four nuclear power plants has no real need as compared to about 200 dams under federal authority. Such dams are characterized by a height larger than 10 m, a reseroir volume larger than 50.000 $m^3$ or a specific potential of damage. Cantonal authorities are in duty for the lots of smaller dams.

## 9.2 PIPELINE-BOUND SUPPLY AND DISPOSAL PLANTS, TRANSPORTATION SCHEMES

The pipeline systems include the water and gas supply, the electricity supply and the sewage disposal schemes. Given their locally fixed installation, their damage by natural disasters is particularly large.

Drinking and industrial *water supply* as used in Switzerland originate by 80% from the groundwater reservoirs and by 20% from lakes. There is a federal law dating from 1991 and relating to the securing of drinking water in emergency situations. It contains minimum criteria regarding quantity and quality of supply waters and also regulates the responsibilities. Details are left over to owners and operators which are members of the Swiss Gas and Water Association SGWA. Accordingly, a SGWA-regulation indicates the design and operation of drinking water emergency supplies, with only a general account to floodings, however.

Spring and lake water intakes arc flood-proof in general, whereas groundwater fountains are often not if they are located in flood plains and thus suffer from floodings. Contaminant flood water can enter a fountain and thiis contaminate a complete supply system. Groundwater fountains are thus elevated to above the flood level, typically on a small artificial hill to inhibit damages.

Earthborne water supply systems are not really in danger during floods, except damages due to scour or erosion. During the Brig disaster in 1993, the supply pipelines were damaged and filled with solid matter, which eventually reached the filters upstream of the buildings and caused clogging. The drinking water supply was practically out of operation and the reoperation was significantly retarded. The current guidelines thus recommend to place filters sufficiently high that sedimentation is impossible and cleaning remains simple.

*Sewers* are typically based on the combined sewer system practice, with stormwater and sewage in the same pipeline. Separation into the treatment facilities and the receiving waters occurs at so-called stormwater overflow structures. The most important guidelines are included in the 1991 federal law on water protection, with details dealt with by the Swiss Water Pollution Association VSA.

**A** sewer with stormwater floods can cause additional floodings, an effect not further elaborated here. The impact of natural floods on sewer systems is of relevance, however. *Stormwater overflows* normally discharging sewage into the receiving waters can become intake structures to a sewer system during high flood levels. Such undesirable events can be countered by clieck valves with some risk for clogging in both **flow** directions. They are therefore not generally accepted. The flood level can even reach lower building iniakes which must be located at a certain minimum elvation, according to VSA-regulations.

Out-of-the-river floodings may cause damage to sewers via submergence and intake by manholes. Examples such as those of Poschiavo in 1987, and Brig in 1993 have led to highly undesirable conditions with a complete *clogging* by mainly sand and gravel. Only after a fortnight, the sewers of Brig for example were flushed with high-pressure systems. It seems that this potential o fdamage cannot really be countered by infrastructural protection.

Outleis of sewage treatment facilities into a receiving water car? also be influenced by floodings, arid the outlet elevation is normally located at the 100 years flood level. Therefore, the sewage is often elevated with screw *pumps* to a sufficiently high level. These pumps are located either at the facility inlet, or between various basins of the plant, or even at the facility outlet. If the treatment facility is still subject to floods, the control installations are at least located at flood-pioof elevation.

The Swiss *gas supply* is based on imported gas. Therefore, this country has a distribution network but no gas production plants. The distribution network is not really sensitive to floods, although there might be a damage potential for pipelines located close to rivers and brooks. An example is the pipeline running along Rhone river which might be scoured by out-of-the-river floods. Then, explosions can occur. At the time of design, it was necessary to demonstrate that the Rhorie river is actually stable in terms of thalweg geometry, and river migration could be excluded. An additional scour arid erosion protection was only exceptionally applied. The control of such pipelines is under the Swiss Federal pipeline inspectorate and details are regulated by SGWA.

The Swiss *electric supply* involves roughly 60% hydropower and 40% nuclear power. The flood-proofing of its infrastructure was already discussed. One may add that practically all dams are related to hydropower and only few are used for **flood** storage. Flood security of river powerplants with a large potential of damage follow similar regulations as dams. The overground powerhouses are usually in the flood regions and must be made floodproof, therefore. The access road and the powerhouse entrance must be elevated at least above the 100 years flood level. For underground powerhouses, flooding danger is low and safety may easily be obtained by a sufficiently high entrance elevation. There are actually no guidelines available. The coordinator of such questions is the Swiss Electricity Supply Association.

The gross distribution of electricity involves high tension networks, whose poles are normally flood-proof. Breakdowns are thus seldom and are not really known for the Swiss electric supply network. The large switchyards are normally erected sufficiently above **flood** elevation, whereas smaller works may be located in flood risk zones. **As** was demonstrated by

recent incidents the corresponding subworks, transformation stations and distribution cabins are often not sufficiently safe against floods. Cable lines may enhance flooding of electricity stations. Due to an old and unfortunate tradition, the measuring, dismbution, regulation **and** safety installations of the buildings are generally installed at the basement where they. may immediately break down.

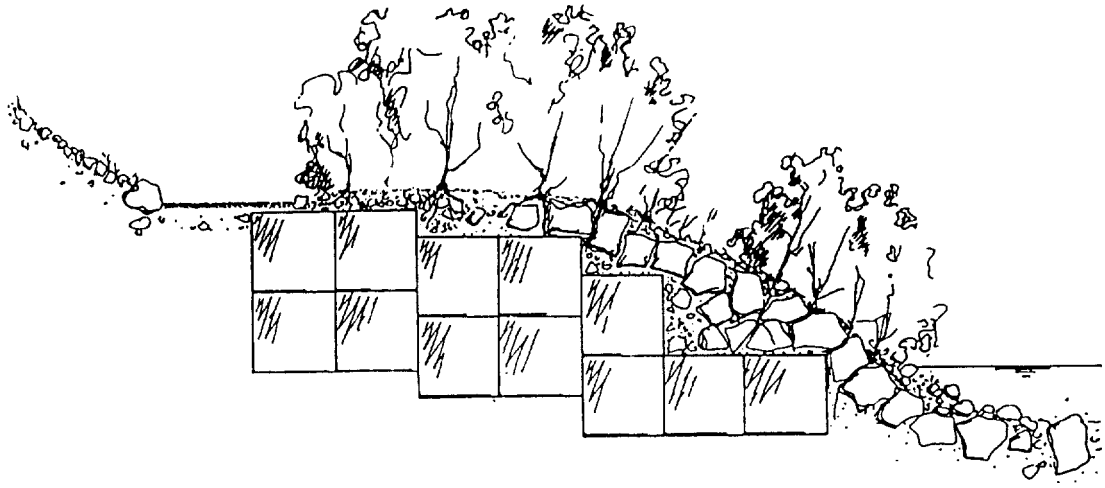

**Fig.Y. 2** Shore protection at Reuss river close to Gotthard **highway** tunnel, with concrete groins of 66 t weight.

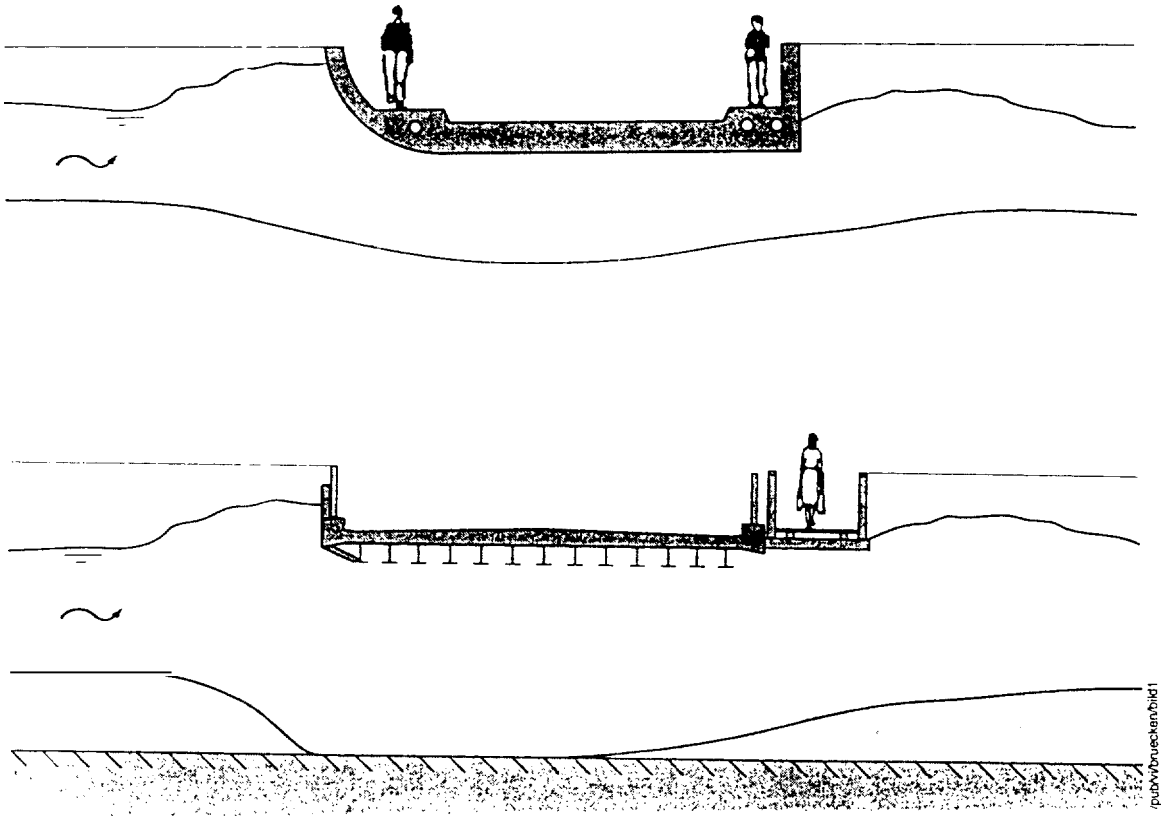

**Fig.9.3** Shield backup bridge, schematic streamwise section, design example of Cimavilla bridge at Poschiavo, Switzerland.

The Swiss *telephone network* is laid practically always underground and seems to be less prone to flood damages as coinpared to the electrical network. Handies increase of course the communication safety during flood flows.

***Traffic infrastructure*** related to flood safety has a widely varying degree of protection. Railroads are particularly safe in this country and critical locations are often crossed with embankments. When running along rivers, the railroads have a special protection against bank erosion. Bridges are so high as to inhibit any overtopping or side flows. Along steep slopes, notorious brooks are often bypassed over galleries. The same methods are also used with highways and other strategic roads (Fig.9.2). In contrast, few works have been erected for normal highways, and damages are particularly numerous with road culverts and **small** bridge openings. These latter may cause significant backwater with corresponding floodings upstream or result in bridge clogging by sediment and float.

Some disasterous river overtoppings were initiated by bridges substructures too close to the water level. Two recent designs were introduced for bridges where their elevation cannot be increased, namely a so-called culvert bridge and the lift bridge. The ***culvert bridge*** (Fig.9.3) inhibits the bridge flooding by forcing water and sediment through the opening, whereas the ***lift bridge*** (Fig.9.4) just releases the water flow. Two lift bridges have been built in **Brig,** one for a public road, the other for the railroad.

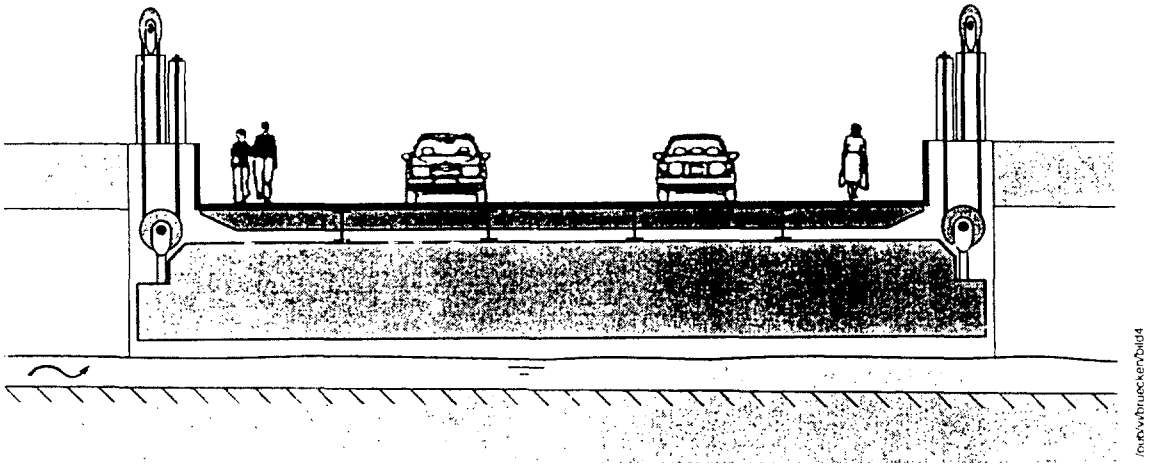

**Fig.9.4** Elevation bridge at Brig, Switzerland. The roadway rests on two abutments that may be elevated by 2.8 m.

Few probleiiis arise with the many suspension railways because their poles can easily be designed flood-proof.

## 9.3 BUILDINGS

Buildings in the Alps are normally not flood-proof, as was already mentioned. Given that all the buildings are insured, and insurances do not apply different risk classes, no particular stimulus is created for building protection. Therefore, even new buildings can be found in out-of-the-river floodplains with garages and other installations in the basement. **A** 1993 federal law is in order that demands a *mapping* of natural hazard zones. Thus, rules and orders are supposed to follow in terms of building protection. The corresponding guidelines are still missing, but may include:

- Hood-proof design of buildings typically on small earth pourings (Fig.9.5), on piers or behind local embankments.
- Omission of basements or addition of seals for entrances, light and air supplies.
- Structural means against erosion and water pressure, including buoyancy.
- Improvement of flood resistance of water anti power supply to the buildings considered.

The realization of structural means seems rather simple, given the general stability of Alpine buildings.

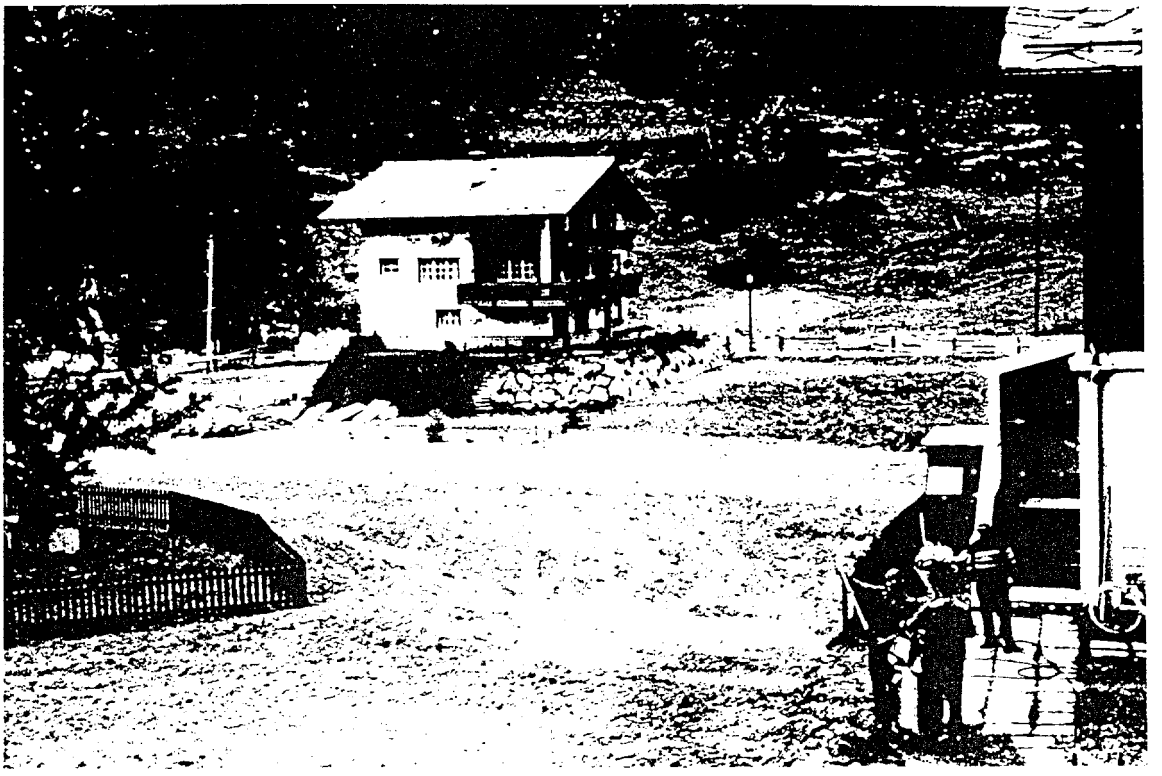

Fig.9.5 Building at Saas Balen, Switzerland, in the flood reach of Saaser Vispa. The earth fill renders it practically floodproof.

It can also be mentioned that Alpine buildings are generally heated with oil, and an oil tank is normally located close to the cellar that may be damaged by floods. It can thus buckle or even swim up. The latter case is particularly dangerous because it leads to a failure of the connecting pipes, and the leaking of oil contaminates the groundwater. Therefore, water authorities require also safety measures against those incidents.