The design of avalanche protection dams. Recent practical and theoretical developments

1-DRAFT

some sections still remain to be written,
the document shows the intended section structure and
contains draft introductory sections, sections about
the design of deflecting and catching dams, and braking mounds,
sections about impact pressures on walls, masts and
other narrow constructions, appendixes about overrun of
avalanches at Ryggfonn, loading of obstacles and a subsection about
Iceland in an Appendix about laws and regulations

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Photographs on the front page:

Top left: Mounds and catching dam in Neskaupstaður, eastern Iceland, photograph: Tómas Jóhannesson. Top right: A deflecting dam at Gudvangen, near Voss in western Norway, after a successful deflection of an avalanche, photograph: ???. Bottom left: A catching dam, deflecting dam, and concrete wedges at Taconnaz, near Chamonix, France, photograph Christopher J. Keylock. Bottom right: A catching dam and a wedge at Galtür in the Paznaun valley, Tirol, Austria, photograph: ???. 
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1 Introduction

Protection measures against snow- and landslides are widely used to improve the safety of settlements in avalanche prone areas. Measures to manage snow- and landslide danger and protect existing settlements include:

**Land use planing:** With proper hazard zoning and long term planning of building activity this is undoubtedly the safest and most cost-effective way to manage danger due to snow- and landslides. It does, however, not solve problems associated with settlements that have already been located in hazard areas.

**Evacuations:** Moving people from threatened areas can be either permanently, usually with some assistance from the government or local authorities to relocate settlements, or temporarily, by evacuating people from their homes and work places during avalanche cycles. Evacuations are usually not considered a viable long-term solution for ensuring the safety of settlements.

**Supporting structures:** Supporting structures in the starting zones of avalanches are the most widely used protection measures in the Alps and have also been used to a lesser extent in many other countries. There is firm evidence that properly designed supporting structures reduce the avalanche hazard substantially, in particular the experience gained during the harsh avalanche winter of 1999 in the Alps.

**Deflecting dams:** If there is sufficient space in the run-out zone and if the endangered area is suitably located with respect to the direction of the avalanches, deflecting dams may be used to divert avalanches away from objects at risk. Deflecting dams are often a cost-effective solution and several examples of successful deflections of medium sized avalanches have been documented.

**Catching dams:** Catching dams are intended to stop avalanches completely before they reach objects at risk. They are typically used for extended areas along the foot of the slope where there is insufficient space for deflecting dams. Large avalanches flowing at high speed can hardly be stopped by catching dams and there are many examples of avalanches overtopping such dams. The effectiveness of catching dams is therefore dependent upon a location near the end of the run-out zone of the avalanches.

**Wedges for the protection of single buildings:** Single buildings may be protected by short deflecting constructions that are either built a short distance away from the building or constructed as a part of the building. Such wedges are widely used and have proven to be an effective protection method against avalanches.

**Braking mounds:** Braking mounds are used to retard avalanches by breaking up the flow and causing increased dissipation of kinetic energy. There is not much observation evidence for the effectiveness of braking mounds from natural avalanches, but laboratory
experiments with granular materials indicate that they can reduce the speed and run-out distance of avalanches.

**Reinforcement of buildings:** Specially designed buildings to withstand the impact pressures of avalanches can increase the safety of the inhabitants considerably. Because of the very high impact pressure of snow avalanches, such buildings must either be built into the slope, so that the avalanches overflow them, or constructed near the end of the run-out zone, where the speed of the velocity has been reduced to lower levels than higher up in the path.

**Measures to reduce snow accumulation in starting areas:** Snow fences in catchment areas for snow drift may be used to collect snow that would otherwise be carried into an adjacent starting zone, thereby decreasing the volume and thus the run-out of the avalanches.

This report is about the design of dams and other protection measures in the *run-out zones of wet- and dry-snow avalanches*. It summarises recent theoretical developments and results of field and laboratory studies and combines them with traditional design guidelines and principles to formulate design recommendations. Hazard zoning, land use planning, evacuations, supporting structures in the starting zones of avalanches, snow fences in catchment areas, and other safety measure outside the run-out zone are not dealt with. Reinforcement of individual buildings also falls outside the scope of the report and so do protection measures against landslides and slushflows.

The report starts with section 2 about the communication between avalanche experts, on one hand, and local authorities and the public, on the other, during the design of avalanche protection measures. This is an important aspect of the preparations of protection measures, where decision makes and the public have a chance to come forward with their views on the problem and are informed about possible alternatives, rest risk, hazard zoning after measures have been implemented and other key concepts. The next two sections, § 3 and § 4, are an overview of traditional design principles for avalanche dams and a summary of avalanche dynamics with an emphasis on the interaction of avalanches with obstacles. Design of deflecting dams, catching dams and breaking mounds is treated in the next four sections, § 5 to § 8. They are followed by a section about dams as protection measures against powder snow avalanches, § 9, and sections about impact loads on walls and on masts and mast-like obstacles, § 10 and § 11, and about static snow loads, § 12. Numerical modeling of snow avalanches with special regard to modeling of flow over or around dams and obstacles is treated in section 13, and geotechnical aspects of dam design in section 14. The report is concluded with Appendix A, with definitions of the variables that are used in the document, Appendix B, with examples of the design of deflecting and catching dams, Appendix C, about combined protection measures, where, for example, braking mounds and a catching dam are constructed in the same run-out area, Appendix D, with geotechnical examples, Appendix E, with a description of overrun of avalanches over the catching dam at Ryggfonn in Norway, Appendix F where of existing
Swiss and Norwegian recommendations about loads on structures are summarised, and Appendix G, with a summary of laws and regulations about avalanche protection measures and hazard zoning in connection with such measures in several European countries.

The design of protection measures in the run-out zones of avalanches needs to be based on an understanding of the dynamics of granular flows against obstructions that lead to a change in the flow direction, slow the flow down or cause it to stop. In spite of some advances in the understanding of the dynamics of avalanches against obstacles in recent years, there remains a substantial uncertainty regarding the effectiveness of deflecting dams, catching dams, breaking mounds, wedges and other defence structures in run-out zones. In particular, analyses of run-up on man-made dams, on one hand, (see section 5.10) and overrun over the catching dam at Ryggfonn in western Norway (see section 7.2 and Appendix E), on the other, give quite misleading indications about the effectiveness of dams to bring avalanches to a halt or shorten their run-out. This uncertainty about the effectiveness of dams must be borne in mind in all planning of protection measures in run-out zones.

One of the most important and difficult steps in the design of dams and other protection measures in the run-out zones of snow avalanches is traditionally the definition of an appropriate design avalanche. This is intimately linked with hazard zoning, which in most cases is the background for the decision to implement protection measures in the first place. This report will only indirectly touch upon hazard zoning and the choice of a design avalanche, assuming in most places that the velocity, flow depth and other relevant properties of the oncoming avalanche under consideration have already been decided. However, some properties of the design avalanche will be considered where appropriate. The section about deflecting dams, for example, contains practical considerations regarding the choice of the deflecting angle for deflecting dams. Hazard zoning below avalanche dams is also briefly discussed in the respective sections.

The report is written as a part of the research project SATSIE, with support from the European Commission, and partly based on results of theoretical analysis, field measurements and laboratory experiments that have been carried out within that project and its predecessor CADZIE, which was also supported by the European Commission.
2 Consultation with local authorities and decision makers

This section will be written by KL with additional input from all.

The section needs to emphasise the importance of close collaboration with local authorities and decision makers about possible alternatives in the choice of protection measures, rest risk, hazard zoning after measures have been implemented, etc.
3 Overview of traditional design principles for avalanche dams

Authors...

Several methods have been used to design avalanche dams, based either on simple point mass considerations pioneered by Voellmy (1955b) and widely used in Alpine countries (Salm and others, 1990a), a description of the dynamics of the leading edge of the avalanche (Chu and others, 1995), or on numerical computations of the trajectory of a point mass on the upstream facing sloping side of the dam (Irgens and others, 1998). Traditional design methods for avalanche dams are described by Salm and others (1990a), Margreth (2004), Norem (1994) and Lied and Kristersen (2003).

The height of avalanche dams, $H_D$, is usually determined from the formula

$$H_D = h_u + h_s + h_f,$$

where $h_u$ is the required height due to the kinetic energy or the velocity of the avalanche, $h_s$ is the thickness of snow and previous avalanche deposits on the ground on the upstream side of the dam before the avalanche falls, and $h_f$ is the thickness of the flowing dense core of the avalanche (Fig. 1). The terms $h_s$ and $h_f$ in the equation for $H_D$ are typically assumed to be a few to several metres each for unconfined slopes and must be estimated based on a knowledge of the snow accumulation conditions and the frequency of avalanches at the location of the dam.

Figure 1: Schematic figure of a catching dam showing the contributions of the velocity of the avalanche, $h_u$, the thickness of snow and previous avalanche deposits on the ground, $h_s$, and the thickness of the flowing dense core, $h_f$, to the dam height, $H_D$. The figure is adapted from Margreth (2004).
The term $h_u$ is usually computed according to the equation

$$h_u = \frac{u^2}{2g\lambda}$$

for catching dams. $u$ is the velocity of the chosen design avalanche at the location of the dam, $\lambda$ is an empirical parameter and $g = 9.8 \text{ ms}^{-1}$ is the acceleration of gravity. The empirical parameter $\lambda$ is intended to reflect the effect of momentum loss when the avalanche hits the dam and the effect of the friction of the avalanche against the upstream side of the dam during run-up. The value of $\lambda$ for catching dams is usually chosen to be between 1 and 2 (and sometimes even higher), with the higher values used for dams with steep upstream faces. Higher values of $\lambda$ (lower dams) are chosen where the potential for large avalanches is considered rather small, whereas lower values of $\lambda$ (higher dams) are chosen for avalanche paths where extreme avalanche with a large volume may be released.

In addition to the requirements expressed by equations (1) and (2), the storage capacity above a catching dam must be large enough to hold the assumed volume of the design avalanche. The storage capacity depends on slope of the terrain above the dam and assumptions about the inclination of avalanche deposits, which have piled up above the dam, and the relative compaction of the snow from the density at release to the deposit density. The inclination of the avalanche deposits is sometimes assumed to be in the range 5–10˚ for slow, moist, dense avalanches, but the storage capacity can be much smaller than corresponding to this for dry, fast flowing avalanches (Margreth, 2004). A value of about 1.5 for the compaction factor from release to deposition density sometimes used [ref?], but this factor is often not used, which is equivalent to adopting a compaction factor of unity.

The height of deflecting dams is traditionally calculated using Equation (1), as for a catching dam, with the term $h_u$ determined according to the equation

$$h_u = \frac{(u \sin \phi)^2}{2\lambda g},$$

where $u$, $\phi$ and $g$ have the same meaning as before and $\phi$ is the deflecting angle of the dam. The terms $h_s$ and $h_f$ are determined in the same manner as for catching dams. The $\lambda$ parameter for deflecting dams is often chosen to be 1. The choice of $\lambda$ equal to 1 is equivalent to neglecting momentum loss when the avalanche hits the dam and the effect of friction of the avalanche against the dam. This leads to higher dams compared with the choice of $\lambda$ higher than 1. This may partly be considered as a safety measure to counteract the uncertainty which is always present in the determination of the deflecting angle and for taking into account internal pressure forces which may lead to higher run-up than assumed in the point mass dynamics.

There exist no accepted design guidelines for braking mounds for retarding snow avalanches although they are widely used as a part of avalanche protection measures (see Fig. 1). Laboratory experiments have been performed in recent years in order to shed light on the dynamics of avalanche flow over and around braking mounds and catching dams and to estimate the retarding effect of the mounds. The experiments and the design criteria for braking
mounds that have been developed on the basis of them are described by Hákonardóttir (2000), Hákonardóttir (2004), Hákonardóttir and others (2003b) and Jóhannesson and Hákonardóttir (2003) and in papers and reports referenced therein, and they form the basis for the treatment of braking mounds in § 8 of this report.

A fundamental problem with the point mass view of the impact of an avalanche with a deflecting dam is caused by the transverse width of the avalanche, which is ignored in the point mass description. As a consequence of this simplification, the lateral interaction of different parts of the avalanche is ignored. Point mass trajectories corresponding to different lateral parts of an avalanche that is deflected by a deflecting dam must intersect as already deflected material on its way down the dam side meets with material heading towards the dam farther downstream. Similarly, it is clearly not realistic to consider the flow of snow in the interior of an avalanche that hits a catching dam without taking into account the snow near the front that has already been stopped by the dam. The effect of this interaction on the run-up cannot be studied based on point mass considerations and a more complete physical description of lateral and longitudinal interaction within the avalanche body during impact with an obstacle must be developed. These flaws of the point mass dynamics are most clearly seen by the fact that no objective method based on dynamic considerations can be used to determine the empirical parameter \( \lambda \) in equations (2) and (3), which nevertheless has a large effect on the design of both catching and deflecting dams.
4 Avalanche dynamics

Authors . . .

Dry, natural snow avalanches are believed to consist of a dense core with a fluidised (saltation) layer on top surrounded by a powder cloud (Fig. 2). The traditional design criteria for catching and deflecting dams (Eqs. (1), (2) and (3)) are based on viewing the avalanche as a point mass as mentioned in the previous section. The important effects of lateral and longitudinal interactions within the avalanche body for run-up on dams cannot be studied from this point of view. The simplest description of a snow avalanche where these interactions are represented is based on a depth-averaged formulation of the dynamic equations for the flow of a thin layer of granular material down inclined terrain. This description is intended to represent the dynamics of the dense core, but the saltation and powder components of the avalanche are neglected.

In the depth-averaged formulation, the dense core is modeled as a shallow, free-surface, granular gravity current (cf. Eglit, 1983), which can be described by a thickness \( h \) and depth-averaged velocity \( u \). The dynamics of shallow, free-surface gravity flows are characterised by the Froude number

\[
Fr = \frac{u}{\sqrt{\cos \psi gh}},
\]

where \( \psi \) is the slope of the terrain and \( g \) is the acceleration of gravity. The flow depth and velocity are here defined in the directions approximately normal and parallel with the terrain,

Figure 2: Schematic figure of a dry-snow avalanche showing the dense core, the fluidised (saltation) layer and the powder cloud. The depth averaged quantities \( u_1 \) and \( h_1 \) apply to the dense core in the sloping \( x,z \)-coordinate system. The figure is adapted from Issler (2003).
respectively, as indicated with the x- and z-axes on Figure 2. The Froude number of the dense core of natural, dry-snow avalanches is in the approximate range 5–10 Issler (2003), which implies that such avalanches are well within the supercritical range defined by \( Fr > 1 \).

Sharp gradients in the flow depth and velocity are now believed to be an important aspect of the dynamics of the dense core during interactions with obstacles (Hákonardóttir and Hogg, 2005; Gray and others, 2003). Such gradients are represented in the depth-averaged description as mathematical discontinuities in the flow depth and velocity across so-called shocks, which are formed upstream of the obstacle. The discontinuities are of course mathematical abstractions but they are believed to reflect real physical aspects of shallow, supercritical flow. “Information” about obstructions can only propagate a short distance upstream in supercritical flows and sharp gradients in flow depth and velocity occur in the transition between the undisturbed flow and flow that is affected by the obstacle. In the following, the subscripts “1” and “2” will be used to denote quantities upstream and downstream of shocks, respectively. Undisturbed flow in the absence of obstacles will thus be denoted by the subscript “1” as in Figure 2. Shock dynamics will be used in the following sections as an important, but until recently ignored, aspect of snow avalanche dynamics to formulate design criteria for avalanche dams.

The depth-averaged formulation cannot represent some processes that may be important in the flow of snow avalanches against obstacles. Among such processes are splashing during the initial impact (see Hákonardóttir and Hogg, 2005), overflow of the saltation and powder components, and the transfer of snow from the dense core into suspension during the impact. Processes related to two-phase dynamics and air pressure in the interstitial air in the avalanche that may cause “hydroplaning”, may also be important during overflow, as well as shearing flow over the dam, where a thick avalanche overflows a dam over a part of the flow depth. These aspects of the dynamics will not be considered in the dam design criteria proposed here.
5 Deflecting and catching dams

Authors . . .

5.1 Introduction

A substantial improvement in the understanding of the flow of snow avalanches against dams and other obstructions has taken place in the last 5–10 years. This improved understanding has been achieved by theoretical analyses, chute experiments, numerical simulations with a new generation of 2D depth-averaged snow avalanche models, and an interpretation of flow marks of snow avalanches that have hit man-made dams and natural obstructions. This development makes it possible to formulate improved design criteria for catching and deflecting dams based on more advanced dynamic concepts, which solve some of the inconsistencies that are associated with the traditional criteria for the design of such dams. In spite of this progress, understanding of the dynamics of the impact of snow avalanches with obstacles remains incomplete, so that some subjective and partly justified concepts are needed in the formulation of the new criteria.

The new criteria are based on the concepts of supercritical overflow and flow depth downstream of a shock and they are formulated in terms of on a detailed description of the geometry of the terrain and the dam and analyses of several aspects of the dynamics of the flow of avalanches against dams. The section starts with a summary of the proposed dam design procedure before the dam height criteria are described in more detail in several subsections dealing with aspects that are common for both deflecting and catching dams. Aspects that are particular to either deflecting or catching dams, such as the determination of the deflecting angle, $\varphi$, and storage space above a catching dam, are treated in two separate sections following the common section. The description of the dam design criteria below is intentionally brief and most of the results are presented without derivations or detailed arguments. Derivations and more detailed arguments are given in a separate report (Jóhannesson and others, 2006), which is intended as an accompanying document to the guidelines.

There is substantial uncertainty about the effectiveness of dams to deflect and, in particular, to stop snow avalanches. Validation of the proposed design criteria based on observed run-up of natural avalanches is discussed below in subsection 5.10 and in more detail in the abovementioned accompanying report to the guidelines. Overflow over catching dams is, in addition, discussed in the section about catching dams, § 7, and in Appendix E, and in more detail by Gauer and Kristensen (2005a).

5.2 Summary of the dam design procedure

It is proposed that the design height of both catching and deflecting dams is determined based on essentially the same dynamic principles and carried out in a stepwise fashion according to the following list. The required dam height, $H$, normal to the terrain, is the sum of the run-up
of the avalanche on the dam side, \( h_r \), and the snow depth on the terrain upstream of the dam, \( h_s \)

\[ H = h_r + h_s . \]  

(5)

The steps are as follows (see Figures 2, 3 and 4 and Appendix A for explanations of the meaning of the variables):

1. Estimate appropriate design values for the velocity and flow depth of the avalanche at the location of the dam, \( u_1 \), \( h_1 \), and for the snow depth on the terrain upstream of the dam, \( h_s \).

2. For a deflection dam, determine the deflecting angle \( \phi \). For a catching dam, \( \phi = 90^\circ \).

3. Compute the Froude number of the flow, \( Fr \), according to Equation (4), and the component of the velocity normal to the dam axis, \( u_\eta = u_1 \sin \phi \). Determine the momentum loss coefficient \( k \) according to Equation (16). The coefficient \( k \) represents the loss of momentum normal to the dam axis in the impact and depends on the angle of the upper dam side with respect to the terrain \( \alpha \).

4. Compute the sum of the critical dam height, \( H_{cr} \), and the corresponding critical flow depth, \( h_{cr} \), according to Equation (9) or (10)(see Figure 5). The dam height above the snow cover must be greater than the run-up height \( h_r = H_{cr} + h_{cr} \). If the dam height above the snow cover is lower than \( H_{cr} \), the avalanche may overflow the dam in a supercritical state. If the dam height is lower than \( H_{cr} + h_{cr} \), the front of the avalanche may overflow the dam while a shock is being formed. Note that some overflow may occur in the initial impact due to splashing even when this criterion is satisfied.

5. Compute the flow depth, \( h_2 \), downstream of a shock above the dam according to Equation (13) (see Figure 7). The dam height above the snow cover, \( h_r \), must also be greater than \( h_2 \).

6. The requirements expressed in the previous two items in the list are expressed graphically in Figure 10, where the design dam height above the snow cover, \( h_r = H - h_s \), corresponding to given values of \( h_1 \) and \( u_\eta \), may be read directly from the higher one of two curves that represent supercritical overflow and flow depth downstream of a shock, respectively.

7. For a deflection dam, check whether an attached, stationary, oblique shock is dynamically possible by verifying that the deflecting angle, \( \phi \), is smaller than the maximum deflecting angle, \( \phi_{max} \), corresponding to the Froude number \( Fr \) (see Figure 8). It is recommended that \( \phi \) is at least 10’ smaller than \( \phi_{max} \). A dam with a deflecting angle \( \phi \) that does not satisfy this requirement must be dimensioned as a catching dam with \( \phi = 90^\circ \) with regard to the flow depth downstream of the shock. The criterion based on
supercritical overflow is computed from Equation (9) or (10) with the original value of \( \varphi \) as before.

8. Compute the vertical dam height, \( H_D \), from \( H = h_r + h_s \) using Equation (6).

9. If the terrain slope normal to the dam axis is in the direction towards the dam, the height of a deflecting dam must be increased by \( \Delta H \) according to Equation (?) in order to take into account the downstream increase in elevation at the location of the shock compared with terrain that does not slope towards the dam.

10. For a catching dam, compute the available storage space normal to the dam axis upstream of the dam per unit length along the dam according to Equation (17). The storage per unit width or storage area must be larger than the volume of the avalanche divided with its width (see Figure 12).

11. For a deflecting dam, evaluate the extent of the region affected by increased run-out distance caused by the interaction of the avalanche with the dam. The construction of the dam leads to increased avalanche risk within this area.

The main new features of the above procedure to compute dam height are that

- shock dynamics are used to derive run-up on dams, which determines the dam height under some conditions,
- a maximum allowable deflecting angle limits the range of possible deflecting angles of deflecting dams,
- momentum loss in the impact with a dam is calculated from the component of the velocity normal to the dam in the same way for both catching and deflecting dams,
- flow above deflecting dams is channelised along the dam, which may lead to a substantial increase in run-out in the direction of the channelised flow.

In practice, these requirements are satisfied in an iterative process, where the dam location, the slope of the upstream face of the dam and the deflecting angle are varied to minimise the construction cost, while taking into account other relevant conditions such as distance to the protected settlement, availability of suitable construction materials and various environmental aspects.

5.3 Dam geometry

Flow depth, dam height and run-up on the dam side are, except in the last stage of the design, defined in the direction normal to the terrain upstream of the dam (see Figure 3). Terrain slope at the dam location in the direction of steepest descent is denoted by \( \psi \) and the slope of the
Figure 3: Schematic figure deflecting dam showing the $x,y,z$- and $\xi,\eta,\zeta$-coordinate systems the deflecting angle, $\varphi$, the slope of the terrain, $\psi$, and the angle between the upper dam side and the terrain, $\alpha$. The figure is adapted from Domaas and Harbitz (1998).

terrain normal to the dam axis by $\psi_\perp$. The (dense core of the) design avalanche has flow depth $h_1$ and depth-averaged velocity $u_1$ at the dam location (that is just upstream of the dam before the dam has any effect on the flow) (see Figure 2). The shape of the terrain and the avalanche flow upstream of the dam are assumed to be sufficiently uniform that spatial variations in $\psi$, $u_1$ and $h_1$ may be ignored. A sloping coordinate system is aligned with the terrain upstream of the dam with the $x$-axis along the flow direction, which is assumed to be directly in the downslope direction. The $y$-axis points away from the dam, and the $z$-axis points upwards in a direction normal to the terrain (see Figure 3). The deflecting angle of the dam is denoted by $\varphi$ and the angle between the upper dam side and the terrain, normal to the dam axis, is $\alpha$. The snow depth on the terrain, $h_s$, is not explicitly considered in the following discussion and simply added in the end, assuming that $h_s$ is sufficiently uniform in space that this is appropriate.

Vertical dam height (and vertical run-up) will in general be slightly different from the corresponding height measured normal to the terrain and may be computed from the following geometric identity

$$H_D = \frac{\cos \psi - \sin \varphi \sin \psi \cot \alpha}{1 - \cos^2 \varphi \sin^2 \psi} H,$$

where $H$ is measured normal to the terrain and $H_D$ is measured in a vertical section normal to the dam axis in a horizontal plane. Protection dams are typically built in the run-out areas of avalanches where terrain slopes are small so this difference is most often not important.
Since vertical dimensions are slightly longer than the corresponding dimensions normal to the terrain, values found for $H$ may be used to determine vertical dam heights with a small error on the safe side. In the discussion that follows, all quantities are expressed in a coordinate system that is aligned with the terrain unless otherwise stated.

5.4 The dynamics of flow against deflecting and catching dams

A dry-snow avalanche will typically flow towards a dam in a supercritical state, that is with $Fr > 1$. The first determining factor for the height of both catching and deflecting dams is that uninterrupted supercritical flow over the dam must be prevented. If supercritical overflow is impossible, shallow fluid dynamics predict the formation of a shock upstream of the dam. This theoretical prediction has been confirmed for fluid and granular flow in several chute experiments, and may have been observed for natural snow avalanches. The second criterion for the height of avalanche dams is that the flow depth downstream of the shock, $h_2$, must be smaller than the dam height. These two requirements in combination form the core of the design requirements that are proposed here.

The dynamics of the formation of a shock upstream of a dam is not well understood. In most, but not all, practical cases, the downstream flow depth, $h_2$, is smaller than the dam height required to prevent supercritical overflow, assuming no loss of momentum in the impact with the dam. Therefore, if the formation of a shock could be guaranteed, the dam could be built substantially lower than required for preventing supercritical overflow. However, there are indications from natural snow avalanches, which have overflowed or scaled high natural terrain obstacles, that avalanches can flow over dams higher than the flow depth downstream of a shock corresponding to likely values of the upstream velocity and flow depth. Therefore, it is proposed here to adopt a worst case scenario, firstly, supercritical overflow must be prevented during the initial interaction so that a shock may form, and then, overflow downstream of a shock must also be prevented.

There is an obvious difference between the flow of avalanches against catching and deflecting dams that hides a fundamental dynamic similarity. This similarity partly shows up in the traditional expressions for the kinetic energy component of the dam height, $h_u$, for catching and deflecting dams (2) and (3). The $\lambda$-factor in these equations represents loss of kinetic energy in the interaction with the dam beyond the potential energy needed to scale the dam. These equations indicate that a deflecting dam is equivalent to a catching dam being hit by an avalanche with a velocity equal to the component of the velocity normal to the dam axis. The equations have an intuitively clear meaning for dams on horizontal terrain in terms of the kinetic and potential energy of a point mass that moves over the dam. In that case, the vertical dam height, $H_D$, is equal to the run-up normal to the upstream terrain, $H$. However, for dams on sloping terrain, the equations do not have a similarly clear interpretation. This is evidenced by the fact that there are “potential streamlines” along the side of deflecting dams in sloping terrain that maintain the same altitude. If avalanches could flow along such streamlines, they would be able to overflow the dam without any loss of kinetic energy due to the scaling of the
Figure 4: Schematic figure of an oblique shock above a deflecting dam showing the deflecting angle, $\phi$, the shock angle, $\theta$, their difference $\Delta = \theta - \phi$, and the $x,y$- and $\xi,\eta$-coordinate systems.

dam. Somehow avalanches “decide” not to flow along such streamlines.

If friction is approximately balanced by downslope gravity, the contact between the terrain and the bottom of the avalanche may be assumed to transmit only normal forces (within the framework of the depth-averaged description). Relative motion between the avalanche and the terrain, parallel with the terrain, has then no influence on the flow of the avalanche. This will be approximately true for regions with sharp gradients in the flow such as shocks, even when friction has some effect, if fluid particles flow through the region in a very short time interval, compared with the time needed for frictional forces to have significant effect on the momentum of the flow. The conservation equations for mass and momentum for shallow fluid flow are equally valid in a uniformly moving coordinate system under these conditions. Let a $\xi,\eta,\zeta$-coordinate system be defined such that the $\xi$-axis is aligned with the axis of a deflecting dam, the $\eta$-axis points in the direction normal to the dam axis in the upstream direction, the $\zeta$-axis in the direction normal to the terrain as the $z$-axis, and the origin moves along the dam axis with speed $u_1 \cos \phi$ (see Figures 3 and 4). It is easy to show that, for supercritical flow over the dam, the dynamics in the $\xi,\eta,\zeta$-coordinate system are exactly equivalent to normal flow with uniform velocity $u_1 \sin \phi$ towards a catching dam. This fact may be used to express
the criterium for supercritical overflow over a catching dam, in a form suitable for a deflecting dam (see Jóhannesson and others, 2006).

The shock relations for a stationary, oblique hydraulic jump upstream of a deflecting dam may be similarly shown to be equivalent to a moving normal shock above a catching dam to a very good approximation (see Jóhannesson and others, 2006). This shows that avalanche flow against catching and deflecting dams are dynamically similar in a fundamental sense. This has the practical implication that theoretical derivations and results of laboratory experiments for catching dams may be used to improve design criteria for deflecting dams and vice versa.

Frictional forces are not considered explicitly in the derivation of the design criteria. However, they are implicitly assumed to balance the downslope component of gravity so that the oncoming flow can be assumed to be non-accelerating and spatially uniform. The role of terrain friction in the dynamics of an impact of an avalanche with a dam is not well understood, as evidenced by the fact that the Coulomb friction coefficient $\mu$ appears in some expressions for the design height of dams but not in others. However, one may expect terrain friction to be comparatively unimportant in the impact of dry-snow avalanches with dams. For each part of the avalanche body, the impact does not last long enough for frictional forces to reduce the momentum of the avalanche significantly. In addition, many dams are located in gently sloping terrain, where friction is partially balanced by downslope gravity. Assuming that frictional forces are approximately balanced by downslope gravity may not be realistic in some situations, in particular for long deflecting dams with acute deflecting angles, where the deflecting process lasts relatively long for each part of the avalanche body. The simplified results where friction is assumed to be balance by gravity may, however, be expected to provide an upper bound for design dam height even when friction cannot be neglected.

Entrainment of snow from the snow cover into the avalanche or deposition of snow from the avalanche onto the terrain is also neglected here. These are poorly understood processes that may affect avalanche/dam interactions to some degree. In particular, deposition may be an important process under some circumstances where a part of the avalanche may pile up in front of a dam and form a platform over which the remainder of the avalanche may flow and overtop the dam. This aspect of avalanche/dam interactions is, however, not considered in the dam design criteria described here.

Many of the above simplifying assumptions may be relaxed in numerical simulations of the depth-averaged shallow fluid equations with shock-capturing algorithms, where complex terrain and dam shapes, and frictional forces and possibly also entrainment/deposition, may be taken into account (see for example Gray and others, 2003). An insight into the simple situation analysed here is, nevertheless, useful in the interpretation of results from numerical simulations. The analytical expressions for dam height that are provided by the simplified analysis are also useful for developing initial ideas for dam geometry in more complex situations that can then be refined by numerical simulations.
5.5 Supercritical overflow

The criterion for supercritical overflow can be derived from a conservation equation for the energy of the flow over the dam (see Hákonardóttir, 2004), which is valid if friction is balanced by gravity and as long as no shocks are formed. The result may be stated in the form of an expression for the critical dam height

\[ H_{cr}/h_1 = \frac{1}{k} + \frac{1}{2} (k \text{Fr} \sin \phi)^2 - \frac{3}{2} (\text{Fr} \sin \phi)^{2/3}, \]  

which is the maximum height of a dam over which supercritical flow may be maintained. The coefficient \( k \) represents the loss of momentum normal to the dam axis in the impact. It is discussed in a separate section below. The momentum loss specified by \( k \) is only meaningful for dams that are higher than several times the upstream flow depth \( h_1 \). In the derivation of Equation (7), it is assumed to take place immediately as the flow crosses the foot of the dam.

The flow depth at height \( H_{cr} \), above the snow cover at the base of the dam, which may be termed critical flow depth, is given by

\[ h_{cr}/h_1 = (\text{Fr} \sin \phi)^{2/3}. \]  

The flow changes from supercritical to subcritical at height \( H_{cr} \), where the flow depth is \( h_{cr} \), so the surface of the flow is then at height \( H_{cr} + h_{cr} \) above the snow cover. If the dam height above the snow cover is lower than \( H_{cr} \), the main core of avalanche may overflow the dam in a supercritical state, and if the dam height is lower than \( H_{cr} + h_{cr} \), the avalanche may partly overflow the dam, while a shock is being formed. Therefore, it is natural to require that the dam height above the snow cover should be larger than \( h_r = H_{cr} + h_{cr} \), which is given by

\[ h_r/h_1 = (H_{cr} + h_{cr})/h_1 = \frac{1}{k} + \frac{1}{2} (k \text{Fr} \sin \phi)^2 - \frac{1}{2} (\text{Fr} \sin \phi)^{2/3}, \]  

according to Equations (7) and (8).

The requirement expressed by Equation (9) may perhaps lead to some overdesign because a dam height of \( H_{cr} \) above the snow cover should be enough to form the shock. Overflow should then only occur temporarily and the bulk of the avalanche should be stopped or deflected. If some overflow can be tolerated, for example if the protected area is some distance away from the dam, it may be possible to require a dam height of only \( H_{cr} \) above the snow cover rather than \( H_{cr} + h_{cr} \). It should, however, be borne in mind that overflow may occur in the initial impact due to splashing for a dam height of \( H_{cr} + h_{cr} \), so that even this dam height may not prevent some overflow of the dense core during the initial impact. In addition, some overflow will occur over most avalanche dams due to the saltation and powder components if the dams are hit by large avalanches. Whether \( H_{cr} \) or \( H_{cr} + h_{cr} \) is the most appropriate dam height cannot be decided without more detailed understanding of the dynamics of the initial impact with the dam. Here, the more conservative choice is made and \( H_{cr} + h_{cr} \) is adopted as a minimum dam height.
Equation (9) may be rewritten in dimensional form as

\[ h_r = H_{cr} + h_{cr} = \frac{h_1}{k} + \frac{(u_1 \sin \phi)^2}{2g \cos \psi} k^2 \left( 1 - k^{-2} (Fr \sin \phi)^{-4/3} \right), \]  

(10)

which facilitates comparison with the traditional dam height expressions (2) and (3).

If a “Froude number” normal to the dam axis, \( Fr_\perp \), is defined as

\[ Fr_\perp = Fr \sin \phi = \frac{|u_\eta|}{\sqrt{g \cos \psi h_1}} \]  

(11)

one may write Equation (9) as

\[ h_r/h_1 = \frac{1}{k} + \frac{1}{2} (kFr_\perp)^2 - \frac{1}{2} (Fr_\perp)^{2/3}, \]  

(12)

which shows that the same fundamental expression, in terms of the component of the velocity normal to the dam axis, \( u_\eta \), may be used for both catching and deflecting dams. These equations are only valid as long as \( (k^{3/2} Fr_\perp) > 1 \), that is as long as the flow in the direction normal to the dam axis is supercritical (after slowing down due to momentum loss in the impact and corresponding thickening of the flow), because they are based on an assumption of energy conservation in flow over the dam, which is only valid for supercritical flow where no shocks are formed.

Figure 5 shows the run-up according to Equation (9) as a function of deflecting angle, \( \phi \), for several values of the upstream Froude number, \( Fr \), (solid curves). The figure also shows run-up according to the traditional formula for the height of deflecting dams (above the snow cover) according to (1) and (3) with \( \lambda = 1 \) (dashed curves). The lowering of the run-up derived from Equation (9), with respect to the corresponding run-up according to the traditional formula, is due to the thickening of the flow as it overflows the dam, and the minimum flow velocity at the top of the dam, which arises from the requirement that the overflow must be supercritical. The resulting reduction in the required dam height is largest in a relative sense for low Froude numbers and low deflecting angles.

### 5.6 Upstream shock

If the dam is high enough to prevent supercritical overflow, a propagating normal shock will form above a catching dam and a semi-stationary, oblique shock may form above a deflecting dam. The velocity and flow depth will change discontinuously across the shock.

The conservation equations for mass and momentum for shallow, incompressible flow in 2D (Whitham, 1999; Hákonardóttir and Hogg, 2005) may be shown to have the following solution for the flow depth downstream of the shock (Jóhannesson and others, 2006) (this solution is exact for normal shocks and a good approximation for oblique shocks)

\[ h_2/h_1 = \frac{2\sqrt{(6Fr_\perp^2 + 4) \cos \delta + 1}}{3}, \]  

(13)
Figure 5: Supercritical run-up, \( h_r/h_1 = (H_{cr} + h_{cr})/h_1 \), according to Equation (9), as a function of deflecting angle, \( \phi \), for horizontal terrain (\( \psi = 0 \)), assuming no momentum loss in the impact (\( k = 1 \)), for several values of the upstream Froude number \( Fr \) (solid red curves). Dashed curves show run-up according to the traditional formulae for the height of deflecting dams (above the snow cover) (3) and (3), also for no friction and no momentum loss in the impact (\( \lambda = 1 \)). The curves are labeled with the Froude number \( Fr \).

where \( \delta \) is defined as

\[
\delta = \frac{1}{3} \left( \frac{\pi}{2} - \tan^{-1} \left( \frac{9Fr^2 - 8}{Fr_{\perp}^{1/2} \sqrt{27(16 + 13Fr_{\perp}^2 + 8Fr_{\perp}^4)}} \right) \right). 
\] (14)

The widening of the oblique shock, \( \Delta \), above a deflecting dam is then approximately given as

\[
\Delta = \frac{\cos \phi \sin \phi}{\cos^2 \phi(h_2/h_1) - 1}. 
\] (15)

from which the shock angle \( \theta = \phi + \Delta \) may be found (see Figure 4).

Figures 6 and 7 show the shock angle, \( \theta \), and the downstream flow depth, \( h_2 \), as functions of the deflecting angle, \( \phi \), for fixed values of the Froude number, \( Fr \). The figures show both the exact oblique shock solution (see Jóhannesson and others, 2006) (thin solid and dashed curves), and the explicit, approximate solution given by Equations (13) to (15) (thick curves). Figure 6 shows that two shock angles are possible for each pair of values of the deflecting angle and the Froude number. The shocks corresponding to the smaller and larger deflecting angle are called “weak” (thin solid curves) The strong shock does typically not occur in real fluid or granular flow, but it has recently been observed experimentally in chute experiments.
Figure 6: Shock angle $\theta$ as a function of deflecting angle $\phi$ for an oblique shock. Thin solid (weak shock) and dashed (strong shock) curves show the shock angle given by the oblique shock relations (see Jóhannesson and others, 2006). Thick green curves show the results given by the approximate solution defined by Equations (13) to (15). The curves are labeled with the Froude number Fr.

with granular flow by adjusting the downstream flow conditions below the lower end of the dam (Xinjun Cui and Nico Gray, personal communication). The normal shock approximation given by (13) to (15) only gives the solution corresponding to the weak shock. Figure 6 shows that the normal shock dynamics provide a good approximation to the exact oblique shock solution for $\text{Fr} \geq 2.5$ and deflecting angles, $\phi$, somewhat below the boundary between the weak and strong shocks. Thus, the normal shock approximation covers the range in Fr and $\phi$ that is relevant for deflecting dams.

For each value of the Froude number, Fr, an attached, stationary, oblique shock is not dynamically possible for deflecting angles, $\phi$, larger than a maximum, $\phi_{\text{max}}$, which represents the boundary between the weak and strong shocks in Figures 6 and 7. The maximum deflecting angle is shown as a function of the Froude number Fr in Figure 8. Chute experiments with granular materials indicate that an attached, stationary shock may perhaps not be maintained for deflecting angles close to the theoretical maximum, $\phi_{\text{max}}$. Therefore, it is recommended here that deflecting angles should be at least 10° lower than $\phi_{\text{max}}$. An avalanche hitting a dam with a deflecting angle $\phi$ that does not satisfy this requirement may not remain attached and start to propagate upstream to form a detached, stationary shock (see ?, 19??). The detached shock will form a larger angle with respect to the oncoming flow than an attached shock and, therefore, the jump in flow depth across the shock will also be larger. It is recommended here that the downstream shock depth for a dam that does not satisfy the above requirement for an
Figure 7: Flow depth downstream of an oblique shock as a function of deflecting angle $\varphi$. Thin solid (weak shock) and dashed (strong shock) curves show the solutions given by the oblique shock relations (see Jóhannesson and others, 2006). Thick green curves show the results given by the approximate solution defined by Equations (13) and (14). The curves are labeled with the Froude number $Fr$.

Figure 8: Maximum deflecting angle of an attached, stationary, oblique shock.

attached, stationary oblique shock be computed as for a catching dam with $\varphi = 90^\circ$.

Figures 5 and 7, which have the same scales and can therefore easily be compared, represent the two dam height requirements proposed in these guidelines. Since both requirements
Figure 9: Supercritical run-up, \( h_r/h_1 = (H_{cr} + h_{cr})/h_1 \), according to Equation (9) (red curve), and flow depth downstream of a normal shock, \( h_2/h_1 \), according to Equations (13) and (14) (green curve), as functions of Froude number, \( Fr \), for a catching dam. The curves are drawn for horizontal terrain (\( \psi = 0 \)), assuming no momentum loss in the impact (\( k = 1 \)). The part of each curve corresponding to larger dam height is drawn as a solid thick curve.

must be satisfied, the larger dam height corresponding to a given pair of Froude number and deflecting angle must be chosen for each dam under consideration. For high Froude numbers and large deflecting angles, the criterium derived from supercritical overflow leads to the higher dam, but for low Froude numbers and small deflecting angles, the shock criterium leads to the higher dam.

Figures 5 and 7 show run-up height for deflecting angles up to \( \varphi = 70^\circ \), and do, therefore, not apply to catching dams, which have \( \varphi = 90^\circ \). Figure 9 shows both supercritical run-up, \( h_r/h_1 = (H_{cr} + h_{cr})/h_1 \), according to Equation (9) and flow depth downstream of a normal shock, \( h_2/h_1 \), according to Equations (13) and (14), for a catching dam, both as functions of the Froude number, \( Fr \). The figure shows that supercritical run-up is the determining factor for the height of catching dams for Froude numbers above approximately 3, but flow depth downstream of the shock determines the dam height for lower Froude numbers.

Shallow fluid shock theory has not been applied to the design of avalanche dams until recently. This theory has, on the other hand, been applied in hydraulics for many decades and it is the basis of the design of numerous hydraulic structures of different types and scales (see for example Chow, 1959; Hager, 1992). The theory has in this context been thoroughly verified for fluid flow. Somewhat unexpectedly, recent chute experiments indicate that the shallow fluid shock theory provides an even better approximation to granular flows than to fluid flows, for which the theory was originally developed (see Håkonardóttir and Hogg, 2005). This
arises because of rapid frictional dissipation in the interaction between grains that can occur in shocks in granular media, which appears to be more efficient than fluid friction. Transition zones with deviations from the theoretically predicted discontinuities in velocity and flow depth are, therefore, narrower in granular flows than in fluid flows. There are of course many aspects of snow avalanche dynamics that are not adequately described by shallow fluid dynamics applied to the dense core as discussed above. Nevertheless, it is clear from the theoretical and experimental studies that have been summarised here that dam height requirements derived from shallow fluid dynamics should definitely be viewed as minimum requirements for avalanche dams.

5.7 Loss of momentum in the impact with a dam

The above discussion has assumed no loss of momentum (or equivalently kinetic energy) in the impact with the dam \((k = 1\) in Equations (9) and (10)). This is of course a worst case scenario and leads to the highest dams, but it is a very pessimistic design assumption as a flow of granular material must lose some momentum in a sharp bend, where it is forced to change direction abruptly. Chute experiments with granular materials, including a few experiments with snow (Hákonardóttir and others, 2003c; Hákonardóttir, 2004, section 6.4), indicate that a substantial reduction in flow velocity occurs in the impact with catching dams that are overflowed by avalanches. This reduction is beyond the reduction in kinetic energy corresponding to the potential energy needed to overflow or scale the dam. These experiments indicate that approximately 50%, or even more (see Hákonardóttir and others, 2003c), of the kinetic energy of an avalanche is lost in an impact with dams that are normal to the bottom of the chute and have heights greater than 2 to 3 times the flow depth. Furthermore, dams that have steep upstream faces with \(\alpha \geq 60^\circ\) seem to be as, or almost as, efficient energy dissipators as normal dams, at least for the granular material that was used in these experiments (glass beads). Dams with \(\alpha = 30^\circ\) were, on the other hand, found to be less efficient. These results provide an estimate of the velocity reduction that takes place as a consequence of the abrupt change in flow direction at the upstream foot of the dam. They can, therefore, be used to estimate the relative reduction in velocity between the oncoming flow and the avalanche as it flows up the dam side after leaving the impact region at the bottom of the dam. The relative reduction in velocity, when an avalanche scales a dam and continues along the path on the other side, is considered in a separate section below. There is considerable uncertainty in these results, and they seem to indicate a somewhat greater reduction in velocity than can easily be reconciled with field observations of run-up of natural snow avalanches on dams and obstacles in the terrain (see below). They are, however, the only available direct evidence on the basis of which values of \(k\) can be estimated.

The \(\lambda\)-factor in the traditional design formula for catching dams (2) has often been chosen approximately 1.5 for catching dams built from loose materials with a slope of the upper side near 1:1.5 \((\alpha = 34^\circ\) on horizontal terrain), and approximately 2 for steep dams with a reinforced upper side with a slope greater than 2:1 \((\alpha = 63^\circ\) on horizontal terrain) (see for
example Margreth, 2004). For deflecting dams, it is often assumed that $\lambda = 1$, that is no loss of momentum in the impact. These $\lambda$-values for catching dams are in rough agreement with the results of the chute experiments described above. In fact they are on the “safe side”, as the energy loss corresponding to these values of $\lambda$ is not as great as indicated by the chute experiments for similar dams, in particular for dams with side slopes corresponding to dams built from loose materials. The $\lambda$-value 1.5 corresponds to $k \approx 0.85$, for catching dams from loose materials with a slope of 1:1.5, and $\lambda = 2$ corresponds to $k \approx 0.75$, for steep catching dams with a slope of 2:1 or greater, in the dam height expression (9). These values take into account the effect of the thickening of the flow during run-up, which leads to $\lambda > 1$ according the supercritical overflow criterium, even when $k = 1$.

Momentum loss in the impact is not well understood dynamically, so not much guidance for the determination of $k$ can be obtained from theory. The approximate dynamic equivalence of catching and deflecting dams, which was discussed in the previous section, indicates, however, that the momentum loss should be applied to both catching and deflecting dams. On the basis of the chute experiments described above and on observations of run-up of natural snow avalanches (see below), it is proposed here that, for dry-snow avalanches, $k = 0.75$ is used for dams with $\alpha > 60^\circ$, and $k = 0.85$ for dams with $\alpha = 30^\circ$, with a linear interpolation for slopes between these points. This variation of $k$ is expressed with the following equation

$$k = 0.75 \text{ for } \alpha > 60^\circ, \quad k = 0.75 + 0.1(60 - \alpha)/30 \text{ for } 30^\circ \geq \alpha \geq 60^\circ. \quad (16)$$

Dams with side slopes lower than $\alpha = 30^\circ$ should, in general, not be built, so that it is not necessary to choose $k$ for lower values of $\alpha$.

Recommended values for wet-snow avalanches are not given here and need to be further discussed in the handbook group.

**5.8 Combined criteria: supercritical overflow and shock flow depth**

The combined requirements derived from supercritical overflow and flow depth downstream of a shock are expressed graphically in Figure 10 for both dams from loose materials ($k = 0.85$, left) and steep dams ($k = 0.75$, right). The design dam height above the snow cover, $h_r = H - h_s$, corresponding to given values of $h_1$ and $u_\eta = u_1 \sin \varphi$, may be read directly from the higher one of two curves in each figure that represent supercritical overflow (red curves) and flow depth downstream of a shock (green curves), respectively. The same curves may be used for both catching and deflecting dams because of the use of the normal shock approximations (13) and (14), according to which run-up on a deflecting dam depends only on the component of the velocity normal to the dam axis in the same manner as for a catching dam. Labeled axes at the top of the figure show the upstream velocity $u_1$ corresponding to three deflecting angles for convenience.

The dependence of the dam height on the upstream flow depth $h_1$ according to the dam height criteria shown in Figure 10 is somewhat different from the traditional criteria (2) and
Figure 10: Design dam height above the snow cover $H - h_s$ as a function of the component of the velocity normal to the dam axis, $u_\eta = u_1 \sin \varphi$, for several different values for the depth of the oncoming flow $h_1$. Momentum loss in the impact with the dam is assumed with $k = 0.85$ (left, corresponding to dams built from loose materials) and $k = 0.75$ (right, corresponding to steep dams). The figures show curves derived from both supercritical overflow (red curves) and shock dynamics (green curves). The design dam height should be picked from the higher of the two curves corresponding to the estimated design flow depth. The part of each family of curves corresponding to the higher dam is drawn with solid, thick curves. The labeled axes at the top of the figures show velocity corresponding to the deflecting angles $\varphi = 15, 25$ and $35^\circ$. Dam height normal to the terrain determined from the figures must be transformed to vertical dam height with Equation (6).

(3). According to the traditional criteria, the upstream flow depth affects the dam height simply as an additional term equal to $h_f = h_1$. The flow depth enters the new criteria in a different way, and at first sight it appears to be a multiplicative quantity in both the criterium that arises from supercritical overflow and flow depth downstream of the shock (Eqs. (10) and (13)). Figure 10 shows, however, that the expression arising from supercritical overflow predicts a weak dependency of the dam height on flow depth, particularly for high velocities. This is due to a partial cancellation of terms in the dam height expression (10). The dam height derived from flow depth downstream of the shock depends, however, approximately linearly on $h_1$, for a given Froude number, but approximately linearly on the square root of $h_1$ for a given upstream velocity $u_1$.

Figure 10 shows that supercritical run-up is the determining factor for the dam height for Froude numbers above a certain value of $Fr$, which depends on the deflecting angle, at which there is a kink in the thick curves (at the point where the color of the thick curve changes from green to red). Flow depth downstream of the shock determines the dam height for lower
Froude numbers. Supercritical overflow becomes less important for low Froude numbers and low deflecting angles, whereas the reverse is true for overflow due to flow depth downstream of the shock above the dam.

A comparison of the new criteria with the traditional dam height formulae (2), (2) and (3) given in Jóhannesson and others (2006) shows that considerably higher dams are required for low deflecting angles at relatively low Froude numbers. As an example, deflecting dams with $\phi = 20^\circ$ corresponding to $Fr = 5$, or $\phi = 10^\circ$ and $Fr = 10$, need to be built approximately one third higher according to the new criteria compared with the traditional formulae. This is, however, not as significant a change as it seems at first sight, because the run-up component of the dam height is much smaller for these combinations of $\phi$ and $Fr$ than for larger deflecting angles. The difference between the new and old criteria may, for example, lead to an increase in run-up, $h_r$, above the snow cover from 6–8 m to 9–10 m.

5.9 Snow drift

*This section remains to be written.*

5.10 Comparison of proposed criteria with observations of natural avalanches that have hit dams or other obstacles

There are few observations run-up of natural avalanches on man-made dams that can be used to validate the proposed dam height criteria. The largest data-set is from Norway where information about the run-up of 15 snow avalanches on dams and natural obstacles has been gathered together with data about the geometry of the obstacles and model estimates of the velocity of the oncoming flow. Similar data about four avalanches from Iceland and the Taconnaz avalanche in France in 1999, which hit three obstacles on its way down the mountainside, were added to the Norwegian data set as a part of the compilation of these guidelines, forming a data set with a total of 21 events. Figure 11 shows a comparison of the run-up expressions derived from supercritical overflow (with $k$ according to Eq. (16) for paths with an abrupt change in slope at the foot of the obstacle) and the flow depth downstream of a shock with these field observations.

These field observations are further described in Jóhannesson and others (2006) and in the references quoted in the figure caption. Many of the obstacles are situated on rather steep terrain where there is a significant difference between run-up normal to the upstream terrain (here denoted by $h_r$) and vertical run-up (here denoted by $r$ and traditionally measured in a vertical cross section normal to the dam axis in the map plane). The figure shows vertical run-up since this is the quantity reported in reports about the avalanches. The theoretically predicted run-up normal to the terrain has been transformed to the corresponding vertical run-up with Equation (6). The flow depth, $h_1$, and velocity, $u_1$, of the oncoming flow are unknown for all the avalanches and must be considered quite uncertain. The velocity was estimated
Figure 11: Run-up of natural snow avalanches in Norway (Harbitz and Domaas, 1997; Domaas and Harbitz, 1998; Harbitz and others, 2001), Iceland (Jóhannesson, 2001) and France (Mohamed Naaim and Francois Rapin, personal communication 2006) on dams and terrain features compared with results of the run-up expressions derived from supercritical overflow (Eq. (9) with $k$ determined from Eq. (16) for the avalanches where momentum loss in the impact is assumed) and the flow depth downstream of a shock (Eqs. (13) and (14)). Momentum loss in the impact with the obstacle is only assumed for paths with an abrupt change in slope at the foot of the obstacle (marked with “(*)” in figure legend). Symbols with numbers denote observed vertical run-up. Overflow, where a substantial part or the entire avalanche went past the obstacle, is denoted with $\triangle$, and slight overflow is denoted with $\Theta$. Double arrows denote (somewhat arbitrary) ranges in the estimates for the flow depth, $h_1$ (typically 1–3 m), and velocity, $u_1$ ($\pm 15\%$), of the oncoming avalanche. Thick arrows correspond to the range in $h_1$ only, using the central estimate for $u_1$ from the above references. Thin arrows correspond to ranges in both $h_1$ and $u_1$. For the avalanches where momentum loss is assumed in the impact, the run-up range corresponding to no momentum loss is shown with dashed thin arrows. Run-up ranges derived from supercritical overflow are shown with red arrows and ranges derived from flow depth downstream of a shock with green arrows. Run-up ranges corresponding to ranges in $u_1$ are in all cases drawn at the location corresponding to the central estimate for $u_1$, so that the symbol, indicating the observed run-up, and both arrows for each avalanche are drawn at the same location on the x-axis in the figure (same value of the normal velocity $u_\eta = u_1 \sin \varphi$).
by modeling and the flow depth subjectively, with some assistance from modeling for some avalanches. In order to highlight the uncertainty due to these estimates, the model results are depicted as ranges corresponding to subjectively chosen ranges in $h_1$ (most often 1–3 m) and $u_1$ ($\pm 15\%$) rather than as single values. The figure clearly shows that the ranges in computed run-up corresponding to “moderate” variations in $h_1$ and $u_1$ are quite large.

The run-up of several of the avalanches is higher than the theoretically predicted run-up ranges, but many of them fall within the predicted ranges as further discussed by Jóhannesson and others (2006). Momentum loss in the impact is only assumed for paths with an abrupt change in slope at the foot of the obstacle (marked with “(*)” in legend of Figure 11). This is the case for all the man-made dams (six avalanches in total), and for six of the Norwegian avalanches hitting natural obstacles, the Kisárdalur and Flateyri avalanches from Iceland in 1995 and the Taconnaz avalanche hitting the glacier moraine (see Jóhannesson and others (2006) for further explanations). Two of those avalanches (no. 4 and 10) overflowed obstacles that are considerably lower than the theoretically predicted run-up. The high run-up on the deflecting dams at Flateyri in 1999 and 2000 (no. 18 and 19) may perhaps be explained by the run-up marks on loose snow on the dam sides being caused by the saltation layer of the avalanche rather than by the dense core. Three of the remaining ten avalanches (no. 13, 21 and 22) overflowed obstacles with height within or lower than the theoretically predicted ranges, six avalanches (no. 2, 8, 14, 15, 17 and 20) produced run-up marks within the ranges or close to them, one avalanche (no. 3, Tomasjorddalen) produced somewhat higher run-up marks than theoretically predicted, and one avalanche (no. 16, Kisárdalur) completely overflowed an obstacle, which is higher than the predicted run-up range, when momentum loss in the impact is assumed.

The run-up data can, thus, only be partially reconciled with the theoretically predicted run-up ranges. Dashed arrows in Figure 11 show the run-up range corresponding to no momentum loss in the impact for the avalanches hitting abrupt obstacles. The difference between the dashed and solid ranges clearly shows the large effect of the assumed momentum loss. Similarly, relatively small modifications in the assumed velocity of the avalanches can result in substantial changes in the predicted run-up ranges. Uncertainty in the flow depth, on the other hand, has little effect on the predicted run-up, except for the Kisárdalur and Taconnaz avalanches, which is estimated and/or modeled to have been unusually thick. The Tomasjordalen and Kinårdalur avalanches (no. 3 and 16) are both in the lower part of the dashed ranges and the Apoldi-L and Indre-Standal-U avalanches (no. 2 and 8), where supercritical overflow is the more important overflow mechanism as for Tomasjordalen, are at the lower end or well below the dashed ranges. Taken together, the assumed momentum loss, thus, leads to run-up ranges that are in rough agreement with this limited data set, with some avalanches within or at the lower end of the ranges, and some above, whereas no momentum loss leads to rather high ranges for the avalanches that hit abrupt obstacles. The Taconnaz avalanche hitting the glacier moraine in 1999 is in the upper part of the range corresponding to supercritical overflow, when momentum loss is assumed. Since this is a very large avalanche and the deflecting angle is rather large ($\approx 40^\circ$), this point on Figure 11 indicates that the theory leads
to reasonable run-up predictions for very large events with large normal velocities, and thus is not limited to laboratory scale granular flows or small snow avalanches. The avalanche at Flateyri in 1995 is also quite large and hits a steep gully wall at a rather large deflecting angle (≈ 30˚) with a run-up that falls within the predicted range. On the other hand, the rather wide spread of the data points compared with the assumed uncertainty of the theoretical predictions clearly indicates an incomplete understanding of the dynamics of the impact process. The Kisárdalur avalanche, in particular, represents a worrisome data point. The three avalanches with the largest run-up in excess of the theoretically predicted run-up ranges (no. 1, 5 and 6) did not hit abrupt obstacles. They are further in Jóhannesson and others (2006).

Another source of information for validating the theoretical run-up ranges is data about overflow of avalanches over the 15 m high catching dam at the full-scale experimental site at Ryggfonna in western Norway (Gauer and Kristensen, 2005a). These data are summarised in section 7 about catching dams and in Appendix E. They show long overrun distances compared with the inferred velocity at the impact with the dam and are difficult to reconcile with the theoretical run-up ranges described in this section. Available data about run-up of natural avalanches on obstacles and man-made dams thus appear to partly inconsistent and cannot be explained within one conceptual framework. The effectiveness of catching dams to completely stop snow avalanches seems to be particularly uncertain as is further discussed in the separate catching dam section. These inconsistencies may to some extent be explained by the uncertainty of the data and of back-calculated velocities and flow depths, but this is unlikely to be the only explanation. Further full-scale experiments and further theoretical analysis are required to improve this unsatisfactory situation.
6 Special considerations for deflecting dams

Authors . . .

6.1 Determination of the deflecting angle

This section remains to be written.

6.2 Curvature of the dam axis

This section remains to be written.
7 Special considerations for catching dams

Authors . . .

7.1 Storage above the dam

There must be sufficient space above a catching dam to store the volume of snow corresponding to the tongue of the design avalanche successfully stopped by the dam. According to traditional dam design principles in Switzerland and some other countries (Margreth, 2004), the storage space per unit width above a catching dam is computed as the area between the snow covered terrain and a line from the top of the dam with a slope 5–10° away from the mountain (see Figure 12). A compaction factor of about 1.5 describing the ratio of deposit density to release density is, furthermore, sometimes used [ref?]. This procedure is, however, not based on any dynamical principles and, therefore, not consistent with the overall design framework described in the previous section for determining the dam height.

Catching dams are usually built in the run-out zone of avalanches where terrain slope may be expected to be smaller than the internal friction angle $\phi$ of avalanching snow, which is, however, not well known and likely depend on the type of snow. In this case, one may expect a shock propagating upstream from the dam to maintain its thickness away from the dam (see Hákonardóttir, 2004), even when the terrain slopes towards the dam. There is, however, considerable uncertainty regarding the propagation of the shock over possibly uneven terrain. The storage volume computed from shock dynamics of this type would, for many dams on sloping terrain, be larger than the volume found with the traditional procedure, because the deposit thickness would not be reduced much with distance away from the dam.

Observations from the catching dam at Ryggfónn indicate that dry-snow avalanches do not pile much up against the dam so that the avalanche deposits slope in many case away from the dam rather than towards the dam (see Fig. 52 in Appendix E).

In the absence of the better choice, it is proposed here to continue to use the traditional methodology, with a deposit slope of 0–10° (see Figure 12), and without a compaction factor.

Figure 12: Schematic figure of the snow storage space above a catching dam. The figure is adapted from Margreth (2004).
The storage volume may then be found from the equation

\[ S = \int_{x_0}^{x_1} (z_l - (z_s + h_s))dx, \]  

(17)

where \( z_l \) is the elevation of a straight line from the top of the dam towards the mountain with a chosen slope in the range 0–10°, \( z_s + h_s \) is the elevation of the top of the snow cover before the avalanche falls, and \( x_0 \) and \( x_1 \) are the locations of the dam and the point where the line intersects the snow covered mountainside, respectively. For dams where dry-snow avalanches are expected, deposit slopes close to 0° should be used, but for locations where wetter avalanches are typical slopes up to 10° can be chosen. This procedure is not very satisfactory because it is not based on dynamic principles and needs to be refined in the future by further studies.

7.2 Overrun of avalanches over catching dams

Laboratory experiments and theoretical analysis have advanced our understanding of the dynamics, but measurements of overrun and velocity of avalanches over the dam at Ryggfonn, see Appendix E, indicate that avalanches are under some conditions able to scale dams more easily than would be expected from the theoretical analysis described in section 5.

More discussion with reference to the Appendix...
8 Braking mounds

Authors . . .

8.1 Introduction

Braking mounds (or retarding mounds) are widely used for protection against dense, wet snow avalanches, but they are often thought to have little effect against rapidly moving, dry snow avalanches (see for example Norem, 1994; McClung and Schaerer, 1993). The design of such mounds has in most cases until recently been based on the subjective judgement of avalanche experts as there exist no accepted design guidelines for braking mounds. There are, furthermore, no accepted methods for estimating the retarding effect of avalanche mounds in a quantitative way. The retarding effect is particularly badly known for dry-snow avalanches.

A number of chute experiments at different scales and with different types of granular materials have recently been performed in order to shed light on the dynamics of avalanche flow over and around braking mounds and catching dams and to estimate the retarding effect of the mounds (Woods and Hogg, 1998, 1999; Hákonardóttir, 2000; Hákonardóttir and others, 2001, 2003c,a,b; Hákonardóttir, 2004). Some of these experiments were carried out as a part of the design of avalanche protection measures for the town of Neskaupstaður in eastern Iceland (Figs. 50 and 51) (Tómasson and others, 1998a,b). A review of available hydro-engineering studies of retarding structures for high speed water flow was also carried out as a part of the design (Tómasson and others (1998b); this review is summarised in the Appendix of Jóhannesson and Hákonardóttir (2003)). The experiments were carried out for dry, supercritical, granular flow in order to analyse the retarding effect of mounds against rapid, dry-snow avalanches.

This section summarises the main results of the abovementioned studies based on Jóhannesson and Hákonardóttir (2003). Several general recommendations for the practical design of braking mounds are given with references to technical articles and reports that contain more detailed descriptions of the experimental results on which the recommendations are based.

There remain open questions regarding the applicability of the experimental results to natural avalanches due to the very different scales. An insignificant braking effect at the scales of the experiments would suggest that this effect would also be small for natural snow avalanches. On the other hand, a result indicating a substantial braking effect does not necessarily apply to natural avalanches due to the different physics and scales of the flows, such as compression of the snow in the impact with the mounds and the effect of air resistance on the flow over the mounds. Nevertheless, the experiments may be used to identify certain types of behaviour, which does not strongly depend on scale or material properties, and which may be exploited in the design of avalanche protection measures. The experiments, thus, provide useful indications for designers of retarding structures for snow avalanches in the absence of data from experiments at larger scales and measurements of natural avalanches.
As described in § 4, the dimensionless Froude number, Fr, given by Equation (4) is commonly used to characterise free-surface fluid flows. The design of the abovementioned chute experiments was based on the conjecture that if the Froude numbers were on the same order of magnitude, dynamic similarity between natural snow avalanches and the smaller-scale experimental avalanches would be maintained (see Hákonardóttir and others (2003a) for a further discussion). The Froude number of the smaller scale experiments in 3, 6 and 9 m long chutes in Reykjavík, Bristol and Davos was \( Fr \approx 10 \). The Froude number in snow experiments in a 34 m long chute at Weissfluhjoch in Davos was in the range 3–6, varying with each experimental run, depending on the condition of the snow. This was the highest Froude number that the experimental setup allowed for, and it was somewhat lower than would have been preferable.

Although there exist no generally accepted guidelines for the design of avalanche mounds, B. Salm has in Salm (1987) proposed an estimate for the reduction in the speed of an avalanche that hits several obstacles, such as buildings, that are spread over the run-out area of the avalanche and assumed to cover a certain fraction, \( c \), of the cross-sectional area of the path. According to this expression, the speed of the avalanche is reduced by the ratio \( c/2 \), assuming that the obstacles are sufficiently strong and are not swept away by the avalanche. If, for example, \( c = 1/2 \), this expression predicts that the speed is reduced by 25%, indicating a substantial effect of the obstructions on the speed of the avalanche. A similar expression for the reduction in the speed of an avalanche that hits several rows of trees was proposed by A. Voellmy in Voellmy (1955b). These expressions are not derived from a conceptual model of the flow around obstacles and it is not clear whether they may be expected to apply to a rapidly moving dry snow avalanche.

Braking mounds designed to retard rapidly moving dry-snow avalanches will in most cases be of a height that is only a small fraction of the height-scale corresponding to the kinetic energy of the avalanche, \( u^2/2g \), where \( u \) is the speed of the flow and \( g \) is the gravitational acceleration. One might expect that having flowed up the mounds, the avalanche could regain the kinetic energy spent when it descends down the backside of the mounds. Substantial energy dissipation by braking mounds must, if the mounds are in fact as effective as assumed Salm and Voellmy, be brought about by irregularities and mixing introduced by the deviation of the avalanche flow over and around the mounds. Such an effect may be expected to depend to a high degree on various details in the layout and geometry of the mounds, making the lack of established guidelines for the design of avalanche mounds particularly acute. One may also note that the volume of the avalanche will typically be so large that only a small fraction of the snow near the front of the avalanche is needed to fill the space upstream of the mounds so that they become effectively buried and the bulk of the avalanche easily overflows the mounds. For braking mounds to be effective while the avalanche passes over them, they must not become buried by the avalanche.

Experiments to study the effect of braking mounds on snow avalanches have not been performed until recently. Similar structures have, however, been studied extensively for supercritical, free-surface water flow in dam spillways and bottom outlets where they are used.
to dissipate the kinetic energy of water before it enters the downstream channel (in this context they are termed baffle blocks and baffle piers). The original experiments are described in Peterka (1984); US Bureau of Reclamation (1987) and they are summarised in many textbooks on hydraulic engineering, for example Roberson and others (1997). The energy dissipation that is induced by baffle piers in dam spillways and bottom outlets is principally a shallow-layer flow phenomenon and does not depend on the frictional properties of the fluid in question. The dense core of rapidly moving snow avalanches is a shallow-layer, gravity-driven flow. Energy dissipation by inelastic granular collisions could play a similar role in avalanche flow around and over dissipating structures as turbulent dissipation by fluid friction in ordinary fluid flow. These studies complement the braking mound experiments with granular materials in an important way because the scale of the hydraulic structures is much larger than the scale of the experimental chutes and therefore closer to the scale of natural avalanches. The speed of the water flow in the spillways is sometimes more than an order of magnitude higher than the speed of the granular materials in the abovementioned chute experiments with mounds. The results of the hydraulic experiments and their implications in the context of snow avalanches, including the importance of the Froude number in both cases, are discussed further in Hákonardóttir and others (2003b).

8.2 Interaction of a supercritical granular avalanche with mounds

The experiments showed that a collision of a supercritical granular avalanche with a row of mounds leads to the formation of a jump or a jet, whereby a large fraction of the flow is launched from the top of the mounds and subsequently lands back on the chute (Figs. 13 and 14). For steep obstacles, particles are initially launched from the top of the obstacle at an angle close to its upstream angle, $\alpha$. The jet rapidly adjusts to a new angle due to the formation of a wedge behind the upstream face of the mound. This angle is termed the throw angle and is denoted by $\beta$. The bulk of the current then passes over the barrier as a coherent, quasi-steady jet (Figs. 13 and 14). This part of the jet lands furthest away from the mounds.

Energy dissipation takes place in the impact of the avalanche with the mounds and also in the interaction of jets from adjacent mounds. Energy dissipation, furthermore, takes place in the landing of the jets on the chute and the subsequent mixing with material flowing in between the mounds.

The airborne jet that is formed by the collision of the flow with the mounds has important practical consequences for the use of multiple rows of mounds or combinations of rows of mounds and a catching dam. The spacing between the rows must be chosen sufficiently long so that the material launched from the mounds does not jump over structures farther down the slope.

The trajectory of the jet launched directly over the mounds can be approximated as a projectile motion in two dimensions (Fig. 13). Conservation of momentum leads to the equation

$$m\ddot{x} = F = mg - m(f/h_j)|\dot{x}|,$$

(18)
where $\mathbf{F}$ is the force exerted on the mass, $m$, $g$ is the gravitational acceleration, $f$ is a dimensionless constant representing turbulent drag caused by air resistance, $h_j$ is the thickness of the core of the jet, $\mathbf{x} = (x, z)$ is the location of the projectile in horizontal and vertical directions, respectively, with the origin at the top of the mound, and a dot denotes a time derivative. Equation (18) can be written as

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

(19)

$$\ddot{z} = -g - \left(\frac{f}{h_j}\right)\dot{z}\sqrt{\dot{x}^2 + \dot{z}^2}.$$  (20)

The initial conditions at $t = 0$ are

$$x = z = 0 \quad \text{at} \quad t = 0$$

and

$$\dot{x}(0) = u_1 \cos (\beta - \psi) \quad \text{and} \quad \dot{z}(0) = u_1 \sin (\beta - \psi),$$

where $\beta$ is the throw angle and $\psi$ is the slope in which the mounds are situated. The horizontal length of the jump, $L$ (Fig. 13), can be found by solving the two equations given appropriate values of $u_1$, $\beta$ and $f/h_j$. Recommended values for these parameters for natural snow avalanches are discussed below.

**$u_1$** The throw speed $u_1$ may be expressed as

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

Figure 13: A schematic diagram of a jet of length $L$ with upstream flow thickness $h$ and jet thickness $h_j$. The jet is deflected at an angle $\beta$ over a mound or a dam of height $H$ positioned in a terrain with inclination $\psi$. The upstream mound face is inclined at an angle $\alpha$ with respect to the slope. $u_0, u_1, u_2, u_3$ and $u_4$ are the speed at different locations in the path.
Figure 14: Photographs of (a) the datum mound configuration and (b) the jet in a quasi-steady state on the 6 m long experimental chute in Bristol (Hákonardóttir and others, 2003b).

where \( u_0 \) is the incoming speed, \( H \) is the height of the mounds and \( k \) is a dimensionless constant representing the energy dissipation involved in the impact of the avalanche with the mounds. The value \( k = 1 \) corresponds to no energy loss in the impact. The experimental results indicate that \( k \) is in the range 0.5–0.8 for mound and dam heights 2–3 times the flow depth (Hákonardóttir and others, 2001, Fig. 38), (Hákonardóttir and others, 2003c, Fig. 7), (Hákonardóttir and others, 2003b, Fig. 10), with most of the values falling in the range 0.6–0.7. For natural snow avalanches, it is recommended that the throw length is computed for the three values \( k = 0.7, 0.8 \) and 0.9 and that the result for \( k = 0.8 \) be used to calculate the minimum distance between a row of mounds and the next retarding or retaining structures below.

Figure 16 shows theoretical curves for the inviscid, irrotational flow of a fluid over an obstacle where gravity effects are neglected Yih (1979). The experimental results indicate
that the theory gives an upper bound for the throw angle, $\beta$. For mounds with $H/h \approx 2–3$ and $\alpha = 90^\circ$, the theoretical $\beta$ should be reduced by $20–25^\circ$, for $\alpha = 75^\circ$, $\beta$ should be reduced by $10–20^\circ$ and for $\alpha = 60^\circ$, it should be reduced by $0^\circ–10^\circ$. It is recommended that the throw angle $\beta$ be chosen based on these considerations.

\[ \frac{f}{h_j} \]  
The turbulent drag on the jet, caused by air resistance, is represented by the dimensionless constant $f$, and depends on the jet thickness, $h_j$, and the speed of the airborne flow (Eqs. (19) and (20)). Air resistance does not affect the flow on the small scale of the granular experiments (including the snow experiments) and the experimental trajectories are therefore well reproduced by using $f = 0$ in equations (19) and (20). On the other hand, full scale experiments with water jets suggest that between 0% to 30% of the initial kinetic energy of the jet may be lost during the jump (see Hager and Vischer, 1995; Novak and others, 1989; US Bureau of Reclamation, 1987). The dense core of an avalanche is less dense (density in the range 100–400 kgm$^{-3}$) than water (density of 1000 kgm$^{-3}$). Therefore, it is reasonable to assume that an avalanche jet will be affected by air resistance at least to the same extent as a jet of water, leading to a shortening of the jet. By taking $f \approx 0.01$ and $h_j \approx 2–4$ m we obtain $f/h_j = 0.0025–0.005$ m$^{-1}$. In order to reduce the kinetic energy of a fluid jet flowing with a speed of 40 ms$^{-1}$ by 30%, as given in Novak and others (1989), $f/h_j$ needs to be given a value of $f/h_j \approx 0.004$ m$^{-1}$ which fits into the range given above. Here it is recommended that the value $f/h_j = 0.004$ m$^{-1}$ is adopted in computations of the throw length.
Figure 16: The throw angle, $\beta$, plotted against the non-dimensional mound height, $H/h$, for different angles between the upstream faces of the mounds and the slope, $\alpha$. The points denote experimental results and the solid lines are theoretical predictions. ‘Series i, ii and iii’ are results from experiments described in Hákonardóttir and others (2003b) using glass particles on 3, 6 and 9 m long chutes. ‘Snow experiments’ denotes experiments with snow on the 34 m long chute at Weissfluhjoch described in Hákonardóttir and others (2003c) and ‘Fluid experiment’ denotes an experiment described in Yih (1979).

8.3 Recommendations regarding the geometry and layout of the mounds

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

1. The height of the mounds, $H$, above the snow cover should be 2–3 times the thickness of the dense core of the avalanche. Increasing the height of the mounds beyond this, for
a fixed width of the mounds, does not significantly reduce the run-out according to the experiments.

2. The upstream face of the mounds should be steep. For the chute experiments with glass beads (ballotini), $\alpha \approx 60^\circ$ was sufficient since a steeper upstream face only marginally improved the energy dissipation. This result may not be appropriate for natural snow avalanches because of the different physical properties of the materials.

3. The aspect ratio of the mounds above snow cover, $H/B$, should be chosen close to 1.

4. The mounds should be placed close together with steep side faces, so that jets launched sideways from adjacent mounds will interact. Many short mounds were found to be more effective than fewer and wider mounds for the same area of the flow path covered by mounds.

If there is sufficient space in the terrain for a second row, it should be staggered with respect to the first row (Fig. 17). As discussed in the following subsection, the retarding effect of the second row may be expected to be somewhat less than the effect of the first row, although this is not well constrained by the available experiments (Hákonardóttir and others, 2001).
8.4 Retarding effect

It is important to be able to quantify the reduction in flow velocity provided by braking mounds in addition to defining an optimum layout and geometry of the mounds. An estimate of this retardation cannot be made from full-scale observations of natural events and must therefore be based on the results of chute experiments. There are many technical difficulties associated with direct measurements of the flow speed of the granular material in the chute experiments, in particular for measurements of the speed of the flow downstream of the landing point of the jet. In most of the experiments, the speed reduction was not directly measured. Rather, the effect of the mounds for reducing the run-out distance of the material beyond the location of the mounds was measured, both the reduction in the maximum run-out and the reduction in the run-out corresponding to the centre of mass.

The most effective single row mound configurations with mound height 2–3 times the flow thickness in the 3, 6 and 9 m long chutes were found to shorten the maximum run-out beyond the mounds by about 30% in the experiments with small glass beads (ballotini) and by a similar amount for sand in the 9 m long chute (Hákonardóttir and others, 2001). The reduction in the run-out corresponding to the centre of mass was greater, i.e. 40–50%. The reduction in the maximum run-out for two staggered rows of mounds was found to be in the approximate range 40–50% for experiments in the 3 and 9 m long chutes, and the reduction in the run-out corresponding to the centre of mass was greater than 50%.

The relative run-out reduction may be crudely interpreted as a relative reduction in the kinetic energy of the granular material by assuming that the slowing down of the avalanche in the run-out zone is brought about by frictional forces between the bed and the moving material that are approximately proportional to the weight of the material (Coulomb friction). A relative run-out reduction of about 30% (the maximum run-out) to 40–50% (the centre of mass run-out) then corresponds to a reduction in the speed of an avalanche by about 15–30% by one row of mounds. For two mound rows, a run-out reduction by 40–50% or more corresponds to a speed reduction of 20–30% or more. This interpretation of run-out reduction in terms of speed reduction is so crude that it is not possible to say whether it is more appropriate to use the relative reduction in the maximum run-out or in the run-out corresponding to the centre of mass.

The velocity of the avalanche was measured in the experiments with snow in the 34 m long chute at Weissfluhjoch (Hákonardóttir and others, 2003c), both the velocity just upstream from the mounds \(v_0\) and the velocity after the landing of the jet \(v_4\) (cf. Fig. 13). The velocity of the control avalanche in the absence of mounds at the landing location of the jet \(v_{cont}\) could furthermore be estimated. There is considerable uncertainty in the measurements of both the velocity and the flow thickness in the Weissfluhjoch experiments, but the results indicate that the ratio \(v_4/v_{cont}\) ≈ 0.8 for mounds that are about 1.3 times higher than the flow thickness.

Although the available results are open to different interpretations, they indicate that braking mounds have a substantial retarding effect on supercritical granular flows. Furthermore, the retarding effect does not seem to vary much with the scale of the chutes over the range.
of scales, velocities and experimental materials covered by the experiments (lengths of the chutes in the range 3–34 m and flow speeds upstream of the obstacles in the range 2.6–7.5 m s\(^{-1}\)). Here it is recommended that the relative velocity reduction corresponding to one row of mounds that is designed according to the recommendations given above is estimated as 20%. It is not possible to specify in detail how the energy dissipation caused by the mounds is divided between the initial impact, the interaction between adjacent jets, air resistance and energy lost in the landing of the jet and mixing with material that flows between the mounds. This will among other things be dependent on the slope and shape of the terrain where the mounds are located. For simplicity, it is recommended that the assumed speed reduction is applied at the location of the upper face of the mounds in a model computation of the flow of the avalanche down the terrain in the absence of the mounds. This assumption should only be used for mounds that are located in the run-out zone of the avalanches. It will not provide reasonable results higher up in the path of the avalanche where the terrain is steeper, but this is not an important restriction since mounds are not likely to be located outside the run-out zone.

It appears from the experimental results that the second row of mounds has less relative effect on the flow velocity than the initial row. This is also indicated by the hydraulic experiments with baffle piers in dam spillways and bottom outlets (Peterka, 1984). Here it is recommended that a second row of mounds is assumed to reduce the velocity of the avalanche by 10% in addition to the 20% reduction provided by the first row.

It needs to be stressed that the above recommendations are based on an incomplete understanding of the complex dynamics of granular avalanches that hit obstructions. Nevertheless, we believe that the chute experiments and the above recommendations that are derived from them, provide useful indications for designers of retarding structures for snow avalanches in the absence of data from experiments at larger scales and measurements of natural avalanches.
9 Dams as protection measures against powder avalanches

A first draft of this section will be written by MN and FN with additional input from DI, …

*How useful are dams for this purpose?*

*Perhaps something in this section could be useful in other sections for the analysis of the rest risk due to the powder part for deflecting and catching dams?*
10 Loads on walls

Authors . . .

Traditional avalanche dams built from loose materials are strong and stable against relevant loads, including dynamic loads from impacting avalanches, if the dams are properly designed according to geotechnical principles as summarized in section 14. Dynamic loads from avalanches do, on the other hand, need to be explicitly taken into account in the design of some dams structures in the run-out zones of avalanches, such as concrete walls and retarding mounds with steep upper sides. This section presents design recommendations for such loads. In addition to being useful for the design of dams and mounds, the recommendations are also of relevance for other structures such as buildings located in avalanche prone terrain, which are otherwise not the subject of these guidelines.

Figure 18: Avalanche impinging upon the catching dam at the NGI test site Ryggfonn. There is an obvious increase in depth as the avalanche hits the dam. (Photo NGI)
10.1 Impact force on a wall-like vertical obstacle

As a first approximation, we consider the impact of an incompressible fluid of width, $W_a$, onto a wall of width, $W_{wall}$, perpendicular to the flow (see Fig. 19). It is assumed that the wall is sufficiently wide that a major part of the avalanche does not flow around it. This is a slight variation of the well known problem of a free jet impinging a wall in hydraulics. Applying the momentum equation in integral form to the control volume, $V$, including a small portion of the fluid in contact with the wall, one obtains the impact force normal to the wall

$$F_{Ix} = \int_{A} (\rho u_x (u \cdot n) + \rho g z) dA = \left[ \rho h \alpha \overline{u}_x^2 + \rho g b \frac{h^2}{2} \right] b .$$  (21)

Here, a hydrostatic pressure distribution is assumed within the flow. $n$ is the normal vector onto the wall surface. The contact area, $A$, is equal to $b h$, where $b$ is the minimum of the avalanche width or the width of the wall, i.e., $b = \min(W_a, W_{wall})$. $\overline{u}_x$ is depth averaged velocity perpendicular to the wall and the factor $\alpha$ accounts for a non-uniform velocity profile and is close to unity.

![Figure 19: A schematic illustration of the impact of an incompressible fluid onto a wall.](image)

Furthermore, if one disregards the hydrostatic component on the right hand side of (21), which may be justified for $Fr = u_x/\sqrt{gh} \gtrsim 2.5$, one obtains

$$\frac{F_{Ix}}{A} \approx \rho \overline{u}_x^2 .$$  (22)

This is a widely used expression for the impact pressure on large obstacles (e.g. Gruber and others, 1999, see also F.1). However, the derivation is a oversimplification of the impact problem. Equation (21) holds true, e.g., in a steady case where the flow is deviated by a angle of 90°, but not during the first milliseconds of a vigorous impact. Schaerer and Salway (1980), for example, observed a short pressure peak, which was serval times the base pressure (cf. Fig. 20).
Let us now consider a flow in a confined setting first (cf. Fig. 21). As soon as the flow hits the wall, the fluid next to the wall will be stopped abruptly and a pressure wave will travel upstream with a celerity $C_p$, causing the fluid to decelerate. Within a short time interval $\Delta t (= t_i - t_0)$ a fluid element of mass $m = \rho C_p \Delta t A$ will be stopped. Applying the principle of linear impulse to this fluid element yields

$$\rho C_p \Delta t A u_x = \int_{t_0}^{t_i} R \, dt,$$

(23)

where $\int R \, dt$ is the impulse of the resultant force, which is given by

$$R = \left[ p A - (p + \Delta p) A \right].$$

(24)

Combining (23) and (24), we get
\[ \Delta p = -\rho C_p u_x, \]  
(25)

In this case, the impact pressure is linear in the velocity perpendicular to the wall. In an elastic medium, the celerity of the pressure (sonic) wave is given by

\[ C_p = \sqrt{\frac{E_b}{\rho}}, \]  
(26)

where \( E_b \) is the bulk elastic modulus of the fluid. For water, \( C_p \) is about 1440 m s\(^{-1}\); for air, \( C_p \) equals 330 m s\(^{-1}\). In multi-phase flows, like an avalanche, \( C_p \) depends on the particle concentration; for an avalanche, it might be as low as approximately 30 m s\(^{-1}\). The so-called water hammer as described above is a well-known problem in hydraulics (cf. Franzini and Finnimore, 1997, Ch. 12.6). It is encountered in hydroelectric plants, during rapid closing of a pipeline. In this case, it is observed that the duration of the pressure peaks depends on the length of the pipe and the celerity. In contrast to the confined setting used in the description above, in the case of an avalanche hitting a wall-like obstacle, the flow is usually only partly confined by the ground surface and the neighbouring flow. At the upper boundary, atmospheric pressure consists, allowing the flow to spread out, which leads to a reduction of the excess pressure throughout the flow. Experiments on water waves (Cooker and Peregrine, 1995) indicate a non-uniform pressure profile with increasing pressure with increasing distance from the free surface. This distribution was also observed for the distribution of maximum impact pressures in full-scale avalanche experiments by Kotlyakov and others (1977). Furthermore, it might also be reasonable to assume that the duration of the peak pressure is on the order of \( O(h/C_p) \), i.e., the time needed by the pressure wave to reach the free surface. In the case of an avalanche, however, the wave propagation might not be fully elastic. It might spread as a plastic wave with a lower propagation speed, or finally as shock wave with increasing propagation speed. A transition from elastic to plastic wave propagation is accompanied with a reduction of the maximum peak pressure, on the other hand the duration of the pressure peak increases, which has to be accounted for in the design of structures.

Hence, if an avalanche suddenly meets a wide obstacle, like a wall, and is thus prevented from moving ahead, it will start to spread out sideways and splash up. Simultaneously, the mixture of snow and air close to the wall will be compressed and stopped. This leads to a piling up in front of the wall and a propagation of a wave, which travels upstream through the incoming avalanche at a speed \( w \). The wave front is a non-material singularity as avalanche snow passes through it. It marks the boundary between the stopped deposit (or deviated avalanche front) and moving avalanche farther upstream. The conservation equation of mass and momentum lead to the following jump conditions across this discontinuity.

\[ \int_{\Sigma} \left\langle \rho (\mathbf{u} - \mathbf{w}) \cdot \mathbf{n} \right\rangle dA = 0, \]  
(27)
Figure 22: Scheme of an impact of an avalanche onto a wall assuming a compressible shock.

\[
\int_{\Sigma} \left[ \rho u (u - w) \cdot n \right] - \left[ t \cdot n \right] dA = 0 ,
\]  

(28)

where the jump bracket \([ f ] = f^+ - f^-\) is the difference between the enclosed function on the forward and rearward sides of the singular surface \(\Sigma\). The evaluation position is denoted by the superscripts + and −, respectively and \(n\) is the normal vector onto the singular surface pointing into the + side. \(w\) is the propagation velocity of the singular surface (wave front) and \(t \cdot n\) describes the normal stress onto its respective side. If one assumes an effective width of the singular surface, \(b\), the jump conditions in (27) and (28) can be written as

\[
\left[ \rho h (u - w) \cdot n \right] b = 0 ,
\]

(29)

\[
\left[ \rho h u (u - w) \cdot n \right] - \left[ \int_0^{b^\pm} \sigma_x dz \right] b = 0 .
\]

(30)

From mass balance, (29), \(w\) can be derived using the approximation \(u^- \cdot n \approx 0\), i.e., normal velocity of the avalanche behind the shock is zero. Then,

\[
w \approx - \frac{u^+ \cdot n}{(\rho^- h^- / \rho^+ h^+) - 1} .
\]

(31)

The ratio \(\rho^- h^- / \rho^+ h^+\) is for certain larger than one. In case the ratio is smaller than two, the wave propagates faster upstream than the incoming avalanche downstream. Using (31) and assuming a hydrostatic pressure distribution within the flowing part of the avalanche and a uniform velocity profile, one obtains from (29)

\[
\left[ \rho^+ h^+ (u^+)^2 \left( 1 - \frac{\rho^+ h^+}{\rho h} \right) + \rho^+ g \frac{(h^+)^2}{2} \right] b = \left[ - \int_0^h \sigma_x^- dz \right] b .
\]

(32)
If one further neglects the influence friction at the bottom and at the upper surface, the right hand side is approximately equal to the normal force imparted to the wall. One has

$$F_{ix} = \rho^+ (u^+)^2 \left[ 1 + \frac{1}{(\rho^+ h^- / \rho^+ h^+ - 1)} \right] + \frac{1}{2 (\text{Fr}^+)^2} h^+ b ,$$  \hspace{1cm} (33)

where $\text{Fr}^+$ is the upstream Froude number ($= u^+ / \sqrt{gh^+}$). The dynamic impact force estimated from (32) is thus greater by approximately $\rho^+ (u^+)^2 ((\rho^+ h^- / \rho^+ h^+ - 1)^{-1} h^+ b$ than the stress estimated by (21).

The jump $[\rho \ h]$ itself depends on the impact pressure and the compressibility of the snow-air mixture and of the ability of the avalanche change direction, which means for laterally extended obstacles primarily to raise and to increase its height ($h^-$). Voellmy (1955a) proposed the following relation for the compressibility of snow.

$$\frac{\rho}{\rho_0} = \frac{1 + \frac{p}{p_0}}{1 + \frac{p_0 \rho}{\rho_F p_0}} ,$$  \hspace{1cm} (34)

where $\rho_0$ is the initial density of the snow, $p_0$ the atmospheric pressure ($\approx 1000 \text{ hPa}$) and $p$ is dynamic overpressure. For the values of the upper limit density, $\rho_F$, Voellmy (1955a) gives the values 800 kg m$^{-3}$ for dry fine-grained snow, 600 kg m$^{-3}$ for dry large-grained snow and 1000

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Figure 23: The ratio between shock speed and the speed of the incoming flow versus incoming Froude number, $\text{Fr}^+$. 
for water-saturated snow. Figure 24 show the densification curve for various initial densities. A comparisons between measurements and Voellmy’s relation is given in (Voellmy, 1955a). From these plots one sees that typical values for the ratio $\rho/\rho_0$ range between 1.5 and 3 during instantaneous compression. Instantaneous compression means the duration is too short for the encapsulated air to escape. Hence, due to consolidation observed density in avalanche deposits can be higher than those during impact. Although the presented curves are measurements from initially intact snow, it is reasonable to assume that curves are similar for flowing densities. Kotlyakov and others (1977) came to similar ratios. They, however, compared the density of the snow deposit in front of their wall (200–600 kg m$^{-3}$) about with the density of snow clods in the avalanche snow (150–400 kg m$^{-3}$).

Figure 24: Densification of snow according to (34) with initial density, $\rho_0$, as parameter. Left hand side, $\rho_F = 800$ kg m$^{-3}$ and right side $\rho_F = 600$ kg m$^{-3}$. The lower row shows the respective densification normalized by the initial density.

There is no available approach to estimate the increase in depth of the avalanche indepen-
dently from the change in density. However, the flow depth by the dam downstream of the shock as function of the Froude number of the incoming flow and a given density ratio was derived by Hákonardóttir and others (2003) based on depth integrated dynamics, assuming hydrostatic stress and lateral confinement.

\[
\frac{\rho^-}{\rho^+} \left( \frac{h^-}{h^+} \right)^2 - \frac{h^-}{h^+} - 1 + \left( \frac{\rho^- h^-}{\rho^+ h^+} \right)^{-1} \left( \text{Fr}^+ \right)^2 = 0 .
\] (35)

There is currently no sound approach to estimate the effect of a non-hydrostatic stress distribution during the impact, or of lateral flow, on the increase in flow depth and density by the dam. The ratio \( h^- / h^+ \) is plotted in Figure 25 as function of \( \text{Fr}^+ \) for different density ratios for a wide obstacle where \( h^- \) is found from (35). An increase in the density ratio lowers the height \( h^- \). The right panel of the figure shows a similar plot for the density ratio \( \rho^- / \rho^+ \).

Figure 25: The ratio between shock depth and depth of the approaching flow versus the incoming Froude number, \( \text{Fr}^+ \), left panel. The density ratios between 1 and 4 are chosen to correspond to possible density ratios in snow avalanches. The right panel depicts the ratio \( \rho^- / \rho^+ \) vs. incoming Froude for varying shock depths. The white area depicts the most likely combinations.

The left panel of Figure 26 shows the dependency of the intensity factor \( 1 + (\rho^- h^- / \rho^+ h^+ - 1)^{-1} \) on \( \text{Fr}^+ \). For \( \text{Fr}^+ > 2.5 \), which is a reasonable value for fast moving catastrophic avalanches, the difference between (33) and (21) is less then 25 %. That is the force onto the wall is similar according to the two expressions. However, the point of action is lifted causing an increase in the moment about the foot point. Due to an increase in height, the pressure onto the wall can be reduced proportional to \( h^+ / h^- \) for Froude numbers greater than 2. For Froude number small than 2 the pressure might be increased by an factor between 1 and 3 and even more.
Figure 26: Intensity factor \( f(Fr^+) = 1 + \left( \rho^- h^- / \rho^+ h^+ - 1 \right)^{-1} \) versus \( Fr^+ \). Left panel, \( \rho^- / \rho^+ \) taken as parameter; \( h^- / h^+ \geq 1 \). The density ratios between 1 and 4 are chosen to correspond to possible density ratios in snow avalanches. Right panel, \( h^- / h^+ \) taken as parameter; densification \( \rho^- / \rho^+ \) ranges between 1 and 5.

Figure 27 depicts the pressure factor as function of the incoming Froude number. This reduction, however, is most likely not uniform as the snow close to the sliding surface is more obstructed by the flow on top to deviate and so will be more confined. In this case, the curve \( h^- / h^+ = 1 \) is more representative.

In addition, it should also be noted that snow-slab avalanches may contain large clods or stones that on impact can cause considerable impact forces locally during short durations (in the order \( O(10–100 \text{ ms}) \)).

As the avalanche changes its direction of flow, it causes not only a normal force onto the wall, it can also impart considerable shear forces, horizontally and vertically. Especially, the vertical component was mentioned by Voellmy (1955a) as a major cause for the observed destructions on buildings. He estimated the vertical force in the range of 0.3 to 0.5 times the normal forces. He also mentioned that the vertical component can also be directed downward in step terrain. In addition to shear forces, up-lifting can be caused by upward motion of the avalanche below balconies or a ledge of a roof. This situation corresponds to a confined setting, \( i.e., \), the ratio \( h^- / h^+ \) is restricted and high pressure can occur. However, no quantitative measurements are known for those effects.

### 10.2 Determining design loads

The construction of a building or wall-like structure in an avalanche prone area requires an assessment of reasonable design loads, \( i.e., \), estimates of the total maximum force, \( F_m \), and moment, \( M \), due to an avalanche. Here, a rectangular wall is regarded that is width enough
that a major portion of an avalanche will not flow horizontally around it and major parts of the avalanche are least laterally confined by neighboring flow.

In the determination of the impact load it is assumed that the parameters of the avalanche (speed, density, structure of the head part of the body, etc.) are known. Such parameters may be defined using numerical models of the flow in accordance with avalanche type or from field observations.

Figure 28 shows a schematic diagram of the impact pressure distribution on a wall due to an avalanche together with the coordinate system and notation used in the following description.

The following recommendations basically follow the proposal by Norem (1991), however some modification concerning coefficients are proposed. Swiss recommendations for the determination of impact forces on walls are described in Appendix F and compared with the recommendations given here in figures and tables below. Three flow regimes are distinguished for the determination of the impact force on a wall-like structures:

- dense flow
- fluidized flow (also referred to as saltation layer)
- suspension flow (powder part)
In addition, the force transmitted through the snowpack is included. **Not considered are static snow loads from the snowpack on the ground or previous avalanche deposits, which have to be considered independently.**

**Pressure transmitted through the snowpack**

Measurements from the full-scale test site Ryggfonn, Western Norway, show that avalanche force can be transferred through the snowpack on the ground (cf. Gauer and others, 2006). For simplicity, a linear pressure distribution will be assumed within snowpack, *i.e.*,

\[
p_s(z) = p_d \frac{z}{h_s} .
\]

(36)

where \( p_d \) is the dynamic pressure of the avalanche calculated at the lower boundary of flow (see (40) next paragraph). Thus, the force normal onto a rectangular wall exert by an avalanche through the snowpack is

\[
F_{sx} = \frac{bh_s}{2} p_d
\]

(37)

and the corresponding moment about the y-axis is

---

Figure 28: Schematic of the impact pressure distribution due to an avalanche on a wall.
\[ M_{sy} \approx b p_d \int_0^{z_h} \frac{z}{h_s} \, dz = \frac{2}{3} h_s F_{sx} \]  \hspace{1cm} (38)

The contribution to a total lift force is negligible, \textit{i.e.},

\[ F_{sz} \approx 0 \]  \hspace{1cm} (39)

Dense flow

The pressure within the dense flow is assumed to be uniformly distribute along the wall. This is certainly a simplification as there is most likely an increasing pressure with depth. On the other hand, a uniform pressure distribution gives an overestimate of the moment and so is more conservative. The pressure is assumed according to (33)

\[ p_d \approx p^+ \left( u^+ \right)^2 \left[ f(Fr^+) + \frac{1}{2 (Fr^+)^2} \right] \frac{h^+}{h^-} \]  \hspace{1cm} (40)

here, the factor \( f(Fr^+) \) accounts for effects due to compressibility of the material and other aspects of the assumed dynamics of the flow by the dam (see Figure 25). A good estimate might be \( f(Fr^+) \) equals 1.2. Reasonable estimates for \( h^-/h^+ \) are in the range of 3 to 8 for vertically unconfined settings. The contact area, \( A \), is equal to \( b H_e \), where \( b \) is the minimum of the avalanche width and the width of the wall, \textit{i.e.}, \( b = \min(W_a, W_{wall}) \), and \( H_e \) the minimum of the shock depth and the effective wall height, \textit{i.e.}, \( H_e = \min(h^-, H_{wall} - h_s) \). The height above ground of the upper boundary of the dense flow is

\[ z_{hd f} = z_{hs} + h^- \]  \hspace{1cm} (41)

and the height of the upper layer still effecting the wall

\[ z_{hd} = \min(z_{hd f}, H_{wall}) \]  \hspace{1cm} (42)

During the first instant of impact, peak pressure of serval times \( p_d \) may occur for durations, \( t_{imp} \), of order \( O(h^+/w) \), which is probably good estimate for the time needed by the pressure wave to reach the free surface. \( w \) is speed of the developing shock wave. This decreases with increasing \( Fr^+ \). For an example, see Figure 27 on the right panel, curve \( h^-/h^+ \) equals 1. A rough estimate based on experiments by Bachmann (1987) with snow blocks for \( 0.5 < Fr^+ < 3 \) might be given by

\[ \frac{p_{peak}}{p_d} = 3 \]  \hspace{1cm} (43)

Similar values are also observed by Schaarer and Salway (1980) in their measurements on full-scale avalanches at Rogers Pass. They mention 3.3 for small sized load cells (645 mm²) and 2.4 for large ones (6450 mm²). \( Fr^+ \) ranged between 6.2 and 8.5. They also noted that
their values are in agreement with other reported values which range from 2 to 5. For example, the reported values in (Kotlyakov and others, 1977) corresponds to a ratio of about 4.8.

The normal force onto the wall is

$$F_{dx} = \rho^+ (u^+)^2 \left[ f(Qr^+) + \frac{1}{2(Qr^+)^2} \right] \frac{h^+}{h^-} (z_{hd} - z_{hs}) b .$$ \hspace{1cm} (44)

The vertical component of the force can be approximated by

$$F_{dz} = c_1 F_{dx} .$$ \hspace{1cm} (45)

According to Voellmy (1955a), \(c_1\) is approximately between 0.3 and 0.5. The moment about the about the y-axis is

$$M_{sy} \approx \int_{z_{hs}}^{z_{hd}} z p_d d_z \approx \frac{z_{hs} + z_{hd}}{2} F_{dx} .$$ \hspace{1cm} (46)

Here, the contribution due to (45) is neglected. However, in the case that a vertical force acts on a balcony or ledge its contribution to the moment can be considerable and has to be accounted for.

**Fluidized layer**

Within the so-called fluidized layer (saltation layer), a decreasing dynamic pressure with increasing height is assumed. The following pressure distribution is assumed

$$p_{fl}(z) = p_{z_{fl}} - (p_{z_{fl}} - p_d) \left( \frac{z_{fl}}{z_{fl} - z_{hd}} \right)^{n_f} .$$ \hspace{1cm} (47)

Based on Norem (1991), the height of the fluidized layer is assumed to be

$$h_{fl}^- = c_e (0.1 s) u^+ ,$$ \hspace{1cm} (48)

where \(c_e\) is an expansion factor and accounts for the increase in flow height at impact; \(c_e = 3\) might be reasonable. *This expression and the chosen value of \(c_e\) have a large effect on the final result. This needs to be further investigated.* The height of the upper boundary of the fluidized layer (saltation layer) over ground is

$$z_{fl} = z_{hd} + h_{fl}^-$$ \hspace{1cm} (49)

and the effective upper limit on the wall is

$$z_{fl} = \min(z_{fl}, H_{wall}) .$$ \hspace{1cm} (50)

The pressure at the upper boundary is approximated by
where the effective density is set to $\rho_e = 15 \text{ kg m}^{-3}$. In this case, the contribution from the fluidized layer to the total force

$$F_{flx} = p_{zhfl} (z_{fl} - z_{hd}) - \frac{(p_{zhfl} - p_d)}{n_f + 1} \left( (z_{hfl} - z_{hd}) - \left( \frac{(z_{hfl} - z_{fl})^{n_f + 1}}{(z_{hfl} - z_{hd})^{n_f}} \right) \right)$$

and the vertical component is

$$F_{flz} = c_1 F_{flx} \ .$$

The moment about the y-axis is

$$M_{fly} = \left[ p_{zhfl} \left( \frac{z^2_{fl} - z^2_{hd}}{2} \right) - \frac{(p_{zhfl} - p_d)}{(z_{hfl} - z_{hd})^{n_f}} \left( \frac{z_{hfl}(z_{hfl} - z_{hd})^{n_f + 1}}{n_f + 1} - \left( \frac{(z_{hfl} - z_{hd})^{n_f + 2}}{n_f + 2} \right) \right) \right] b \ .$$

For the shape factor exponent, $n_f$, a value of 0.25 is recommended. Originally, Norem (1991)\(^1\) proposed a value of 4, which would give a rapid decrease in pressure above the dense flow within the fluidized layer (saltation layer), however measurements indicate that the decrease might be slower than proposed by (Norem, 1991). The choice of $n_f$ equals 0.25 is also more conservative. However, also this choice needs to be further investigated.

The pressure at the lower boundary, $p_d$, and at the upper boundary, $p_{zhfl}$, is given by equations (40) and (51), respectively.

If the avalanche is proceeded by an fast moving fluidized head, (40) to (42) with appropriate density $\rho^+$ should be used instead of (47) through (54). In this case, $p_{zhfl}$ is given by (40) and used as lower boundary for the powder part.

**Powder part**

Within the powder part, it is assumed that the dynamic pressure decreases rapidly with height, *i.e.*, 

$$p_p(z) = \max \left( p_{zhfl} \left( \frac{z_{hp} - z}{z_{hp} - z_{hfl}} \right)^3, p_a \right) \ .$$

\(^1\)There is a misprint in the original formula in (Norem, 1991, Eq. (9)).
The dynamic pressure at the lower boundary is given by (51) or (40), respectively. The pressure at the upper boundary is

\[ p_a = \frac{\rho_a u^+}{2}, \tag{56} \]

where the air density is approximately \(1.2 \text{ kg m}^{-3}\). The height of the snow cloud, \(h_p\), is assumed to depend on the travel distance along the track, \(l_{\text{track}}\), and is given by

\[ h_p = (10^{-5} \text{ s}^{-2}) l_{\text{track}} (u^+)^2. \tag{57} \]

It follows that

\[ z_{hp} = z_{hf} + h_p \tag{58} \]

and the effective upper limit on the wall

\[ z_p = \min(z_{hp}, H_{\text{wall}}). \tag{59} \]

The normal force is given by

\[ F_{px} \approx \frac{p_{zhf} l_{\text{track}}}{4} \left[ (z_{hp} - z_{hf}) - \frac{(z_{hp} - z_p)^4}{(z_{hp} - z_{hf})^3} \right] b, \tag{60} \]

the vertical force is set to

\[ F_{py} \approx 0, \tag{61} \]

and the corresponding moment about the y-axis is

\[ M_{py} \approx p_{zhf} \left[ \frac{z_{hp}(z_{hp} - z_{hf})}{4} - \frac{(z_{hp} - z_{hf})^2}{5} - \frac{(z_{hp} z_{hp} - z_p)^4}{4 (z_{hp} - z_{hf})^3} - \frac{(z_{hp} - z_p)^4}{5 (z_{hp} - z_{hf})^5} \right] b. \tag{62} \]

### 10.3 Example: Load on a wall

In the following, an example is given for the determination of the design force for a 15 m wide wall in the lower part of an avalanche track. The wall is assumed approximately 900 meter downstream from the starting zone. The input parameter are summarized in Table 1. No considerations about return periods are given.

Figure 29 depicts the pressure distribution according to the recommendation. Table 2 gives the calculated forces.
Table 1: Example input: Load on a wall

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
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<td>Height of the wall</td>
<td>$H_{wall}$ (m)</td>
<td>20</td>
</tr>
<tr>
<td>Width of the wall</td>
<td>$W_{wall}$ (m)</td>
<td>100</td>
</tr>
<tr>
<td>Distance along the track</td>
<td>$l_{track}$ (m)</td>
<td>900</td>
</tr>
<tr>
<td>Height of snowpack / deposits</td>
<td>$h_s$ (m)</td>
<td>1.5</td>
</tr>
<tr>
<td>Front velocity</td>
<td>$u^+$ (m s$^{-1}$)</td>
<td>25</td>
</tr>
<tr>
<td>Flow height (dense flow)</td>
<td>$h^+$ (m)</td>
<td>2</td>
</tr>
<tr>
<td>Density (dense flow)</td>
<td>$p^+$ (kg m$^{-3}$)</td>
<td>150</td>
</tr>
</tbody>
</table>

Table 2: Comparison of the calculated loads on a wall for the example according to the recommended approach and the Swiss recommendation. Dynamic forces and moments per meter length of the wall are given.

<table>
<thead>
<tr>
<th>Force</th>
<th>Recommend. (kN m$^{-1}$)</th>
<th>Swiss (kN m$^{-1}$)</th>
<th>Moment</th>
<th>Recommend. (kN m$^{-1}$)</th>
<th>Swiss (kN m$^{-1}$)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F'_{sx}$</td>
<td>39.8</td>
<td>–</td>
<td>$M'_{sy}$</td>
<td>39.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$F'_{dx}$</td>
<td>248.8</td>
<td>375</td>
<td>$M'_{dy}$</td>
<td>956.3</td>
<td>937.5</td>
<td></td>
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<td>$F'_{stau}$</td>
<td>–</td>
<td>1195.0</td>
<td>$M'_{stau}$</td>
<td>–</td>
<td>2986</td>
<td></td>
</tr>
<tr>
<td>$F'_{fix}$</td>
<td>325.6</td>
<td>–</td>
<td>$M'_{fix}$</td>
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<td></td>
<td></td>
</tr>
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<td>$F'_{px}$</td>
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<td>–</td>
<td>$M'_{px}$</td>
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<td></td>
</tr>
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<td>$F'_{totx}$</td>
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<td>1570.6</td>
<td>$M'_{toty}$</td>
<td>4267.0</td>
<td>3924</td>
<td>$\lambda = 2.5$</td>
</tr>
<tr>
<td>$F'_{totz}$</td>
<td>172.3</td>
<td>470.9</td>
<td></td>
<td></td>
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</tbody>
</table>
Figure 29: Distribution of the dynamic pressure for the example according to the recommendation. For impact pressure 3 times $p_d$ is used. Also shown is a comparison with Swiss recommendations.
11 Loads on masts and mast-like obstacles

Authors . . .

Impact forces by snow avalanches on narrow obstacles are important for the design of many constructions in avalanche prone terrain, such as masts of electrical power lines, ski lifts, and cable cars. The design of such objects are mostly outside the scope of these guidelines, but the design of some retarding objects, such as short concrete walls to break up the flow of the avalanche, is in principle similar to the design of other narrow obstacles against dynamics loads. This section, therefore, presents design recommendations for dynamic loads due to snow avalanches on narrow obstacles.

An important question in connection with such impact forces on high obstacles that extend through the flow is how they depend on the width and cross-sectional shape of the obstacle for a given velocity and thickness of the oncoming flow. Widely used engineering guidelines imply that a significant fraction of the dynamic pressure of the avalanche impacts the obstacle simultaneously over a substantial part of the full height range corresponding to the run-up of the avalanche.

11.1 Forces on immersed bodies

The drag force, \( F_D \), on a body submerged (or partly immersed) in a flow can be viewed as having two components: a pressure drag, \( F_p \), and a friction drag, \( F_f \) (e.g. Franzini and Finnimore, 1997, Ch. 9). The pressure drag is also referred to as form drag because it depends largely on the form or shape of the immersed body. It is equal to the integral of all pressure components in the direction of motion exerted on the surface of the body. Commonly, the pressure drag is related to the dynamic pressure, \( \rho U_\infty^2/2 \), acting on the projected area, \( A \), of the body normal to the flow. Thus,

\[
F_p = C_p A \frac{\rho U_\infty^2}{2},
\]

(63)

where \( \rho \) is the density of the flow and \( U_\infty \) the flow velocity upstream of the body. The coefficient \( C_p \) depends on the geometry of the body and factors that define the flow like the Reynolds number, \( \text{Re} \), or the Froude number, \( \text{Fr} \).

The friction drag, along a body is equal to the integral of the shear stress along the surface of the body in the direction of motion. Similar to the pressure drag, the friction drag, also referred to as skin friction, is commonly expressed as function of the dynamic pressure. Thus,

\[
F_f = C_f B L \frac{\rho U_\infty^2}{2},
\]

(64)

where \( L \) is the length of the surface parallel to the flow and \( H \) the width of the surface. Similarly to \( C_p \), \( C_f \) depends on the geometry of the body and factors that define the flow. (64)
gives only the drag on one side of a immersed body. Hence, the total frictional component of 
the drag force is twice that if two sides of the body are flowed around.

The total drag force on a body is the sum of both, the friction drag and the pressure drag:

\[ F_D = F_p + F_f \]  \hspace{1cm} (65)

However, it is customary to express the total drag on a body by a single equation

\[ F_D = C_D A \frac{\rho U_\infty^2}{2} \]  \hspace{1cm} (66)

Again, \( \rho \) is the density of the fluid, \( U_\infty \) the upstream flow velocity, and \( A \) is the projected area of the obstacle perpendicular to the flow. Thus, the drag factor, \( C_D \), describes the combined

Figure 30: A mast built for studying impact forces on electrical power lines (left), and an 
instrument tower (right) that has just been hit by an avalanche in the Ryggfonn avalanche path in Western Norway (Photos by NGI). The power line mast was broken several times by avalanches during the investigation period.
action of dynamic pressure and friction on the body. Consequently, it is a function of the flow regime and depends on factors like the Reynolds number, $\text{Re}$, Froude number, $\text{Fr}$, and the geometry of the obstacle. If one considers a granular flow, $C_D$ might also depend on the particle concentration, size, and restitution coefficient as well as on the ratio of particle diameter to size of the obstacle. Generally, the drag coefficients have to be determined experimentally.

Figure 31: Fluid “vacuum” behind partly immersed obstacles in water (Photos by P. Gauer).

In the case of a free surface flow, *i.e.*, the obstacle is only partly immersed, a fluid free zone, a "vacuum", can develop behind the obstacle (cf. Fig. 31). The depth and extend of this zone depends on the flow velocity and properties of the flow. In addition to the dynamic drag, a unbalanced static load is imparted onto the obstacle. The additional quasi-static load is given by

$$F_{\text{static}} = W \int_{z_{h2}}^{z_{h1}} \Delta \rho \ g \ (z_{h1} - z) \ dz = \Delta \rho \ g \ W \left(\frac{z_{h1} - z_{h2}}{2}\right)^2,$$

(67)

where $\Delta \rho$ is the difference between the fluid density $\rho$ and the density, $\rho_a$, of air. $W$ is the obstacle width across the flow. $z_{h1}$ and $z_{h2}$ are the flow depths upstream and downstream of the obstacle, respectively.

The total drag force can then be rewritten as

$$F_D^* = \left(C_D + \frac{f_s(h_1/h_{\infty}, h_2/h_{\infty}, W/h_{\infty}, \Delta \rho/\rho)}{\text{Fr}_{\infty}^2} \right) A_{\infty} \frac{\rho U_{\infty}^2}{2},$$

(68)
where the Froude number, \( Fr_\infty = \frac{U_\infty}{\sqrt{g h_\infty}} \), \( h_\infty \) is the upstream flow depth, the area, \( A_\infty = W h_\infty \), is the cross-sectional area of the upstream flow, \( h_1 \) the run-up height upstream of the obstacle, and \( h_2 \) the flow depth immediately behind the obstacle. It is reasonable to assume that the function \( f_h(h_1/h_\infty, h_2/h_\infty, W/h_\infty, \Delta \rho/rho) \) is also a function of another dimensionless groups, which may involve \( U_\infty \). The contribution from the quasi-static component might become negligible for \( Fr_\infty \gg 1 \). However, for \( Fr_\infty \) lower than one, the quasi-static load may dominate the drag. This is for example the case during snow creep and gliding (see Section 12). In the case of, e.g., water the “vacuum” zone diminishes as the flowing velocity goes to zero and the static force upstream and downstream of the obstacle balance each other. In contrast to this, avalanche snow has a cohesive strength that might prevent the "vacuum" behind the obstacle to close causing a static load from avalanche deposits on the obstacle even after the avalanche passage.

On the other hand, the overall drag on a small obstacle might be reduced in a free surface flow compared to a confined one due to the flexibility given to the stream to flow more easily around the obstacle.
11.2 Dynamic drag coefficients

From fluid mechanics, it is well known that \( C_D \) can vary by several orders of magnitude depending on the flow regime. There is only a limited set of reliable measurements for avalanches available. Those, however, indicate that \( C_D \) values can vary considerably depending on the stage of the avalanche flow, \textit{i.e.}, whether the flow is more or less fluidized or frictional dominated; if the avalanche is a dry type or wet, or even slush like.

Despite this, the value used for a rectangular cross section in dry flow avalanches is commonly set to 2, \textit{cf.} (Mellor, 1968). This holds true for the powder part as well as for the dense part, even if not explicitly stated. Norem (1991) sets up equations for computing the coefficient, connecting it with the Reynolds number, \( \text{Re} \). As he notes, even though \( \text{Re} \) is often in the range of 4 to 1000, it can be in the range of 0.1 to 4 when snow avalanches are coming to a halt in their run-out area. If \( \text{Re} \) is expected to range from 4 to 1000, the coefficient, \( C_D \), may be expected to lie in the range of 1 to 4. Finally, Norem proposed a value of 2.5 for dry snow avalanches and 6.3 for wet snow avalanches based on impact pressure measurements from the Ryggfonn test site. Salm and others (1990b) recommend a \( C_D \) of 2 for small rectangular obstacles (\( C_D = 1 \) for cylindrical ones) in combination with a density of 300 kg m\(^{-3}\).

The authors are not aware of any systematic investigation of drag factors in avalanches. Some considerations can be found in (Bozhinskiy and Losev, 1998, Chapter 5.6). Schäerer and Salway (1980) reported values ranging from 2 to 3.4 for the front part and from 0.86 to 0.96 for the body (values are adapted to the form of Eq. (66)). However, they related those values to the front velocity, which probably overestimates the velocity within the body and so causes significant underestimation of the \( C_D \) values. Also McClung and Schäerer (1985) provided some considerations.

Pfeiff and Hopfinger (1986) conducted laboratory experiments with dense suspensions of polystyrene particles in water. They found good agreement with the classical correlation \( C_D(\text{Re}) \) that is valid in Newtonian fluids, if they calculated the Reynolds number using the apparent viscosity of the suspension. Gauer and Kvalstad (unpublished) used numerical simulations and experimental results to determine the drag coefficient for mud flows hitting a cylinder. They obtained the relationship \( C_D = 24/\text{Re} + 1 \) with \( \text{Re} = \rho \ U_\infty^2 / k \), where \( k \) is the yield stress of the mud in simple shear. This means there are two contributions to the drag force, one independent of the velocity and the other growing with the square of the velocity. Pazwash and Robertson (1975) gained similar results for the force on bodies immersed in a Bingham fluid doing experiments with discs, spheres, an ellipsoid, and flat plates. They propose the formulation \( C_D = C_{D0} + k_p \text{He}/\text{Re} \), where \( C_{D0} \) is a constant depending on the form of the body, \( k_p \) is a plasticity factor also depending on the form of the body, \( \text{He} (= \rho L^2 / \mu B^2) \) is the Hedstrom number and \( \text{Re} (= \rho U_\infty L / \mu B) \) their Reynolds number. \( L \) is a length scale and \( k \) and \( \mu_B \) the yield stress and the Bingham viscosity, respectively.

Chehata and others (2003) conducted experiments with dense granular flows around an immersed cylinder in a confined setting and found that \( C_D \propto \text{Fr}^{-2} \), resulting in a velocity independent drag force. The Froude number was defined by \( \text{Fr} = U_\infty / \sqrt{g(D+d)} \), where
$D$ is the cylinder diameter and $d$ the particle diameter. The velocities in their experiments were less than 1 m s$^{-1}$ and $F_r$ was less than 1. For similar conditions but with a free surface, Wieghardt (1975) made experiments moving rods in sand. In his case, the drag factor might be approximated by $C_D \approx 24/5 Fr^{-2} \sqrt{h/D}$, where $h$ is the flow height and $F_r = U_\infty/\sqrt{gh}$.

Wassgren and others (2003) performed numerical simulations of dilute granular flows around an immersed cylinder in a confined setting. They found that $C_D$ increases with increasing Knudsen number (ratio between the upstream particle free path length to the cylinder diameter; $Kn = \pi d/(8c_\infty D)$, where $c_\infty$ is the upstream particle concentration, ranging from 0.08 to 0.3 in their simulations) and decreases with increasing upstream Mach number. For both cases, $C_D$ reaches an asymptote. Beside this, they conclude that the drag coefficient decreases with decreasing restitution coefficient, $e$, of the particles. Taking parameters that might be relevant in dilute dry avalanches ($D = 0.6$ m, $d \approx 0.05$ m, $c_\infty \approx 0.2$, $e \approx 0.1$–0.3) $C_D$ would vary only between 1 and 3 so that 2 seems to be a reasonable approximation for this flow regime.

Hauksson and others (2006) found rather low $C_D$ values ranging around 0.5 for cylindrical obstacles and 0.8 for rectangular ones in their laboratory experiments. They used small granular material (glass beads) in a free surface flow at a Froude number of approximately 13 and upstream volume fraction of approximately 0.55 ($Kn$ numbers between $8 \cdot 10^{-4}$ and $3 \cdot 10^{-3}$). There is also a slight difference in their experimental setup in that they measured the total force on the obstacle in the free surface. If one plots their results according to (68) (see Fig. 33), one finds

$$\frac{2L}{A_\infty \rho_\infty u_\infty} = C^*_D \approx \left(1.02 + \frac{1}{Fr^*_\infty} \left(1 - 0.26 \frac{(h_1)^2 - (\Delta h)^2}{h^2_\infty}\right)\right).$$

(69)

Here, $L$ is the measured load and $Fr^*_\infty$ the slope corrected Froude number. $C^*_D$ is a combined drag for dynamic and static loading. $\Delta h = \max(h_1 - h_{obs}, 0)$, which is the difference by which the climbing height $h_1$ exceeded the obstacle height in several cases. For those cases, the experiment number is set in parenthesis in the figure. The difference is used to correct the static load that actually was imparted on the load, i.e.,

$$F_{sl-cor} = \Delta \rho g W \left(\frac{h_1^2 - (\Delta h)^2}{2}\right).$$

(70)

Using (69) in (68) would also predict the approximate static load as $u_\infty$ goes to zero. Note, for their configuration $h_2$ has to be regraded always as zero. Figure 33 also shows the observed climbing height $h_1 - h_\infty$ normalized by $u_\infty^2/(2g*)$, where $(g* = g \cos \psi)$ and $\psi$ the slope angle.

If one now assumes that $(h_1 - h_\infty) \sim u_\infty^2$, then $C^*_D$ is function of $u_\infty^2$, rather than a function of $Fr^*_\infty$. In the fitting above no distinguishing was made between the geometry of the obstacle, which is a simplification. However, the amount of data is too sparse to do better and to be conclusive. On the other hand, Hauksson and others (2006) reports laboratory experiments with impact forces on debris flow breakers which could be interpreted similarly.

Yakimov and others (1979) measured forces acting on wedge-shaped obstacles located in an avalanche path (cf. Eglit, 2005). The wedges had angles of $\alpha = 60^\circ, 90^\circ, 120^\circ, 150^\circ,$
where the angle between the flow velocity and the wedge surfaces being 0.5 $\alpha$. The flow depth was equal to the height of the wedge $H$, or (in some experiments) larger than $H$. The normal component of the force on the surface was measured as a function of time. Yakimov and others proposed the following empirical formulae for the drag factor

$$C_{D_{max}} = 0.025\alpha; \quad C_{D_{\infty}} = 1.2 \quad \text{if} \quad \alpha > 90^\circ.$$  \hspace{1cm} (71)

$C_{D_{max}}$ corresponds to the drag factor at maximum force and $C_{D_{\infty}}$ to the one during stationary flow. According to (71) both are independent of the Froude number nor on the flow width.

It should be noted that it is not always is distinguished between the static component ant the dynamic drag factor in the cited experiments above. Hence, the cited drag factors may rather represent a combined one, i.e.,

$$C_D^* = \left( C_D + \frac{f_s(h_1/h_{\infty}, h_2/h_{\infty}, W/h_{\infty}, \Delta\rho)}{Fr^2} \right)$$  \hspace{1cm} (72)

### 11.3 Determining design loads

The construction of a mast-like structure in an avalanche prone area requires an assessment of reasonable design loads, i.e., the assessment of the total maximum force, $F_m$, due to an avalanche. Actually, even more important is the assessment of the maximum moment about the foot point of the mast or about the footing of the foundation, respectively.

In the determination of the impact load it is assumed that the parameters of the avalanche (speed, density, structure of the head part of the body, etc.) are known. All such parameters
may be defined using numerical models of motion in accord with avalanche type or from field observations.

Figure 34 shows schematic diagram of the impact pressure distribution due to an avalanche on a mast. Here, \( p_s \) denotes the pressure transmitted from the avalanche through the snowpack on ground and \( h_s = z_{hs} \) is the snow depth; \( p_d \) is the impact (dynamic) pressure due to the dense part and \( h_d = z_{hd} - z_{hs} \) the flow height of the dense layer. Similarly, \( p_{fl} \) is the impact (dynamic) pressure due a fluidized layer and \( h_{fl} = z_{hfl} - z_{hd} \) is the height of this layer. Finally, \( p_p \) is the dynamic pressure within the suspension layer (powder part) and \( h_p = z_{hp} - z_{hfl} \) the height of the suspension layer. One should keep in mind that all pressures might vary with height.

![Figure 34: Schematic diagram of the impact pressure distribution due to an avalanche on a mast-like structure.](image)

Total force in \( x \) direction can be given by
\[ F_{\text{max}} = \int_{0}^{z_{hs}} A(z) p_s(z) \, dz + \int_{z_{hs}}^{z_{hd}} C_D(z) A(z) p_d(z) \, dz \\
+ \int_{z_{hd}}^{z_{hfl}} C_D(z) A(z) p_{fl}(z) \, dz + \int_{z_{hfl}}^{z_{hp}} C_D(z) A(z) p_p(z) \, dz. \] (73)

As mentioned in 11.2, the drag coefficient \( C_D \) depends on the flow. Hence, it might not be constant within the whole avalanche and vary may from layer to layer. The moment, \( M \), about the foot point of the mast is given by

\[ M = \int_{0}^{z_{hs}} z p_s(z) A(z) \, dz + \int_{z_{hs}}^{z_{hd}} z C_D(z) A(z) p_d(z) \, dz \\
+ \int_{z_{hd}}^{z_{hfl}} z C_D(z) A(z) p_{fl}(z) \, dz + \int_{z_{hfl}}^{z_{hp}} z C_D(z) A(z) p_p(z) \, dz. \] (74)

The following recommendation basically follows the proposal by Norem (1991), as in the previous section about loads on walls, however, some modification concerning coefficients are proposed. Swiss recommendations are, as for the wall loads, described in Appendix F and compared with the recommendations given here in figures and tables below. Three avalanche flow regimes are distinguished for the determination of the force on a small obstacles:

- dense flow
- fluidized flow (also referred to as saltation layer)
- suspension flow (powder part)

In addition the force transmitted through the snowpack is included. Not considered are static snow loads from the snowpack on the ground or previous avalanche deposits, which have to be considered independently.

Full-scale experiments (Gauer and others, 2006) show that the heights pressures not necessarily occur during the passage of the front. This has to be taken into account by a choice of reasonable velocity and density.

**Pressure transmitted through the snowpack**

Measurements from the full-scale test site Ryggfønn, Western Norway, show that avalanche force can be transferred through the snowpack on the ground (cf. Gauer and others, 2006). For simplicity, a linear pressure distribution will be assumed within snowpack, \( i.e., \)

\[ p_s(z) = p_d \frac{z}{h_s}, \] (75)
Figure 35: Schematic diagram of the impact pressure distribution due to an avalanche on a mast-like structure according to the recommendations.

where \( p_d \) is the dynamic pressure of the avalanche calculated at the lower boundary of flow (see 78). Hence, the force on the mast exerted by an avalanche through the snowpack is

\[
F_{sx} = \frac{W h_s}{2} p_d ,
\]

(76)

where \( W \) is the width across the flow. The corresponding moment about the y-axis is

\[
M_{sy} \approx \frac{2}{3} W h_s F_{sx} .
\]

(77)

**Dense flow**

The dynamic pressure within the dense flow is assumed to be

\[
p_d = \rho_d \frac{u_f^2}{2} ,
\]

(78)
where $\rho_d$ is set to a rather high value of 300 kg m$^{-3}$ for safety reasons. The height above ground of the upper boundary of the dense flow is

$$z_{hdf} = h_d + z_{hs},$$

(79)

and the effective height effecting the mast

$$z_{hd} = \min(z_{hdf}, H_{mast}).$$

(80)

Then, the force created by the dense flow is

$$F_{dx} = C_{Deff} W (z_{hd} - z_{hs}) p_d.$$

(81)

The corresponding moment about the y-axis is

$$M_{dy} \approx \frac{z_{hs} + z_{hd}}{2} F_{dx}.$$

(82)

The effective drag coefficient $C_{Deff}$ is given by

$$C_{Deff} = C_D + \frac{f_s(h_d)}{2Fr^2}.$$

(83)

Table 3 gives recommendations for the dynamic drag coefficient $C_D$. Measurements on a wet snow avalanche (cf. Gauer and others, 2006) indicate that $f_s(h_d)$ might be approximated be

$$f_s(h_d) \approx \frac{48}{5} \sqrt{\frac{h_d}{W}},$$

(84)

which is similar to the proposed value by Wieghardt (1975) and also comparable to loads due to snow-creep and gliding (see Section 12).

**Fluidized layer**

Within the so-called fluidized layer (saltation layer) a decreasing dynamic pressure with increasing height is assumed. The following pressure distribution is assumed

$$p_{fl}(z) = p_{zhl} - (p_{zhl} - p_d) \left( \frac{z_{zhl} - z}{z_{zhl} - z_{hd}} \right)^{nf}.$$

(85)

The pressure at the lower boundary, $p_d$, is given by equations (78) and the one at the upper boundary is set to

$$p_{zhl} = \rho_e \frac{u_f^2}{2},$$

(86)

where the effective density is assumed to be $\rho_e = 15$ kg m$^{-3}$. The height of the fluidized layer is set to
\[ h_{fl} = (0.1 \, \text{s}) \, u_f \]  
\[ (87) \]

It follows that
\[ z_{hfl} = z_{hd} + h_{fl} \]  
\[ (88) \]

and the effective upper limit on the mast
\[ z_{fl} = \min(z_{hfl}, H_{mast}) \]  
\[ (89) \]

In this case, the contribution from the fluidized layer to the total force is
\[ F_{f1x} = C_D W \left[ p_{zhfl} (z_{fl} - z_{hd}) - \frac{(p_{zhfl} - p_d)}{n_f + 1} \left( \frac{z_{hfl} - z_{fl}}{n_f + 1} \right) - \frac{(z_{hfl} - z_{fl})^{n_f + 1}}{n_f + 1} \right] \]  
\[ (90) \]

and the corresponding moment about the y-axis is
\[ M_{fly} = C_D W \left[ \frac{(z_{fl}^2 - z_{hd}^2)}{2} - \frac{(p_{zhfl} - p_d)}{n_f + 1} \left( \frac{z_{hfl} (z_{hfl} - z_{hd})^{n_f + 1}}{n_f + 1} \right) - \frac{(z_{hfl} - z_{fl})^{n_f + 2}}{n_f + 2} \right] \]  
\[ (91) \]

For the shape factor exponent, \( n_f \), a value of 0.25 is recommended. Originally, Norem (1991)\(^2\) proposed a value of 4, which would give a rapid decrease in pressure of above the dense flow within the fluidized layer (saltation layer). However, measurements indicate that the decrease might be more slowly than proposed by Norem (1991). The choice of \( n_f \) equals 0.25 is hence also more conservative.

If the avalanche is proceeded by an fast moving fluidized head, (78) to (80) with appropriated density \( \rho \) should be used instead of (85) through (91). In this case, \( p_{zhfl} \) is given by (78) and used as lower boundary for the powder part.

**Powder part**

Within the powder part, it is assumed that the dynamic pressure rapidly decrease with height, \( i.e. \),
\[ p_p(z) = \max \left( p_{zhfl} \left( \frac{z_{hp} - z}{z_{hp} - z_{hfl}} \right)^3, p_a \right) \]  
\[ (92) \]

\(^2\)There is a misprint in the original formula in (Norem, 1991, Eq. (9)).
The dynamic pressure at the lower boundary, $p_{zhfl}$, is given in Equation (86) or (78), respectively.

$$p_a = \rho_a \frac{u^2_f}{2}$$  \hspace{1cm} (93)

and the air density is approximately $1.2 \text{ kg m}^{-3}$. $u_f$ is the front velocity of the avalanche. The height of the snow cloud $h_p$ is assumed to depend on the travel distance along the track, $l_{track}$, and is given by

$$h_p = \left(10^{-5} \text{ s}^{-2}\right) l_{track} u^2_f.$$  \hspace{1cm} (94)

It follows that

$$z_{hp} = z_{hfl} + h_p$$  \hspace{1cm} (95)

and the effective upper limit on the mast

$$z_p = \min(z_{hp}, H_{\text{mast}}).$$  \hspace{1cm} (96)

The force due to the powder part is then

$$F_{px} \approx C_D W p_{zhfl}\left[ \frac{z_{hp} - z_{hfl}}{4} - \frac{(z_{hp} - z_p)^4}{5(z_{hp} - z_{hfl})^3} \right]$$  \hspace{1cm} (97)

and the corresponding moment about the foot point is

$$M_{fly} \approx C_D W p_{zhfl}\left[ \frac{z_{hp} (z_{hp} - z_{hfl})}{4} - \frac{(z_{hp} - z_{hfl})^2}{5} - \frac{(z_{hp} - z_p)^4}{4(z_{hp} - z_{hfl})^3} - \frac{(z_{hp} - z_p)^4}{5(z_{hp} - z_{hfl})^5} \right].$$  \hspace{1cm} (98)

**Recommended drag coefficient $C_D$**

As discussed in Section 11.2, there is a uncertainty for the choice of the right dynamic drag coefficient, $C_D$, for avalanche flows. Table 3 gives an overview of recommended values for $C_D$ for design purposes. **For safety reason, a dense flow density, $\rho_d$, of $300 \text{ kg m}^{-3}$ is recommended.**

**11.4 Example: Load on mast**

In the following, an example is given for the determination of the design force for a 1.5 m diameter mast in an avalanche track. The mast is assumed approximately 900 meter downstream from the starting zone. The further input parameter are summarized in Table 4. The
Table 3: Recommended drag coefficients $C_D$ for various geometries.

<table>
<thead>
<tr>
<th>Flow regime</th>
<th>Obstacle form</th>
<th>$C_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>dry</td>
</tr>
<tr>
<td>Through snowpack</td>
<td>no distinction</td>
<td>1.0</td>
</tr>
<tr>
<td>Dense flow</td>
<td>⬜</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>△</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>□</td>
<td>2.0</td>
</tr>
<tr>
<td>Fluidized layer</td>
<td>⬜</td>
<td>1.0</td>
</tr>
<tr>
<td>(Saltation)</td>
<td>△</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>□</td>
<td>2.0</td>
</tr>
<tr>
<td>Powder Part</td>
<td>⬜</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>△</td>
<td>1.2</td>
</tr>
<tr>
<td></td>
<td>□</td>
<td>1.5</td>
</tr>
</tbody>
</table>

Table 4: Example input: Load on a mast

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
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<td>Diameter of the round mast</td>
<td>$W$</td>
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</tr>
<tr>
<td>Height of the round mast</td>
<td>$H_{\text{mast}}$</td>
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</tr>
<tr>
<td>Distance along the track</td>
<td>$l_{\text{track}}$</td>
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</tr>
<tr>
<td>Height of snowpack</td>
<td>$h_s$</td>
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</tr>
<tr>
<td>Front velocity</td>
<td>$u_f$</td>
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</tr>
<tr>
<td>Density (dense flow)</td>
<td>$\rho_d$</td>
<td>300</td>
</tr>
<tr>
<td>Flow height dense flow</td>
<td>$h_d$</td>
<td>2</td>
</tr>
</tbody>
</table>

values are regarded as typical for the path, no extreme values. No consideration about return periods are done. The example gives also an comparison to the recommendation based on (Gruber and others, 1999) (see Appendix F.2).

Figure 36 depicts the pressure distribution according to recommendation proposed here and compares it with the Swiss recommendation (see F.2). Table 5 gives the calculated forces for both cases.
Figure 36: Distribution of the dynamic pressure on a mast for the example according to recommended approach and the Swiss recommendation

Table 5: Comparison of the calculated loads on a mast for the example according to recommended approach and the Swiss recommendation.

<table>
<thead>
<tr>
<th>Force</th>
<th>Recommend. (kN)</th>
<th>Swiss (kN)</th>
<th>Moment</th>
<th>Recommend. (kNm)</th>
<th>Swiss (kNm)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_S$</td>
<td>152</td>
<td>0</td>
<td>$M_S$</td>
<td>152</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>$F_D$</td>
<td>608</td>
<td>405</td>
<td>$M_D$</td>
<td>1519</td>
<td>1012</td>
<td></td>
</tr>
<tr>
<td>$F_{stau}$</td>
<td>–</td>
<td>1301</td>
<td>$M_{stau}$</td>
<td>–</td>
<td>8602</td>
<td>$\lambda = 2.5$</td>
</tr>
<tr>
<td>$F_{fl}$</td>
<td>492</td>
<td>–</td>
<td>$M_{fl}$</td>
<td>2383</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>$F_p$</td>
<td>21</td>
<td>–</td>
<td>$M_p$</td>
<td>432</td>
<td>–</td>
<td></td>
</tr>
<tr>
<td>$F_{tot}$</td>
<td>1272</td>
<td>1706</td>
<td>$M_{tot}$</td>
<td>4487</td>
<td>9615</td>
<td></td>
</tr>
</tbody>
</table>
12 Loads due to snow pressure

Static loads due to snow pressure were originally not within the scope of these guidelines. However, such loads are often important for the design of narrow obstacles, which are the subject of the preceding section. For completeness, it is therefore of interest to include a section on static snow pressure, especially because loads due to static snow pressure can exceed dynamic loads due to avalanche impacts in areas with an abundant snow cover. The knowledge of snow pressure on narrow structures, such as masts of electrical power lines, ski lifts or cableways, is still limited and the recommendations for design loads are based on an empirical formulation.

12.1 Static snow pressure

As the snowpack moves slowly and continually down slope it generates forces onto obstacles parallel and perpendicular to the slope. Two types of movement of the snowpack can be distinguished: snow creep and snow glide (see Figure 37). The gliding velocity $u_0$ can vary in a wide range from zero to several meter per day. Snow creep ($v$) is the resultant of vertical settlement ($w$) of the snow cover and internal shear deformation parallel to the slope ($u$). Typical creep rates are mm to cm per day. At the ground the snow creep is zero.

During the downward motion the snow causes a pressure on any obstacle. The snow pressure depends mainly on the snow depth, snow density, slope angle, gliding factor and efficiency factor. The efficiency factor accounts for extension of the influence zone, which is depends on the strength of the snowpack and can be much larger then the obstacle size. The gliding factor is a measure for the speed of motion of the snowpack. Higher speeds give the snowpack less time to relax stress around an obstacle and so cause higher loads.

12.2 Determining design loads

The following recommendation follows the approach by Margreth (2006) based on the Swiss Guidelines (1990). The snow load per unit meter length due to snow creep and gliding on a mast-like obstacle is

$$ S_{N,M}' = \frac{\rho_s \ h_s^2 \ g}{1000 \ 2 \ K \ N \ \eta_F} \frac{W}{D} \quad \text{(in kN m$^{-1}$)} \quad (99) $$

The moment per unit meter length is about the foot point is

$$ M_{N,M}' = \frac{h_s^2}{2} S_{N,M}' \quad \text{(in kNm m$^{-1}$)} \quad (100) $$
To get the total snow pressure $S_{N,M}$, $S'_{N,M}$ has to be multiplied by the snow thickness, $D$. Similarly, $M'_{N,M}$ has to be multiplied by $D$. $\rho_s$ is the density of the snowpack in kg m$^{-3}$, $g$ the acceleration due to gravity, and $h_s$ the vertical snow depth. The creep factor, $K$, depends on the snow density and the slope angle, $\psi$, and is approximate by (cf. Swiss Guidelines (1990))

$$K = \left(2.5 \left(\frac{\rho_s}{1000}\right)^3 - 1.86 \left(\frac{\rho_s}{1000}\right)^2 + 1.06 \left(\frac{\rho_s}{1000}\right) + 0.54\right) \sin(2\psi).$$ (101)

The glide factor is given in Table 6 according to the Swiss Guidelines (1990). The efficiency $\eta_F$ is defined in relation to the snow thickness measured perpendicular to the ground, $D (= h_s \cos \psi)$ and the width of the structure $W$. It accounts for end-effect force, which are higher for small obstacles relation to those on larger ones as the influence width of a small obstacle is much larger compared to the width of the object itself mainly because of the three-dimensional viscous flow of the snow pack around the object.

$$\eta_F = 1 + c \frac{D}{W},$$ (102)

where additional factor $c$ accounts for the intensity of snow gliding and the snow depth. A recommendation for Switzerland can be found in Table 7.

---

**Figure 37:** Schematic diagram of the creep and glide movement of the snowpack and snow pressure acting on a mast.
Table 6: Gliding factor, $N$, in relation of the ground classification according to the Swiss Guidelines (1990)

<table>
<thead>
<tr>
<th>Ground classification</th>
<th>Gliding factor N</th>
<th>Gliding intensity:</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Slope exposition:</td>
<td></td>
</tr>
<tr>
<td></td>
<td>WNW-N-ENE</td>
<td>ENE-S-WNW</td>
</tr>
<tr>
<td>I</td>
<td>Big boulders, rocks $&gt;$ 0.3 m</td>
<td>1.2</td>
</tr>
<tr>
<td>II</td>
<td>Large bushes $&gt;$ 1 m, bumps, mounds $&gt;$ 0.5 m</td>
<td>1.6</td>
</tr>
<tr>
<td>III</td>
<td>Short grass</td>
<td>2.0</td>
</tr>
<tr>
<td></td>
<td>- Bushes $&lt;$ 1 m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Fine rubble alternating with grass and small shrubs</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Grass with indistinct cow trails</td>
<td></td>
</tr>
<tr>
<td>IV</td>
<td>Smooth long-bladed grass</td>
<td>2.6</td>
</tr>
<tr>
<td></td>
<td>- Smooth rock plates with stratification planes parallel to the slope</td>
<td></td>
</tr>
<tr>
<td></td>
<td>- Swampy depressions</td>
<td></td>
</tr>
</tbody>
</table>

Table 7: The c-factor for the calculation of the efficiency $\eta_F$ in relation of the snow gliding intensity and the slope exposition (cf. Margreth, 2006).

<table>
<thead>
<tr>
<th>Gliding intensity and situation</th>
<th>Ground classification</th>
<th>c-factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Small</td>
<td>Class I-III</td>
<td>0.6</td>
</tr>
<tr>
<td>Medium</td>
<td>Class IV</td>
<td>1.0</td>
</tr>
<tr>
<td>Strong</td>
<td>Class IV</td>
<td>2.0</td>
</tr>
<tr>
<td>Strong</td>
<td>Class IV</td>
<td>6.0</td>
</tr>
<tr>
<td>Extreme and big snow depth ($&gt;2$-3 m)</td>
<td>Class IV</td>
<td>2.0</td>
</tr>
<tr>
<td>Extreme and small snow depth ($&lt;2$-3 m)</td>
<td>Class IV</td>
<td>6.0</td>
</tr>
</tbody>
</table>
12.3 Example: Snow-creep load

The following example shows the determination of the design force for a 1.5 m diameter mast in an 30° slope. The further input parameter are summarized in Table 8. The example gives also an comparison to Larsen (1998) who based his recommendations on experiments on snow creep loads on two masts with different diameter at the Norwegian test-site Fonnbu (see Appendix F.3).

Table 8: Example of snow-creep load calculation according to Swiss recommendations (cf. Margreth, 2006) and according to Larsen (1998). The first case according to the Swiss Guidlines corresponds to situation with low gliding, the second to extreme glide conditions.

<table>
<thead>
<tr>
<th>Input parameter</th>
<th>Symbol</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of the round mast</td>
<td>$W$ (m)</td>
<td>1.5</td>
</tr>
<tr>
<td>Slope angle</td>
<td>$\psi$ (°)</td>
<td>30</td>
</tr>
<tr>
<td>Snowpack density</td>
<td>$\rho$ (kg m$^{-3}$)</td>
<td>300</td>
</tr>
<tr>
<td>Snow depth</td>
<td>$h_s$ (m)</td>
<td>1.5</td>
</tr>
<tr>
<td>Snow thickness</td>
<td>$D$ (m)</td>
<td>1.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Model parameter</th>
<th>Symbol</th>
<th>Value Swiss</th>
<th>Value Larsen</th>
</tr>
</thead>
<tbody>
<tr>
<td>Creep factor</td>
<td>$K$</td>
<td>0.66</td>
<td>0.66</td>
</tr>
<tr>
<td>Gliding factor</td>
<td>$N$</td>
<td>1.2</td>
<td>2.6</td>
</tr>
<tr>
<td>c-factor</td>
<td>$c$</td>
<td>0.6</td>
<td>6</td>
</tr>
<tr>
<td>Efficiency factor</td>
<td>$\eta_F$</td>
<td>1.52</td>
<td>6.2</td>
</tr>
<tr>
<td>Coefficient Factor</td>
<td>$C_L$</td>
<td>1.52</td>
<td>1.69</td>
</tr>
<tr>
<td>Factor</td>
<td>$K_L$</td>
<td>1.2</td>
<td>1.2</td>
</tr>
<tr>
<td>Total snow creep load</td>
<td>$S_{N,M}$ (kN)</td>
<td>5.95</td>
<td>52.6</td>
</tr>
<tr>
<td>Total moment</td>
<td>$M_{N,M}$ (kNm)</td>
<td>3.87</td>
<td>34.2</td>
</tr>
</tbody>
</table>

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13 Numerical modeling of flow around obstacles

This section will be written by MN.

This section will describe special considerations regarding avalanche modeling for flow over or around dams and obstacles. It will also briefly describe general advances in avalanche modeling in recent years that have practical implications for design of avalanche dams.
14 Geotechnical issues

Authors . . .

14.1 Introduction

For an avalanche expert it is of importance to have a basic knowledge about the construction principles of retaining and deflecting dams.

Taking geotechnical issues into account when building dams and walls is important to ensure stability of the construction, reduce maintenance costs and increase lifetime. Although the avalanche specialist himself does not perform the geotechnical analysis, she should know which building principles must be applied and what to recommend to the client concerning the principle build up of a dam. In addition to have some knowledge about geotechnical principles, the avalanche specialist must also be aware of the fact that geotechnical investigations and calculations should always be performed by specialists in the geotechnical field, to ensure sufficient stability of the dam and the ground below it.

Avalanche dam constructions usually have heights ranging from 10–25 m and lengths from 50–500 m or more. The volumes are large, usually on the order of $10^4$ to $10^5 \text{ m}^3$, construction costs are therefore high, and a dam constructed without applying geotechnical principles may lead to fatal failures of the dam, with serious damage as a result. See Figure 38.

Figure 38: Failure in an avalanche retaining dam. Dam height 8 m. (To be made by KL.)

When dams are planned, the clients are usually a municipality, another public institution, building consultants, contractors, architects or private persons. Few, or none of these are experts in avalanche dam construction, and it is of importance to the builder or client to get an overview and realistic plans and cost estimates for the project as early as possible in the process.

14.2 Location and design

Location of the dam is usually the first issue in dam planning. The dam must be located in such a way that it protects the whole exposed object or area in question. The dam must have the correct dimensions to secure the exposed object. It should not be too large, and of course not too small, and it should be planned in such a way that the cost/benefit is optimised.

To optimise the height and length of the dam, and therefore the costs, it is of importance to locate the dam as far down in the avalanche path as possible and as near the protection area as possible. This is also an important issue concerning the construction itself as it is usually cheaper to perform the construction work on flat ground instead of at a steep mountain slope.

Both deflecting and retaining dams can be made both shorter and lower as they are moved closer to the object to be protected. If a dam is located farther uphill one must ensure that the
dam is long enough to prevent the avalanche to circumvent the dam and hit the area or object to be protected. This possibility increases rapidly with increasing distance between the dam and the exposed object.

When the best location has been found, the specialist must ensure that the avalanche is stopped or deflected and calculate:

- length of the dam crown
- effective, vertical height of the dam
- storage volume above the dam

These calculations are described in detail in sections 5 to 7.

14.3 Construction materials

Many different types of materials are used for avalanche deflecting and retaining dams or walls, depending on what is found to be the most cost/effective solution in each case. The construction materials normally consist of:

- loose deposits: rocks, gravel, sand
- reinforced earth
- concrete

14.4 Dams made of loose deposits (earth materials)

Ground investigations

Before the construction starts, ground investigations must be accomplished to ensure that the soils are usable for the construction and that the stability of the underlying ground is sufficient. By the ground investigations one must:

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

A common way is to make pits in the construction area both at the dam site itself and in the excavation area and collect samples of the materials. Core drillings may also be used in fine grained soils. The soil samples must be analysed in a geotechnical laboratory. A sieve analysis is important and should always be performed. By the sieve analysis one constructs a grain-distribution curves which clarifies the relative amounts of the different types of materials in the sample (clay, silt, sand, gravel), see Figure 39. In special cases, triaxial tests will be performed to calculate the angle of repose of the masses.

Based on the soil samples, geotechnical experts must calculate the global stability of the ground, the stability of the dam itself and make a detailed plan for the construction. (Slope angles of the fill, build up of the dam, erosion protection, drainage of the dam area, etc.).
Figure 39: Grain distribution curves, two examples.

Figure 40: Catching dam of earth materials. Vertical section (Sketch to be made by KL).

**Dam construction**

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

Mass balance means that the excavation is done in situ, just above the dam, and that all the excavated masses are used in the dam fill, see Figure 40. By such a procedure, the fill volume may be reduced also, as the effective dam height is the sum of the fill itself and the depth of the excavated masses.

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

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

**Quality of the earth materials**

All kinds of loose materials, from clay, silt, sand, gravel and rocks may in principle be used for the construction of a dam. In fine-grained, cohesive materials as clay and silt, the drainage of water is a very slow process. Pore pressures might build up during the construction phase or later during heavy rainfall and reduce the stability of the dam.
A rule of thumb says that if more than 10% of the dam fill consists of fine-grained material one has to make extra precautions in the construction to ensure that the water drainage from the body of the dam is sufficient, to obtain satisfactory stability of the dam. This induces extra costs for the construction. It is therefore a clear advantage to use coarse grained, frictional materials as gravel and rocks for dams made of loose deposits.

**Organic materials**

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

**Design of the dam**

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

In steeper dams one should use dry walls, reinforced earth or concrete, see section below. The steeper inclination is a clear advantage for the stopping and deflecting effect as described in sections 5 to 7.

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

- Fine grained materials max 1:2
- Sand, gravel max 1:1.5
- Loose layered rocks 1:1.25
- Dry walls with rocks 3.5:1–4:1
- Reinforced earth, geotextiles 4:1

Fine grained masses must be sorted out from the excavation. If one decides to use fine grained material in the dam, the fines must be built into the dam in succession with coarser material to ensure sufficient drainage. A common practise is to make a layered construction with horizontal thin layers, coarse grained alternating with fine grained layers, see Figure 41. The layers should not exceed a thickness of 0.5 m, and be levelled out and compacted by heavy machinery.
Figure 41: Principle sketch of a dam with a dry wall.

**Design of the excavation area**

The excavation area above the dam must be made broad enough to prohibit the avalanche masses to jump over the dam from the natural terrain surface above the excavation. A minimum width of the excavation area should be about 50 m. The width depends on the avalanche velocity and avalanche volume and must be calculated in each single case. The layout of the excavation must ensure that the effective height of the dam is retained; the excavation must not be so deep and narrow that the dam ends up in a “ditch”.

The sides of the cut must be gentle enough to ensure stability of the earth masses along the cut, and should normally not be steeper than 1:1.5. Coarser deposits (gravel, boulders) will be stable up to 1:1.25, and if clay and silt makes up for most of the cut, the inclination should not be steeper than 1:2.

**Water, drainage and erosion protection**

If water occurs in the excavation, precautions must be taken to keep the construction masses as dry as possible as water soaked masses are difficult to handle. Because of the high water content in such masses, the angle of repose is lower and it will be difficult to obtain the designed inclinations of the fill during the construction. Water built into the dam will reduce the stability of the dam also. Usually, the water content in the dam is at the highest during the construction, especially if much fine grained soils are used. The construction period is therefore often a critical phase for the stability of the dam.

Surface streams and brooks must be diverted from the dam area. If possible, the flowing water should be directed around the dam along the base of the upper fill, or kept completely away from the dam area. The dam and the excavated area should be designed in such a manner that ponding of water above the dam cannot occur. If necessary, one could lead the water under the dam in culverts, but there is always a possibility that such culverts may be
blocked, either by avalanche snow or earth materials. If a possibility exists for water build up behind a dam, the dam should be designed for hydrostatic pressures. Some countries have special regulations concerning this problem.

The weight of the dam itself may block natural drainage channels in the ground and force groundwater upwards into the dam itself and by this reduce the stability of the dam itself. A high ground water table can be avoided by making ditches under the base of the dam to ensure sufficient drainage. In addition, the bottom layer of the dam should always be constructed from self-draining materials.

Both the dam sides and the sides of the cut should be protected against water erosion. This could be done by use of different kinds of vegetation or geotextiles to stabilise the surface. Water courses in the dam area must be protected against erosion by (stones, boulders, etc.) unless the water flows on the bedrock itself.

**Advantages and disadvantages**

The advantages of using natural loose deposits for the construction are mainly:

- materials are often at hand
- natural loose deposits are cheaper than other materials
- maintenance costs are low
- the appearance of nature like constructions are more easily accepted by the public as the visual impact is less than for an artificial structure such as a concrete dam.

The disadvantages of dams made purely of loose deposits are many:

- dams require much space. A 15 m high dam with inclination 1:1.5 on horizontal ground is 45 m wide at the base, plus the width of the dam crown (2–4 m). When the excavation area is included one needs at least about 100 m for the construction. As dams are built in the run-out zone, the terrain is often sloping, and with increasing terrain inclination the lower fill will rapidly increase in width and volume.

- the volume of a dam is roughly proportional to \( h^2 \cot \alpha \), per unit length, where \( h \) is the vertical dam height and \( \alpha \) the inclination of the dam sides. As can be seen, the volume increases rapidly with the dam height. Although unit prices per m\(^3\) will decrease with the volume of the dam, high dams with natural inclination of the dam sides will be costly.

- by using earth materials it is difficult to obtain steep enough dam sides, and they are therefore less effective than dams made of concrete or reinforced earth.
for deflecting dams in steep terrain, the effective inclination of the dam sides (measured perpendicular to the dam axis) will decrease with increasing terrain inclination. The angle of repose will in such cases be found along a plane between the direction of the cross section and the longitudinal dam axis.

14.5 Dams with steeper sides

It is possible to increase the inclination of the dam sides by the use of materials as:

- dry walls
- reinforced earth
- concrete, steel

As earlier mentioned, a steep dam at the avalanche side of the dam will increase the effect of the dam.

Dry walls consist of a “masonry” of boulders with a back fill of other earth materials, see Figures 42 and Fig. 43. The boulders should not be smaller than abut 0.5 m$^3$ and be built up in bonded layers. Experience has shown that dry walls with inclinations up to 4:1 (76˚) are stable, provided that the foundation is adequate and the dry wall itself is designed to withstand the earth pressure from the backfill. To withstand the earth pressure, the thickness of the dry wall must increase with the height of the dam. The ratio of the thickness of the dry wall to the dam height should not be less than 1:5, i.e., a 10 m high dam will need a 2 m thick dry wall. To increase the stability, it is advantageous to tilt the boulders a little into the wall.

The foundation of the dry wall must consist of materials not subjected to frost heave, (sand, gravel, boulders) and both the foundation and back fill must be drained. Calculations to ensure sufficient global stability and stability of the dam itself are necessary.

As for the use of reinforced earth, many solutions are possible, and different commercial products are available on the market. The reinforcement may consist of nets or gabion boxes of galvanised steel, or net constructions of polymers, as polyethylene, polypropylene, polyester, etc. The reinforcement is applied as the outer cover of the dam, and makes it possible to build dams with inclinations of 4:1 or more. Earth materials (usually gravel or stones) are embedded into the cases or into the nets and kept in place by additional anchors built into the dam. All such constructions must be designed by geotechnical experts to ensure a safe and optimal layout.
A combined defence structure system consisting of two rows of 10 m high braking mounds and a 17 m high steep catching has been constructed above the town of Neskaupstaður, eastern Iceland, see Figures 44, 45 and 46. The dam and mounds are designed in combination with 1200 m of supporting structures in the starting zone of the avalanches (not shown on the map). The dam is 400 m long and the mounds are 10 m wide at the top and 30 m wide at the bottom. The uppermost 14 m of the upstream side of the dam has a slope of 4:1. It is built from reinforced loose materials with a front constructed from 0.5 m high steel-mesh steps, which are anchored into the dam fill with long steel rods. The lowest 3 m of the face of the dam are built from loose materials with a slope of 1:1.5. The upstream sides of the mounds have the same slope as the steep part of the dam and are built with the same kind of steel reinforcement.

Concrete constructions are well known as deflectors and for catching purposes. The advantages in preference to constructions made of earth materials are firstly; that it is possible to obtain a vertical wall construction which is more effective concerning kinetic energy dissipation from the moving avalanche. Secondly, the concrete walls are much slender than dams made of natural deposits only, and consequently needs much less space. The major drawbacks
Figure 46: Vertical section of the dam/breaking mounds in Neskaupstaður (see map in Figure 44).

Figure 47: Concrete diverting dam, height 8 m, length 200 m. Odda, Norway. *(To be made by KL.)*

Figure 48: Principle sketch of a concrete slab dam built on loose deposits. *(To be made by KL.)*

Figure 49: Concrete retaining dam made of 20 m wide shells. Designed avalanche impact force: max 92 kPa. Each shell has four vertical steel anchors drilled 12–14 m into the bedrock with a capacity of 2800 kN. Tilting moment on each shell is 59 kNm, and shear force is 8500 kN *(check)*. Ullensvang, Norway. *(To be made by KL.)*

are high costs and unpleasant visual impacts.

The concrete walls are usually made as slab concrete dams reinforced with ribbing, see Figure 47. For such slender constructions the avalanche impact pressure must be carefully calculated as the walls must withstand the pressure without tilting or being displaced. For these reasons, the foundations are especially important. In bedrock, the foundations are usually made by steel tension anchors (ribbed bars) in boreholes with cement grout. In loose deposits one must ensure that the ground is able to withstand the weight of the concrete construction plus the loads from the avalanche impact. A foundation platform of concrete is normally used as a base for the wall, see Figure 48. The base must be frost proof and the area around it well drained.

A 10 m high and 113 m long retaining wall made as a shell construction, founded on bedrock, is shown in Figure 49. A shell construction can take higher loads than a straight wall for the equivalent amount of concrete, and was therefore more cost effective.
Steel constructions may also be used for deflecting and retaining purposes. In special cases steel has been used for this purpose, but such constructions are not common, as they tend to be more costly than constructions from other materials.
15 Acknowledgements

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A  Notation

The following list defines the variables used to describe the geometry of the terrain and the dam and the flow of the (dense core of the) avalanche at the dam location. Figures 2, 3 and 4 provide schematic illustrations of the meaning of the variables.

\(u_1, h_1\)  Velocity and flow depth of the oncoming flow upstream of any disturbance to the flow caused by the dam.

\(F_r\)  Froude number, \(F_r = \frac{u}{\sqrt{g \cos \psi h_1}}\).

\(F_r\)  “Froude number” corresponding to the component of the velocity normal to the flow, \(F_r = F_r \sin \psi\).

\(H\)  Dam height measured in the direction normal to the terrain. This quantity is used to simplify the formulation of some equations.

\(H_D\)  Vertical dam height measured in a vertical section normal to the dam axis in a horizontal plane.

\(h_u, h_s, h_f\)  Contributions to the dam height, \(H_D\), in the traditional design criterion for dam height, Equation (1). \(h_u\) is the required height due to the kinetic energy or the velocity of the avalanche, \(h_s\) is the thickness of snow and previous avalanche deposits on the ground on the upstream side of the dam before the avalanche falls, and \(h_f\) is the thickness of the flowing dense core of the avalanche.

\(r\)  Vertical run-up of an avalanche measured in a vertical section normal to a dam or obstacle axis in a horizontal plane.

\(H_{cr}\)  Critical dam height. The maximum height of a dam over which uninterrupted supercritical flow may be maintained.

\(h_{cr}\)  Critical flow depth. Depth of flow over a dam with height \(H_{cr}\) at the top of the dam.

\(h_r\)  Run-up height, \(h_r = H_{cr} + h_{cr}\).

\(u_2, h_2\)  Velocity and flow depth downstream of a shock that is formed in the flow against a dam.

\(\psi\)  Slope of the terrain at the location of the dam.

\(\psi_\perp\)  Slope of the terrain in the direction normal to the dam axis.

\(\alpha\)  Angle of the dam side with respect to the terrain in the direction normal to the dam axis.

\(\alpha_s\)  The steepest inclination of the dam side.
Deflecting angle of the dam (\( \varphi = 90^\circ \) for a catching dam).

Shock angle for a stationary, oblique shock upstream of a deflecting dam.

Widening of a shock along a deflecting dam, \( \Delta = \theta - \varphi \).

Relative reduction in normal velocity in the impact with the dam.

Friction coefficient for Coulomb friction.

Internal friction angle of avalanching snow.

A coordinate system with the \( x,y \)-axes in the plane of the terrain near the dam location with the \( x \)-axis in the direction of the oncoming flow upstream of the dam.
B  Practical examples, deflecting and catching dams

This section will be written by NGI and TóJ with additional input from LR and ?

General: NGI,
Flateyri: VST?/NGI/IMO,
some Norwegian? catching dam: NGI,
comments: all.
C Practical examples, combined protection measures

This section will be written by TóJ and MN with additional input from LR, ?

General: TóJ, MN,
Neskaupstaður: VST?/IMO,
Taconnaz: Cemagref.

Two figures that should fit in somewhere:
Figure 50: Plan view of the protection measures in the Drangagil area in Neskaupstaður, eastern Iceland. The map shows the position of the supporting structures in the starting zone, two rows of braking mounds beneath the gully and a dam just above the uppermost houses.
Figure 51: A photograph of the braking mounds in Neskaupstaður and the catching dam behind them. Each mound is 10 m high and the catching dam is 17 m high.
D Geotechnical examples

This section will be written mainly by NGI with assistance from others regarding maps, drawings and photographs for examples from outside of Norway.

This appendix makes it possible to treat common geotechnical aspects of deflecting and catching dams in one section. It will contain 2–3 examples of geotechnical design of dams built from loose deposits, reinforced earth and concrete. The previous two appendixes with examples can therefore focus exclusively on snow-technical issues, such as determination of deflecting angle, dam height and the choice of dam type (steep dam sides vs. the slope of the dam sides determined by the angle of repose of loose materials).
E Analysis of overrun of avalanches at the catching dam at Ryggfonn

It will be rarely possible to design a catching dam in a manner that all avalanches can be stopped in front of the dam at all times; parts of the avalanche will overflow the dam at some times. Observation from full-scale experiments on a 16 m high and 70 m wide (crown) dam in Ryggfonn / Western Norway imply that in those cases when the avalanche topped the dam the overrun length of the avalanche can be expressed by a simple relationship (Gauer and Kristensen, 2005b). The slope angle of the dam is 40°.

Figure 53 shows the normalized overrun length vs. the normalized kinetic energy. The overrun length of the avalanches that surpassed the dam crown can be fit by

\[
\frac{l_{ovr}}{h_{fb}} \approx b_1 \frac{u_b^2}{2gh_{fb}} + b_0, \tag{103}
\]

where \(l_{ovr}\) is the overrun length measured from the top of the dam, \(h_{fb}\) the free board height, \(u_b\) the front velocity at the upstream base of the dam and \(g\) is the gravitational acceleration. \(u_b^2/(2gh_{fb})\) is the kinetic energy, \(E_n\), normalized by the potential energy (the energy a lumped

Figure 52: Deposition pattern of the 19970208 12:38 avalanche at Ryggfonn (# 17). Left side, image of the deposit and right side, map of the deposit. Due to deposits of previous avalanches the effective free board height was only 5 m and the estimated front velocity 40 m s\(^{-1}\). (Photo NGI)
Figure 53: Correlation between normalized kinetic energy and normalized overrun length (left panel). ◇ marks the best estimates for avalanches that surpassed or topped the dam crown \((l_{ovr} \geq 0)\). Crosses indicate the range of uncertainty. Numbers mark the individual avalanches. The dashed line shows a linear fit according to (103) for those avalanches using robust fitting. The dash-dotted red line marks the critical energy \(E_c\). The right panels presents the mass distribution ratio vs. normalized kinetic energy. Dashed line shows a linear fit.

mass block would need to climb up the effective dam height). The parameter \(b_1\) is approximately 2.56 and \(b_0\) is -1.41. The ratio \(b_0/b_1\) is a measure for the energy, \(E_c\), dissipated by the “effective” dam in this case. The fitting line is also shown. A similar relations can also be found in granular experiments (cf. Gauer and Kristensen, 2005b).

Equation (103) can also be physically motivated. If one rewrites the equation in from of an energy balance

\[
\frac{u_p^2}{2} = -\frac{b_0}{b_1} g h_{fb} + \frac{g}{b_1} l_{ovr}.
\]  

(104)

The term on the left hand side corresponds to the kinetic energy of the incoming avalanche front. The first term on the right hand side, is the energy dissipation of the front during the ascend of the dam. Finally, the second term on the right describes the dissipation of the remaining energy due to friction. In this sense, \(1/b_0\) corresponds to an effective friction parameter.

Figure 53 also shows the mass distribution ratio, \(m_r\), vs. normalized kinetic energy, \(E_n\). The mass distribution ratio is defined as the estimated fraction of the total deposit mass that surpassed the dam crown, \(M_{ovr}\). It is based on the measured deposit mass above the dam and the total mass, \(M_{tot}\) from the field surveys. The fit (dashed line) is given by
\[ m_r = \frac{M_{\text{ovr}}}{M_{\text{tot}}} = c_1 \frac{u_b^2}{2gh_{fb}} + c_0 \]  

(105)

The fitting coefficients are \( c_1 = 0.053 \) and \( c_0 = -0.0254 \). Again, the ratio \( c_0/c_1 \) is a measure for the energy, \( E_c \), dissipated by the “effective” dam.

Although Equation (103) is derived only considering those avalanches that overtopped the dam, the observations from the full-scale experiments imply that energy dissipation by the dam is less efficient than traditionally assumed (at least for dams with low steepness). Due to limited information about the avalanches which did not surpass the dam it is not possible to say whether they were stopped by the dam or just were at the end of their run-out. However, from a total of about 70 to 80 observed avalanches, twelve are known to have surpassed the dam crown since the dam at Ryggfonna was built in 1980.

If one calculates the required dam height to stop an avalanche based on equation (103) one finds that only avalanche with \( u_b < 13 \text{ m s}^{-1} \) could be stopped by the dam at Ryggfonna, provided the total height of the dam is available. Figure 54 shows further example calculations for the catching dam at Ryggfonna. The left panel shows the calculated overrun length versus the front velocity for given effective dam heights. The right panel depicts the required dam height if a certain overrun length can be permitted. Taken at face value, this example indicates that the application of dams as protective measures for endangered areas is limited to the end of the run-out zone or against small avalanches with typically low velocity, e.g. against small slides along roads to reduce road clearing work. This conclusion is clearly not consistent with the dam design recommendations described in sections 5 to 7 in the main text of the report, and it is difficult to reconcile with observations of run-up of natural avalanches on dams and other obstacles that are described in subsection 5.10. This inconsistency reflects our current lack of understanding of the dynamics of snow avalanches that hit obstacles. It indicates considerable uncertainty about the effectiveness of avalanche dams, in particular the effectiveness of catching dams to stop or reduce the run-out of rapid dry-snow avalanches.
Figure 54: Overrun length vs. front velocity $U_b$ calculated for the catching dam at Ryggfonn, left panel. On the right, required dam height vs. front velocity $U_b$ given an allowable overrun length, $l_{ovr}$. Dashed line indicates the height of the dam at Ryggfonn.
F Loads on walls and masts, summary of existing Swiss and Norwegian recommendations

Authors . . .

F.1 Load on wall like structure

Swiss recommendation

According to Gruber and others (1999) the following approach for the determination of the force on a extended obstacles is recommended in Switzerland. Extended means that a considerable amount of snow particle of the avalanche is reflected by an angle of \( \alpha \).

Dense flow

\[ p_{dn} = \rho u^2 \sin^2 \alpha, \]  

(106)

Figure 55: Load on a large obstacle.
where $p_n$ is the pressure normal to surface $A$, $\rho$ the density of the avalanche, and $u$ the speed of the approaching flow. In the case of a vertical wall $\alpha$ would be $90^\circ$. The tangential pressure is assumed to be

$$p_{dt} = \mu p_{dn},$$

(107)

**For safety reason a flow density, $\rho$, of 300 kg m$^{-3}$ is assumed.**

It follows that the normal force acting on a wall with width, $b$, is

$$F_{dn} = p_n (z_{hd} - z_{hs}) b,$$

(108)

the tangential force

$$F_{dt} = \mu F_{dn},$$

(109)

and the moment

$$M_{dn} = \frac{(z_{hd} + z_{hs})}{2} F_{dn},$$

(110)

Above the flow height of the avalanche pressure is assumed to decrease linear within a stagnation (climbing) height, which is given by

$$h_{stau} = \frac{u^2}{2g\lambda}.$$

(111)

For dry mostly fluidized flows $\lambda = 1.5$ is proposed; for dense flows, it is assumed that $2 \leq \lambda \leq 3$. The pressure is given by

$$p(z) = \frac{\rho u_f^2}{2} \frac{(z_{tot} - z)}{(z_{tot} - z_{hd})}.$$

(112)

The force component is

$$F_{fl} = C_D W p_d \frac{(z_{tot} - z_{hd})}{2}$$

(113)

and the moment

$$M_{fl} = \frac{z_{tot} + z_{hd}}{3} F_{fl},$$

(114)

where $z_{tot} = z_{hd} + h_{stau}$.  

116
Fluidized layer (saltation layer) and powder part

In Gruber and others (1999), Issler gives some consideration on the effect of powder snow avalanches and the saltation layer. Briefly summarized,

\[ p_{pn} = f \rho u^2 \sin^2 \alpha, \] (115)

where the factor \( f \) is between 0.5 and 1. It is closer to 1, (i) the higher the velocity, \( u \), (ii) the higher the deflection angle, (ii) higher the density of the powder part, and (iv) the larger the particle size within the flow. For perpendicular impact, \( f \) equals one is recommended. No profile is specified.

The density, \( \rho \) within a saltation layer is assumed to range between 10 and 50 kg m\(^{-3} \) and in the powder part between 1 and 10 kg m\(^{-3} \). The height of the saltation layer is assumed range between 1 to 5 m, the one of the powder part several 10 m.

It follows that the force acting on a wall is

\[ F_{pn} = p_n (z_{hp} - z_{hd}) b \] (116)

and the moment (no profile is specified)

\[ M_{pn} = \frac{(z_{hp} + z_{hd})}{2} F_{pn}. \] (117)

F.2 Load on mast like structure

Swiss recommendation

According to Gruber and others (1999) the following approach for the determination of the force on a small obstacles is recommended in Switzerland:

\[ F_m = C_D A p(z) . \] (118)

Here, no distinction between different flow regimes is made. Also no distinction between dry or wet snow avalanches is made. The proposed values for \( C_D \) are summarized in Table 9.

<table>
<thead>
<tr>
<th>Flow regime</th>
<th>Obstacle form</th>
<th>( C_D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>No distinction</td>
<td>( \circ )</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>( \triangle )</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>( \Box )</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 9: \( C_D \) according to the Swiss recommendation.

The projected area, \( A \), is defined as
\[ A = h_{\text{tot}} W, \quad (119) \]

where \( W \) is the width of the obstacle. The total impacted height, \( h_{\text{tot}} \), is given by (see also Fig. 56)

\[ h_{\text{tot}} = h_d + h_{\text{stau}}. \quad (120) \]

\( h_d \) is the flow height of the avalanche. The second term on the right hand side describes the climbing height and is give by

\[ h_{\text{stau}} = \frac{u_f^2}{2 g \lambda} f(W/h_d). \quad (121) \]

For dry mostly fluidized flows \( \lambda = 1.5 \) is proposed; for dense flows, \( 2 \leq \lambda \leq 3 \) is assumed. \( f(W/h_d) \) is a reduction factor, which depends on the ratio between obstacle width and flow height. As a guide line, proposed values are summarized in Table 10.

<table>
<thead>
<tr>
<th>( W/h_d )</th>
<th>0.1</th>
<th>0.5</th>
<th>1</th>
<th>2</th>
<th>( \geq 3 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f(W/h_d) )</td>
<td>0.1</td>
<td>0.4</td>
<td>0.7</td>
<td>0.9</td>
<td>1</td>
</tr>
</tbody>
</table>

Table 10: Reduction factor in dependency of the ration \( W/h_d \).

Within the flow height the pressure is assumed to be constant and given by

\[ p_d = \frac{\rho u_f^2}{2}. \quad (122) \]

Hence, the force component on the mast from the dense flow is

\[ F_d = C_D W p_d (z_{hd} - z_{hs}) \quad (123) \]

and the moment

\[ M_d = \frac{z_{hd} + z_{hs}}{2} F_d. \quad (124) \]

Above the avalanche a linear decreasing pressure is assumed, \( i.e., \)

\[ p(z) = \frac{\rho u_f^2}{2} \frac{(z_{\text{tot}} - z)}{(z_{\text{tot}} - z_{hd})}. \quad (125) \]

Here, the force component is

\[ F_{fl} = C_D W p_d \frac{(z_{\text{tot}} - z_{hd})}{2} \quad (126) \]
and the moment

\[ M_{fl} = \frac{z_{tot} + z_{hd}}{3} F_{fl} . \]  

(127)

Figure 56: Schematic diagram of the impact pressure distribution due to an avalanche on a mast-like structure according to the Swiss recommendation.

For safety reason a flow density, \( \rho \), of 300 kg m\(^{-3}\) is assumed.

F.3 Loads due to snow pressure

Larsen (1998) Based on experiments on snow creep loads on two masts with different diameter at the NGI test-site Fonnbu, Larsen (1998) proposes for the design load on mast like constructions the following relation.

\[ S'_{N,M} = K_L C_L \frac{\rho}{1000} D^2 g \sin \psi \quad \text{(in kN m}^{-1}\text{)} , \]  

(128)
where $\rho$ is the average snow density, $D$, the thickness of the snowpack measured perpendicular to the ground, $\psi$ the slope angle, and $g$ is the acceleration due to gravity. The factor, $K_L$, depends on $D$: 1.2 for snow thickness of 4 m and 0.7 for snow thickness of 5 m.

$$C_L = 0.98d^{0.63} + 0.42,$$  \hspace{1cm} (129)

where $d$ is the mast diameter. Margreth (2006) notes that this model disregard effects due to snow gliding and hence limited to situation with no snow gliding. The moment is

$$M'_{N,M} = \frac{h_2}{2} S'_{N,M} \quad \text{(in kNm m}^{-1}) \hspace{1cm} (130)$$
Laws and regulations about avalanche protection measures in Austria, Switzerland, Italy, France, Norway and Iceland

Laws and regulations regarding the adaptation of hazard zoning after avalanche protection measures have been constructed are different in different countries. In France, no changes are made in the zoning, so that no relaxation of land use restrictions is made in spite of the improved safety provided by the protection measures. This is underlines the policy that protection measures are only intended to improve the hazard situation in existing settlements and should not lead to increased population density in potentially hazardous areas, especially considering the inherent uncertainty about the effectiveness of avalanche protection measures. In most other countries, the hazard zoning is modified after protection measures have been completed in order to reflect the improved hazard situation, but the detailed manner in which the modifications are made differs between countries. The design requirements for the protection measures are usually expressed in terms of a minimum return period of avalanches, which can reach the settlement with a given impact pressure, or a maximum acceptable risk in the settlement after protection measures have been constructed. The following sections summarise the laws and regulations that concern avalanche protection measures in some European countries where snow avalanches constitute a natural hazard.

G.1 Austria

...

G.2 Switzerland

...

G.3 Italy

...

G.4 France

...

G.5 Norway

...
Table 11: Definition of Icelandic hazard zones

<table>
<thead>
<tr>
<th>Zone</th>
<th>Lower level of local risk</th>
<th>Upper level of local risk</th>
<th>Building restrictions</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>$3 \cdot 10^{-4} \text{yr}^{-1}$</td>
<td>–</td>
<td>No new buildings, except for summer houses*, and buildings where people are seldom present.</td>
</tr>
<tr>
<td>B</td>
<td>$1 \cdot 10^{-4} \text{yr}^{-1}$</td>
<td>$3 \cdot 10^{-4} \text{yr}^{-1}$</td>
<td>Industrial buildings may be built without reinforcements. Homes have to be reinforced and hospitals, schools etc. can only be enlarged and have to be reinforced. The planning of new housing areas is prohibited.</td>
</tr>
<tr>
<td>A</td>
<td>$0.3 \cdot 10^{-4} \text{yr}^{-1}$</td>
<td>$1 \cdot 10^{-4} \text{yr}^{-1}$</td>
<td>Houses where large gatherings are expected, such as schools, hospitals etc., have to be reinforced.</td>
</tr>
</tbody>
</table>

*If the risk is less than $5 \cdot 10^{-4}$ per year.

G.6 Iceland

The Icelandic regulation on snow- and landslide hazard zoning is based on individual risk 
Ministry for the Environment (2000); Jónasson and others (1999); Arnalds and others (2004),
i.e. the probability of death as a consequence of a snow avalanche or a landslide. The so-called local risk (i.e. ignoring exposure, see Arnalds and others (2004)) of $0.3 \cdot 10^{-4}$ per year is defined to be acceptable for residential areas, and three types of hazard zones are defined where the risk is progressively higher, see Table 11. The guidelines for the zoning and utilisation of the hazard zones are tailored to attain the acceptable risk level in residences when the exposure and increased safety provided by reinforcements have been taken into account. For industrial buildings the guidelines probably correspond to a somewhat higher risk, but this may be justified by the absence of children.

According to the Icelandic hazard zoning regulation Ministry for the Environment (2000)\(^3\), which is based on a law from 1997 about avalanches and landslides Alþingi (1997)\(^4\), protective structures “shall only be built to increase the safety of people in areas already populated.” The effect of protective structures shall be assessed and/or calculated and this effect is reflected in an updated hazard zoning, which is issued by the government after the protection measures are completed. This leads to a (partial) relaxation of previous restrictions on the use of land in the protected area. This applies in particular to catching and deflecting dams in

avalanche run-out areas and to supporting structures in starting zones. In areas with protective structures, both local risk in the absence of such measures and local risk taking the structures into consideration shall be shown on the hazard map. Protection measures shall be designed with the aim to increase the safety so that the risk to people in the protected area is as near as possible to the acceptable risk as specified by the hazard zoning regulation (see above), but this goal is not an absolute requirement. Due to the large uncertainty in the design assumptions of avalanche protection measures, the adaptation of the hazard zoning is to a large extent based on the subjective judgement of experts involved in the design of the structures. In order to reflect this uncertainty, the outer boundary of the A hazard zone is typically not moved higher up than corresponding to the previous location of the C hazard line.

Figure 57 shows a hazard map for the Drangagil area in Neskaupstaður, eastern Iceland Arnalds and others (2001), where protection measures consisting of supporting structures, braking mounds and a catching dam have been constructed Tómasson and others (1998a,b). The map shows the estimated isorisklines in the absence of protection measures and the estimated local risk after the structures have been fully completed.
Figure 57: A hazard map for the area below Drangagil in Neskaupstaður, eastern Iceland, showing the estimated local risk both in the absence of protection measures (solid lines) and the estimated local risk after the structures have been fully completed (dashed lines, the B and the C lines coincide below the dam) (cf. Table 11). The protection measures consist of supporting structures (not shown in this figure, but shown in figure 50), 10 m high braking mounds and a 17 m high catching dam (see also figures 50 and 51).