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> Defense structures in avalanche starting zones

Technical guideline as an aid to enforcement

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Technical guideline as an aid to enforcement

Editors:
Federal Office for the Environment FOEN
WSL Swiss Federal Institute for Snow and Avalanche Research SLF
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Legal status of this publication
This publication is an implementation guide issued by FOEN in its capacity as a supervisory authority, and is addressed primarily to the enforcement authorities. It seeks to clarify undefined legal concepts contained in the relevant Acts and ordinances so as to facilitate consistent enforcement practices. Authorities who give due consideration to these guides can safely assume that federal law is being correctly implemented. Alternative approaches are, however, permissible provided they comply with the legal requirements. Guides of this kind (until now also referred to as guidelines, guidance, recommendations, handbooks, enforcement aids, etc.) are published by FOEN in the series entitled “Environment in Practice”.

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> Abstracts

The technical guideline regulates the planning of snow supporting structures and the dimensioning of separated structures. The procedures and criteria for type approval, together with the requirements for supporting structures and anchor grout are specified. Further, an overview of the effects of snow pressure and instructions on the planning of defense structures in permafrost are given. The guideline draws heavily on past experience gained with supporting structures, and is complementary to the relevant SIA standards. It is directed towards designers and project engineers.

Keywords:
Defense structures,
avalanche protection,
type approval,
guideline,
permafrost

Die vorliegende technische Richtlinie regelt die Projektierung von Stützverbauungen und die Bemessung von gegliederten Stützwerken. Der Ablauf der Typenprüfung und die Prüfkriterien sowie Anforderungen an Stützwerke und Ankermörtel werden festgelegt. Weiter wird eine allgemeine Übersicht über die Schneedruckwirkung gegeben sowie Angaben gemacht, wie Lawinenverbauungen im Permafrost zu planen sind. Die technische Richtlinie stützt sich stark auf die in der Vergangenheit im Stützverbau gemachten Erfahrungen ab und ergänzt die einschlägigen SIA-Normen. Sie richtet sich an Konstrukteure und Projektverfasser.

Stichwörter:
Stützverbau,
Lawinenschutz,
Typenprüfung,
Richtlinie,
Permafrost

La présente directive réglemente l'élaboration du projet de construction de paravalanches et le dimensionnement des ouvrages de stabilisation composés. Elle définit le déroulement de l'examen des types d'ouvrages, les critères du test ainsi que les exigences liées aux ouvrages de protection et aux mortiers d'ancrage. Un aperçu général des effets de la pression de la neige et des indications sur la planification des paravalanches dans le pergélisol y sont également présentés. Largement inspirée de l'expérience acquise, cette directive complète les normes SIA en vigueur. Elle s'adresse aux constructeurs et aux auteurs de projets.

Mots-clés :
Ouvrage de stabilisation,
protection contre les avalanches,
examen des types d'ouvrages,
directive,
pergélisol

Questa direttiva tecnica disciplina la progettazione delle opere di premunizione e il dimensionamento di opere di sostegno strutturate, stabilisce lo svolgimento dell'omologazione dei tipi di strutture e i criteri di esame e fissa i requisiti posti per le opere di sostegno e la malta di ancoraggio. Inoltre, fornisce una panoramica generale della pressione esercitata dalla neve sulle opere di sostegno e indica come pianificare le opere di premunizione contro le valanghe nel permafrost. La direttiva poggia in gran parte sulle esperienze acquisite in passato nell'ambito delle opere di premunizione e integra le vigenti norme SIA. Si rivolge a costruttori e progettisti.

Parole chiave:
opere di premunizione,
protezione contro le valanghe,
omologazione dei tipi di strutture,
direttiva,
permafrost

> Foreword

Alongside protective forest – a biological protective measure – supporting structures represent the primary form of protection from avalanches in Switzerland. Technical and biological protective measures are often combined. Today, over 500 km of permanent supporting structures are in service. In addition, about 150 km of temporary supporting structures are in use in combination with reforestation measures. The modern supporting structures withstood the severe test in the avalanche winter of 1999, during which numerous avalanches having high damage potential could be prevented. In Switzerland, the most important supporting structures have now been realized, so that the principal challenge for the future will be the maintenance of existing works.

Present-day supporting structures, which started life as terrace walls, to be followed by concrete and aluminum supporting structures, and finally by modern snow bridges fastened to anchors and micropiles, requiring a long period of development. Current building materials together with new research knowledge and experience all reflect the continually changing status of technology. Work on the technical guideline, a recognized work both at home and abroad, began in the 1950s by Dr. Bruno Salm and was later influenced substantially by the work of Stefan Margreth of the Federal Institute for Snow and Avalanche Research (SLF) in collaboration with the Federal Laboratories for Materials Testing and Research (EMPA) and specialists from the Expert Commission for Avalanches and Rockfall (EKLS). The present updated version of the technical guideline is the product of over 50 years' development. The previous edition of 1990 was extended to include the latest SIA structural codes, the layout has been revised, knowledge resulting from the avalanche winter of 1999 included, and the chapters on type approval tests and the use of anchor grout in supporting structures added.

When applying for federal subsidies for avalanche supporting structures according to art. 36 WaG (Law on Forests), officially tested and approved types of structure and anchor grout must be implemented. The requirements for this are specified in the present guideline. The Federal Office for the Environment maintains a list of approved types of structure and anchor grout.

The effect of snow pressure on supporting structures is complex. To permit simple implementation of the guideline by engineers, loads and analytical load models have been heavily simplified. Note, however, that in practical cases other loads and load cases may occur that are not covered by the present guideline. Those using the technical guideline must always remain aware of this fact, which makes a corresponding high level of competence on their part essential.

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> Purpose and legal basis of the technical guideline

The present technical guideline for defense structures in avalanche starting zones issues from the Federal Law on Forests (WaG, SR 921.0) of 4 October 1991, which specifies the general and specific conditions for the granting of federal subsidies for measures for the protection of humans and material assets from natural hazards (arts. 35 and 36 WaG). The Ordinance relating to Forest (WaV; SR 921.01) of 30 November 1992 specifies particular conditions for the granting of federal subsidies, and also covers the competency of the FOEN to issue guidelines in this field (art. 39, para. 3 WaV). Where applications are made for federal subsidies for avalanche defense structures under art. 36 WaG, these must basically implement officially tested and approved types of structure and anchor grout. The present technical guideline specifies the relevant requirements. The following objectives are thereby pursued:

> Advice to those responsible for the planning, building and maintenance of supporting structures
> Overview of snow pressure effects
> Procedure for dimensioning separated supporting structures
> Specification of the requirements for anchor grout
> Specification of requirements for avalanche defense structures in permafrost
> Specification of procedures for type approval tests

Avalanche supporting structures are mostly erected at high altitudes on highly inaccessible slopes having a variety of different ground characteristics. Simple, inexpensive, robust and well-proven structural methods are therefore essential for successful, durable, implementation of avalanche defense structures. The technical guideline draws heavily on the experience obtained in the past with supporting structures. For this reason, differences have arisen from SIA 267 Geotechnology, particularly in connection with the dimensioning of foundations and anchors.

The effects of snow pressure on supporting structures are very varied. Often, situations occur that are not well understood, and it is not always possible to clarify these despite careful observation and measurement. The information contained in this guideline is based on heavy simplifications of the true situation. Users should be aware that this requires a high level of competency on their part.

The technical guideline is aimed at designers and project engineers. Section 4 “Dimensioning of separated supporting structures” and Section 8 “Type approval tests” are addressed particularly to designers. Section 3 “Planning” and in relevant situations, Section 7 “Avalanche defense structures in permafrost”, must be observed by project engineers.

Federal subsidies may be granted for measures other than those given in the present technical guideline provided that the applicant can show in the application that the minimum requirements of the guideline are complied with.

1 > Scope

1.1 Delimitation

The technical guideline applies to the planning of supporting structures in the avalanche starting zone.

The computational and dimensioning procedures apply to separated supporting structures having rigid or flexible supporting surfaces installed normal to the line of slope, or which deviate from the normal by an angle δ.

The technical guideline specifies:

- > the planning of supporting structures in the terrain
- > requirements on building materials
- > determination of loads on the supporting structures resulting from snow pressure
- > dimensioning of supporting structures and their foundations/anchors
- > use of anchor grout in avalanche defense structures
- > the installation of avalanche supporting structures in permafrost
- > type approval tests on avalanche defense structures

1.2 Relationship to the SIA standards

1.2.1 General

The present technical guideline supplements SIA 261 and/or 261/1. Where not otherwise stated, the relevant SIA standards apply. The SIA standards are the recognized codes of building practice in Switzerland and form the official set of building standards (cf. *www.sia.ch*).

1.2.2 Dimensioning of the superstructure of supporting structures

Where no further information is given in the technical guideline, the SIA standards 262, 263 and 265 are applicable to the dimensioning of the superstructure of supporting structures.

1.2.3 Dimensioning of the foundations of supporting structures

For the dimensioning of the foundations of supporting structures, the provisions of the guideline apply. In special cases, SIA 267 (Geotechnology) can be used.

1.3 Other protective measures

Under certain site conditions, other protective measures may supplement, or, indeed, replace, the supporting structures:

1.3.1 Anti-drifting structures

Structures (walls, panels, fences, etc.), which exploit wind effects to control snow deposition with the objective either of

> preventing the formation of cornices, or
> reducing the deposition of snow in starting zones.

1.3.2 Deflecting structures

Structures designed to withstand avalanche forces (dams, walls, wedges, sheds, ramp roofs), whose purpose is to guide over, divert, divide or restrict the lateral extent of an avalanche in motion.

1.3.3 Braking structures

Structures designed to withstand avalanche forces placed directly in the path of the avalanche with the objective of restraining its mass (using retention dams) or shortening the runout zone (using retarding wedges, retarding mounds or flow retarders).

2 > Nomenclature

2.1 Organizations

FOEN	Federal Office for the Environment, Bern
EKLS	Expert Commission for Avalanches and Rockfall, Bern
EMPA	Federal Laboratories for Materials Testing and Research, Dübendorf and St.Gall
SIA	Swiss Society of Engineers and Architects, Zurich
SLF	Swiss Federal Institute for Snow and Avalanche Research, Davos (The SLF forms a part of the Swiss Federal Institute for Forest, Snow and Landscape Research (WSL), Birmensdorf)
VSE	Swiss Electricity Supply Association
WSL	Swiss Federal Institute for Forest, Snow and Landscape Research, Birmensdorf

2.2 Technical Terms

General

Effect	Reaction of the supporting structure to actions (loading, stresses, internal forces, reactions, deformations, etc.; according to SIA 260: 2003).
Total ground resistance	Limiting strength of the ground (ground resistance, base failure resistance, glide resistance, shear strength; according to SIA 267: 2003).
Dimensioning	Specification of dimensions, building materials (incl. material properties) and the structural design of a supporting structure on the basis of structural or implementational considerations and/or computational verification procedures (according to SIA 260: 2003).
Design value	Value derived from a characteristic or other representative value, or from a function of design values in conjunction with partial and conversion factors or (where appropriate) directly specified value used in a verification procedure (according to SIA 260: 2003).
Characteristic value	Value of an action, a geometrical dimension or property of a building material or the ground (average, upper or lower value) normally determined by statistical methods, or (where appropriate) the nominal or tentative (anticipated) value (according to SIA 260: 2003). Characteristic values do not include coefficients of resistance. The values for snow pressure given in this guideline are characteristic values.
Influence factor	The influence factor of an element of finite width is the ratio of the snow pressure effectively sustained by the element to the snow pressure that would impinge on a section of a continuous wall of equal width.
Single structure	Independent structure usually having 2 supports and girders.
Load	Gravitational force impinging on a supporting structure (according to SIA 261: 2003).
End of structure	Area over which the end-effect loads impinge with a distance between structures of 2 m.
Solifluction	Ground creep, downward creep or creep in the loose upper ground layers saturated with water.
Supporting structure	Arrangement of several supporting structures.
Ultimate limit state	Maximum resistance (according to SIA 260 or SIA 262, 263, 265 and 267: 2003).
Variable action	Action that is not continuously present, not constant, or not changing monotonically (according to SIA 260: 2003); e.g. snow pressure.
Unprotected end of a structure	Area on which the end-effect loads impinge.

Superstructure

Crossbeam	Grate element of snow bridge and snow rake
Net	Supporting surface formed by wire ropes.
Purlin	Part of the supporting structure not touching the ground to which the steel or timber crossbeams of a snow rake are attached.
Grate	Supporting surface consisting of ribs, steel or timber crossbeams.
Snow bridge	Structure with crossbeams parallel to the ground.
Snow net	Structure with a supporting surface formed by a net.
Snow rake	Structure with crossbeams at right angles to the ground.
Support	Part of the supporting structure used to brace the girder or the purlin at the underside.
Supporting surface	Total surface available to support the snow cover (surface within the periphery of a grate or net).
Supporting structure	Aggregate of structural elements that transfer the forces from the grate or net to the foundations.
Girder	Part of the supporting structure to which the crossbeams of a snow bridge or the purlins of a snow rake are attached.

Foundation

Anchor	Drilled foundation element for the transfer of tension forces.
Concrete foundation	Foundation fabricated on site (e.g. with concrete).
Ground anchor	Drilled anchor for the transfer of tension forces to the ground.
Rock anchor	Drilled anchor for the transfer of tension forces in compact or slightly fissured rock.
Prefabricated foundation	Prefabricated foundation, e.g. ground plate consisting of steel profiles that is installed at the site.
Foundations	Totality of the measures for transferring the loads and forces of a structure to the ground (according to SIA 267: 2003).
Micropile	Drilled foundation element for the transfer of compression forces.
Net anchor	Non-explosive anchor with a stocking to prevent loss of grout.
Non-explosive anchor	Ground anchor for coarse gravel or ground with one or more large outcrops of rock.
Surface zone	Zone parallel to the slope with a thickness of 0.5 m in which the load-bearing capacity of the ground is very marginal.
Pressure bar	Connecting element between the girder and lower foundations to resist compression and tension forces.
Sleeper	Part of the supporting structure lying on, or in, the ground to support the steel or timber crossbeams (snow rake).
Explosive anchor	Ground anchor for gravelly or sandy ground, whose lower end is placed in a blasted cavity subsequently filled with grout.
Anchor length	Length over which the force is transferred to the body of the anchor (according to SIA 267: 2003).

Fig. 1 > Snow bridge.

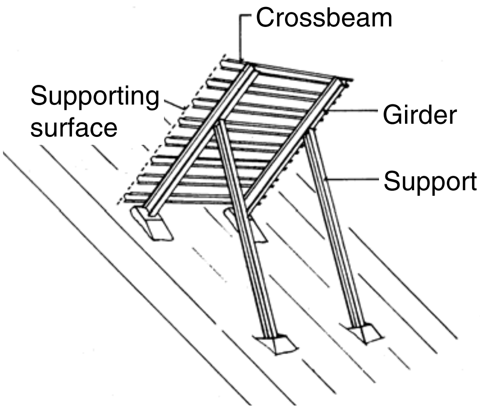


Fig. 2 > Snow rake.

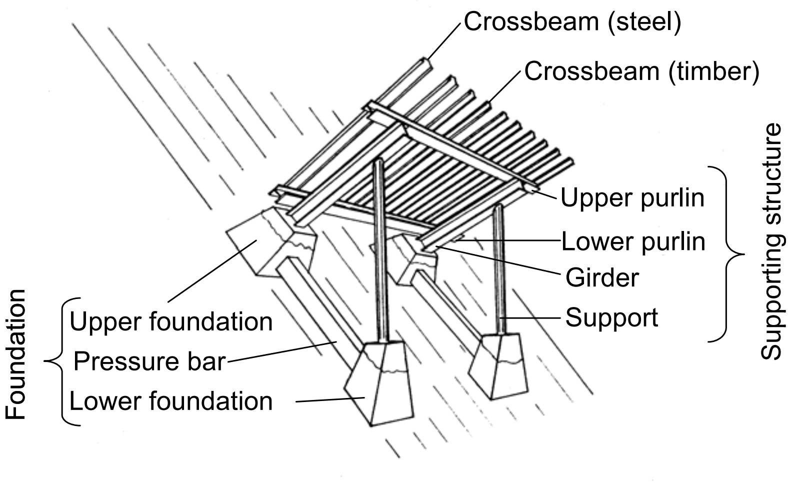
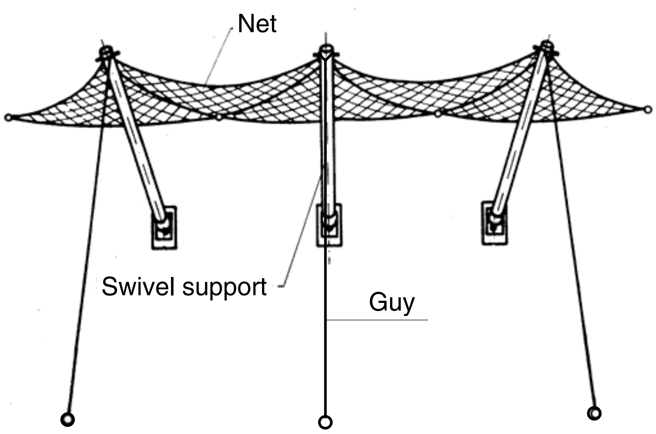


Fig. 3 > Snow net.



2.3 Units and comments on terminology

SI units are used throughout this technical guideline as follows:

> actions: kN, kN/m, kN/m²
> stresses and strengths: N/mm², kN/m²
> the density is defined as mass per unit volume 1 t/m³ = 1000 kg/m³.

Comments on the terminology and notation used in this guideline:

> angles are given in degrees (a circle has 360°).
> a dash (') in designating forces always signifies force per unit length (distributed load).
> forces not designated with a dash refer to resultant forces over a specified length.
> forces in upper case apply to the whole height of the structure, whereas those in lower case apply to elements of the structure or the load per unit area (pressure).
> the technical terms relating to avalanches were taken from the Avalanche Atlas, an illustrated international avalanche classification published in 1981 by the UNESCO.

2.4 Symbols

The symbols used in the present technical guideline may differ from those used in the SIA standards.

Symbol	Unit	Description	Section
A	m	Lateral distance between structures (measured along the contour line)	3.8.1, 5.5.2.4, 8.2.1
a	-	Coefficient for the determination of ε (dependent on the type of snow)	4.3, 5.5.2.2
B_K	m	Height of grate or net (average height of the supporting surface normal to the contour line)	3.6.3, 5.6.1.2, 5.6.1.4
b	m	Loading width for crossbeams	5.6.1.2, 5.8.1.1, 5.8.2.1.1
D_{ext}	m	Extreme snow thickness (peak value of the maximum snow thickness over a period of many years at a particular point)	3.5.3, 3.6.3
D_K	m	Effective height of grate or net (measured average distance of the upper edge of the supporting surface from the ground – analogous to the snow thickness)	3.6.3, 5.5.2.3, 5.5.2.4, 5.6.1.2, 5.8.1.2.1, 8.2.1
D_{max}	m	Maximum snow thickness (maximum snow thickness during the winter at a particular point)	3.5.3
D	m	General snow thickness (measured at right angles to the slope)	3.5.3, 3.6.3, 4.4
E	N/mm²	Elasticity module of the anchor grout	6.2.1.3, 6.2.1.4
E_d	kN	Design value of an action (loading)	5.2.2.1, 5.9.7.1.8
FS	-	Frost resistance of anchor grout	6.2.1.3, 6.2.1.4, 6.2.2.9, 6.3.1.5
F_C	m²	Area of foundation	5.9.5.3.1, 5.9.6.5
F_k	kN	Characteristic value of the tension or compression force in an anchor or micropile	5.9.7.1.6, 5.9.7.1.8, 7.5.4.4, 7.5.4.5, 7.5.4.7
fc	N/mm²	Compressive strength of anchor grout	6.2.1.4, 6.2.2.9, 6.3.1.5

Symbol	Unit	Description	Section
f_C	-	Height factor (accounts for the dependency of the density and the creep factor on altitude)	3.10.1, 3.10.6, 5.5.2.1, 5.5.4, 5.7.4.1, 8.2.1
f_L	-	Distance factor (for the determination of L)	3.7.2
f_R	-	End-effect factor (for the determination of end-effect loads)	3.10.1, 5.5.2.4, 5.5.3.3
f_S	-	Reduction factor for the components of snow pressure parallel to the slope with flexible supporting surface	5.7.4.1
G'	kN/m'	Weight of snow prism bounded by the supporting surface and the vertical plane passing through the intersection of the supporting surface and the ground	4.4, 5.5.2.3, 5.7.4.4
G'_N, G'_Q	kN/m'	Components of G' parallel and normal to the slope respectively	4.4, 5.5.2.5
g	m/s²	Gravitational acceleration	4.2, 4.4
H_{ext}	m	Extreme snow height (peak value of the maximum snow height over a period of many years at a particular point)	3.5.2, 3.5.4, 3.6.2, 3.10.3, 5.5.1
$\overline{H}_{ext}$	m	Extreme snow height averaged over the area (average of the extreme snow heights H_{ext} over a section of the terrain, analogous to H_{max})	3.5.2, 3.5.4
H_K	m	Height of structure (vertical height)	3.4.2.1, 3.6.2, 3.7.2.1, 3.10.3, 5.5.2.1, 5.5.3.1, 5.5.3.4, 5.5.4, 5.7.4.1, 5.8.1.3.3, 5.8.2.3.2, 5.8.3.4
H_{max}	m	Maximum snow height (maximum snow height during the winter at a particular point)	3.5.1, 3.5.2, 3.5.4
$\overline{H}_{max}$	m	Maximum snow height averaged over the area (average of the maximum snow heights H_{max} over a section of the terrain)	3.5.2, 3.5.4
H	m	General snow height (vertical height)	3.10.1, 4.2,
h	m	Snow height corresponding to the snow pressure in load case 2	5.5.3.1, 5.5.3.2
K	-	Creep factor (dependent on the density and the inclination)	3.10.1, 3.10.4, 4.2, 5.5.2.1
L	m	Distance between structures (measured along the line of slope)	3.4.5.2, 3.7.2.1, 3.8.2
l	m	Length of structure (effective length of a single structure measured along the contour line)	3.9.1, 5.8.1.3.4, 5.8.3.5
Δl	m	Length over which the end-effect loads impinge (measured along the contour line)	4.5, 5.5.2.4, 5.5.3.3
N	-	Glide factor (dependent on ground roughness and slope exposure)	3.7.2.3, 3.10.1, 3.10.5, 4.2, 4.3, 4.6.1, 5.5.2.1, 5.5.2.2, 5.5.2.4, 5.5.4, 5.7.4.1, 8.2.1
P'	kN/m'	Component of R' normal to the supporting surface	5.6.1.2
p'_B	kN/m'	Force on a crossbeam normal to the supporting surface	5.6.1.2, 5.8.1.2.2, 5.8.1.2.4, 5.8.2.2
p_h	kN/m²	Snow pressure normal to the supporting surface in load case 2	5.6.1.2, 5.6.1.3, 5.8.1.2.2, 5.8.2.2
Q'	kN/m'	Component of R' parallel to the supporting surface	5.8.1.2.1
Q_k	kN	Characteristic value of a variable action	5.2.2.1
q'_B	kN/m'	Load on a crossbeam parallel to the supporting surface	5.8.1.2.1, 5.8.1.2.2, 5.8.1.2.3, 5.8.1.2.4
q_h	kN/m²	Snow pressure parallel to the supporting surface in load case 2	5.8.1.2.1
q'_S	kN/m'	Lateral loading of support normal to the axis of the support	4.6.1, 5.5.4
R'	kN/m'	Resultant of all snow pressure forces	5.5.2.5, 5.5.2.6, 5.6.1.2, 5.8.1.2.1,
R_d	kN	Design resistance as specified in the SIA standards	5.2.2.1, 5.2.2.2, 5.2.2.4, 5.2.3.2, 5.2.3.3, 5.9.7.1.8
R_k	kN	Characteristic value of the load-bearing capacity according to the SIA standards	5.2.2.1, 5.2.3.3
$R_{a,k}$	kN	Characteristic external resistance of an anchor	5.9.7.1.5, 5.9.7.1.8, 5.9.7.2.5,

Symbol	Unit	Description	Section
			5.9.7.4.4, 5.9.7.5.5, 7.5.4.4
S'_N	kN/m'	Component of snow pressure in the line of slope (creep and glide pressure)	4.2, 4.3, 4.5, 4.6.1, 5.5.2.1, 5.5.2.2, 5.5.2.4, 5.5.2.5, 5.5.6, 5.7.4.1
S'_Q	kN/m'	Snow pressure component normal to the slope (creep pressure)	4.3, 5.5.2.2, 5.5.2.5, 5.7.4.3
S'_R	kN/m'	Additional snow pressure component in the line of slope at the end of a supporting surface (end-effect force)	4.5, 5.5.2.4, 5.5.2.5, 5.6.1.4
S_S	kN	Lateral load of a supporting structure (parallel to the contour line)	4.7, 5.5.6, 5.7.4.3, 5.9.7.3.2
s_B	kN/m²	Ultimate shear resistance in the undisturbed ground along the surface of a concrete foundation (tension load)	5.9.5.4, 5.9.6.4
s^*_B	kN/m²	Ultimate shear resistance in the refilled ground material along the surface of a prefabricated foundation (tension loading)	5.9.6.4
T_k	kN	Characteristic value of the resultant foundation force impinging on the upper foundation	5.9.5.3.1, 5.9.5.3.2, 5.9.5.4, 5.9.6.3, 5.9.6.4
t	m	Foundation depth (measured in the vertical)	5.9.5.4, 5.9.6.4
U_k	kN	Characteristic value of the resultant foundation force impinging on the lower foundation	5.9.4.2, 5.9.6.5, 5.9.6.6
w	m	Width of opening between members of the grate	5.8.1.3.1, 5.8.2.3.1, 5.8.3.3
Z	m a.sl	Altitude	3.5.4, 3.10.6
α	°	Angle between direction of force and the line of slope (refers to foundations)	8.9.6.6, 5.9.4.4, 5.9.4.5
δ	°	Angle between the supporting surface and the plane normal to the slope	4.4, 5.3.2, 5.5.2.3, 5.6.1.2, 5.8.1.2.1
γ_M	-	Coefficient of resistance	5.2.2.1, 5.2.2.2, 5.2.2.4, 5.2.3.2, 5.2.3.3, 5.9.4.1, 5.9.7.1.8
γ_Q	-	Load coefficient for variable action	5.2.2.1, 5.2.3.1, 5.9.4.1, 5.9.7.1.8
ε	°	Angle between the snow pressure resulting from S'_N and S'_Q (vectorial addition) and the line of slope	4.3, 5.5.2.2,
ε_R	°	Angle between the resultant of all snow pressure forces and the line of slope	5.5.2.6, 5.6.1.2, 5.8.1.2.1
ε_{cs}	%	Change in length (shrinkage) of anchor grout	6.2.1.4
η	-	Influence factor of a supporting structure in regard to snow pressure	4.6.1, 4.6.2, 5.5.4
ρ_H	t/m³	Average density of snow corresponding to snow height H_{ext}	3.10.2, 5.5.2.1, 5.5.3.4
ρ_h	t/m³	Average density of snow corresponding to snow height h	5.5.3.4
ρ	t/m³	General average density of snow	3.10.1, 4.2, 4.4, 5.7.4.4
σ_α	kN/m²	Specific total ground resistance	5.9.5.3.1, 5.9.4.4, 5.9.6.5
$\sigma_{90°}$	kN/m²	Total ground resistance normal to the slope	5.9.4.4, 5.9.4.6
σ_O	kN/m²	Total ground resistance in the line of slope	5.9.4.4
φ	°	Angle of friction for glide motion of snow over the ground	3.7.2.1, 3.7.2.2, 3.7.2.3
φ_{Ek}	°	Characteristic angle of friction for transfer of pressure forces (applies to foundations)	5.9.5.4, 5.9.6.4, 5.9.6.6
ψ	°	Inclination of slope	3.5.3, 4.2, 4.3, 4.4, 5.5.2.2, 5.5.2.3, 5.9.4.4, 8.2.1

3 > Planning of supporting structures

3.1 Avalanche formation mechanisms

3.1.1 Snow slab avalanches

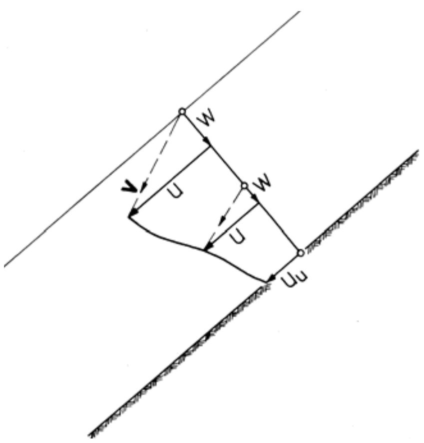
3.1.1.1 Creep and glide formation

Fig. 4 shows a layer of snow resting on a slope. In the layer, creep movement takes place and – under certain conditions between the ground and the snow – glide motion may occur at the ground surface.

The motion depends on the following factors:

> inclination of slope
> snow thickness
> ground roughness
> snow characteristics (deformability, friction, and in particular wetting of the boundary between the ground and the snow).

Fig. 4 > Creep and glide velocities in the snow cover.



v (u,v,w) resultant velocity vector
u Velocity component in the line of slope
u_u Glide velocity
$u-u_u$ Glide velocity in the line of slope
w Creep velocity normal to the slope

3.1.1.2 Neutral zone

Where no local changes in these factors occur, the velocity profiles are identical from one point to another. In this case, the weight of the snow cover is transferred directly to the ground by normal pressure and shear stress at each point. These conditions characterize the so-called neutral zone, in which no changes in stress occur in directions parallel to the line of slope. As opposed to this, local changes in these factors result in zones having increased tension, compression and shear stresses in planes normal to the slope.

3.1.1.3 Initiation of snow slab avalanches

In snow slab avalanches, a slab of snow glides down in its entirety, rapidly gaining speed. This can only occur when a compact layer of snow lies above a thin, weak, layer or boundary. The break – characterized by a primary shear fracture – starts in the weak layer or boundary, where the local stresses exceed the strength of the snow. Starting from this initial fracture, the break spreads rapidly in all directions. With increasing propagation of the break, secondary cracks occur in the upper snow layer. These result in an upper tensile and a lateral shear fracture. The lower edge of the moving slab (the snow slab as such) forms a stauchwall. The initial break may be triggered either by natural mechanisms (e.g. additional loading by fresh snow, or a reduction in strength caused by a rapid rise in temperature), or by artificial causes such as skiers.

3.1.2 Loose snow avalanches

Loose snow avalanches occur in very loose snow over a minute area when a small packet of snow is loosened spontaneously or by a weak action (falling stone or lump of snow), thereby setting snow particles below it in motion. This movement propagates over a narrow (pear-shaped) region, whereby the mass of snow involved continually increases.

3.1.3 Avalanche formation and inclination

The lowest inclination at which avalanches have been observed is 17° (31%). This particular case may be neglected for practical purposes. Fractures seldom occur at inclinations below 30° (58%). For inclinations above 45°, loose snow avalanches predominate. These lead to a more frequent relief of the slope and hinder the formation of a stressed snow cover, thereby preventing the occurrence of snow slab avalanches.

3.2 Purpose and function of supporting structures

3.2.1 Purpose

The purpose of supporting structures is to prevent avalanches being triggered, or at least to prevent snow movements occurring that could lead to damage. Snow movements cannot be completely prevented. In fully developed avalanches, forces arise that cannot normally be withstood by the supporting structures.

3.2.2 Function

Avalanche supporting structures are designed to withstand the creeping and (at times) sliding snow layer. The structures are anchored in the ground approximately normal to the slope and extend up to the surface of the snow. Thus a restraining effect occurs, so that the creep and glide velocities decrease steadily in the downslope direction towards the structure. Within this so-called back-pressure zone, which normally extends over a distance measured in the line of slope of at least 3 times the vertical snow height (depends to a large extent on the sliding motion), additional compressive stresses in the line of slope develop. These are withstood by the supporting surface, leading to a reduction of the shear (and possibly tension) stresses in the back-pressure zone in front of the supporting structure that are responsible for the formation of snow slabs.

When fractures occur, the supporting structure prevents the old snow pack being dragged downwards, and limits the area of the region in which shear cracks can propagate. Through their braking effect, the supporting structures keep the velocity in check, the chief variable responsible for the occurrence of damage. Finally, the retention capacity of the supporting structures has a beneficial effect.

3.2.3 Freedom in designing and dimensioning the structures

The present technical guideline allows considerable leeway in laying out and dimensioning the structures. This should be exploited to configure the structures in accordance with the requirements of the objects to be protected and/or with the **acceptable residual risk.** In determining these requirements, both the **characteristics of the objects to be protected** (e.g. occupied or unoccupied) and their **topographical siting** in relation to the starting zone, the avalanche track and deposition zone must be considered (NB: special requirements apply when the object to be protected lies within the avalanche track).

3.3 Structure types

3.3.1 Rigid structures

Where the creep and sliding motion of a snow layer is arrested by a supporting surface that is subject to only slight elastic deformation, it is referred to as a rigid supporting surface or a rigid supporting structure (e.g. snow bridge with steel crossbeams, see Fig. 1).

3.3.2 Flexible structures

If the supporting surface is to a certain extent able to follow the movement of the snow layer, the surface or supporting structure is said to be flexible (e.g. snow nets, see Fig. 3).

3.3.3 Loading of a supporting structure

As explained in Section 3.2.2, a supporting structure must withstand both the **snow pressure** and the **dynamic forces.** Whereas dimensioning of the structures is based on the static snow pressure (Section 5), the magnitude of the dynamic forces may be influenced by suitable arrangement of the structures (see Section 3.7) to ensure that they suffer no or very little damage.

3.3.4 Choice of structure

The structure should be chosen in accordance with the requirements of the objects to be protected (Section 3.2.3) and in relation to the local snow, terrain and ground conditions. Snow nets are less sensitive to creep movement and rockfall (cf. Section 7.4.3.1), but more difficult to anchor in loose ground.

3.4 Extent and positioning of a supporting structure

3.4.1 Slopes to be controlled by structures

Supporting structures are generally required for slope inclinations between 30° and 50° (58% and 119%).
In exceptional cases, flatter or steeper terrain in a starting region may need to be controlled, e.g. flatter shoulders above steeper slopes, or flatter sections of the slope.

3.4.2 Positioning of the uppermost structures

3.4.2.1 General

Supporting structures should mainly be installed below the **highest** observed or anticipated **fracture lines** of snow slab avalanches (Section 3.1.1), in such a way that these still lie within the actual back-pressure zone of the structures. As explained in Section 3.2.2, this is the case when the structures are installed not more than 2–3 H_K below the fracture lines.

3.4.2.2 Cornices

Where the slope to be controlled is bounded by a ridge known to form a heavy cornice, the uppermost structures should be positioned as near as possible to the foot of the cornice, without, however, coming to lie within the cornice itself. The structures should be dimensioned very generously to accept the large volume of snow and withstand falling sections of the cornice. In many cases, the mass of the cornice can be reduced by anti-drifting structures. If appropriate, these should be installed prior to erection of the supporting structures.

3.4.2.3 Rocky terrain

Where the upper edge of the slope to be controlled is bounded by very steep, rocky, terrain, the uppermost structures should likewise be very generously dimensioned. Furthermore, where there is a danger of rockfall, they should be provided with a supporting surface having the highest possible resistance to rockfall (for example: snow nets, massive steel grates or timber covering). Where there is a danger of damage to the supporting structures from snow, rock or ice falls from higher ground that cannot be secured, this may be reduced with the help of deflecting or retaining structures (earth dam or rockfall protection net).

3.4.2.4 Secondary starting zones

Supporting structures should mainly be located at the highest observed or anticipated starting zones of snow slab avalanches. Depending on the situation, a check should be made whether avalanches could be triggered in secondary starting zones further above, and which could impinge on the supporting structure. For this, an extreme avalanche situation should be assumed.

3.4.3 Positioning of the lowermost structures

As a result of the supporting structures, new, secondary, starting zones usually occur further down, so that the supporting structure should be extended downslope until either

- the inclination of the slope has finally dropped below approx. **30°** (58%)
- it may be safely assumed that **no damage effect** could arise from avalanches triggered further below and/or from snow volumes originating from within the controlled area.

In making this assessment, the topographical situation and the characteristics of the objects to be protected should be taken into account (see Section 3.2.3).

3.4.4 Arrangement of the structures in relation to the direction of the snow pressure

In plan view, the supporting surfaces of the structures should be positioned as far as possible normal to the anticipated direction of the resultant snow pressure (especially important in narrow gullies).

3.4.5 Lateral extent of supporting structures

3.4.5.1 Fundamental principles

It should always be the objective to position supporting structures well above in the starting zone and design it wide enough to cover an entire natural terrain unit so that it abuts the natural, lateral, boundary lines (i.e. the terrain ribs, Fig. 5). Where the structures terminate in open terrain, reinforced end structures should be used (Section 5.5.2.4).

3.4.5.2 Tapering-back of structures, and separating walls

If owing to the circumstances of the terrain or for economic reasons it is not possible to secure an entire natural terrain unit, the unprotected flank should be **heavily tapered back** in the downward direction. This is to ensure that the lower structures are not damaged by avalanches descending immediately adjacent to the defense structure. To hinder adjacent snow slab avalanches from spreading into the defense zone, additional structures may be placed at the edge of the zone. These should be positioned in the gap between the normal structures (distance L) and have a total length of at least 2 D_K. Separation walls arranged in the line of slope at the side of the structure should have a vertical height of approx. $H_K/2$ to prevent full-depth avalanches spreading to the structure. They substantially reduce the end-effect loads as shown in Section 45. Also, to prevent damage to the supports, the separation walls should be extended down to the downslope foundations (Fig. 6).

Fig. 5 > Complete coverage of a natural terrain unit.

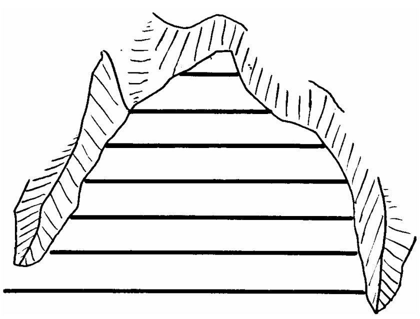
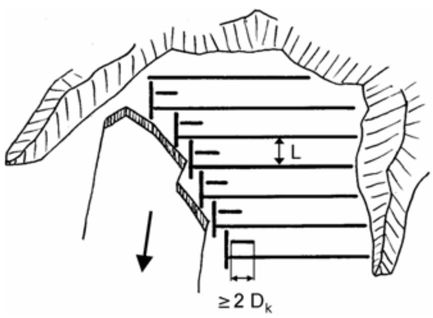


Fig. 6 > Partial coverage. Tapering back and delimitation of the unprotected flank of a structure.



3.4.6 General arrangement of structures

3.4.6.1 Continuous structures

With continuous structures, these consist of long horizontal rows of structures extending across the entire controlled area. They are interrupted only in those sections of the terrain that are unaffected by starting zones (Fig. 7). Continuous structures are the **preferred arrangement** for permanent protection.

3.4.6.2 Separated structures

With separated structures, a distinction must be made between interrupted and staggered arrangements.

3.4.6.2.1 Separated, interrupted structures

With interrupted structures, the arrangement is derived from that of continuous structures by inserting gaps in the horizontal rows (Fig. 8).

3.4.6.2.2 Separated, staggered structures

Staggered structures differ from interrupted structures in that the individual sections alternate in height (Fig. 9).

Fig. 7 > Continuous structure.

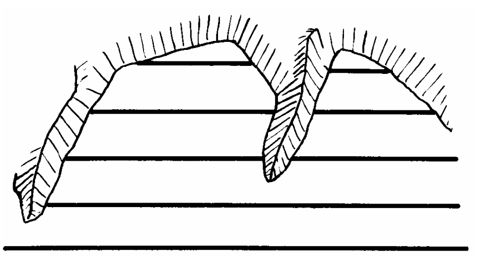


Fig. 8 > Separated, interrupted structure.

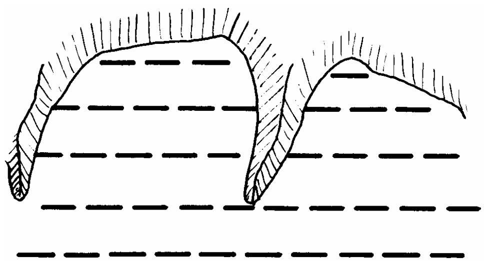


Fig. 9 > Separated, staggered structure.

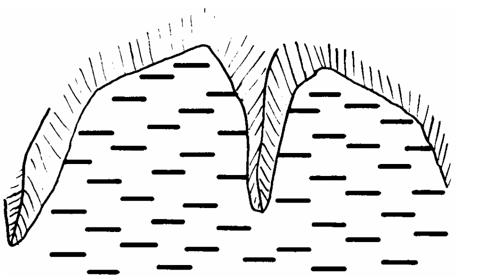
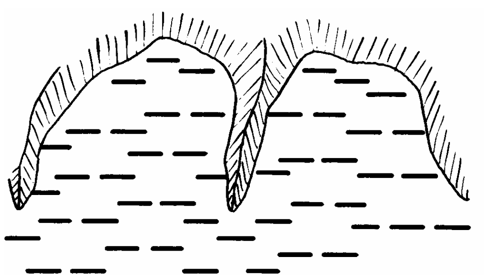


Fig. 10 > Separated, combined structure.



3.4.6.3 Advantages and disadvantages of the different arrangements

All three arrangements have their advantages and disadvantages. These are listed in the following Table 1.

3.4.6.4 Choice of arrangement

The arrangement should be chosen in accordance with the requirements of the objects to be protected (Section 3.2.3) and take account of the local snow conditions and the terrain. Where the safety demands are high, and where loose snow avalanches frequently occur (e.g. at high altitudes and with north-facing starting zones), continuous structures are strongly recommended.

Tab. 1 > Advantages and disadvantages of the various arrangements.

Arrangement	Advantages	Disadvantages	Application
Continuous (Section 3.4.6.1)	• Propagation of shear fractures in the snow cover largely hindered beyond the rows both in the upward and downward directions • Continuous barrier against snow slides • Tension stresses in the snow cover largely avoided • Loading of the structures by end-effect loads only at the ends of the rows (minimum total loading caused by snow pressure)	• Large-scale lateral distribution of remaining shear and tension zones in the snow cover • Possible lateral propagation of damage to the structures • Limited adaptability to heavily irregular terrain and large local variations in snow conditions (more or less relevant depending on the type of structure used)	• Normal case
Separated, interrupted (Section 3.4.6.2.1)	• Good horizontal adaptability to the horizontal terrain features and to locally changing snow conditions • Restriction of damage to individual sections • Possible cost savings (as against continuous structures)	• Partial penetration of snow between the gaps in the structures • Loading of the structures by end-effect loads as a function of the distance between the structures • More prone to propagation of shear fractures in the snow cover beyond the rows both in the upward and downward directions (as against continuous construction)	• In exceptional cases in zones with (e.g.) rock ribs or local steps in the terrain
Separated, staggered (Section 3.4.6.2.2)	• Good adaptation to the terrain in all directions • Distribution of remaining tension and shear stress zones • On average, reduced snow glide as against continuous, and separated, interrupted, arrangements	• Loading of the structures by end-effect loads corresponding to those on an independent structure • Higher cost per m (as against continuous, and separated, interrupted, structures) • Possible propagation of shear fractures in all directions	• In exceptional cases in very steep and heavily irregular terrain, and also where there is a concentration of older supporting structures not conforming to the guideline

3.5 Snow height

3.5.1 General definition

The snow height H is measured in the vertical direction. It is characteristic of the snow cover in the terrain. When the snowfall is uniform and vertical (no wind), the snow height is **independent of the inclination**.

3.5.2 Definition of snow heights

> **Maximum snow height H_{max}:** maximum height of snow during the winter at a particular point (e.g. at the site of a supporting structure).
> **Maximum snow height $\bar{H}_{max}$ averaged over the area:** average of the maximum snow heights H_{max} over an extended section of the terrain at the time of occurrence of the general maximum snow height during the winter.
> **Extreme snow height H_{ext}:** the anticipated maximum value of the maximum snow heights H_{max} over a long period at a particular point (e.g. at the site of a supporting structure).
> **Extreme snow height $\bar{H}_{ext}$ averaged over the area:** average of the extreme snow heights H_{ext} over an extended section of the terrain at the time of occurrence of extreme snow cover (occurs on average not more than once in 100 years).

3.5.3 Definition of snow thickness

The snow thickness is the height of snow cover measured normal to the surface of the ground and is designated by the symbol D (D, D_{max}, D_{ext}, etc.). The snow thickness D is a function of snow height H as follows:

$$\boxed{D = H \cdot \cos\psi} \qquad [m] \qquad (1)$$

3.5.4 Determination of extreme snow height

The extreme snow heights H_{ext} at the site of the structure are decisive in planning it (Section 3.6.2). The effectiveness of a supporting structure depends primarily on a reliable determination of these values. However, in most cases long-period observations of snow heights at the sites of supporting structures are not available, so that the required measurement series must be taken from neighbouring observation stations. For this purpose, the SLF reference stations may, for example, be used (see SLF winter reports). The snow heights or the precipitation measured there are representative of a wider area, largely enabling perturbations due to local topographical conditions to be avoided (e.g. with a station in a horizontal location at the foot of a valley). Values measured in this way at a single point may therefore be regarded as average values (area average). The large-scale distribution of the area averages of the extreme snow heights $\bar{H}_{ext}$ measured in this way is shown in Fig. 11 for the region of the Swiss Alps.

The figure is based on measurements of snow height at SLF reference stations and the automatic ENET stations (SLF/MeteoSchweiz), for which measurements over periods of between 10 and 66 years are available. The snow heights quoted do not take account of wind effect. The chart was converted to a **common recurrence interval of 100 years,** and applies to the region of the Swiss Alps. The dependency of $\bar{H}_{ext}$ on altitude in the four zones is as follows (see Fig. 11):

$$\text{Zone 1: } \bar{H}_{ext} = 1.00\,(0.15 \cdot Z–20) \qquad (2)$$
$$\text{Zone 2: } \bar{H}_{ext} = 1.30\,(0.15 \cdot Z–20) \qquad (3)$$
$$\text{Zone 3: } \bar{H}_{ext} = 1.65\,(0.15 \cdot Z–20) \qquad (4)$$
$$\text{Zone 4: } \bar{H}_{ext} = 2.00\,(0.15 \cdot Z–20) \qquad (5)$$

$\bar{H}_{ext}$ is the area average of the extreme snow heights in cm and Z is the altitude in m a.s.l.

The calculation of the extreme snow height to be used at the site of the supporting structure is performed as follows:

> Measurement of the **maximum snow height H_{max}** at the site of the intended structure, if possible during several winters, with the aid of depth probes or with snow stakes. We are concerned here with local variations. The number of measurement points should therefore be adjusted to suit the terrain in such a way that any local changes in snow height (e.g. in narrow gullies) can be detected. As a general rule 25–100 depth probes or snow stakes per hectare should be taken. Useful observations of the variation in snow height can often be made during the snow melt period.
> Simultaneously with the measurement of the maximum snow heights, the **area average of the maximum snow height $\bar{H}_{max}$** in a section of the terrain must be measured, and this should as far as possible be representative of the snow height over a wider area. In addition, the observations from one or more SLF reference stations in the vicinity, or values from suitably positioned snow stakes, can be incorporated. In general, the area covered by the supporting structures is not suitable for these measurements since the whole of this is located at an exceptional point, e.g. on the windward or leeward side of a slope (medium-scale distribution).
> Determination of the **area average of the extreme snow height $\bar{H}_{ext}$** using Fig. 11 or employing other reliable data (large-scale distribution). Further information may be obtained on request from the SLF.
> Calculation of the **extreme snow height H_{ext}** at the site of a structure on the assumption that the distribution of snow heights remains similar from one year to the next independently of the snow height:

$$\boxed{H_{ext} = H_{max}\,\frac{\bar{H}_{ext}}{\bar{H}_{max}}} \qquad [m] \qquad (6)$$

Where measurements are made over several years – which should always be the objective – the values of H_{ext} will be found to vary from year to year. In these cases, the most reliable value is that calculated from the largest measured value of $\overline{H}_{max}$. Where, however, the snow heights remain approximately the same over several years, the largest value of H_{ext} calculated should be used for dimensioning purposes.

Example:

On the Dorfberg above Davos at an altitude of 2266 m a.s.l. at the site of a future supporting structure, the maximum snow heights $\overline{H}_{max}$ were measured during 3 winters with a snow stake. The SLF test area on the Weissfluhjoch at 2540 m a.s.l., which lies not far from the site, provides snow height values H_{max} valid over a wide area for the same days as measured at the above site (this assumes, however, that the snow stake measurements are in accordance with the large-area snow heights!).

Fig. 11 shows that the SLF test area belongs to zone 2. The area average of the extreme snow heights $\overline{H}_{ext}$ may therefore be calculated from (3) as follows:

$\overline{H}_{ext} = 1.30\ (0.15 \cdot 2540 - 20) = \underline{469\ \text{cm}}$

Date Snow heights m:	8.2.1961	7.4.1962	17.1.1963
- H_{max}	1.50	2.20	1.20
- $\overline{H}_{max}$	2.38	2.75	1.40
- $\overline{H}_{ext}$	4.69	4.69	4.69
Daraus: - H_{ext}	$1.50\frac{4.69}{2.38} = \underline{\underline{2.96}}$	$2.20\frac{4.69}{2.75} = \underline{\underline{3.75}}$	$1.20\frac{4.69}{1.40} = \underline{\underline{4.02}}$

A design value of 3.75 m should be used. The largest absolute value of 4.02 m is insufficiently reliable since it was calculated from a much smaller value of H_{max}.

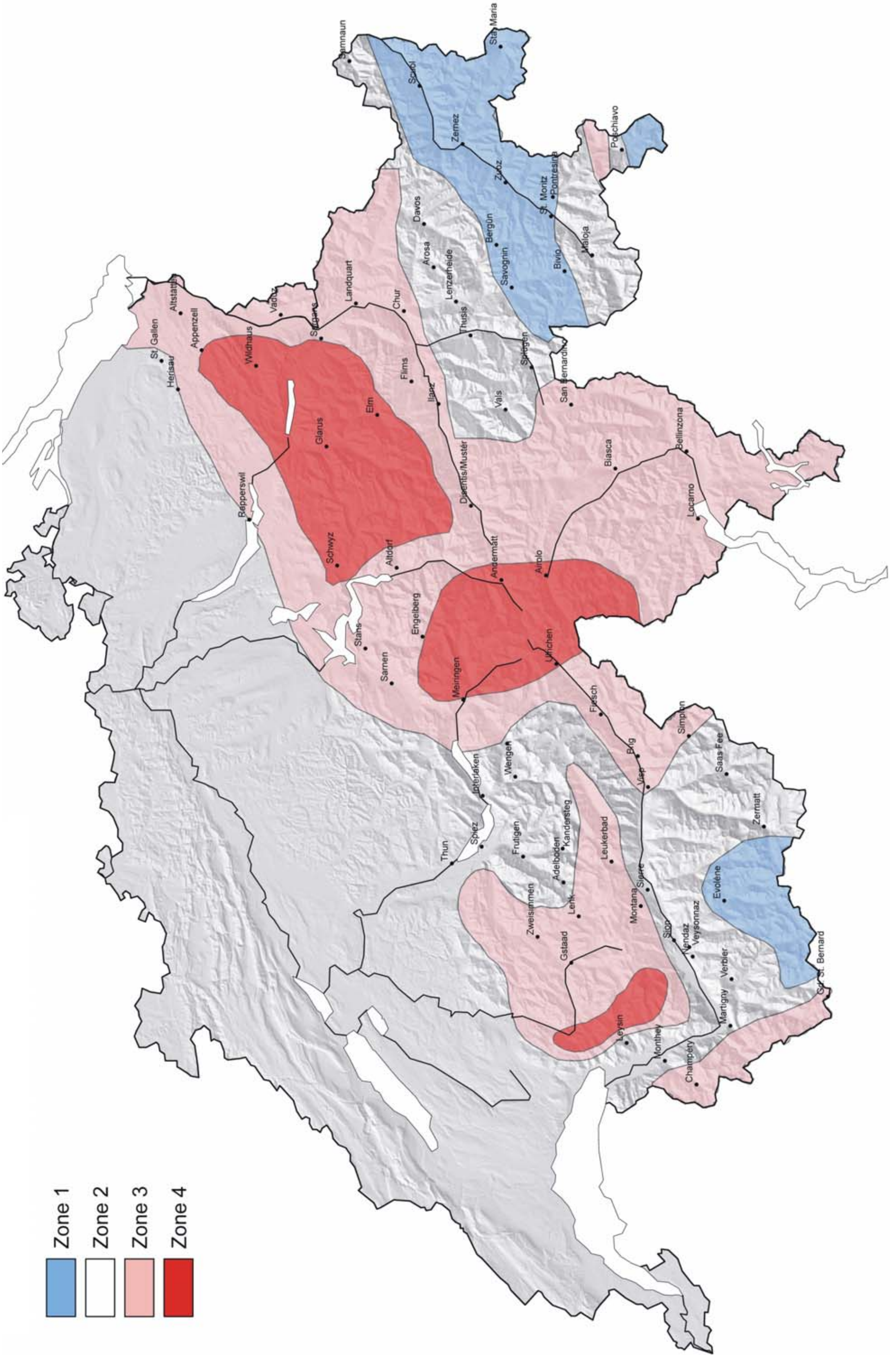


Fig. 11 > Area average of the extreme snow heights $\overline{H}_{ext}$.

3.6 Height of structure

3.6.1 Definition of height of structure

The height of the structure H_K is defined as the average vertical distance from the upper edge of the supporting surface to the ground. Definitions of the different types of structure are given in Section 5 (snow bridge: Section 5.8.1.3.3; snow rake: Section 5.8.2.3.2 and snow net: Section 5.8.3.4).

3.6.2 Determination of height of structure

The height of the structure H_K must be at least as great as the extreme snow height anticipated at the site of the supporting structure.

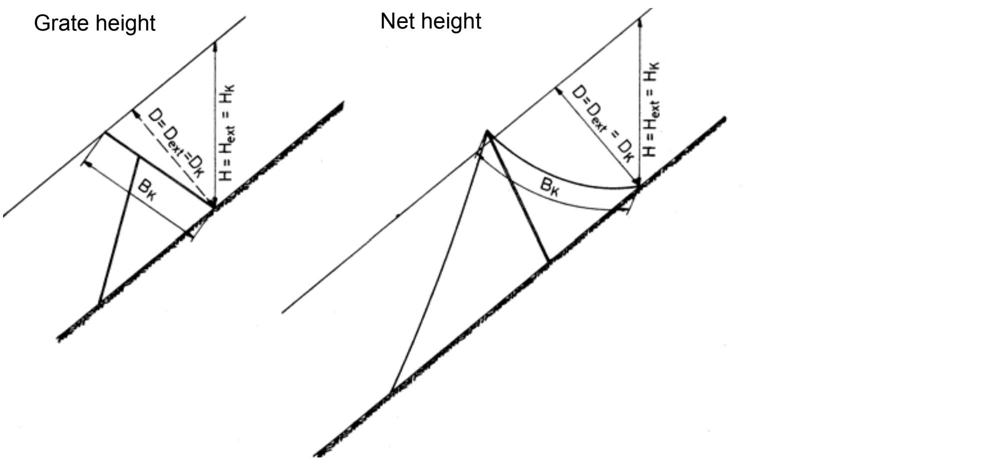
$$\boxed{H_K \geq H_{ext}} \quad [m] \qquad (7)$$

This is the fundamental condition to be fulfilled to provide protection from avalanches during natural catastrophes, and dictates the procedures for dimensioning the defense structures. Where $H_K > H_{ext}$ is chosen, the supporting structures must be dimensioned based on H_K throughout. Note that depending on the design of the supporting structures and the wind conditions, these may have a substantial influence on the quantity of snow deposited.

3.6.3 Definition of grate and net heights

The grate or net height B_K is defined as the average width of the supporting surface normal to the contour line. It is bounded at the lower end by the surface of the ground (Fig. 12).
The effective grate or net height D_K is defined in a similar way to the snow thickness as the average distance of the upper edge of the supporting surface from the ground normal to the line of slope.

Fig. 12 > Grate and net heights.



3.7 Distance between structures in the line of slope

3.7.1 Determination of the distance between structures

The distance between structures and rows of structures in the line of slope should be so dimensioned that in addition to fulfilling the objective of the supporting structure according to Section 3.2.1, the following three conditions are all met:

- the structures should suffer no damage from the static effect of the maximum snow pressure
- likewise, the dynamic loads resulting from snow movement should be sustained without damage
- the velocity of snow movement within the supporting structure should not exceed a certain limiting value. The energy of motion is limited by the structure to a value below that which would cause damage to buildings etc. lying below the structure.

3.7.2 Calculation of distance between structures

3.7.2.1 Calculation of the distance in the line of slope

The distance L in the line of slope is calculated from:

$$\boxed{L = f_L \cdot H_K} \qquad [m] \qquad (8)$$

The distance factor f_L depends on the inclination of the slope and – in accordance with the three conditions in Section 3.7.1 – on the angle of friction φ between the ground and the snow, on the glide factor N and on the height of the structure H_K. Fig. 13 shows f_L as a function of the parameters mentioned. The value of f_L under the given conditions may be obtained from the 3 families of curves tan φ, N and H_K = const.

The distances L (in the line of slope) and L' (plan view) may also be obtained directly as a function of D_K from Tables 2.1 and 2.2, or 3.1 and 3.2, respectively.

3.7.2.2 Safety requirements and ground surface

- With smooth ground (N > 2) or for higher safety requirements, values of tan φ = 0.55 and 0.50 should be used.
- For rough ground (N < 2) and where no particular safety requirements are imposed, tan φ should be chosen as 0.60.

3.7.2.3 Maximum permissible values of the distance factor

The curves for tan φ = 0.60, N > 1.3 and f_L = 13 give the highest permissible values for the distance factor f_L.

3.7.2.4 Lowest glide factor for dimensioning the structures

Where the structures are dimensioned based on a glide factor N = 1.2, the distance factor must not be chosen to lie above the curve for this value.

3.7.2.5 Large structure heights

Where the vertical structure heights H_K exceed 4.5 m, the maximum permissible values for f_L lie on the curves correspondingly designated.

3.7.2.6 Freedom of action

The freedom of action permitted in the distance calculation according to Section 3.2.3 should be exploited to configure the supporting structure in a way commensurate with the requirements of the objects to be protected. It is normally recommended to choose f_L for tan φ between 0.55 and 0.50.

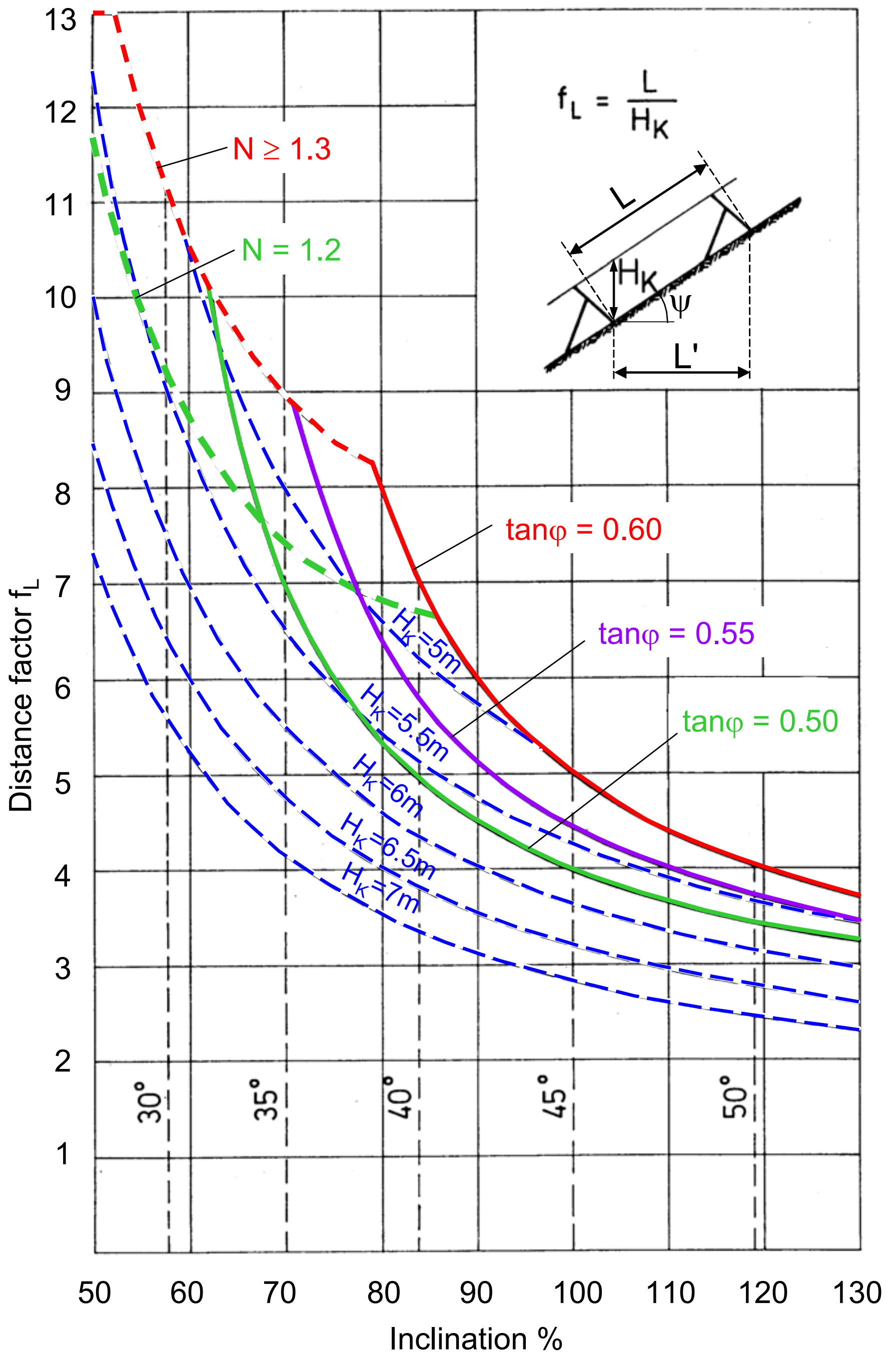
3.7.2.7 Climate

To achieve sufficient protection from avalanches being triggered, the climate should also be taken into account in determining the distance between structures. Particularly on north-facing slopes and/or in pre-Alpine regions with heavy precipitation, values lower than for $\tan \varphi = 0.50$ may in certain circumstances have to be chosen.

3.7.2.8 Variable slope inclination

Where the inclination varies within the structures, ψ is chosen as the inclination of the straight line between the foundations of the relevant structures in calculating L.

Fig. 13 > Distance factor f_L.



Tab. 2.1 > Distance between structures in the line of slope L [m] according to Fig. 13.

Inclination of slope	D_K [m]	H_K [m]	L [m]					
			N = 1.2			N ≥ 1.3		
			tan φ =			tan φ =		
			0.60	0.55	0.50	0.60	0.55	0.50
60 % (31°)	1.5	1.75		15.3			18.4	
	2.0	2.33		20.3			24.6	
	2.5	2.92		25.4			30.7	
	3.0	3.50		30.5			36.9	
	3.5	4.08		35.6			43.1	
	4.0	4.66		40.7			49.2	
	4.5	5.25		45.8			49.1	
	5.0	5.83		43.3			43.3	
70 % (35°)	1.5	1.83		13.6	12.8		16.4	12.8
	2.0	2.44		18.1	17.1		21.8	17.1
	2.5	3.05		22.7	21.4		27.3	21.4
	3.0	3.66		27.2	25.6		32.7	25.6
	3.5	4.27		31.8	29.9		38.2	29.9
	4.0	4.88		36.3	34.2		43.6	34.2
	4.5	5.49		35.9			35.9	
	5.0	6.10		32.5			32.5	
80 % (38.7°)	1.5	1.92	13.1	12.3	10.2	15.4	12.3	10.2
	2.0	2.56	17.4	16.4	13.7	20.5	16.4	13.7
	2.5	3.20	21.8	20.5	17.1	25.6	20.5	17.1
	3.0	3.84	26.2	24.6	20.5	30.7	24.6	20.5
	3.5	4.48	30.5	28.7	23.9	35.9	28.7	23.9
	4.0	5.12		32.1	27.3		32.1	27.3
	4.5	5.76		28.6			28.6	
	5.0	6.40		26.4			26.4	

Tab. 2.2 > Distance between structures in the line of slope L [m] according to Fig. 13.

Inclination of slope	D_K [m]	H_K [m]	L [m]		
			N ≥ 1.2		
			tan φ =		
			0.60	0.55	0.50
90 % (42°)	1.5	2.02	12.1	10.4	9.1
	2.0	2.69	16.1	13.8	12.1
	2.5	3.36	20.2	17.3	15.1
	3.0	4.04	24.2	20.8	18.2
	3.5	4.71	28.2	24.2	21.2
	4.0	5.38		26.5	24.2
	4.5	6.05		24.1	
	5.0	6.73		22.4	
100 % (45°)	1.5	2.12	10.6	9.4	8.5
	2.0	2.83	14.1	12.6	11.3
	2.5	3.54	17.7	15.7	14.1
	3.0	4.24	21.2	18.9	17.0
	3.5	4.95	24.7	22.0	19.8
	4.0	5.66		22.8	22.6
	4.5	6.36		21.0	
	5.0	7.07		19.7	
110 % (47.7°)	1.5	2.23	9.8	8.9	8.2
	2.0	2.97	13.1	11.9	10.9
	2.5	3.72	16.3	14.9	13.6
	3.0	4.46	19.6	17.8	16.3
	3.5	5.20	22.5	20.8	19.1
	4.0	5.95		20.2	
	4.5	6.69		18.8	
	5.0	7.43		17.7	
120 % (50.2°)	1.5	2.34	9.4	8.6	8.0
	2.0	3.12	12.5	11.5	10.7
	2.5	3.91	15.6	14.4	13.4
	3.0	4.69	18.7	17.3	16.1
	3.5	5.47		20.1	18.7
	4.0	6.25		18.3	
	4.5	7.03		17.1	
	5.0	7.81		16.2	
130 % (52.4°)	1.5	2.46	9.1	8.5	8.0
	2.0	3.28	12.2	11.4	10.7
	2.5	4.10	15.2	14.2	13.3
	3.0	4.92	18.3	17.1	16.0
	3.5	5.74		18.3	
	4.0	6.56		16.8	
	4.5	7.38		15.8	
	5.0	8.20		15.1	

Tab. 3.1 > Distance between structures L' [m] in plan view according to Fig. 13.

Inclination of slope	D_K [m]	H_K [m]	$L' = L \cdot \cos \psi$ [m]					
			N = 1.2			N ≥ 1.3		
			tan φ =			tan φ =		
			0.60	0.55	0.50	0.60	0.55	0.50
60 % (31°)	1.5	1.75		13.1			15.8	
	2.0	2.33		17.4			21.1	
	2.5	2.92		21.8			26.4	
	3.0	3.50		26.2			31.6	
	3.5	4.08		30.5			36.9	
	4.0	4.66		34.9			42.2	
	4.5	5.25		39.3			42.1	
	5.0	5.83		37.1			37.1	
70 % (35°)	1.5	1.83		11.1	10.5		13.4	10.5
	2.0	2.44		14.9	14.0		17.9	14.0
	2.5	3.05		18.6	17.5		22.3	17.5
	3.0	3.66		22.3	21.0		26.8	21.0
	3.5	4.27		26.0	24.5		31.3	24.5
	4.0	4.88		29.7	28.0		35.7	28.0
	4.5	5.49		29.4			29.4	
	5.0	6.10		26.6			26.6	
80 % (38.7°)	1.5	1.92	10.2	9.6	8.0	12.0	9.6	8.0
	2.0	2.56	13.6	12.8	10.7	16.0	12.8	10.7
	2.5	3.20	17.0	16.0	13.3	20.0	16.0	13.3
	3.0	3.84	20.4	19.2	16.0	24.0	19.2	16.0
	3.5	4.48	23.8	22.4	18.7	28.0	22.4	18.7
	4.0	5.12		25.1	21.3		25.1	21.3
	4.5	5.76		22.4			22.4	
	5.0	6.40		20.6			20.6	

Tab. 3.2 > Distance between structures L' [m] in plan view according to Fig. 13.

Inclination of slope	D_K [m]	H_K [m]	L' = L · cos ψ [m]		
			N ≥ 1.2		
			tan φ =		
			0.60	0.55	0.50
90 % (42°)	1.5	2.02	9.0	7.7	6.7
	2.0	2.69	12.0	10.3	9.0
	2.5	3.36	15.0	12.9	11.2
	3.0	4.04	18.0	15.4	13.5
	3.5	4.71	21.0	18.0	15.7
	4.0	5.38		19.7	18.0
	4.5	6.05		17.9	
	5.0	6.73		16.7	
100 % (45°)	1.5	2.12	7.5	6.7	6.0
	2.0	2.83	10.0	8.9	8.0
	2.5	3.54	12.5	11.1	10.0
	3.0	4.24	15.0	13.3	12.0
	3.5	4.95	17.5	15.6	14.0
	4.0	5.66		16.1	16.0
	4.5	6.36		14.8	
	5.0	7.07		13.9	
110 % (47.7°)	1.5	2.23	6.6	6.0	5.5
	2.0	2.97	8.8	8.0	7.3
	2.5	3.72	11.0	10.0	9.2
	3.0	4.46	13.2	12.0	11.0
	3.5	5.20	15.1	14.0	12.8
	4.0	5.95		13.6	
	4.5	6.69		12.6	
	5.0	7.43		11.9	
120 % (50.2°)	1.5	2.34	6.0	5.5	5.1
	2.0	3.12	8.0	7.4	6.9
	2.5	3.91	10.0	9.2	8.6
	3.0	4.69	12.0	11.1	10.3
	3.5	5.47		12.8	12.0
	4.0	6.25		11.7	
	4.5	7.03		10.9	
	5.0	7.81		10.4	
130 % (52.4°)	1.5	2.46	5.6	5.2	4.9
	2.0	3.28	7.4	6.9	6.5
	2.5	4.10	9.3	8.7	8.1
	3.0	4.92	11.1	10.4	9.7
	3.5	5.74		11.1	
	4.0	6.56		10.2	
	4.5	7.38		9.6	
	5.0	8.20		9.2	

3.8 Lateral distance between structures

3.8.1 Interrupted arrangement

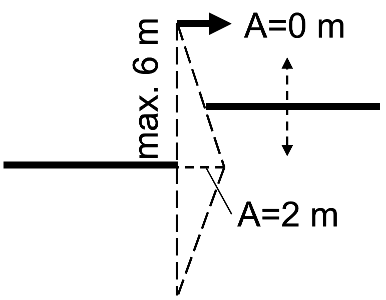
With interrupted arrangement, the lateral distance A between neighboring structures (does not apply to sections of the terrain not affected by avalanches) is limited to 2 m.

$$\boxed{A \leq 2\,\mathrm{m}} \qquad [m] \qquad (9)$$

The structures must be fully protected from above by structures spaced at a distance of L (does not apply to the upper row of structures).

Where laterally adjacent structures are displaced slightly with respect to one another in the line of slope, the gap (or more precisely its projection in the line of slope) must be closed increasingly in relation to the displacement as shown in Fig. 14.

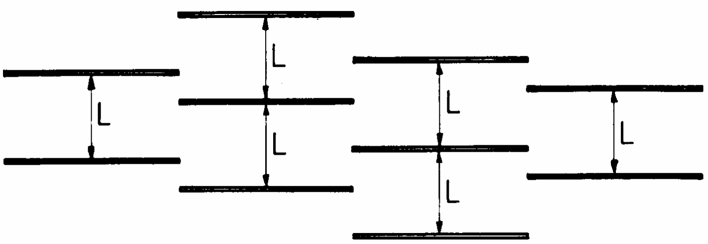
Fig. 14 > Partial closure of the gap between structures.



3.8.2 Staggered structures

With staggered structures, the gaps may be chosen at will, whereby gaps of over 2 m must either be fully protected from above by structures with the normal gap L (Fig. 15), or partially closed as given in Section 3.8.1.

Fig. 15 > Distances between structures for staggered arrangement.



3.9 Lengths of continuous support grates

3.9.1 Definition

Continuous support grates consist of a continuous arrangement of single structures. The length l of a single structure (without intermediate structures) is the average effective length of the supporting surface measured along the contour line (snow bridge: see Section 5.8.1.3.4; snow net: see Section 5.8.3.5).

3.9.2 Maximum and minimum lengths

Normally, the minimum length of a continuous support grate should not be less than 16 to 22 m. This applies to all categories of arrangement.

For practical reasons (i.e. to permit access) they should not exceed a length of approx. 50 m.

3.10 Site factors for snow pressure

3.10.1 Definitions

The snow pressure on a supporting structure depends on the following site factors:

> ρ average density of snow*
> H vertical snow height at site of structure
> K creep factor*, dependent on density and inclination of the slope
> N glide factor, dependent on vegetation, roughness and solar exposure of the ground
> f_c altitude factor, characterizing the dependency of the density on altitude
> f_R end-effect factor, dependent on the lateral distance between structures (and on the arrangement of the structures) and on the glide factor.

Certain of these factors must be determined for all avalanche defense projects, and in some cases for every structure in the terrain. Certain other factors are set based on generally valid relationships. The latter are marked by an asterisk* in the above list. The calculation of snow pressure from the above factors is given in Sections 4 and 5.

3.10.2 Snow density

The average snow density is set to a uniform value of $\rho_H = 0.270$ t/m³, a value which would occur in the case of an **extreme snow height.** This value applies in the Swiss Alps at an altitude of 1500 m a.s.l. and an exposure of WNW-N-ENE. The variation of this basic value with altitude and slope exposure is expressed by the altitude factor f_c (Section 3.10.6) and the glide factor N (Section 3.10.5). The increase in density as a result of settling of the snow cover, starting from the above basic value, is accounted for in the dimensioning instructions (Section 5.5.3).

3.10.3 Snow height at site of structure

The basic starting value for the calculation of snow pressure is the structure height H_K, which is calculated from the extreme snow height H_{ext} as given in Section 3.6.2.

3.10.4 Creep factor

The values for the creep factor K as a function of the density and the inclination of the slope are given in Section 4.2 (Tab. 6). In practice, the dependency on inclination in the region 35°–45° is neglected (assumption: $\sin 2\psi = 1$).

3.10.5 Glide conditions and glide factor

The glide factor N, which expresses the increase in snow pressure for movement of the snow cover along the ground (see Section 3.1.1.1), depends on the ground roughness and the slope exposure (solar exposure). It is classified in 4 ground classes and 2 exposure sectors (see Tab. 5).

For surface types lying between the specified classes, intermediate values of N can be interpolated. When the inclination of the terrain lies above 45°, fairly strict conditions must be applied in determining N; for inclinations below 35°, the conditions can be somewhat relaxed. At high glide factors, an assessment should always be made as to whether an **artificial increase in ground roughness** (terracing, piling etc.) might be more economical than more generous dimensioning of the structures. In cases where one of the usual types of wooden snow rake are erected temporarily, whose upper foundations can normally only withstand small tension forces, an increase in roughness should in any case be provided (applies only under these particular circumstances).

3.10.6 Altitude factor

The altitude factor f_c is not an independent constituent of the snow pressure formula, but is coupled to the determination of the density. It represents the generally observed increase in average density with altitude Z (m a.s.l.) and includes the related increase in the creep factor. The increment in snow pressure with altitude between 1500 and 3000 m a.s.l. is set to 2% per 100 m as follows:

$$\boxed{f_c = 1 + 0.02\left(\frac{Z}{100} - 15\right)} \qquad (10)$$

Tab. 4 > Altitude factor as a function of altitude.

For altitudes below 1500 m a.s.l., f_c is set to 1.00, and above 3000 m a.s.l. to 1.30.

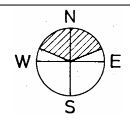
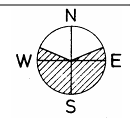
Z:	m a.s.l.	1500	1600	1800	2000	2200	2400	2600	2800	3000
f_c:	-	1.00	1.02	1.06	1.10	1.14	1.18	1.22	1.26	1.30

3.11 Foundation conditions

The planning procedure comprises, among other things, a thorough assessment of the foundation conditions. This must include

> An assessment of the geological structure of the ground (depth of rock, type and fissuring of the rock, type of rock cover, humidity and frost conditions, movement of loose ground [solifluction], possible chemical reactions in the ground, and its compatibility with the foundation material).
> Determination of the total ground resistance (e.g. by means of anchor tests).
> Choice of structure type. As the different types of structure place different demands on the foundations, the foundation conditions must be assessed prior to the choice of structure type, and these taken into account (e.g. by means of exploratory drillings and test anchors).
> Type of foundations (anchors, micropiles, or concrete or prefabricated foundations).

Tab. 5 > Ground classes and glide factors.

Ground classes	Glide factor	
	 Exposure WNW-N-ENE	 Exposure ENE-S-WNW
Class 1		
• Coarse scree (d* ≥ 30 cm) • Terrain heavily populated with smaller and larger boulders	1.2	1.3
Class 2		
• Areas covered with larger alder bushes or dwarf pine at least 1 m in height • Prominent mounds covered with grass and low bushes (height of mounds over 50 cm) • Prominent cow trails • Coarse scree (d* ca. 10–30 cm)	1.6	1.8
Class 3		
• Short grass interspersed with low bushes (heather, rhododendron, bilberry, alder bushes and dwarf pine below approx. 1 m in height) • Fine scree (d* ≤ 10 cm) alternating with grass and low bushes • Smallish mounds of up to 50 cm in height covered with grass and low bushes, and also those alternating with smooth grass and low bushes • Grass with shallow cow trails	2.0	2.4
Class 4		
• Smooth, long-bladed, compact grass cover • Smooth outcropping rock plates with stratification planes parallel to the slope • Smooth scree mixed with earth • Swampy depressions	2.6	3.2

d* is the boulder diameter characteristic of the roughness of the ground surface.

4 > Overview of snow pressure effects

4.1 General

This section provides only a general overview of the forces arising. Dimensioning of the structures is covered in Section 5. In general, the snow pressure in a plane perpendicular to the slope, is attributable to the pressure arising from local retardation of the

> creep movement (creep pressure) and, where present,
> glide movement (glide pressure).

4.2 Pressure component in the line of slope

The component of creep and glide pressure in the line of slope on a rigid supporting surface lying normal to the slope and of infinite length in the contour line amounts to

$$\boxed{S'_N = \rho \cdot g \cdot \frac{H^2}{2} \cdot K \cdot N} \qquad [\text{kN/m}'] \qquad (11)$$

S'_N snow pressure component in the line of slope per meter run of the supporting surface along the contour line [kN/m']
ρ average density of the snow cover (dependent on altitude and slope exposure) [t/m³]
g gravitational acceleration (=10 m/s²)
H vertical snow height [m]
K creep factor (dependent on the slope inclination ψ and the density ρ given in Tab. 6)
N glide factor as given in Section 3.10.5

The values given in Tab. 6 multiplied by $\sin 2\psi$ give the approximate K values at the densities stated.
In general, S'_N is assumed to be uniformly distributed over the height (simplification of the complex pressure distribution present both in homogeneous and non-homogeneous snow cover).

Tab. 6 > Creep factor K as a function of average snow density (ρ) and slope inclination (ψ).

ρ [t/m³]	0.2	0.30	0.40	0.50	0.60
$K/\sin 2\psi$	0.7	0.76	0.83	0.92	1.05

4.3 Pressure component normal to the slope

The pressure component normal to the slope on a rigid supporting surface normal to the slope occurs when the settling movement of the snow at the surface is prevented by adhesion and surface roughness. It has the value:

$$\boxed{S'_Q = S'_N \frac{a}{N \cdot \tan\psi}} \qquad [\mathrm{kN/m'}] \qquad (12)$$

$$\frac{a}{N \cdot \tan\psi} = \tan\varepsilon = \frac{S'_Q}{S'_N} \qquad (13)$$

S'_Q Snow pressure component normal to the slope per meter run of the supporting surface along the contour line [kN/m']

ε Angle between the resultant snow pressure arising from vectorial addition of S'_N and S'_Q and the line of slope [°]

a Coefficient dependent on snow type (can vary within the region 0.2 to 0.5)

S'_Q is likewise assumed to be distributed uniformly over the height.

4.4 Increment for non-normal supporting surface

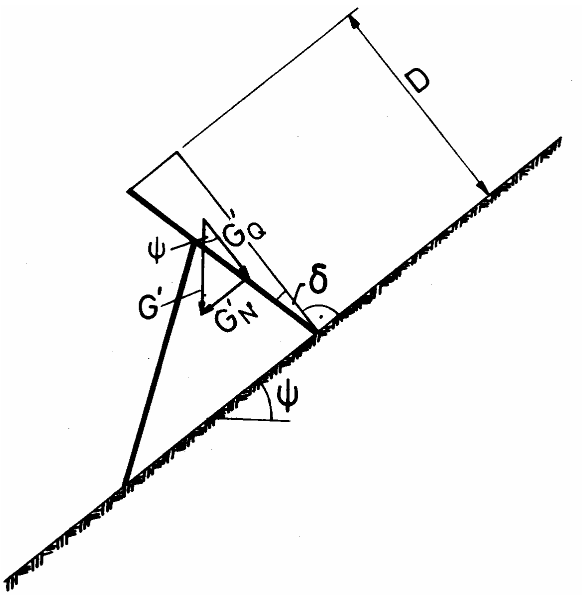
When the supporting surface is not normal to the slope, the components S'_N and S'_Q must be incremented by the weight G' of the snow prism formed between the supporting surface and the plane normal to the slope. When the supporting surface is tilted downslope, this second plane passes through the intersection of the supporting surface and the ground, whereas when it is tilted upslope, it passes through the upper edge of the supporting surface (snow fence).

For a plane supporting surface (see Fig. 16):

$$\boxed{G' = \rho \cdot g \cdot \frac{D^2}{2} \cdot \tan\delta} \qquad [kN/m'] \qquad (14)$$

G' weight of snow prism per meter run (vertical force in the contour line) [kN/m']
D snow thickness measured normal to the slope [m]
δ angle between supporting surface and the normal to the slope [°]
G'_N, G'_Q components of G' parallel and normal to the slope, respectively [kN/m']
ρ average density of the snow cover [t/m³]

Fig. 16 > Snow pressure increment with non-normal supporting surface.



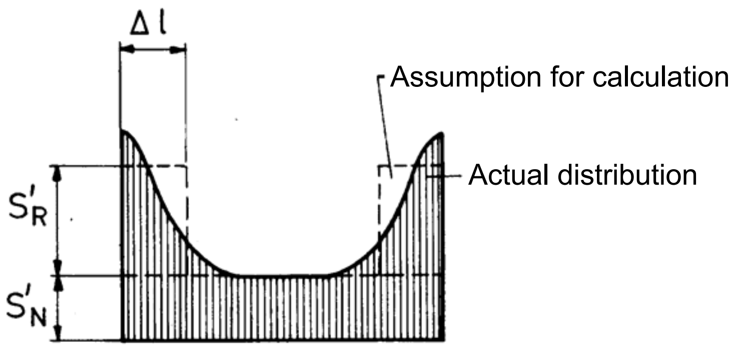
4.5

End-effect loads

With finite width of the supporting surface in the contour line, incremental end-effect loads occur by virtue of the fact that the snow can flow laterally around the surface, so that a lateral restraining effect occurs. The end-effect loads are dependent not only on the factors determining the snow pressure on an infinitely wide surface, but also on the dimensions, shape and surface roughness of the grate, and even more so on the glide factor. The basic snow pressure distribution is shown in Fig. 17. For practical calculations, the end-effect force is substituted by an equivalent, constant, force per meter run S'_R acting over the length Δl (see Section 5.5.2.4).

The influence factor η relating to the pressure transfer to a supporting structure or a slender element can be defined as the ratio of the effective snow pressure including end-effect loads to the snow pressure excluding end-effect loads.

Fig. 17 > Snow pressure distribution on a supporting surface of finite width.



4.6 Snow pressure on slender elements of a supporting structure

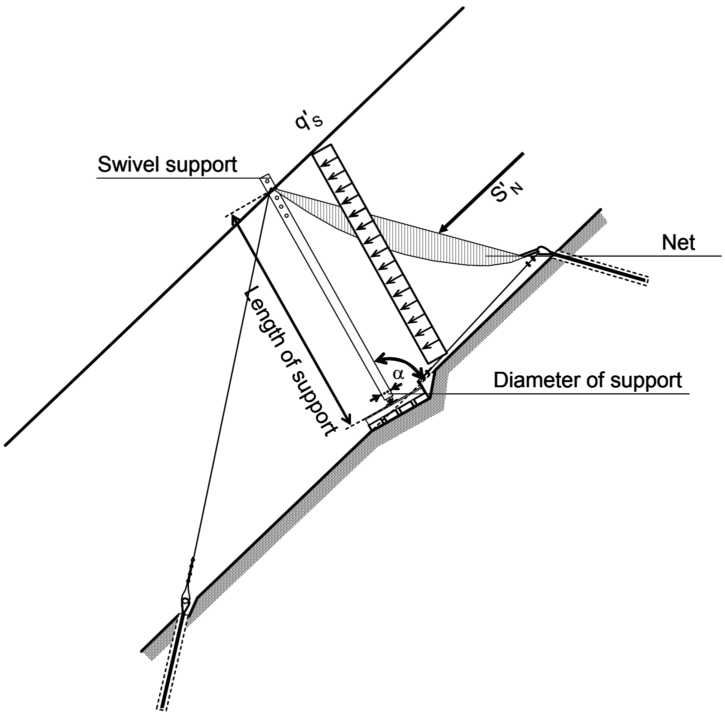
4.6.1 Snow pressure on supports

The supports of rigid supporting structures and snow nets are subject to downslope forces due to snow masses attached to the underside of the structure (Fig. 18). The magnitude of the transverse load is strongly dependent on the influence factor η of the support. With heavy snow glide, the influence factor increases. The snow pressure on the supports can be assumed as a uniformly distributed line load q'_S:

$$\boxed{q'_S = \eta \cdot S'_N \cdot \frac{\text{support diameter}}{\text{support length}} \cdot \sin\,\alpha} \qquad [\text{kN/m'}] \qquad (15)$$

q'_s	snow pressure on support represented by a line load. The direction of q'_s is normal to the axis of the support. The load impinges along the axis of the support [kN/m'].
η	influence factor of the support.
S'_N	snow pressure component in the line of slope per meter run of the supporting surface [kN/m'], equation (11).
-	diameter and length of support [m].
α	Angle between the support axis and the surface of the ground [°].

Fig. 18 > Snow pressure q'_S on the support of a snow net.



The influence factor η can be assumed to be 1. For extreme snow glide, the influence factor η can increase up to a value of 5. At sites with low snow glide ($N<1.6$, or effective snow glide protection), the transverse forces are usually negligible.

4.6.2 Snow pressure on wire ropes and bars

With thin wire ropes or bars subject to snow pressure (e.g. lateral guys), **heavily augmented end-effects** must be expected. These depend not only on the factors that determine snow pressure on infinite structures, but also on the wire rope or bar diameter, the position relative to the back-pressure zone of the supporting structure and, above all, on the glide factor. The snow pressure can be estimated using equation (15), whereby an increased influence factor η must be applied. Only rough estimates are available for the determination of the influence factor. For a snow thickness of 200 cm and a wire rope diameter of 1 cm, the order of magnitude of the influence factor η can be assumed to be around 50.

4.7 Lateral loads

Owing to irregularities in the terrain and fluctuations in the height of the snow, the resultant of the loads acting on the supporting surface given in Sections 4.2–4.4 in plan view is not always normal to the supporting surface (see condition specified in Section 3.4.4). A lateral load S_s parallel to the contour line should be assumed (Section 5.5.6). Note that a higher lateral load must be expected in the influence zone of the end-effect loads.

5 > Dimensioning of separated supporting structures

5.1 Materials

5.1.1 Steel

5.1.1.1 Steel quality class

The choice of steel quality class is made according to SIA 263 for the field of application A2 (e.g. buildings). Table 18, p. 81 of SIA 263 (2003 edition) specifies use of quality class JR or higher.

5.1.1.2 Safety from brittle fracture

Where special designs, components with sensitive welds, large metal thicknesses, cold-drawn components, internal stresses etc., are involved, steel quality classes having adequate resistance to brittle fracture should be chosen.

5.1.2 Timber

5.1.2.1 Timber selection:

Timber selection is performed based on SIA 265/1, Section 5: Selection of round and sawn timber.

5.1.2.2 Resistance of timber types

The service life of a supporting structure can be substantially increased by the choice of **fungus resistant timber,** for example sweet chestnut, oak and English tree. With heartwood from the larch, which is less resistant to fungus, a service life of at least 10 years can be achieved depending on the site. However, the sapwood of these woods is equally vulnerable to fungus as the entire wood mass of spruce, fir, Douglas fir, red beech and ash.

5.1.2.3 Chemical timber preservation

With spruce, fir and pine, **industrial impregnation** based on the so-called alternating pressure process is used to achieve the required minimum impregnation depth (15 mm) of the preservative in these woods. A substantial increase in the service life of the supports and timber crossbeams may be achieved by mechanical pre-treatment (e.g. drilled perforation) or additionally applied two-layer protection (Ger.: Doppelstock-schutz) in the transition region between the air and the ground as is common practice with telephone masts (see for example VSE/SWISSCOM, 1999: Guideline for use of the drilling process for the mechanical pre-treatment of telephone masts [German: Richtlinie für die Anwendung des Bohrverfahrens als mechanische Vorbehandlung von Leitungsmasten], VSE no. 2.59, Swiss Electricity Supply Association (VSE), Zurich. Note that the legislation on the use of toxic substances as well as environmental legislation require that products compatible with human health and the environment be used. Where impregnated timber is used, it is essential to ensure that the timber is labeled with the LIGNUM quality seal for pressure impregnated timber, ensuring that it contains the required quantity of preservative. The Ordinance on Air Pollution Control stipulates that pressure impregnated timber must be disposed of in plant especially designed for this purpose (municipal waste incineration plant or cement factory). The manual impregnation of timber should be avoided for technical and ecological reasons. Furthermore, manual impregnation is prohibited in the "Chemikalien-Risikoreduktions-Verordnung" (ChemRRV, 2005), except with specialist cantonal approval.

Non-impregnated structures must be made exclusively from the timber of the sweet chestnut, English tree or oak. Where a service life of less than 20 years suffices, larch (without sapwood) from slow-growth sites can also be used.

5.1.3 Other building materials

Where materials such as wire ropes, light metals, cement, and plastics are used, the strength and deformation characteristics of these must be precisely specified.

5.2 ## Structural analysis and dimensioning

5.2.1 ### Principles

5.2.1.1 Approval procedure

In distinction to the SIA standards, only the **ultimate limit state** of the structures must be complied with in accordance with the loads specified in the present technical guideline. Proof of the **serviceability** is not required. The service life of the materials used must accord with the intended duration of use.

5.2.1.2 Loads

The loads due to snow pressure calculated according to the technical guideline should be regarded as characteristic values.

5.2.2 ### Verification of ultimate limit state of the structural system and grate

5.2.2.1 Dimensioning

The load assumptions in the present technical guideline are to be regarded as variable actions, Q_k. For approval purposes, the **load coefficient $\gamma_Q = 1.5$.** The **ultimate limit state** is fulfilled when the following dimensioning criterion is complied with:

$$\boxed{E_d \leq R_d} \tag{16}$$

$E_d = \gamma_Q \cdot Q_k$: Design effect of actions (loading), where Q_k is the characteristic value of the variable action (e.g. snow pressure) and γ_Q =1.5 load coefficient.
$R_d = R_k/\gamma_M$: Design resistance, whereby R_k is the characteristic value of the resistance (e.g. of the steel profile) and γ_M the coefficient of resistance.

5.2.2.2 Design resistance for steel

For **steel,** the design resistance R_d is calculated as specified in SIA 263. Normally, the coefficients of resistance are as follows:

> γ_{M1} = 1.05 for strength and stability approval purposes
> γ_{M2} = 1.25 for connections and verification in net section.

5.2.2.3 Design resistance for timber

For **timber,** the design resistance f_d according to SIA 265 are to be used. In dimensioning the structural system, these values must be reduced by a timber humidity coefficient η_w of 0.7. In dimensioning the crossbeams, the strengths specified need not be reduced. The crossbeams, which are easy to replace, therefore have a slightly lower breaking strength than the supporting structure.

5.2.2.4 Design resistance of wire ropes

For **wire ropes,** the design resistance R_d is determined using a coefficient of resistance γ_M of 1.35 times the minimum breaking strength.

5.2.2.5 Design resistance of other building materials

For **other building materials,** the design resistance is determined based on the data given in Section 5.1.3 on a case-by-case basis in consultation with professional advisors.

5.2.3 Verification of ultimate limit state of the foundations

5.2.3.1 Dimensioning

The ultimate limit state of the foundations is also fulfilled using the dimensioning criterion given by equation (16). For the purposes of simplification, for all loading conditions (permanent and variable actions), a uniform load coefficient $\gamma_Q = \gamma_{G,sup} = 1.5$ is applied. For permanent actions (e.g. earth load), a higher load coefficient is therefore used than specified in SIA 261.

5.2.3.2 Internal ultimate limit state

Proof of the **internal ultimate limit states** of the foundations is made analogous to that of the supporting structure. The internal design resistance R_d of prefabricated foundations in steel, and steel tensioning members (anchors and micropiles), is determined using a constant coefficient of resistance $\gamma_M = 1.05$.

5.2.3.3 External ultimate limit state

Proof of the **external ultimate limit states** of the foundations is made using a simplified procedure in relation to SIA 267, in that the total ground resistance R_k is determined using the characteristic ground parameters and/or total ground resistance. The external design resistance R_d is determined using a constant coefficient of resistance for shallow foundations, anchors and micropiles, of $\gamma_M = 1.35$.
A safety factor of 1.5 is to be used to take account of excessively large deformation rates.

5.2.4 Notes on the dimensioning and execution of steel structures

5.2.4.1 Determination of internal forces

In verifying the ultimate limit state, the internal forces must be determined **elastically.** The structures must be supported on statically determined bearings.

5.2.4.2 General corrosion resistance

In general, the superstructure need **not be corrosion resistant** and no allowance is necessary for rust. However, the structure should be designed in accordance with anti-corrosion principles (e.g. to ensure effective runoff of water).

5.2.4.3 Corrosion resistance at and in the ground

At the ground (i.e. up to 40 cm above ground level) non-replaceable parts (e.g. anchors) and parts of the foundations in direct contact with the ground must be provided with corrosion protection. This can be achieved via a **rust allowance of 2 mm** per external surface. The galvanization of anchor bars is not recommended.

For anchors that are located in a chemically aggressive environment and/or subject to a critical stray current load, **corrosion protection level 2** according to SIA 267 must be provided (provision of an additional sleeve pipe in plastic).

5.2.4.4 Note to designers

In designing the structure, note that for certain types of steel, heavy corrosion must be expected at welded joints (if any) or overlaps.

5.2.4.5 Requirements for crossbeam profiles

Crossbeams should not have a **material thickness less than 5 mm**. They can also be subjected to an impact test with an impact energy of 3.5 kNm. The resultant reduction of the inertia moment must not exceed 15%. This provision does not apply to the structural system.

5.2.4.6 Wall thickness of supports

With hollow profiles, **a wall thickness of 4 mm** must be maintained to avoid damage during transport.

5.2.5 Notes on the dimensioning and execution of wooden structures

5.2.5.1 Principles

The service life of the wooden components not in contact with the ground can be substantially increased by paying attention to the details of structural design. The primary objective should be to avoid the ingress and retention of rainwater in the timber and/or to promote drying. Attention should be paid to good runoff of the water. For supporting structures in wood, a snow rake is preferable to a snow bridge.

5.2.5.2 Mechanical protection measures for wood

For wood, mechanical protection is extremely important. The following principles must always be followed:

> use only sound timber.
> avoid unnecessarily large diameters, thereby facilitating rapid drying.
> use of upright or at least inclined arrangement.
> covering of horizontal timbers (e.g. purlins).
> types of timber with highest possible durability to be used for the supporting structure (supports, purlins). Sweet chestnut should be used as far as possible for horizontal purlins, even when the remaining structural elements are (for example) in larch or impregnated spruce.

However, design measures of this type are not a substitute for impregnation with wood preservatives or the choice of resistant heartwoods. This is particularly the case for timber in contact with the ground.

5.2.6 Notes on the dimensioning and execution of structures with wire ropes

5.2.6.1 Deflection

For intermediate supports, the wire ropes should be passed over circular segments with a radius not less than 2.5 times the diameter of the wire rope. For angles of deflection less than 5°, no restrictions apply to the radius. The lateral load (line load) on the support must not exceed 1 kN/mm'.

5.2.6.2 Connections

Connections using cable clips, loops and cable eye stiffeners must be designed as stipulated in the relevant EN and DIN standards.

5.2.6.3 Wire ropes

The steel strands of replaceable wire ropes and nets must be zinc plated as specified in EN 10264, Class B, or alternatively galvanized as specified in DIN 2078, or provided with equivalent corrosion protection.

5.2.6.4 Wire rope anchors

For wire rope anchors, spiral cables must be used as the tension element. The steel strands must be zinc plated as specified in EN 10264, Class A, or alternatively, heavily galvanized as specified in DIN 2078. The head of the wire rope anchor must be protected additionally with a closed steel pipe imbedded in the anchor grout, or with equivalent corrosion protection.

For wire rope anchors located in a chemically aggressive environment and/or subject to a critical stray current load, **corrosion protection level 2** according to SIA 267 must be provided (provision of an additional sleeve pipe in plastic).

5.3 Structural design

5.3.1 General

In principle, the design of the supporting structure may be chosen at will. This also applies to the geometry (inclination and connection points of supports, angle to the terrain, field widths, etc.). For optimum solutions, not only the external forces and the slope inclination must be considered, but also the foundations and the erection procedure. To achieve a uniform safety level for all components of the structure (incl. foundations) with varying slope inclination, the angles of the triangle formed by the grate, support and ground surface should be kept constant.

5.3.2 Inclination of the supporting surface to the plane normal to the slope

5.3.2.1 Rigid supporting surface

For rigid supporting surfaces, the angle to the plane normal to the slope should be chosen as **δ = 15°** in the downslope direction (Fig. 16).

5.3.2.2 Flexible supporting surface

For flexible supporting surfaces (nets), the angle δ of the plane connecting the lower edge and upper fastenings of a net of approx. 30° is used.

5.3.2.3 Steep terrain

In very steep terrain, the angle δ should be chosen somewhat smaller than the values given in Sections 5.3.2.1 and 5.3.2.2, since otherwise the grate would lie too flat.

5.4 Execution and maintenance

5.4.1 Execution

5.4.1.1 Materials and dimensions

All dimensions and materials used must correspond to the drawings/plans in the type approval procedure.

5.4.1.2 Service life

The design value of the service life of permanent supporting structures is 80 years.

5.4.2 Maintenance

5.4.2.1 Annual inspection

Normally, the structures should be inspected visually once yearly.

5.4.2.2 Periodic inspection

The physical condition of the supporting structures must be inspected in detail at least every 3–5 years and after each major loading event. For critical components (e.g. connection points between the anchors and the superstructure), the inspection should be performed at close range.

5.4.2.3 Assessment of physical condition and planning of measures

Any damage identified is to be assessed as given in Tab. 7 and, where necessary, repaired within a reasonable period.

Tab. 7 > Assessment of the physical condition of supporting structures.

Assessment of the need for repairs and action to be taken	Effect on structural safety (maximum resistance reached and/or loss of overall stability of the supporting structure)	Time frame for appearance of consequential damage	Consequences for the viability of the supporting structure (serviceability)	Examples:
Condition Class 1 "good"				
Not urgent: keep under observation	Low	> 5 years	No impairment	• Deformed crossbeams • Erosion of foundation block < 10–20 cm • Collection of debris on grate, thickness < 50 cm • Uniform surface corrosion (rust)
Condition Class 2 "damaged"				
Moderately urgent: repair within 1–3 years	Average	2–5 years	No immediate impairment	• Slightly deformed supports • Displaced cable clips • Micropile anchors pushed into the ground • Exposed anchors > 20–40 cm (still intact)
Condition Class e 3 "poor"				
Very urgent: immediate repairs or replacement before the winter	Large, danger of collapse	1 year	Extreme impairment: supporting function nil or very limited	• Buckled supports • Heavily deformed or broken girder • Broken or pulled out anchors • Buckled micropiles • Broken wire ropes

5.5 Loads on the structural system

5.5.1 General

When $H_K > H_{ext}$ is chosen, the dimensioning must be based entirely on H_K (cf. Section 3.6.2).

5.5.2 Snow pressure in load case 1

5.5.2.1 Snow pressure component in the line of slope

Load case 1 assumes that the structure is subject to full snow loading with snow height H_K. The snow pressure component in the line of slope at points where no end-effect loads act is:

$$\boxed{S'_N = H_K^2 \cdot N \cdot f_c} \qquad [\text{kN/m}'] \qquad (17)$$

H_K vertical structure height [m]
N glide factor as specified in Section 3.10.5.
f_c altitude factor as specified in Section 3.10.6.

Equation (17) was derived from equation (11), whereby a relatively small value of ρ_H = 0.270 t/m³ was chosen for the average density. This value applies to a basic site altitude of 1500 m a.s.l. and an exposure of WNW-N-ENE (cf. Section 3.10.2). Furthermore, K = 0.74 and $\sin 2\psi$ = 1.00 were set (cf. Section 4.2). These values apply to a slope inclination of 45°.

5.5.2.2 Snow pressure component normal to the slope

The snow pressure component normal to the slope is:

$$\boxed{S'_Q = S'_N \frac{a}{N \cdot \tan\psi}} \qquad [\text{kN/m}'] \qquad (18)$$

$$\frac{a}{N \cdot \tan\psi} = \tan\varepsilon = \frac{S'_Q}{S'_N} \qquad (19)$$

whereby the most unfavorable case as between a = 0.35 and a = 0.50 must be chosen.

5.5.2.3 Supplementary value for non-normal supporting surface

The vertically acting weight of the snow prism on a plane grate (whereby a somewhat higher density is assumed at the supporting surface) amounts to:

$$\boxed{G' = 1.50 \cdot D_K^2 \cdot \tan\delta} \qquad [kN/m'] \qquad (20)$$

D_K Effective grate height in m, where $D_K = H_K \cdot \cos\psi$

δ Angle between supporting surface and the normal to the slope

5.5.2.4 End-effect loads

The end-effect loads S'_R are applied as a supplementary distributed load in the line of slope over a distance Δl (no supplementary end-effect loads are applied normal to the slope) as follows, Fig. 20.

$$\boxed{S'_R = f_R \cdot S'_N} \qquad [kN/m'] \qquad (21)$$

where f_R is the end-effect factor:

$$\boxed{f_R = (0.92 + 0.65 \cdot N)\frac{A}{2} \leq (1.00 + 1.25 \cdot N)} \qquad (22)$$

N glide factor as specified in Section 3.10.5

A distance between structures [m]

The upper limit on the right of equation (22) applies to a separated structure (A > 2 m) and must not be exceeded (Fig. 19).

$$\boxed{\Delta l = 0.60 \cdot \frac{A}{2} \leq \frac{D_K}{3}} \qquad [m] \qquad (23)$$

Δl length of applied load of S'_R [m]

D_K effective grate or net height [m]

The upper limit value on the right of equation (23) applies to a separated structure (A > 2 m) and must not be exceeded.

Examples:

The relevant values for f_R and Δl (underscored) are obtained from equations (22) or (23):

Calculation of the end-effect factor f_R using equation (22).

N = 2.4 A = 2 m $f_R = (0.92 + 0.65 \cdot N)\frac{A}{2} = \underline{2.48}$

$f_R \leq 1.00 + 1.25 \cdot N = 4.00$

N = 2.4 A = 4 m $f_R = (0.92 + 0.65 \cdot N)\frac{A}{2} = 4.96$

$f_R \leq 1.00 + 1.25 \cdot N = \underline{4.00}$

Calculation of the length Δl of the end-effect force using equation (23.

A = 2 m D_K = 4 m $\Delta l = 0.6\frac{A}{2} = \underline{0.60m}$

$\Delta l \leq \frac{D_K}{3} = 1.33m$

A = 2 m D_K = 1.5 m $\Delta l = 0.6\frac{A}{2} = 0.60m$

$\Delta l \leq \frac{D_K}{3} = \underline{0.50m}$

Fig. 19 > End-effect factor as specified by equation (22).

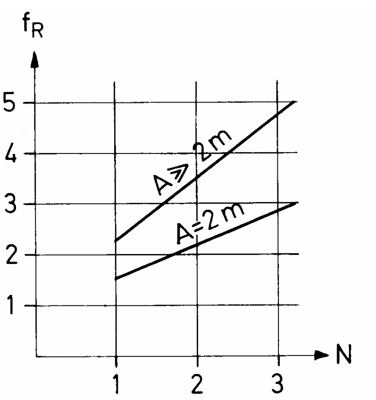
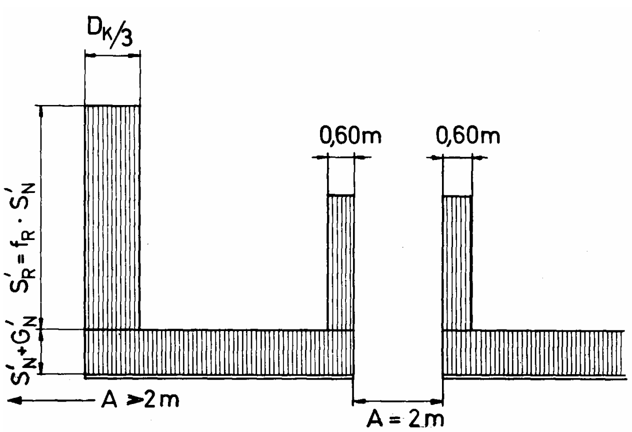


Fig. 20 > Distribution of the end-effect loads at the unprotected end of the structure and for a distance between structures of 2 m (ends of structures).



In cases where adjacent structures are slightly displaced in the line of slope (as specified in Section 3.8.1), the same end-effect loads are applied as on non-displaced structures.

In certain cases, a **symmetrical layout** should be chosen despite non-uniform loading of the two ends of the structure based on the higher of the two end-effect loads. This is particularly the case for shorter structures at their unprotected ends when there is an increased risk from dynamic loads.

5.5.2.5 Magnitude of resultant

The magnitude of the resultant R' is obtained by vectorial addition of the sums of the components parallel and normal to the slope obtained from Sections 5.5.2.1, 5.5.2.2, 5.5.2.3 and 5.5.2.4 (Figs. 20 and 21).

For an infinite wall:

$$R'_N = S'_N + G'_N \qquad (24)$$

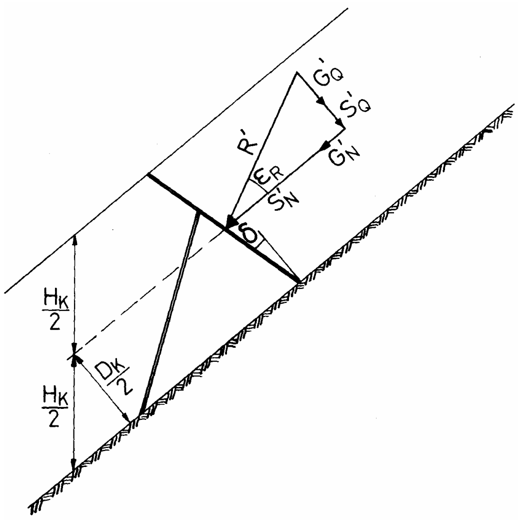
$$R'_Q = S'_Q + G'_Q \qquad (25)$$

$$\boxed{R' = \sqrt{R'^2_N + R'^2_Q}} \qquad [kN/m'] \qquad (26)$$

In the region where the end-effect loads are active, the end-effect force S'_R should be added to the components S'_N and G'_N that act in the line of slope.

$$R'_N = S'_N + S'_R + G'_N \qquad (27)$$

Fig. 21 > Resultant snow pressure.



5.5.2.6 Direction of the resultant

The direction of the resultant in the plane normal to the contour line is obtained from

$$\boxed{\tan \varepsilon_R = \frac{R'_Q}{R'_N}} \qquad (28)$$

where ε_R is the angle between the resultant and the line of slope. Note that within the region affected by the end-effect loads, R' is inclined differently from the region where no end-effect loads act. In verifying the ultimate limit state of the structure, the direction of the resultant should be determined in proportion to the areas over which the loads are applied.

5.5.2.7 Point of application of the resultant

The point of application of the resultant R' may be assumed at the **center height of the structure**.

5.5.3 Snow pressure in load case 2

5.5.3.1 Specification

Load case 2 assumes partial snow loading of the structure with snow height h of

$$\boxed{h = 0.77 \cdot H_K} \qquad [m] \qquad (29)$$

and a resultant R' having the same magnitude and direction as with load case 1 (Fig. 22).

5.5.3.2 Exceptions

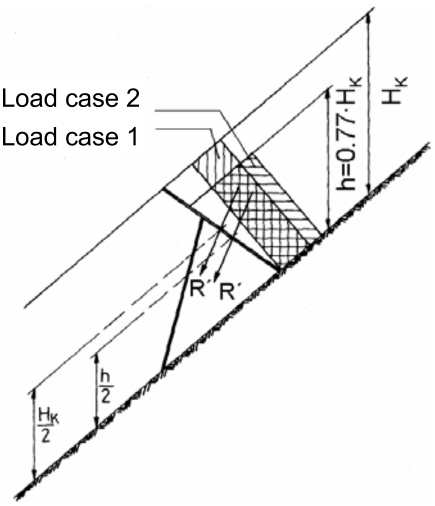
Load case 2 differs from load case 1 in that

> the point of application of the resultant lies lower, namely at height $h/2 = 0.385 \cdot H_K$,
> the snow pressure [kN/m²] is higher (increased by a factor 1/0.77 = 1.3).

5.5.3.3 End-effect loads

The end-effect factors f_R and the lengths Δl over which the loads are applied are assumed equal in both load cases.

Fig. 22 > Points of action of the resultants and snow pressure distribution for the two load cases.



5.5.3.4 Note

The snow conditions of load case 2 are derived from load case 1 through settlement of the snow cover and a supplement for additional snowfall. The resulting increased average density is $\rho_h = 0.400$ t/m³. This value applies to a basic altitude of 1500 m a.s.l. and an exposure of WNW-N-ENE. Note that in this case $\rho_h \cdot h > \rho_H \cdot H_K$.

5.5.4 Snow pressure on supports

The snow layer below the structure can cling to the supports of rigid supporting structures and snow nets. As a result, downslope pressure forces occur. These are relatively small and are applied in the form of a uniformly distributed load q'_S as follows (cf. Fig. 18):

$$\boxed{q'_S = \eta \cdot S'_N \cdot \frac{\text{support diameter}}{\text{support length}} \cdot \sin\ \alpha} \qquad [\text{kN/m'}] \qquad (30)$$

η influence factor: depends mainly on the glide factor and is assumed to be 1.0. At sites of extreme snow glide, higher influence factors must be expected (cf. Section 4.6.1).
H_K height of structure [m]
– support diameter and length [m]
α angle between support axis and ground surface [°].

The transverse load q'_s acts at right angles to the support axis (where the rotation of the support about the lower pivot is restrained, the direction is in the line of slope). The line of action lies in the support axis.

5.5.5 Dead loads

The dead loads of the structures should be taken into account where these are significant.

5.5.6 Lateral loads

To ensure sufficient lateral stiffness of the structures, a lateral force S_s should be assumed to act at both ends of the structure in a direction parallel to the contour line (see Section 4.7). Where the resultant snow pressure on the protected or unprotected end of a structure seen in plan view does not act normally to the supporting surface (e.g. in depressions), the lateral force S_S must be multiplied by the end-effect factor f_R.

The lateral force on each single structure of length l amounts to:

$$\boxed{S_S = 0.10 \cdot S'_N \cdot l} \quad [\mathrm{kN}] \qquad (31)$$

The point of action of the force is assumed at mid-height of the structure (load uniformly distributed over the height).

Steps should be taken to ensure adequate transfer of this load from the grate (or net) to the supporting structure and the foundations.

For foundations with anchors and micropiles, Sections 5.9.7.1.4, 5.9.7.2.1, 5.9.7.3.2 and 5.9.7.3.3 apply.

5.5.7 Lifting loads

Supporting structures can be subject to upslope **wind loads.** Steps should be taken to ensure adequate transfer of these lifting forces from the supporting structure to the foundations and the ground (cf. Section 5.9.3.6). The wind loads are determined as specified in SIA 261, Chapter 6: Wind.

5.6 Loads on the grate

5.6.1 Loads normal to the plane of the grate

5.6.1.1 Principles

In nature, the pressure distribution on the supporting surface is often irregular. This makes more stringent assumptions necessary for the specific loading of the elements of the grate.

5.6.1.2 Specific loading

In both load cases, a base load corresponding to the **snow pressure in load case 2** is assumed. For a plane grate (see Fig. 23):

$$P' = R' \cdot \cos(\delta - \varepsilon_R) \qquad [\text{kN/m'}] \qquad (32)$$

P' component of R' normal to the grate (equation 26)
ε_R angle between R' and the line of slope, calculated as specified in Section 5.5.2.6 with a = 0.35

From this, the snow pressure p_h normal to the grate is given by

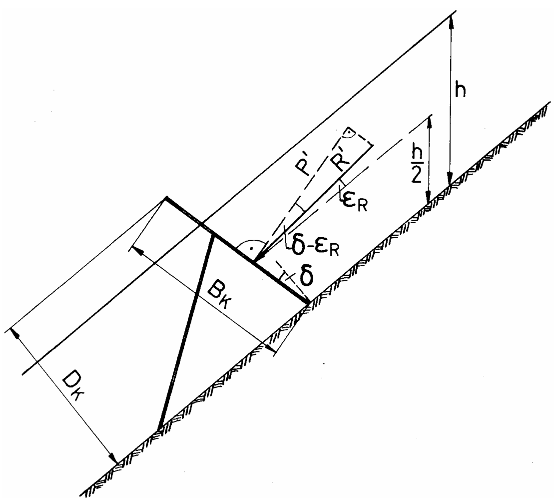
$$p_h = \frac{P' \cdot \cos\delta}{0.77 \cdot D_k} = \frac{P'}{0.77 \cdot B_K} \qquad [\text{kN/m}^2] \qquad (33)$$

(a higher value applies within regions where the end-effect loads act)

The required distributed normal load on a crossbeam applied over a width b (= cross-beam width + percentage of the distance to the neighboring structure) amounts to:

$$p'_B = p_h' \cdot b \qquad [\text{kN/m'}] \qquad (34)$$

Fig. 23 > Load normal to the grate.



5.6.1.3 Supplementary load

In addition to the specific loading in Section 5.6.1.2, a supplementary load should be applied over an area extending from the surface of the ground up to ¼ of the grate height and over the entire length of the structure. The **supplementary load amounts to 25%** of the snow pressure p_h lying outside the region where the end-effect loads act (Fig. 24).

5.6.1.4 End-effect loads and load cases

For grates for which the conditions for the occurrence of end-effect loads apply (Section 4.5), two distinct load cases apply (Fig. 25):

> load case **with** end-effect loads S'_R calculated as specified in Section 5.5.2.4
> load case **without** end-effect loads S'_R

5.6.2 Loads parallel to the grate surface (transverse loads)

The magnitudes of the transverse loads to be applied depend on the design of the grate, i.e. on the type of structure. These are therefore treated under the characteristics of the individual types of structure.

Fig. 24 > Load case 1 for the grate.

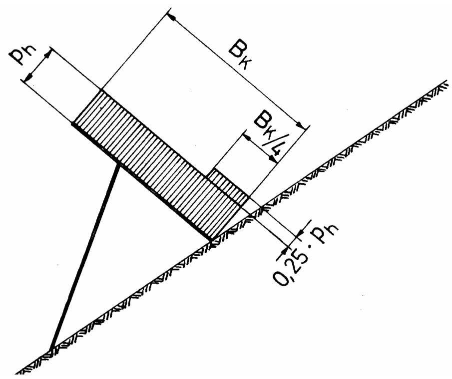
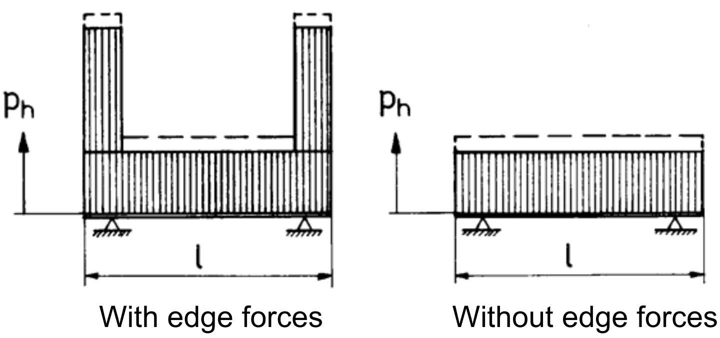


Fig. 25 > Load cases with and without end-effect loads.



5.7 Dimensioning and execution of the structural system

5.7.1 General

5.7.1.1 Principles

In dimensioning the structural system, the loads and load cases as specified in Sections 5.5.2, 5.5.3, 5.5.4, 5.5.5 and 5.5.6 apply. Where appropriate, the load cases with and without end-effect loads S'_R specified in Section 5.6.1.4 must also be taken into account.

5.7.1.2 Span

For components attached rigidly to the upper foundation, the span extends downwards to the point B, about which the structure is assumed to be freely rotatable (see Sections 5.9.5.3.1 and 5.9.6.3). Between the ground surface and the pivot B, the component concerned may be assumed to be load-free.

5.7.1.3 Single structure decisive

In general, single structures with a distance between structures of A = 2.0 m are the reference case for dimensioning of the supporting structure.

5.7.2 Dimensioning of the supports

5.7.2.1 Transverse loads

In dimensioning the supports of flexible and rigid supporting structures, both the central pressure force with its line of action in the axis of the support, and the transverse loads resulting from snow pressure, must be considered as specified in Section 5.5.4. Both loads act simultaneously in full measure (interaction of bending and normal forces).

5.7.2.2 Excess length

Both supports and pressure bars must be dimensioned with a minimum excess length of 0.5 m.

5.7.3 Special conditions for snow rakes

In dimensioning the lower purlin to withstand normal loads, load case 2 is applicable, whereby p_h is increased by 25% (as specified in Section 5.6.1.3).

5.7.4 Special conditions for snow nets

5.7.4.1 Reduction of snow pressure in the line of slope

The reduction in the snow pressure component in the line of slope resulting from the flexibility of the supporting surface is achieved by applying a reduction factor f_s. Strictly speaking, f_s depends on numerous factors such as: glide of the snow cover over the ground (f_s increases with N), sag, shape, inclination and mesh width of the net (the smaller the sag and the mesh width, the higher f_s).

The snow pressure component in the line of slope (modification of Section 5.5.2.1) is given by

$$\boxed{S'_N = f_S \cdot H_K^2 \cdot N \cdot f_C} \qquad [\text{kN/m'}] \qquad (35)$$

f_s reduction factor for a flexible (slack) supporting surface. For average glide conditions, f_s can be taken be to 0.8.

H_K vertical height of structure in [m]

5.7.4.2 Sag

The loading of a snow net depends significantly on the sag. When erecting the structure, and also following major loading events (with stretching of the wire ropes), it is therefore necessary to inspect the net. The sag must correspond to the value specified by the designer of approx. 15% of the chord of the net.

5.7.4.3 Snow pressure component normal to the slope and lateral loads

The snow pressure component normal to the slope (Section 5.5.2.2) and the lateral load (Section 5.5.6) are neglected.

5.7.4.4 Supplementary load

The snow prism whose weight G' (ρ = 0.3 t/m³) must be added to the snow pressure, is formed by the net area and the area normal to the slope passing through the upslope edge of the net.

5.7.4.5 Load case 2

In dimensioning the supporting structure of snow nets, load case 2 is applicable.

5.7.4.6 Net supports

When the net partly lies on the supports under full load, the lateral load on these is taken to be the lateral load resulting from the full snow pressure calculated from equation (35) on the corresponding part of the net (also see Section 5.7.4.2).

5.7.4.7 Eccentric swivel support

Where as a result of the given design eccentric loading of the swivel support may occur, the pressure force should be applied with the maximum possible eccentricity.

5.7.4.8 Guys

Lateral guys not protected by the net surface (also see Section 4.6.2) are subject to the full snow pressure (increased influence and end-effect factors dependent on the distance between the structures). This must be taken into account in dimensioning.

5.7.4.9 Base of support

In calculating the internal forces at the base of the support, both the lateral load as specified in Section 5.5.4 and an exceptional oblique position of the support in the line of slope of 10° (downslope) is assumed. The resultant lateral load must amount to at least 20% of the maximum compression force on the support.

5.8 Dimensioning and execution of the grate

5.8.1 Grate dimensioning of snow bridges (crossbeams parallel to the contour line)

5.8.1.1 Normal loads

5.8.1.1.1 Loading width

The crossbeams must be dimensioned in accordance with their **effective loading widths b.** Excepted is the upper crossbeam, which must not be dimensioned smaller than the neighboring crossbeams.

5.8.1.1.2 Lower crossbeam

The loading width of the lower crossbeam extends to the ground surface (Fig. 26).

5.8.1.2 Transverse loads

5.8.1.2.1 Specific loading and distributed transverse load

In dimensioning the crossbeams, a constant distributed load q_B' acting in the upslope or downslope directions is applied (Fig. 28).

From Fig. 26,

$$Q' = R' \cdot \sin(\varepsilon_R - \delta) \qquad [\text{kN/m'}] \qquad (36)$$

Q' component R' (Section 5.5.2.5) parallel to the grate
ε_R angle between R' and the line of slope calculated according to Section 5.5.2.6 with a = 0.5

The uniformly distributed specific transverse load amounts to:

$$q_h = \frac{Q' \cdot \cos\delta}{0.77 \cdot D_K} = \frac{Q'}{0.77 \cdot B_K} \qquad [\text{kN/m'}] \qquad (37)$$

The required distributed load acting on the crossbeam amounts to:

$$q'_B = q_h \cdot b \qquad [\text{kN/m'}] \qquad (38)$$

5.8.1.2.2 Minimum value of transverse load

The minimum value of the transverse load must be applied as follows:

$$\boxed{q'_B = 0.20 \cdot p'_B} \qquad [\text{kN/m'}] \qquad (39)$$

($p'_B = p_h \cdot b$; p_h as given by equations (33) and (34))

For larger glide factors and slope inclinations, this minimum value must be applied in almost all cases.

5.8.1.2.3 Line of action

The line of action of the transverse load q'_B is situated at the extreme upslope edges of the crossbeam (Fig. 28).

5.8.1.2.4 Normal load

The normal load p'_B must be varied between its maximum value and q'_B, whereby the transverse load q'_B is assumed to act simultaneously. A check should be made to determine whether a less favorable load combination might occur, and if so, this should be applied.

5.8.1.2.5 Torsional loading

Torsional buckling resulting from transverse loads must be taken fully into account. This may be done approximately by doubling the transverse load q'_B given by equation (39).

Fig. 26 > Loading widths for crossbeams.

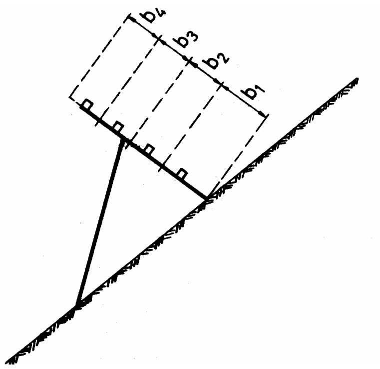


Fig. 27 > Forces parallel to the plane of the grate.

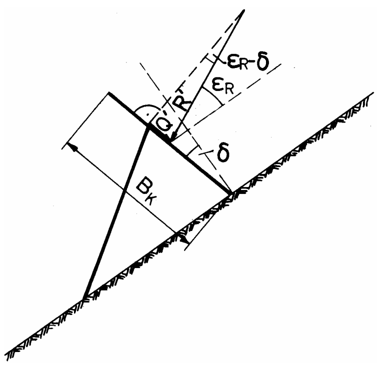
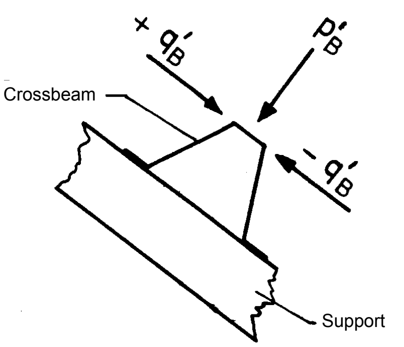


Fig. 28 > Transverse load on crossbeam.



5.8.1.3 Further provisions

5.8.1.3.1 Open width

The ideal value of the open width between the crossbeams is 250 mm.

The maximum permissible deviations from this are:

> $200 \text{ mm} \le w \le 280 \text{ mm}$ in the upper 3/4 of the grate height,
> $150 \text{ mm} \le w \le 280 \text{ mm}$ in the lower 1/4 of the grate height.

In the area between the ground and the lowest crossbeam, w should not be chosen greater than 250 mm.

5.8.1.3.2 Uppermost crossbeam

The uppermost crossbeam should be firmly fastened to counteract possible upward dynamic forces.

5.8.1.3.3 Structure height

Where the upper edges of the crossbeams vary in height, the effective structure height H_K is taken as the arithmetic mean of the vertical distances of the upper edges of the higher and lower crossbeams from the ground.

5.8.1.3.4 Structure length

The structure length l is defined as the average distance between the straight lines connecting the ends of the crossbeams.

5.8.2 Grate dimensioning of snow rakes (steel or timber crossbeams normal to the contour line)

5.8.2.1 Normal loads

5.8.2.1.1 Loading widths

The crossbeams should be dimensioned in accordance with their **effective loading widths b.** Excepted are the outermost crossbeams, whose loading width should be taken equal to the axial distance from the neighboring crossbeam, and on which the increased snow pressure at the ends acts.

5.8.2.1.2 Lower loading width

The loading width of a crossbeam extends to the surface of the ground.

5.8.2.1.3 Supplementary load

The 25% supplement to the snow pressure as specified in Section 5.6.1.3 is not applicable to snow rakes (see, however, Section 5.7.3).

5.8.2.1.4 Load case 2

For snow rakes, load case 2 must also be considered as applicable.

5.8.2.2 Transverse loads

The most unfavorable transverse loading of a crossbeam should be assumed to occur over the grate surface parallel to the contour line in the form of a distributed load q'_B with its line of action at the outer (upslope) end of the crossbeam. The magnitude of this load amounts to:

$$\boxed{q'_B = 0.10 \cdot p'_B} \qquad [kN/m'l \qquad (40)$$

p'_B maximum normal load on a crossbeam
($p'_B = p_h \cdot b$; p_h from equation (33) and Sections 5.8.2.1.1 to 5.8.2.1.3.)

The lateral load due to settlement (component of R' normal to the slope) must be considered in fastening the crossbeams.

5.8.2.3 Further provisions

5.8.2.3.1 Open width

> The ideal value for the open width w between the crossbeams is 300 mm.
> The maximum permissible deviation from this is: 250 mm ≤ w ≤ 330 mm.
> Between the ground and the lower ends of the crossbeams, w should not exceed 200 mm.

5.8.2.3.2 Height of structure

The effective height of the structure H_K is defined as the vertical distance of the straight line connecting the upper ends of the crossbeams and the ground.

5.8.3 Special conditions for snow nets (flexible supporting surface formed by wire ropes)

5.8.3.1 Specific loading

In dimensioning the nets, the specific loading of load case 2 must be assumed over the entire height of the net as specified in Section 5.6.1.2 in conjunction with the amendments given in Sections 5.7.4.1 and 5.7.4.3. This applies particularly to those parts of the net responsible for the transfer of forces to the structural system or foundations.

5.8.3.2 Distribution and direction of the specific load

The snow pressure is assumed to be uniformly distributed over the height of the net surface in a direction parallel to the resultant R', resulting from S'_N, G'_N, G'_Q , and, if present, S'_R.

5.8.3.3 Open width

The open width w between the wire ropes or wires forming the supporting surface (mesh width) is subject to the following conditions:

> where no covering of **wire netting** is used, the open width w of the wire ropes should not exceed **100 mm.**
> where a covering of **wire netting** having a mesh of 50 mm is used, a wire rope mesh of **200 to 250 mm** will suffice.
> to ensure an **adequate braking effect** in low-cohesion, moving, snow, the nets can be covered either with wire netting having a mesh width of 50 mm or an open 'patchwork' of metal sheeting, fine-mesh wire netting or similar materials. In these cases, a side length of the cover materials of **200 to 250 mm** is recommended.

5.8.3.4 Height of the structure

The effective height of the structure H_K is taken to be the arithmetic average of the largest and smallest vertical distance of the upper edge of the net surface of an average field in the loaded condition from the ground.

5.8.3.5 Length of the structure

For trapezoidal or triangular net surface, the length of the structure l is defined as the arithmetic average of the base length and the distance between the support heads.

5.9 Execution and dimensioning of the foundations

5.9.1 Principles

In dimensioning the foundations, the two load cases as specified in Sections 5.5.2 and 5.5.3, and the loads as specified in Sections 5.5.4 to 5.5.7, apply.

5.9.2 Types of foundation

5.9.2.1 Rigid supporting structures (Section 3.3.1)

For permanent supporting structures in loose ground, the foundations may consist of anchors, micropiles, prefabricated foundations (ground plates) or concrete foundations (Figs. 1, 2, 29 and 30). In general, two separate foundations are used (Figs. 29 and 30): an upper and a lower foundation. Where the ground has low resistance and is unstable, a pressure bar resistant to compression and tension can be used to connect the upper and lower foundations. Data on permissible creep movement is given in Tab. 13.

5.9.2.2 Flexible supporting structure (snow nets) and special structures (fences, suspended grates).

The tension forces can be sustained by anchors (see Section 5.9.7). Permafrost slopes subject to tolerable creep movement must be secured with snow nets. Snow nets are less sensitive to creep movement than rigid supporting structures (see Section 7.4.3.1).

5.9.2.3 Temporary supporting structures

Several of the customary designs for wooden snow rakes can only accept very limited tension forces in the upper foundations (see Fig. 32). To reduce these forces to a minimum,

> at high glide factors, the ground roughness should be increased, for example by terracing or piling
> the erection of these structures in excessively steep terrain or with excessive snow heights should be avoided.

Otherwise, either foundations specially designed to withstand tension forces (e.g. using anchors as specified in Section 5.9.7), or permanent supporting structures, should be chosen.

Fig. 29 > Supporting structure with separate foundations. The graphical determination of the foundation forces is shown for a support pivoted at both ends and a girder pivoted in B (three-point frame).

The lower foundation consists of a ground plate and the upper foundation of a micropile and ground anchor.

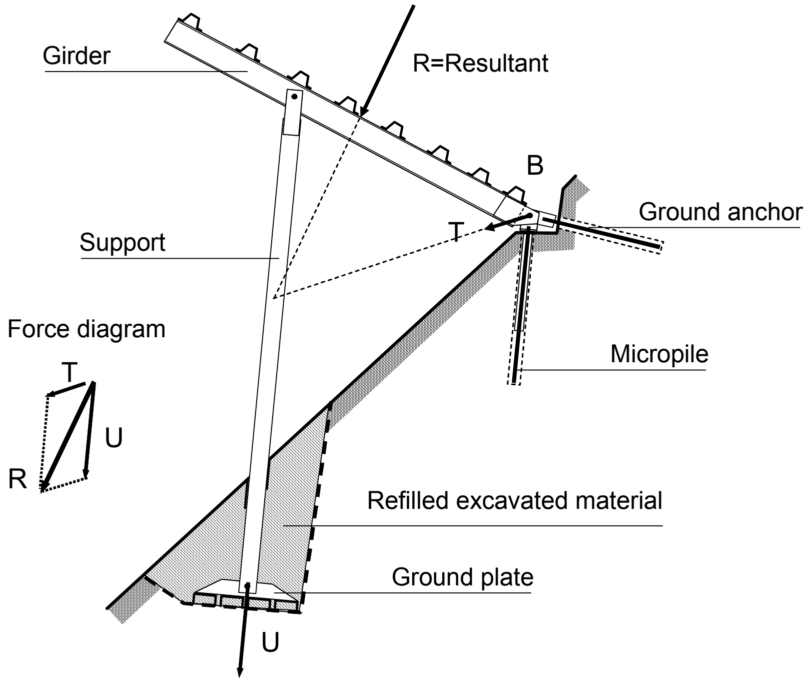
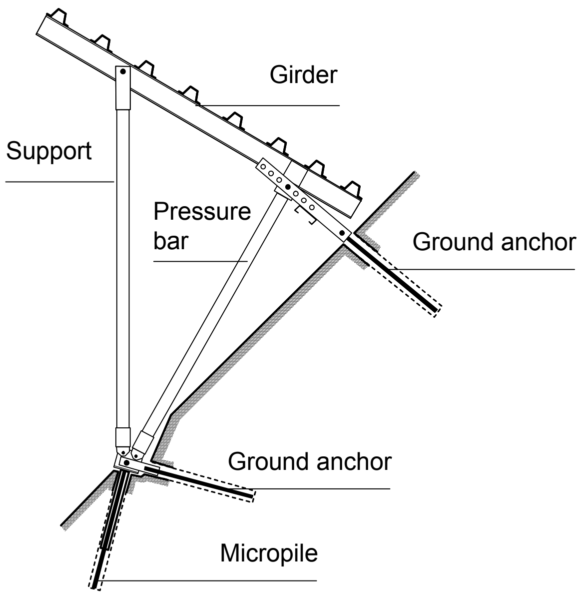


Fig. 30 > Supporting structure with pressure bar, where the lower foundation consists of a micropile and ground anchor, and the upper foundation of a ground anchor.



5.9.3 Connection of supporting structure to foundations

5.9.3.1 Principles

Basically, both rigid and pinned connection of the supporting structure to the foundations may be used.

5.9.3.2 Connection to upper foundations

With upper foundations with concrete or prefabricated foundations as specified in Section 5.9.5 and 5.9.6, pinned fastening is only permissible when outcropping rock plates lie at the surface of the terrain, or are only at a shallow depth, so that the cantilever beam carrying the pivot can be rigidly fixed in the rock plate. In other cases, i.e. in loose ground, a pinned fastening may lead either to unsuitable loading of the ground or to uneconomic dimensions of the foundations. Thus in loose ground, rigid fastening of the girder to the foundations is to be recommended, whereby, however, an increase in the span must be accepted. Note that clamping forces (due to solifluction, etc.) that might relieve the forces on the superstructure must not be taken into account.

5.9.3.3 Connection to lower foundations

For the lower foundations, pinned support fastening is preferable, and leads neither to unsuitable loading of the ground nor to uneconomic dimensions of the foundations.

5.9.3.4 Connection to ground anchors and micropiles

Supporting structures supported on ground anchors and micropiles must be provided with pinned connections.

5.9.3.5 Connection of the support to the girder

With separate upper and lower foundations (Section 5.9.2.1, Fig. 29), the support must normally be fixed with a pin-joint to the girder. When, however, a pressure bar (Section 5.9.2.1, Fig. 30) is used, or a rock foundation can be used, it is not essential to use a pinned connection between girder and support.

5.9.3.6 Lifting loads

In designing the connection between the foundations and the supporting structure, lifting forces must be considered as specified in Section 5.5.7.

Fig. 31 > Snow net.

Snow net anchored with two wire rope anchors and a ground plate.
The ground plate is secured using a retaining cable (see Section 5.9.4.2).

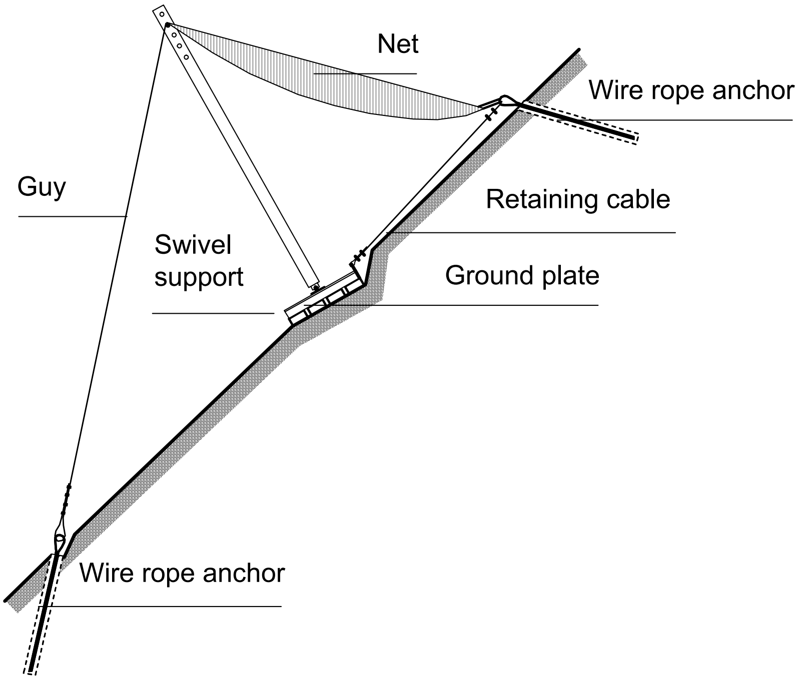
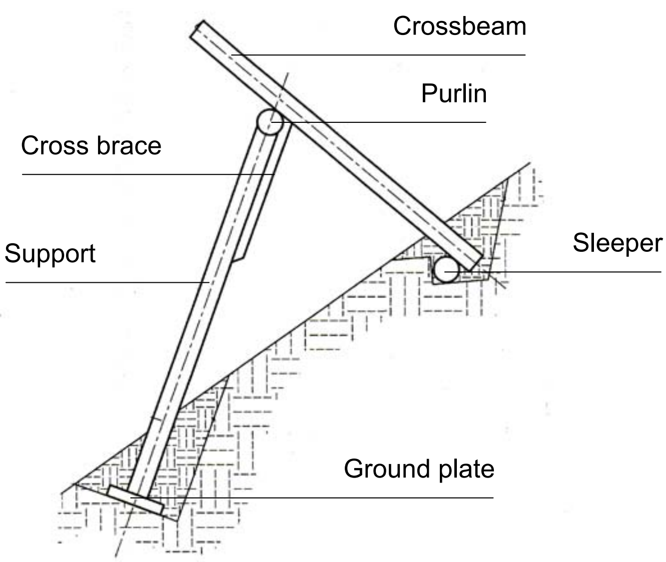


Fig. 32 > Wooden snow rake with upslope sleeper foundation.



5.9.4 General notes on dimensioning

5.9.4.1 Verification of ultimate limit state

The following simplified formulae for the verification of the ultimate limit state of the foundations were derived from equation (16), whereby a load coefficient $\gamma_Q = 1.5$ and coefficient of resistance $\gamma_M = 1.35$ were applied (assumption: $\gamma_Q \cdot \gamma_M = 1.5 \cdot 1.35 \approx 2.0$).

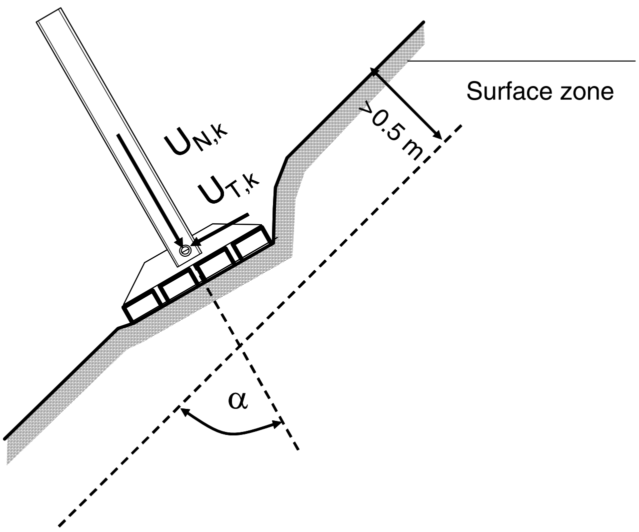
5.9.4.2 Surface zone

Where ground surfaces are subject to pressure loading, these must be completely interred beneath a **surface zone of at least 0.5 m** measured normal to the surface of the slope (cf. Figs. 33 and 40), provided that the angle α between the force normal to the support $U_{N,k}$ and the line of slope is less than 75° (Fig. 33).

5.9.4.3 Transverse forces

When the foundations transmit transverse forces to the ground, the ground surfaces subject to shear must be fully interred beneath a surface zone of at least 0.5 m measured normal to the slope. When, for example, the ground plates of snow nets are installed close to the surface, the shear forces cannot be transferred directly to the ground (Figs. 31 and 50, or Section 7.4.3.4.3).

Fig. 33 > Ground plate of a snow net anchored at the surface of the ground.



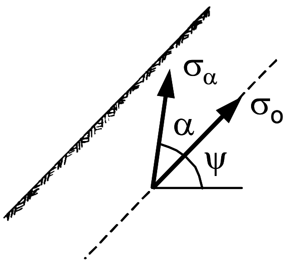
5.9.4.4 Directional dependency of the total ground resistance

For concrete or prefabricated foundations as specified in Sections 5.9.5 and 5.9.6, the dependency of the total ground resistance on the direction of the resultant force is assumed as given in Tab. 8. Here, α is the angle between the resultant force and the line of slope, σ_o the total ground resistance in the line of slope, σ_α the specific total ground resistance in the direction of the force (see Fig. 34) and ψ the slope inclination. The total ground resistance is a maximum normal to the slope ($\sigma_{90°}$) and a minimum in the line of slope ($\sigma_{0°}$).

Tab. 8 > Determination of the specific total ground resistance.

α [°]	0°	15°	30°	45°	60°	75°	90°
σ_α / σ_o [-]	0.40	0.53	0.66	0.80	0.90	0.97	1.00

Fig. 34 > Specific total ground resistance σ_α as a function of the direction α of the applied force.



5.9.4.5 Ground tension forces

Tab. 8 applies only to ground pressure forces. Whenever ground tension forces occur, the foundations must be dimensioned in a similar way to those of masts (for details see the following sections). For anchors and micropiles, see Section 5.9.7.

5.9.4.6 Total ground resistance normal to the slope

The total ground resistance (maximum load-bearing capacity) normal to the slope $\sigma_{90°}$ depends on the inclination of the slope, the ground characteristics, the dimensions of the foundations and the encastrated depth of the foundations. The ground resistance must be carefully determined in relation to the local conditions. Experience with avalanche supporting structures shows that a ground resistance normal to the slope $\sigma_{90°}$ of between 500 kN/m² and 1000 kN/m² may be expected.

5.9.4.7 Refilling of excavated material

Following installation of the foundations, the excavated material must be refilled and carefully compacted.

5.9.5 Concrete foundations in loose ground

5.9.5.1 Definition

Concrete foundations are foundations fabricated on site.

5.9.5.2 Risk of corrosion

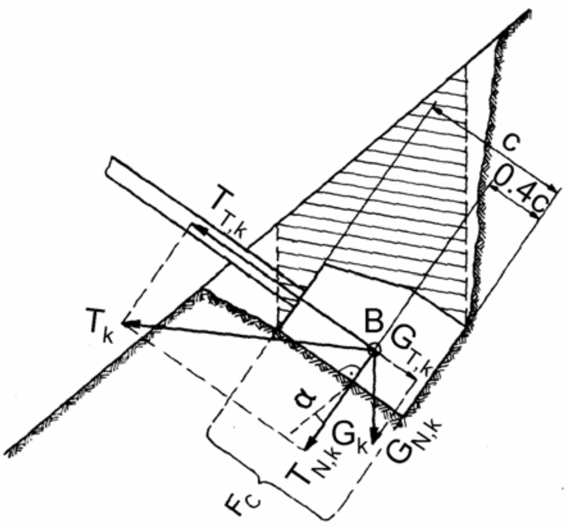
For components imbedded in concrete, particularly those in aluminum alloys, the risk of corrosion must be considered.

5.9.5.3 Dimensioning of upper concrete foundations to withstand compression

5.9.5.3.1 Rigid connection between supporting structure and foundations.

The loading of a foundation consists of a single force T_K. The point of action B of T_K is assumed to be at a distance of 0.4 c above the level of the foundations (c = height of foundations, Fig. 35). B is the hypothetical support point of the respective component and determines the span. In dimensioning the girder, its end should be assumed to be freely rotatable at point B.

Fig. 35 > Dimensioning of upper concrete foundations to withstand compression forces.



The lower foundation area F_c is subject to the following condition:

$$\boxed{F_c \geq \frac{2 \cdot (T_{N,k} + G_{N,k})}{\sigma_\alpha}} \quad [m^2] \qquad (41)$$

$T_{N,k}$ component of the characteristic value of the resultant reaction force normal to the foundation area F_c
$G_{N,k}$ component of the characteristic value of the weight of the foundations (including hatched ground area in Fig. 35) normal to the foundation area F_c
σ_α specific total ground resistance normal to F_c (cf. Tab. 9 and Section 5.9.4.6)

5.9.5.3.2 Pinned connection between supporting structure and foundation.

The loading of the foundation consists of the eccentrically acting characteristic force T_k. The point of action of T_k is at the pivot point. As explained in Section 5.9.3.2, this type of fastening should not normally be used with separate foundations in loose ground.

5.9.5.4 Dimensioning of upslope concrete foundations to withstand shear forces

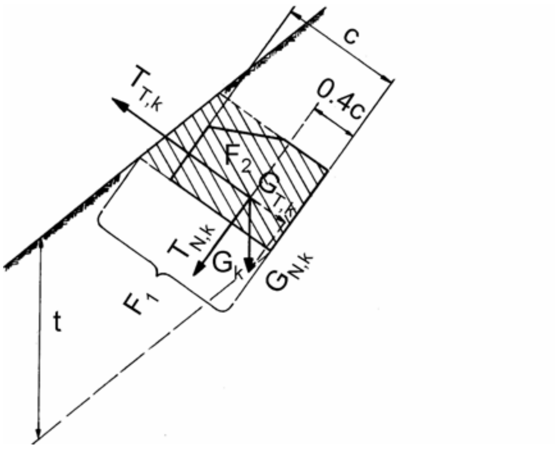
The characteristic value of the tension force $T_{T,k}$ must accord with the following condition (also see Section 5.5.5):

$$\boxed{T_{T,k} \leq \frac{(F_1 + 2F_2) \cdot s_B + G_{T,k} + (T_{N,k} + G_{N,k}) \cdot \tan \varphi_{Ek}}{2}} \quad [kN] \qquad (42)$$

$T_{N,k}$ component of the characteristic value of the resultant reaction force normal to the foundation area F_c
$G_{N,k}$ component of the characteristic value of the foundation weight including superimposed earth load (hatched area in Fig. 36) normal to the foundation area F_1
$G_{T,k}$ component of the characteristic value of the foundation weight including superimposed earth load (hatched area in Fig. 36) parallel to the foundation area F_1
F_1 downslope surface of the foundation up to the surface of the ground
F_2 Lateral surface of the foundation up to the surface of the ground (hatched in Fig. 36)
s_B ultimate shear resistance along the surface in undisturbed ground according to Tab. 9
φ_{Ek} characteristic angle of friction for transfer of compression forces (assumed constant)

$$\boxed{\tan \varphi_{Ek} = 0.8} \qquad (43)$$

Fig. 36 > Dimensioning of the upper concrete foundation to withstand shear forces.



In the absence of particular tests for the determination of the ultimate shear resistance s_B, the following values that apply to a total foundation depth t of 1 m are to be used:

Tab. 9 > Ultimate shear resistance along the surface of the foundation in the undisturbed ground.

Type of ground	s_B [kN/m²]
Sound, compact, rock	> 800
Unsound, fissured, rock	80–800
Heavily pre-stressed ground, moraine	20–80
Very coarse, densely bedded, gravel	20–40
Limey and densely bedded shingle sand	20–25
Loosely bedded shingle sand and rock debris	15–20

The increase in the s_B values with depth of foundation t can be included as given in Tab. 10:

Tab. 10 > Increase in the ultimate shear resistance s_B with depth of foundation.

Foundation depth t in m, measured vertically	Effective s_B value as a function of the s_B value for 1 m depth of foundation
1.0	$1.0 \cdot s_{B\,(1\,m)}$
1.5	$1.2 \cdot s_{B\,(1\,m)}$
2.0	$1.3 \cdot s_{B\,(1\,m)}$
3.0	$1.4 \cdot s_{B\,(1\,m)}$

The intermediate values for other depths can be interpolated.

Example:

With very coarse, densely bedded, gravel, the s_B value for 1 m foundation depth amounts to 30 kN/m². For a foundation depth of 1.25 m, the ultimate shear resistance is

$$s_B = 1.1 \cdot 30 = 33 \text{ kN/m}^2$$

5.9.5.5 Upper concrete foundations with thin, loose ground cover over sound rock

Where the rock anchors are suitably dimensioned as given in Section 5.9.7.2, the tension forces for the latter may be used (see Fig. 37). A pinned connection of the supporting structure to the foundations is permissible and mostly more economical.

Fig. 37 > Foundation in sound rock.

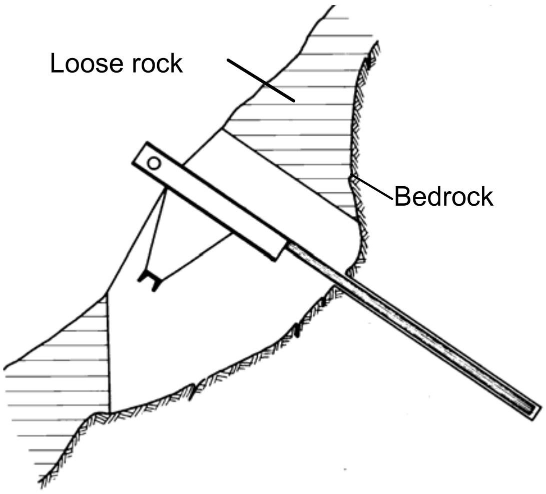
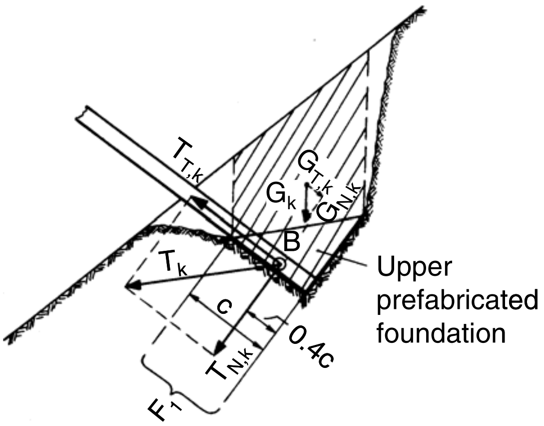


Fig. 38 > Dimensioning of upper prefabricated foundation to withstand compressive forces.



5.9.6 Prefabricated foundations in loose ground

5.9.6.1 Definition

Prefabricated foundations are foundations produced under factory conditions (e.g. ground plates fabricated from steel profiles) and installed at the site.

5.9.6.2 Corrosion

With prefabricated foundations, the **risk of corrosion** must be specially considered (where necessary, ground samples to be tested for corrosive constituents).

5.9.6.3 Dimensioning of the upper prefabricated foundations to withstand compressive forces

Rigid fastening between supporting structure and foundations (Fig. 38): as specified in Section 5.9.5.3.

5.9.6.4 Dimensioning of upper prefabricated foundation to withstand shear forces

For non-undercut prefabricated foundations, the s_B values as specified in Tab. 9 are not valid, since when the foundation is pulled out, the break takes place in refilled material, whose cohesion is reduced. When refilling, the material should be compacted as far as possible, and this has been assumed in making the following assumptions for the calculation.

The characteristic value of the tension force $T_{T,k}$ must accord with the following conditions (also see Section 5.5.5):

$$\boxed{T_{T,k} \leq \frac{(F_1 + 2F_2) \cdot s^*_B + G_{T,k} + (T_{N,k} + G_{N,k}) \cdot \tan \varphi_{Ek}}{2}} \quad [kN] \qquad (44)$$

$T_{N,k}$ component of the characteristic value of the resultant reaction force normal to the foundation area F_1 (assumption: rigid connection between the supporting structure and the foundations).

$G_{N,k}$ component of the characteristic value of the weight of the ground (hatched area in Fig. 39) normal to the foundation area F_1

$G_{T,k}$ component of the characteristic value of the weight of the ground (hatched area in Fig. 39) parallel to the foundation area F_1

F_1 downslope surface of the foundation up to the surface of the ground

F_2 Lateral surface of the foundation up to the surface of the ground (hatched in Fig. 39)

s^*_B ultimate shear resistance along the surface in the refilled material. For a foundation depth t of 1 m:

$$\boxed{s^*_{B(1m)} = 10 \text{ kN/m}^2} \qquad (45)$$

The increase of the s^*_B values with foundation depth t [m] may be accounted for using the following equation:

$$\boxed{s^*_{B(t)} = \frac{s^*_{B(1m)}}{2} \cdot (1+t)} \qquad [\text{kN/m}^2] \qquad (46)$$

φ_{Ek} Characteristic angle of friction for the transfer of compressive forces (assumed constant; $\tan\varphi_{Ek} = 0.8$ cf. equation 43)

Fig. 39 > Dimensioning of upper prefabricated foundation to withstand shear forces.

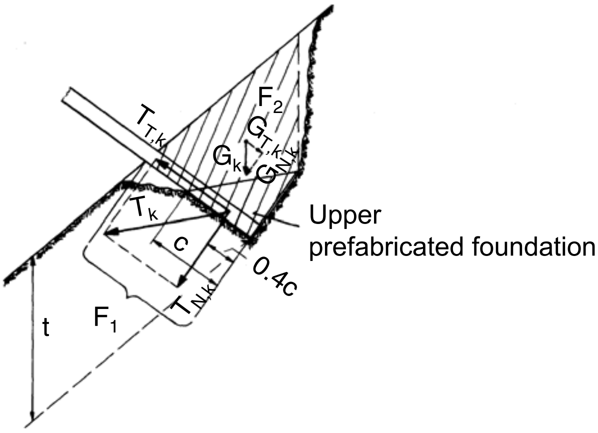
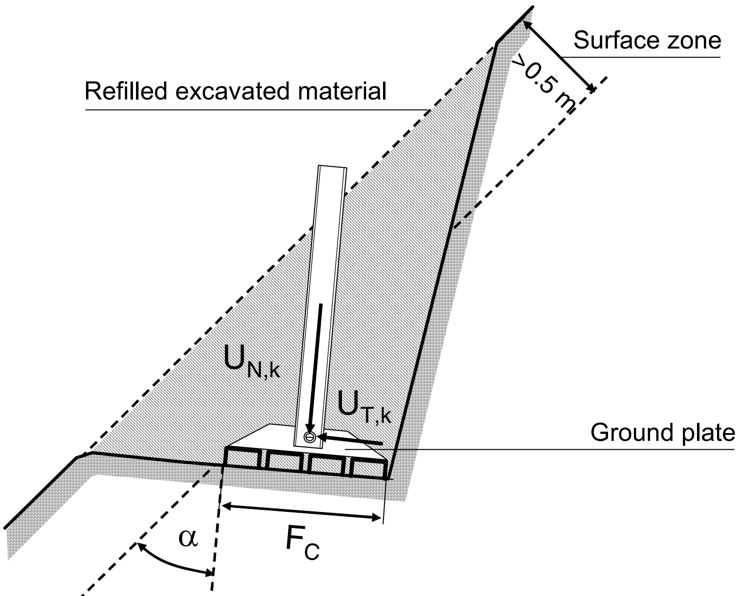


Fig. 40 > Dimensioning of the lower prefabricated foundation.

It is preferable to design the connection between the support and the foundations as pinned. The load on the foundations consists of a normal force $U_{N,k}$ and a transverse force $U_{T,k}$.



5.9.6.5 Dimensioning of the lower prefabricated foundation to withstand compressive forces

The lower foundation area F_C, which must be completely interred beneath a surface zone of 0.5 m (Fig. 40), must accord with the following condition:

$$\boxed{F_C \geq \frac{2 \cdot U_{N,k}}{\sigma_\alpha}} \qquad [m^2] \qquad (47)$$

$U_{N,k}$ characteristic value of the axial support force normal to the foundation area F_c
σ_α specific total ground resistance in a direction normal to F_c (cf. Tab. 9 and Section 5.9.4.6)

5.9.6.6 Dimensioning of the lower prefabricated foundation to withstand shear forces

The shear force $U_{T,k}$ must accord with the following condition:

$$\boxed{U_{T,k} \leq \frac{U_{N,k} \cdot \tan\varphi_{Ek}}{2}} \qquad [kN] \qquad (48)$$

$U_{T,k}$ Characteristic value of the transverse force at the base of the support parallel to the foundation area F_c
φ_{Ek} Characteristic angle of friction for transfer of compressive forces (assumed constant; $\tan\varphi_{Ek} = 0.8$ cf. equation 43)

5.9.7 Anchors and micropiles

5.9.7.1 General and terminology

5.9.7.1.1 General

It is usually preferable to design the foundations of permanent supporting structures using anchors and micropiles, as opposed to the methods specified in Sections 5.9.5 und 5.9.6.

5.9.7.1.2 Definition of anchor

Anchors are relatively short, drilled, slender, load-bearing elements, designed to withstand tension. For the purposes of this technical guideline, they are usually not prestressed.

With anchors, a distinction is made between **rock anchors** and **ground anchors.** Ground anchors are further divided into **non-explosive anchors** (incl. net anchors) and **explosive anchors.** For **wire rope anchors,** a wire rope is used as the tension member (cf. Section 5.2.6.4).

5.9.7.1.3 Definition of micropile

Micropiles are relatively short, drilled, load-bearing elements of small diameter that are usually subject to compressive loading.

With micropiles, a distinction is made between **non-explosive** and **explosive** micropiles.

5.9.7.1.4 Load transfer

The transfer of loads from the supporting structure to the anchor or micropile must take place **at ground level,** i.e. not projecting above the ground.

5.9.7.1.5 Pull-out test

With larger projects and under difficult ground conditions, pull-out tests should be carried out prior to the choice of structure and/or the tendering procedure, in order to determine the characteristic pull-out resistance $R_{a,k}$ of the anchors and micropiles. For each type of ground having comparable geotechnical properties, 3 test anchors should normally be installed. Where only 1 or 2 test anchors are installed for each type of ground, the smallest measured value of the external resistance R_a should be reduced by 40% or 20% respectively.

5.9.7.1.6 Inspections and quality testing

Where an inspection (see Section 5.4.2) shows **heavy deformation or damage,** this must be checked and/or eliminated immediately. Where necessary, tests of the load-bearing capacity are recommended up to 1.35 times the maximum expected tension force F_k. If necessary, the anchors or micropiles should be replaced. To verify the quality of the work, acceptance tests should be carried out on approx. 5% of all anchors, or at least on three anchors per ground category having comparable geotechnical characteristics. The anchor work should be monitored and logged.

5.9.7.1.7 Corrosion protection

Corrosion protection: see Sections 5.2.4.3 and 5.2.6.4

5.9.7.1.8 Dimensioning

Verification of ultimate limit state: also see Sections 5.2.2 and 5.2.3.
The **external ultimate limit state** of an anchor or micropile must comply with the following condition:

$$\boxed{E_D \leq R_D} \qquad (49)$$

$E_d = \gamma_Q \cdot F_k$: design effect of actions (loading), where F_k is the characteristic value of the tension or compression force (e.g. that resulting from snow pressure), and the load coefficient γ_Q =1.5.
$R_d = R_{a,k} / \gamma_M$: design resistance of the anchor or micropile, whereby $R_{a,k}$ is the characteristic external resistance of the anchor (pull-out resistance) or micropile (pile resistance) and the coefficient of resistance γ_M =1.35.

The **internal ultimate limit state** of an anchor or micropile must be demonstrated for the statically effective steel cross-section as specified in Section 5.2.3.2.

5.9.7.2 Rock anchors

5.9.7.2.1 Principles

Rock anchors can accept both tension and shear forces. Where the anchors are subject to both tension and shear forces (tension direction ≠ anchor direction), this must be considered in dimensioning the anchors. In situations of this kind, it is mostly preferable to use flexible wire rope anchors.

5.9.7.2.2 Installation

In sound rock, a hole is drilled in which a tension member (ribbed bar or wire rope) is fastened with or without additional wedge fastening. The borehole is filled with grout from the bottom up, whereby the anchor is riddled. Also, the anchor should be centered as far as possible in the borehole.

5.9.7.2.3 Implementation

Sound rock is compact to slightly fissured, non-weathered, rock that may be drilled in the normal way.

5.9.7.2.4 Essential requirements

The borehole must have a diameter of at least 1.5 times the diameter of the anchor, and the anchor must be covered by at least 10 mm of grout. Prior to injection of the grout, the borehole must be blown clean.

5.9.7.2.5 Dimensioning

Rock anchors are to be dimensioned as specified in Section 5.9.7.1.8. The length of the anchor must be designed in accordance with the quality of the rock. The anchor resistance depends on the weathering, extent of mineral cohesion, type of rock and the spacing of the parting plane. In strata close to the surface, the strength of the rock is often reduced, so that the anchor length within the rock should be **at least 1.5 m.** In rock largely free of fissures, the following values of the ultimate pull-out resistance $R_{a,k}'$ per meter anchor length can be taken as follows:

Tab. 11 > External specific anchor resistance in tension in rock largely free of fissures as a function of borehole diameter.

Borehole diameter (mm)	Ultimate skin friction between anchor grout and rock s_k (N/mm²)	Pull-out resistance per meter anchor length $R_{a,k}'$ (kN/m')
30	1.00	95
45	2.00	280

5.9.7.3 Ground anchors

5.9.7.3.1 Principles

Ground anchors can accept tension forces in the axial direction.

5.9.7.3.2 Neglect of shear forces

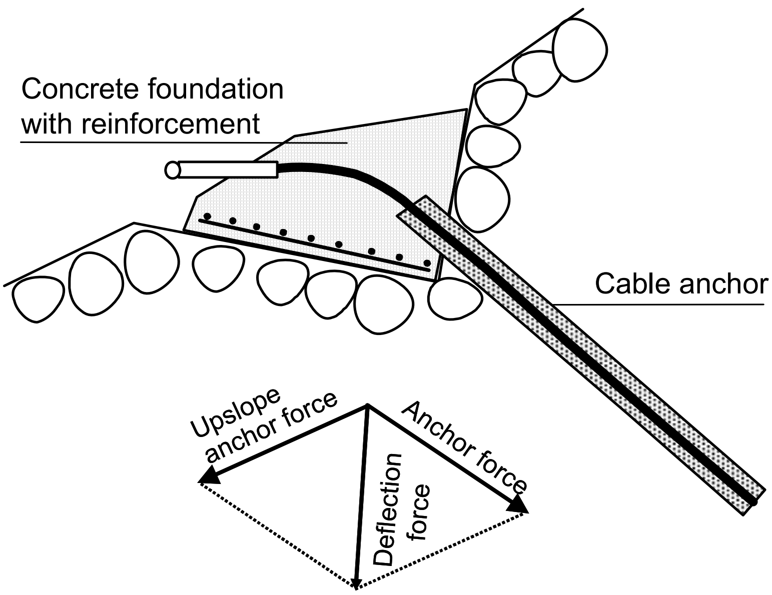
The possibility of shear forces arising from lateral loads as specified in Section 5.5.6 is acknowledged, but is ignored for dimensioning purposes.

5.9.7.3.3 Consideration of shear forces

By contrast, shear forces arising through upslope wire rope anchors of snow nets through deflection of the anchor force must be considered in dimensioning the anchors. At favorable sites (densely bedded, compact, ground), the installation of an additional stabilizing pipe will suffice. At unfavorable sites (humus-like, loose, ground; large snow glide), the installation of a concrete foundation is recommended (Fig. 41).

If it must be expected that the resultant snow pressure in plan view is not at right angles to the supporting surface (owing to irregularities in the terrain or in the distribution of the snow height), the share forces caused by the additional loading as specified in Section 5.5.6 must be taken into account in dimensioning the anchoring of **single structures** and of **short rows of structures.**

Fig. 41 > Concrete foundation with reinforcement to withstand deflection forces, ensuring that the compression force caused by the wire rope anchor alone is reduced.



5.9.7.4 Net anchors

5.9.7.4.1 Installation

The anchor and the injection pipe are introduced down to the lowest point of the borehole, which, if necessary, is lined with a stocking. The injection pipe is then retracted slowly until the borehole has been filled with grout.

5.9.7.4.2 Implementation

This anchor type is suitable for the following types of ground:

> coarse gravel
> permeable, bolder-filled, ground, such as with scree slopes or rock debris

5.9.7.4.3 Essential requirements

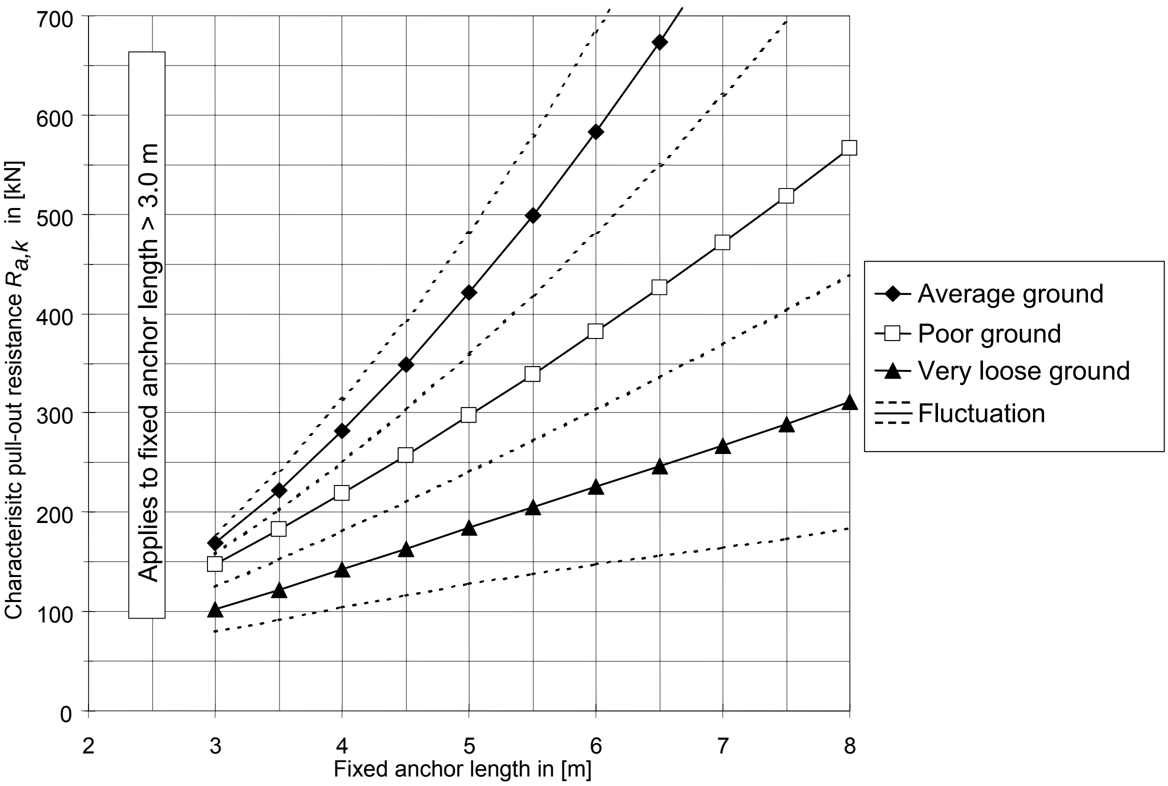
The following essential requirements apply:

> the minimum diameter of the borehole is 90 mm.
> the **maximum mesh** of the stockings is **10 mm.**
> as far as possible, the anchors to be **centered** in the borehole.
> the anchor grouts applied must comply with Section 6 of this technical guideline.
> the **required anchor length** must be determined separately for each project (cf. Sections 5.9.7.1.5 and 5.9.7.4.4).
> the anchors should not be inclined less than *15°* to the horizontal.
> the **grout cover** of the tension member (anchor bar, wire rope) must be a minimum of **20 mm.** The grout cover of sleeve pipes must be a minimum of **10 mm.**

5.9.7.4.4 Dimensioning

Net anchors must be dimensioned as specified in Section 5.9.7.1.8. For the purposes of **pre-dimensioning,** and where no anchor tests or practical experience in comparable ground are available, the characteristic pull-out resistance $R_{a,k}$ of the anchor can be estimated from the following diagram (Fig. 42) as a function of anchor length and ground category. A reliable distinction between average, poor or very loose ground conditions is hardly possible at the site. For final determination of the anchor lengths, pull-out tests must be carried out. A minimum of 3–5 tests must be carried out per hectare site area (cf. Section 5.9.7.1.5). Where the ground conditions in the substratum within the site perimeter are very inhomogeneous, the number of tests must be increased accordingly.

Fig. 42 > Characteristic pull-out resistance $R_{a,k}$ as a function of anchor length and ground category for non-explosive anchors and net anchors.



Average ground:	Densely bedded with fine cohesive material, e.g. coarse scree with fine, binding, material constituent and/or dry shingle sand.
Poor ground:	Loosely bedded ground with non-cohesive, non-binding, fine, material with low angle of friction (e.g. damp, fine-grained, weathered, constituents, permitting only poor bonding between the anchor and the ground, and scree slopes).
Very loose ground:	Very loosely bedded, humus-like, ground with cavities.

5.9.7.5 Explosive anchors

5.9.7.5.1 Installation

Through detonation of an explosive at the lower end of the borehole, a pear-shaped cavity is created in the borehole. Prior to blasting, the borehole is reinforced with a sleeve pipe, so that subsequently the anchor can be inserted. The anchor should traverse the entire cavity. Cavity and borehole must be completely filled with grout by means of an injection pipe.

5.9.7.5.2 Implementation

This type of anchor is suited to densely bedded, gravelly to sandy, ground.

The decision whether or not use explosive anchors, and the dimensioning of the explosive charge should, however, be made with great care.

5.9.7.5.3 Essential conditions

The following essential conditions apply:

> the minimum diameter of the borehole is 90 mm.
> for **sleeve pipes,** steel tubes with at least 60 mm external diameter and at least 2 mm wall thickness must be used. The tubes should be cross-slit at their lower end in the axial direction over a length of 100 to 150 mm and pinched together. Furthermore, the tubes are to be slit or drilled so that the grout can easily run out and surround the tube. The slit width or hole diameter must be at least 20 mm. The openings should be positioned at intervals of 500 mm and displaced relative to one-another.
> the **explosive charge** must be inserted up to the tip of the tube.
> in general, it is recommended to use an **electric detonator:** this is absolutely essential where several boreholes lie close together (twin anchors: simultaneous detonation essential).
> the anchor must be 300 mm **longer** than the sleeve pipe, to be absolutely sure that it traverses the entire pear-shaped cavity. It must also be **centered** in the tube as far as possible.
> the **injection of grout** must be such that the grout in and outside the tube rises to ground level. The anchor grouts used must comply with Section 6 of this technical guideline.
> the **necessary anchor length** must be determined separately for each project (cf. Sections 5.9.7.1.5 and 5.9.7.5.5).

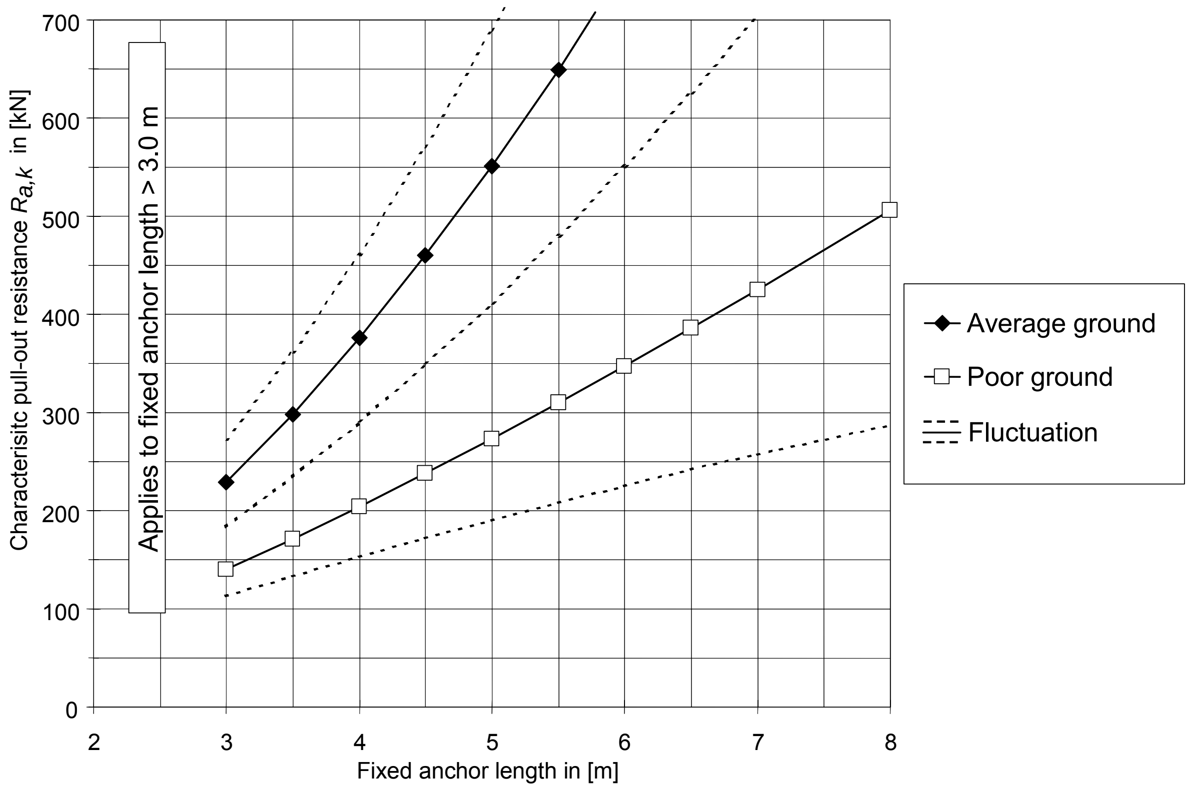
5.9.7.5.4 Note

Experience shows that 50 to 100 g of slow explosive provide a cavity volume of 30–50 l.

5.9.7.5.5 Dimensioning

Explosive anchors are to be dimensioned as specified in Section 5.9.7.1.8. For **pre-dimensioning** purposes, and where no anchor tests or practical experience in comparable ground are available, the characteristic pull-out resistance $R_{a,k}$ of the anchor can be estimated from the following diagram (Fig. 43) as a function of anchor length and ground category. The diagram is also applicable to twin anchors. A reliable distinction between average and poor ground conditions is hardly possible at the site. For final determination of the anchor lengths, pull-out tests must be carried out. A minimum of 3–5 tests must be carried out per hectare site area (cf. Section 5.9.7.1.5). Where the ground conditions within the site perimeter are very inhomogeneous, the number of tests must be increased.

Fig. 43 > Characteristic pull-out resistance $R_{a,k}$, as a function of the anchor length and ground category for explosive anchors.



Average ground:	Densely bedded with cohesive fine material (e.g. coarse scree with a percentage of cohesive fine material, or dry shingle sand).
Poor ground:	Loosely bedded, with non-cohesive, non-binding fine material with low angle of friction (e.g. humid, fine-grained, weathered, constituents permitting only a poor bond between the anchor and the ground, and scree slopes).

5.9.7.6 Micropiles

5.9.7.6.1 Principles

Micropiles can sustain the forces acting in the axial direction.

5.9.7.6.2 Shear forces

Shear forces resulting from supplementary loads (drilling errors, transverse load on support according to Section 5.5.4) must be considered when dimensioning the foundations (the transverse loads must amount to at least 20% of the central pressure force acting along the axis of the support). The shear forces must be carried either by supplementary anchors or by ground resistance.
Where either the ground is poor or heavy snow glide or very steep terrain are present, the micropile must be secured by a supplementary anchor.

5.9.7.6.3 Installation

Micropiles are fabricated in the same way as anchors.

5.9.7.6.4 Essential conditions

To prevent buckling of the micropile head, this must be reinforced over a length of at least 1.5 m by means of stiffening pipes, concrete base or similar means. The stiffening pipes must extend up to the top of the micropile, so that the anchor bar does not project above the stiffening pipe. The grout cover of the stiffening pipes must be at least 10 mm thick.

5.9.7.6.5 Dimensioning

Micropiles must be dimensioned according to Section 5.9.7.1.8. The resistance of a micropile under compression is 50% higher than under tension (anchor) and can be provisionally dimensioned using Figs. 42 and 43 for non-explosive anchors, net anchors or explosive anchors.

In calculating the internal ultimate limit state, the steel cross-section only may be considered (see Section 5.2.3.2).

5.9.7.6.6 Note

In applying compression forces whose direction is not precisely known (e.g. swivel support with snow nets attached to the foundation via a ball joint), the use of micropile/anchor foundations in loose ground is very questionable. In these situations, the use of concrete foundations or ground plates is to be preferred.

5.9.7.7 Special foundation methods

Under difficult ground conditions (e.g. ground of limited resistance, large loss of grout and unstable boreholes), injection drilled anchors, sack anchors, subsequent injection or casing drilling may be used. Injection drilled anchors are dimensioned in the same way as net anchors (see Section 5.9.7.4). To determine the final anchor length, anchor tests are essential.

6 > Use of anchor grout in avalanche defense

6.1 General

6.1.1 Objective

The following sections prescribe the procedures for suitability tests for anchor grout and the verification procedure during on-site grouting work. The procedures are obligatory for the use of anchor grout for avalanche defense structures subsidized by the Confederation.

6.1.2 Suitability tests

The suitability of the anchor grout must be verified by means of a suitability test. This must be carried out by a neutral laboratory. The last suitability test or report must have been performed within the **last 3 years.** The suitability test forms the basis of the type approval test for anchor grout (cf. Section 8.3).

6.1.3 Test of conformity

During the grouting work at the site, the conformity of the grout used must be continually checked. The frequency of tests should be appropriate to the quantity of grout processed, the importance of the site, the experience of the building company and the combination of grout and mixing pump used, and be such that the fluctuation of the grout properties may be ascertained. The test must be carried out by a neutral laboratory. Since the test results take between 1 and 2 months to obtain, it is advisable to carry out preliminary tests prior to commencing building work, and to monitor the properties of the fresh grout by measuring the air void content.

6.2 Normal anchor grout

6.2.1 Suitability test

6.2.1.1 Test procedure

The suitability test is performed with grout whose consistency is adjusted so that it may be pumped. The test must consist of the following procedures:

6.2.1.2 Properties of fresh grout

The settling and spreading properties, the void content and the bulk density are determined using fresh grout based on the EMPA method.

6.2.1.3 Tests on set grout

The following tests must be carried out on set grout. The tests are carried out on prisms of dimensions 40x40x160 mm, prepared according to the EMPA method:

> bulk density, bending and compressive strength after 24 hours and after 3, 7, 14 and 28 days (air storage at 20°C, 90% relative humidity) prepared in analogy to SIA 215.001.
> elasticity module after 28 days according to SIA 262/1, Annex G (stress range σ = 0.5 ... 5.0 N/mm²).
> determination of the frost resistance FS according to the older SIA standard 162/1, Test no. 7, on prismatic sections (test begin after 21 days).
> measurement of the shrinkage analogously to SIA 262/1, Annex F (test conditions: 20°C, 70% relative humidity), test duration up to 90 days.

6.2.1.4 Requirements for anchor grout

The anchor grout must comply with the following limiting values:

> ompressive strength:	7 days:	$f_c \geq 22$ N/mm²
	28 days:	$f_c \geq 35$ N/mm²
> elasticity module:		$E \leq 25'000$ N/mm² (stress range $\sigma = 0.5 \dots 5.0$ N/mm²)
> frost resistance:		$FS \geq 1.5$
> linear shrinkage:	nach 28 days	$\varepsilon_{cs} \leq 2.0‰$

6.2.2 Test of conformity

6.2.2.1 Sample preparation

The grout and the sample are prepared on site. The grout to be tested is prepared in the normal way by a batch mixer. The sample material is taken at the end of the supply pipe or at the point of installation.

6.2.2.2 Samples

The following samples are required for the test:

> 9 prisms 40 x 40 x 160 mm or
> 2 cylinders D = h = 200 mm or
> 2 cubes 200 x 200 x 200 mm or samples with comparable volume

6.2.2.3 Marking and transport

The samples must be clearly and permanently marked. Transport of the samples to the test laboratory must not take place before setting, and at the latest 2 days after setting.

6.2.2.4 Storage

Following preparation, and prior to delivery to the laboratory, the samples must be stored in such a way as to prevent loss of humidity and ensure a temperature of at least 10°C (wrapped in plastic sheeting, water storage, storage in the builder's shack).

6.2.2.5 Data required by test laboratory

To ensure that the test values may be properly interpreted, the following data must be supplied to the test laboratory together with the samples:

> customer
> details of grout preparation
> type and date of sample preparation
> temperatures of the air and the grout at time of preparation
> time of ejection from the mold
> storage conditions

6.2.2.6 Test sample

The test is carried out on samples selected from those delivered to the laboratory from the site in the form of prisms of dimensions 40 x 40 x 160 mm or bore samples with D = 50 mm (diameter = height).

6.2.2.7 Storage in the laboratory

Prior to performing the tests, the samples are stored in the laboratory at an air temperature of 20°C at 90% relative humidity.

6.2.2.8 Scope of the test of conformity

The test of conformity must comprise the following procedures:

- bulk density, compressive strength after 7 and 28 days analogously to SIA 215.001 or EN 12504-1.
- determination of the frost resistance FS according to the older SIA standard 162/1, Test no. 7 (test begin after 21 days).

6.2.2.9 Requirements for anchor grout

The test of conformity is based on the following limiting values:

> compressive strength:	7 days:	$fc \geq 22$ N/mm²
	28 days:	$fc \geq 35$ N/mm²
> frost resistance:		$FS \geq 1.5$

6.3 Special anchor grout for use in permafrost

6.3.1 Suitability test

6.3.1.1 General

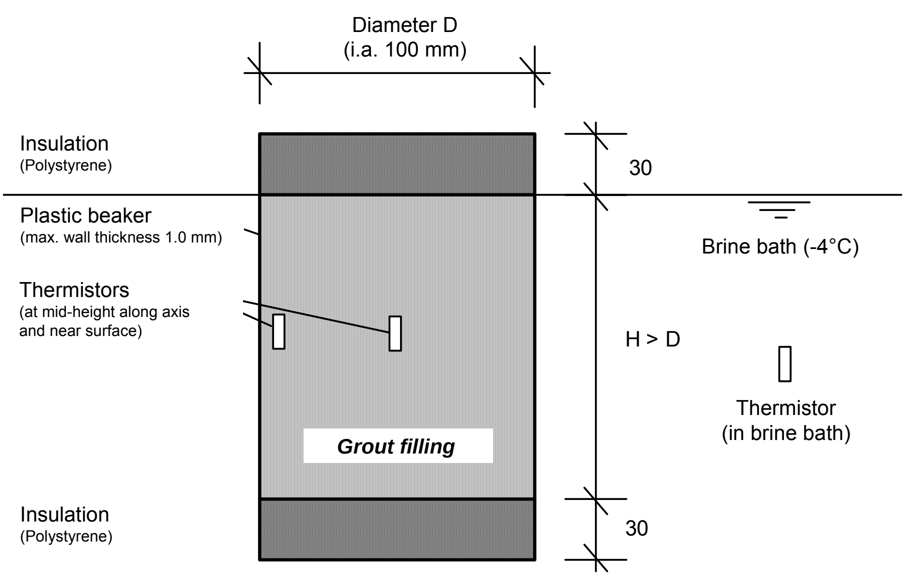
The quality of the special grout must comply with the requirements of the suitability test for normal anchor grout. The grout is tested as given in Section 6.2.1.

In addition, the specific suitability test for use in permafrost ground or rock must be performed. The tests are performed down to temperatures of –4°C in the ground. To ensure comparability of the results, the tests are performed using the EMPA method.

6.3.1.2 Sample preparation

The special grout used in the test is mixed according to the supplier's instructions at a temperature of 20°C (cf. Section 7.6.2). To prepare the samples, the grout is filled into plastic beakers having a diameter corresponding to the borehole (usually 100 mm) and a height > D mm (cf. Fig. 44). The samples are insulated at top and bottom with 30 mm polystyrene. The maximum wall thickness of the beaker is 1.0 mm. A total of 8 samples of this type must be prepared. A temperature sensor (thermistor) must be placed in one of the samples at mid-height along the axis, and another at mid-height near the surface of the sample.

Fig. 44 > Dimensions of sample.



6.3.1.3 Cooling curve of fresh grout

The 8 samples must be placed immediately in a brine bath (water containing 10% salt) cooled to a temperature of –4°C. The samples should be immersed to just below the upper insulation. The cooling curve of the grout must be measured continuously and the temperature of the brine bath likewise. The volume of the brine bath should be 5 times that of the sample. Using this arrangement, an inert sample will be cooled from 20°C to 0°C within 1–1¼ hours. An inert sample is one whose heat of hydration has escaped. The cooling rate so obtained corresponds to that of grout in contact with permafrost ground or rock.

6.3.1.4 Sub-zero storage

The brine baths containing the samples are maintained during the entire storage period of up to 28 days at a constant temperature of –4°C using closed-cycled air circulation cooling.

6.3.1.5 Requirements on grout after sub-zero storage

Following sub-zero storage, the grout must conform to the following limiting values specified in Section 6.2.2. The compressive strength is measured on all test samples. Prior to the test, the test samples are thawed out at 20°C for a period of 24 hours.

> compressive strength:	7 days:	$f_c \geq 22$ N/mm² (3 test samples)
	28 days:	$f_c \geq 35$ N/mm² (3 test samples)
> frost resistance:		$FS \geq 1.5$ (1 series cut from 1 single test sample, test according to the older SIA standard 162/1 no. 7, test begin after 21 days)

6.3.1.6 Workability

Using practical pumping tests, the workability of the special grout must be established using the usual mixing and injection appliances. The consistency necessary to permit pumping and the duration of this should be established.

6.3.2 Test of conformity

The test of conformity of the special grout is performed according to Section 6.2.2.

7 > Avalanche defense in permafrost

7.1 General

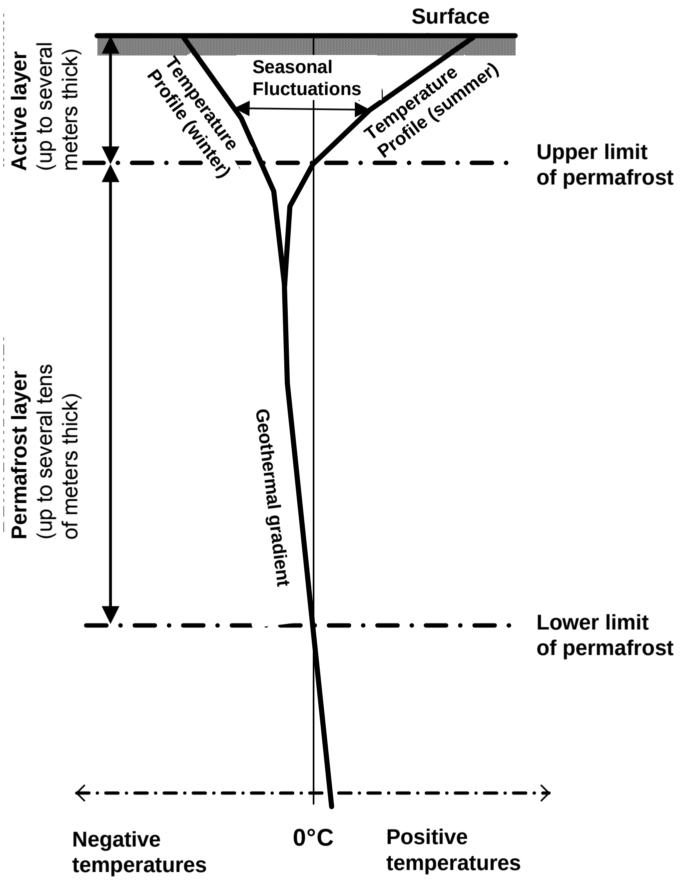
The publication Avalanche Defense in Permafrost (final report and comments; obtainable from: Swiss Federal Institute for Snow and Avalanche Research, Flüelastrasse 11, CH-7260 Davos) contains comments on the following sections.

7.1.1 Definitions and terminology

7.1.1.1 Permafrost

Permafrost or permanently frozen ground signifies ground near the surface that is subject to negative temperatures during the whole year (Fig. 45). The definition of permafrost is based on temperature alone, i.e. irrespective of the ice content.

Fig. 45 > Typical temperature distribution in a permafrost layer.



7.1.1.2 Upper permafrost limit

The **upper permafrost limit** is the upper limit of the permafrost layer at the depth where the actual permafrost begins. Below that, the temperature of the ground is negative over the whole year. The lower limit of the permafrost layer is the so-called **lower permafrost limit.** The layer above the upper permafrost limit, which thaws out in summer and freezes again in winter, is described as the **active layer.**

7.1.1.3 Block glaciers

Block glaciers are a typical feature of permafrost and signify rock debris oversaturated with ice. Owing to the viscosity of the ice, so-called **active block glaciers** creep slowly down the slope under gravity. **Fossil or inactive block glaciers,** on the other hand, are former block glacier flows, in which the ice has melted away. These have come to a standstill, and are today no longer in motion.

7.1.1.4 Continuous and patchy permafrost

In areas with **continuous permafrost,** the ground is subject to uninterrupted permafrost. As opposed to that, in areas with **patchy permafrost,** frozen and unfrozen zones alternate.

7.1.2 Occurrence and properties of permafrost

7.1.2.1 Occurrence

Permafrost occurs in the Alps at altitudes upwards of 2500 to 3000 m a.s.l. The lower line of permafrost, i.e. the altitude to which the permafrost descends, depends on various factors. The main factors are slope exposure, local climatic conditions (i.e. air temperature, solar irradiation, wind conditions), local ground conditions, snow thickness in winter and duration of snow cover. In the transition regions between 2500 and 3000 m a.s.l., permafrost does not occur continually but in patches.

7.1.2.2 Ground conditions

The ground in a permafrost area consists either of loose ground (in the high mountains, often in the form of rock debris or moraine) or of rock. In permafrost slopes, the layers near the surface often consist of weathered boulder-filled debris lying above the bedrock.

7.1.2.3 Ice content

Permafrost loose ground is classified according to its ice-content:

1. Dry permafrost (ice content = 0)
2. Permafrost undersaturated with ice (ice content lower than the cavity volume of the loose ground)
3. Permafrost saturated with ice (ice content equal to the cavity volume)
4. Permafrost oversaturated with ice (ice content greater than the cavity volume)

Loose ground that is almost saturated with ice or oversaturated is designated as **ice-rich permafrost,** and loose ground that is dry or undersaturated as **ice-deficient permafrost.**

7.1.2.4 Creep

Under slope conditions, ice-rich loose ground, including block glaciers (Section 7.1.1.3), tend to creep owing to their viscous characteristics.

7.1.2.5 Fissures

The fissures in rock subject to permafrost are often filled with ice.

7.1.3 Interaction between defense structures and permafrost

Avalanche defense structures have no identifiable warming effect on the permafrost layer. No measurable heat input takes place in summer from the steel structures (snow bridges or nets) via the foundations to the ground. Long-term computer simulations have shown that the changed snow conditions upslope of the structures in winter had a negligible influence on the temperature distribution in the permafrost. The gap in the snow cover that normally forms below the support grates or nets as a result of creep and glide of the snow cover has a mild cooling effect on the permafrost layer over the decades, thereby favoring the formation of permafrost.

7.2 Inspection of the ground

7.2.1 Estimating the occurrence of permafrost in the ground

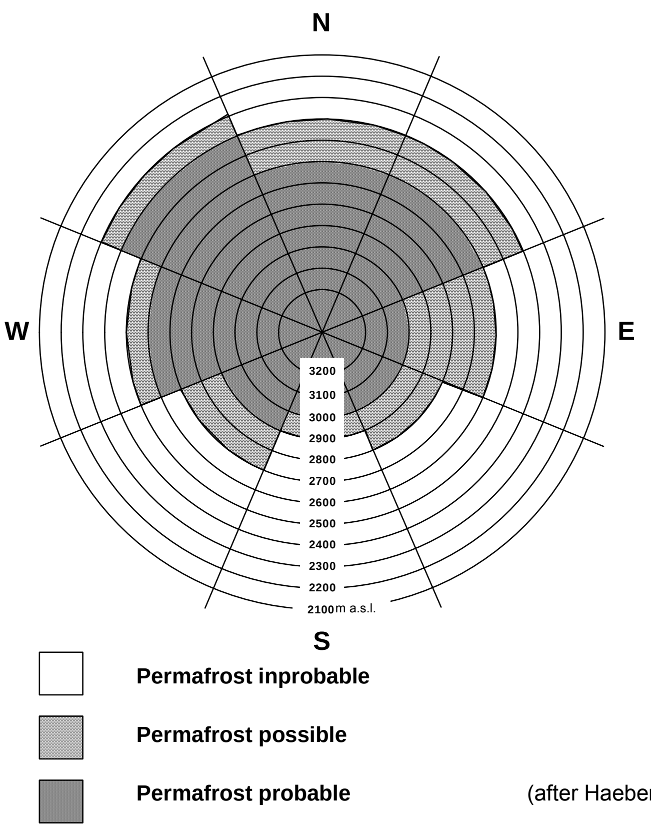
7.2.1.1 Minimum altitude of permafrost

Since the occurrence of permafrost depends on numerous factors (Section 7.1.2.1), it is not possible to specify a general minimum altitude for permafrost.

7.2.1.2 Probability of permafrost

In the preliminary phase of a defense project, the probability of permafrost in the ground may be roughly estimated using the following diagram (Fig. 46), which is based on practical experience. The categories “improbable”, “possible” and “probable” are assigned as a function of the slope exposure and height above sea level. Deviations from the diagram may occur in the particular circumstances of the terrain. Particularly in extremely shaded areas or at wind exposed points, the lower permafrost limit can extend further down in places.

Fig. 46 > Diagram to estimate the occurrence of permafrost.



(after Haeberli, 1975)

7.2.1.3 Aerial photographs and their interpretation

Aerial photographs (obtainable from: Federal Office of Topography, CH-3084 Wabern/BE or Flight Service, KSL, CH-8600 Dübendorf) may be used to identify creep phenomena (block glaciers or solifluction) resulting from permafrost in the ground. Loose ground oversaturated with ice tends to display gravitational creep movement owing to its viscosity, and the corresponding surface patterns may partly be identified in the photographs. The photographs may also be used to distinguish between active and fossil block glacier zones with or without vegetation respectively.

7.2.1.4 On-site inspection

A geomorphological inspection will provide an indication whether permafrost may occur in the ground or not. With the help of a geomorphologist or geologist/geotechnician the occurrence and extent of permafrost may be estimated. Surface flow patterns resulting from permafrost may be identified from the vantage point of the opposite slope. The principal determining factors are given in the following Tab. 12:

Tab. 12 > Determining factors.

Determining factor:	Permafrost likely for:	Permafrost unlikely for:
Vegetation	• none or very sparse	• continuous Alpine sward
Surface patterns	• visible creep patterns, solifluction phenomena • boulder-filled debris	• no visible creep patterns
Block glaciers	• active – active creep movement visible – no vegetation – steep block glacier front	• fossil – no visible creep movement, but fossil creep patterns visible – vegetation present – ground fissures
Moraines	• visible creep patterns	• stable ground conditions
Snow patches	• no thawing in summer (perennial)	• thawing in summer
Temperature of well water in summer	• <2 °C	• >2 °C

7.2.1.5 Indirect and semi-direct methods in permafrost identification

None of the known indirect methods such as the BTS method (measurement of the base temperature of the snow cover), or semi-direct geophysical methods such as the geoelectric, seismic or radar methods, are entirely satisfactorily for the detection of permafrost in the ground when applied to a specific building project. Often, the construction site coincides with the transition between permafrost and permafrost-free areas. Under these conditions, neither the indirect nor the semi-direct methods give unique results that would permit final conclusions to be drawn. Furthermore, in slopes, the permafrost is often dry or undersaturated with ice. In undersaturated conditions, the ice content is too low to distinguish between permafrost and permafrost-free areas using geophysical methods, and in particular, with geoelectric radar methods.

7.2.2 Secure identification of permafrost

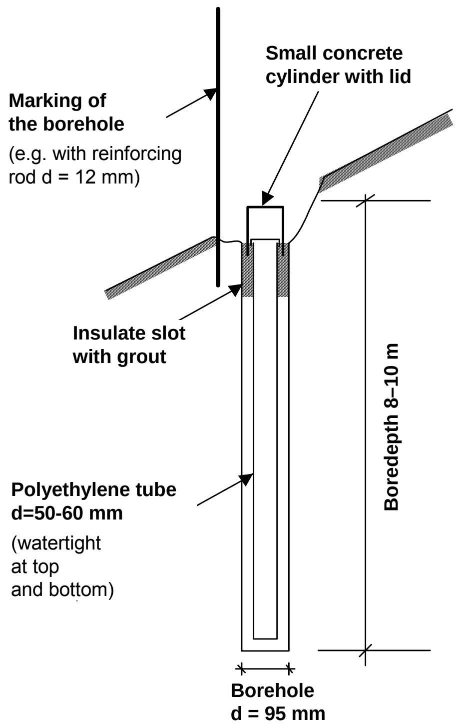
7.2.2.1 Sampling

When in the course of the preliminary assessment and the on-site inspection (Sections 7.2.1.2–7.2.1.4) permafrost is suspected in the ground, exploratory drillings should be made during the project phase (Fig. 47). The drillings should be made vertically up to a depth of 8–10 m, using a conventional drill (down the hole hammer), commonly used with defense structures. The arrangement and number of drillings within the site perimeter is chosen to be representative of the entire site. As a rule of thumb, 1–2 drillings per hectare should suffice with homogenous ground conditions. Under very inhomogeneous conditions, the number should be increased accordingly.

7.2.2.2 Temperature measurement

After a period of 3–4 weeks following boring, the temperature in the borehole will have stabilized, and can be measured using a manual temperature measuring instrument. For this, the temperature probe attached to a cable is lowered into the borehole. To determine the temperature profile, the temperature is measured at intervals of, for example, 1 m (Fig. 47). The probe should be dry (not wet) and must be lowered into the borehole with care to avoid disturbing or intermixing the air strata in the pipe. Sufficient time should be allowed for the temperature to stabilize before making further measurements.

Fig. 47 > Boring to detect permafrost.



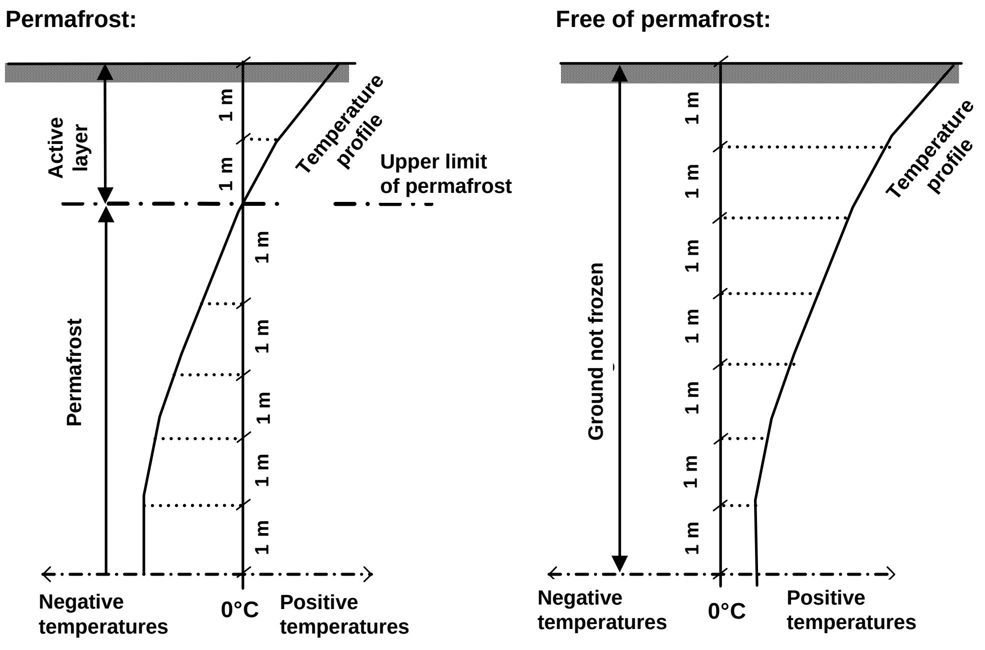
7.2.2.3 Thermistor string

Alternatively, the borehole may be fitted with a thermistor string connected to a data logger. This permits continuous measurement of the temperature, but is more time consuming than using a manual temperature appliance.

7.2.2.4 Time intervals of temperature measurement

Ground temperature measurements must be made at least once in September and October. During this period, the highest temperatures occur at a depth of 4–6 m. Where temperatures of < 0°C are found at this depth, this signifies the presence of permafrost. Negative temperatures near to the surface must be interpreted with care. Here, cooling of the ground occurs from October onwards. Thus negative temperatures here may simply indicate re-freezing of the active layer or seasonal winter frost, and not, therefore, the existence of permafrost in the ground. Only negative temperatures measured at a depth of approx. 2–5 m indicate the existence of permafrost in the proper sense.

Fig. 48 > Temperature profiles in permafrost and permafrost-free areas.



7.2.2.5 Distinguishing between permafrost and permafrost-free areas

Based on the temperature measurements and the temperature profiles obtained from them, permafrost and permafrost-free areas may be distinguished (Fig. 48). The temperature profile of the permafrost may also be used to determine the thickness of the active layer and the depth of the upper limit of permafrost.

7.2.2.6 Precision of temperature measurement appliances

An appliance suitable for the measurement of air temperature should be used. The temperature sensor should not be too sluggish, and should react quickly to temperature changes. The appliance should be tested periodically in an ice bath, and if necessary the temperature offset determined. To do so, pieces of ice are crushed in cold water and stirred constantly. Owing to the high latent heat of ice, a constant temperature of precisely 0°C results in the ice bath. Snow may also be used in place of ice.

7.2.2.7 Problems of drilling the ground where ice occurs

The exploratory drillings using a down the hole hammer are also useful in determining the suitability of the ground for drilling. The presence of ice can make drilling difficult and impede, or completely prevent, extraction of the sample material (Section 7.5.1.1). The results can also be used to estimate the geotechnical profile, enabling zones of loose ground and bedrock to be distinguished. The presence of ice in the ground can be discerned by the presence of ice particles in the extracted material.

7.2.2.8 Core sampling

To obtain precise information on the ground conditions, rotation core samplers can be used.

7.3 Assessment of creep probability in the ground

7.3.1 General

The assessment of the likelihood of creep in a slope forms the basis of reference for the planning of defense structures. Slopes that are stable or have only a slight tendency to creep can be permanently protected. With slopes having a moderate to high tendency to creep, the service life must be expected to be shorter, and in this case, alternative protection measures must be considered. Tab. 13 shows the creep rates and the measures appropriate to selected creep ranges.

7.3.2 Qualitative assessment

7.3.2.1 Creep movement in permafrost

The geomorphological and geological assessment in the terrain (Section 7.2.1.4) provides an indication of the extent of creep movement resulting from permafrost in the ground. Aerial photographs (Section 7.2.1.3) show the creep patterns of block glaciers and solifluction effects. Owing to their viscous characteristics (ice-debris mixture), active block glaciers and smaller block glacier snouts are subject to downward creep of the order of several cm to several dm per year. Defense structures may not be built on these. The geomorphological and geological assessments, and the evaluation of aerial photographs, only permit a qualitative assessment as to whether the slope is stable or whether it can creep. Quantitative assessments of the degree of creep are hardly possible.

7.3.2.2 Unstable equilibrium of loose slope debris

Generally speaking, loosely piled debris on steep slopes is often in unstable equilibrium, since the internal friction angle of the debris is similar in magnitude to the inclination of the slope. For this reason, even in the absence of permafrost soil, slope debris of this kind tends to display slip movement of several mm to several cm per year.

7.3.3 Measurement of creep movement

7.3.3.1 General

Where there is uncertainty concerning the stability of a slope, the creep movement can be monitored either by **engineering surveying** or with **inclinometer measurements** (Fig. 49). The measurements should be continued for a minimum of 2–3 years.

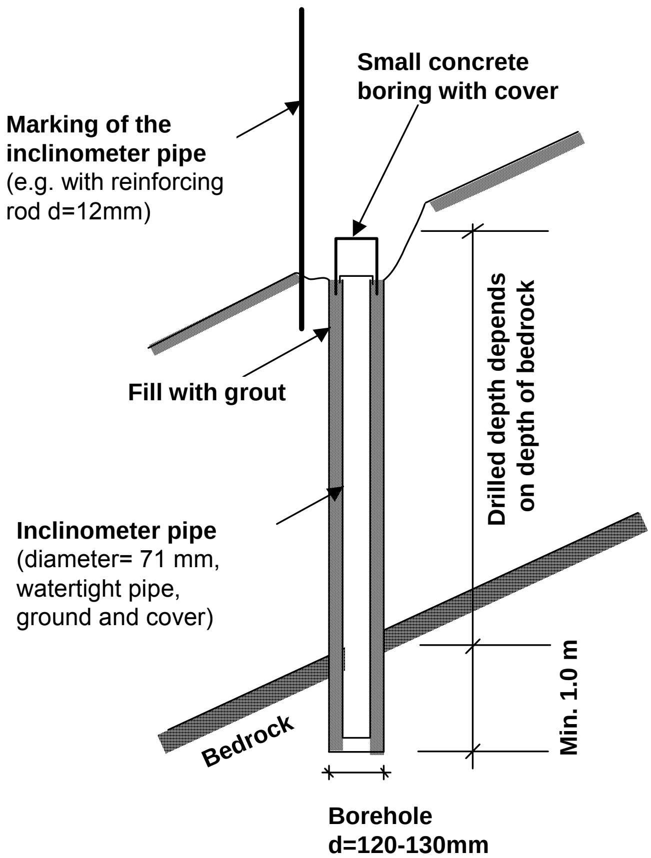
7.3.3.2 Engineering surveying

Using **engineering surveying,** selected points in the terrain (e.g. large boulders) are fitted with gauging pins. The site of the instrument should be chosen in stable terrain (e.g. in bedrock) and provided with sufficient securing points.

7.3.3.3 Inclinometer method

With the inclinometer method the boreholes can be drilled using a conventional drill (down the hole hammer) typically used in avalanche defense structures. The inclinometer pipe, the ground and the cover (Fig. 49) must be watertight, since any water entering the pipe will freeze. The measurements provide information on the magnitude of creep movement and the possible existence of a slip plane. The base of the inclinometer pipe should be fastened in stable rock to provide a point of reference not subject to movement. If this is not possible, the upper end of the inclinometer pipe should be monitored throughout the measuring period using engineering surveying, in order to determine the creep movement of the slope near the surface.

Fig. 49 > Inclinometer pipe for the measurement of slope movement.



7.4 Defense structures in permafrost

7.4.1 Assessment of creep movement in the ground

7.4.1.1 Creep rate

The creep rate (displacement in cm/a) of the slope determines whether defense structures can be erected on it, or if this would involve excessive cost or prove technically impossible.

7.4.1.2 Permissible creep rates for the erection of defense structures

At present, no long-term experience with defense structures under creep conditions in permafrost are available. The creep rates given in the following table are based on estimates. They were taken from known cases of damage and on inclinometer measurements performed in permafrost.

7.4.1.3 Creep slopes

Creep movement fluctuating near the surface and creep varying in magnitude over the area is problematic for avalanche defense structures. By contrast, creep slopes with slip plains at greater depth (greater than 10 m) having a uniform rate of creep are less problematic for avalanche defense structures.

Tab. 13 > Creep rates and appropriate protective measures.

Ground conditions at site	Estimated creep rate	Measures
Stable		
(mainly in rock)	0 cm/a	• Structures according to Section 7.4.2
Slight creep		
(ice-rich loose ground, fairly stable slope debris)	0.5–2 cm/a	• Structures according to Section 7.4.3: – additional expenditure for maintenance probable
Moderate creep		
(ice-rich loose ground, unstable slope debris)	2–5 cm/a	• Structures according to Section 7.4.3: – erection of supporting structures only permitted at high cost effectiveness – additional expenditure for maintenance necessary, since damage must be expected in the middle term – reduced service life of the supporting structures must be expected – alternatives to supporting structures in the starting zone to be considered
Heavy creep		
(ice-rich loose ground, block glaciers, very unstable slope debris)	> 5 cm/a	• Erection of supporting structures only permitted where no alternatives are available: – middle- and long-term service life not assured – alternative measures to supporting structures in the starting zone must be taken

7.4.1.4 Project planning

During project planning, consideration should be given to tolerable creep rates and to whether or not the project is cost effective.

7.4.2 Defense measures in permafrost rock without creep movement

Under stable rock conditions with no creep, all of the protective systems appropriate to the site as given in the “Type Approval List for Avalanche Supporting Structures” (see Section 8.9) may be used. The quality requirements for anchor grout are given in Section 6.3.

7.4.3 Defense measures in permafrost loose ground with tolerable creep movement

7.4.3.1 Flexible supporting structures

The securing of starting zones in slopes with tolerable creep movement as shown in Tab. 13 must be by means of flexible structures, e.g. with snow nets. Snow nets are less sensitive to slight creep movement than rigid snow bridges. Snow nets can be readjusted following displacement of the upslope retention and downslope guys. The types of structure permissible in permafrost are specified by the FOEN in a type approval list (see Section 8.9).

7.4.3.2 Rigid supporting structures

Rigid supporting structures are not permitted, since with these, creep movements would lead to distortion and excessive loads.

7.4.3.3 Dimensioning of the supporting structures

The snow pressure on the supporting structures is the same as in areas without permafrost. The dimensioning of the superstructure to resist snow pressure can therefore be performed as given in Section 5. Experience shows that snow glide is only slight in permafrost areas (low solar irradiation, ground frequently boulder-filled).

7.4.3.4 Foundations

7.4.3.4.1 General

The foundations of the swivel supports of snow nets should be provided with micropiles with steel pipes (i.e. *not* anchor bars) or with steel ground plates.

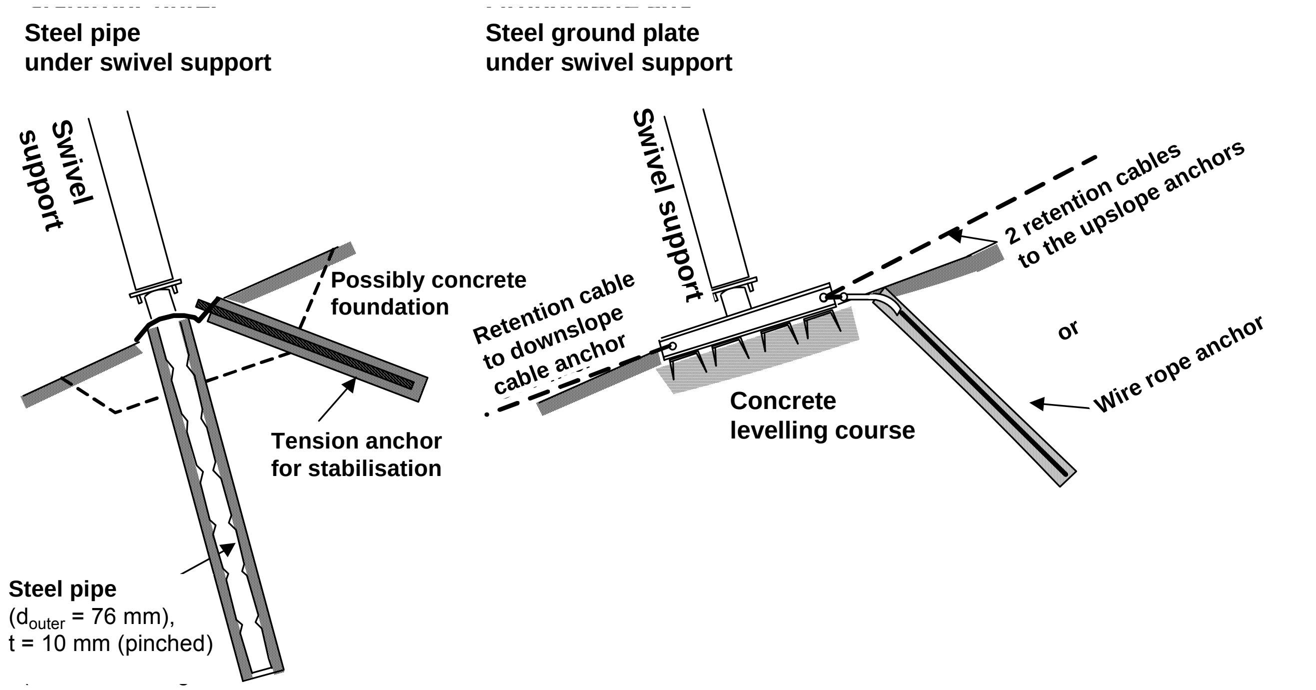
7.4.3.4.2 Micropiles

Micropiles must be designed as stiff steel pipes (Fig. 50) (diameter d_{outer} = 76 mm, wall thickness = 10 mm, steel quality S. 355). To ensure bonding between the surface of the pile and the grout, the steel pipe must be profiled (e.g. using mechanical pinching). The transverse force that may arise from slight tilting of the swivel supports is taken up by this stiff steel pipe. This must be retained at its upper end by an additional tension anchor. In loose slope debris, the use of additional concrete foundations at the upper end on the micropile should be considered.

7.4.3.4.3 Ground plate

The steel ground plate (Fig. 50) is relatively insensitive to tilting of the swivel support. The geometry of the snow nets can be readily readjusted. The plate should be imbedded in a concrete leveling course and either fixed with retaining ropes attached to the upslope and downslope cable anchors, or with a wire rope anchor in the terrain. In calculating the internal forces at the base of the support, not only the transverse force should be considered according to Section 5.5.4, but also an exceptional (upslope) tilting of the support relative to the line of slope by 10°. The resultant transverse force must amount to a minimum of 20% of the maximum support force. Soft ground layers near the surface must be removed in advance. The area of the ground plate must be dimensioned as a function of the total ground resistance (see Section 5.9.4.6).

Fig. 50 > Swivel support foundations under creep conditions in permafrost.



7.4.3.4.4 Deflection forces with upslope wire rope anchors

With upslope wire rope anchors in very loose slope debris that must sustain high loads, the deflection forces must be transferred to the ground via reinforced concrete foundations (Fig. 41). The total ground resistance is determined according to Sections 5.9.4.4 and 5.9.4.6.

7.4.3.4.5 Downslope wire rope anchors

The downslope anchoring can be done conventionally with wire rope anchors.

7.4.4 Protective measures on slopes with non-tolerable creep movement

7.4.4.1 Non-allowed protecting structures

In slopes with non-tolerable creep movement according to Tab. 13, non-allowable deformation and damage to the defense structures (including flexible snow nets), occur in the short to middle term. Stabilization of the slope by structural measures is not possible. In slopes of this nature, protection by means of supporting structures is not allowed, since owing to the heavily reduced service life they are not cost effective.

7.4.4.2 Alternative measures

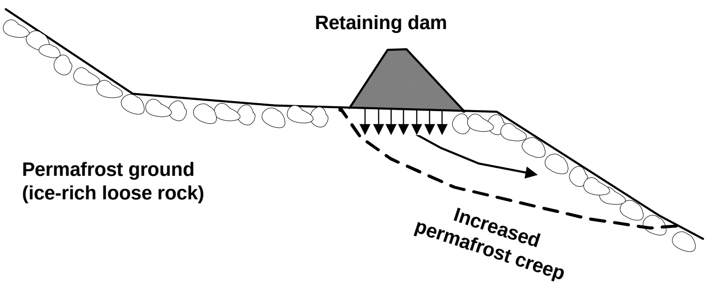
The endeavor should be made to achieve avalanche protection by alternative measures such as: zonal planning, deflecting structure, retaining dam, evacuation, etc.

7.4.5 Dam in permafrost loose ground

7.4.5.1 Ice-rich loose ground

In ice-rich loose ground (including active block glaciers), dams may not be built, since the high dead load of the viscous ice/debris mixture can cause increased permafrost creep in the ground (Fig. 51). By virtue of the increase in stress in the ground, slope creep can progressively increase.

Fig. 51 > Dam in permafrost ground (loose ground).



7.4.5.2 Low-ice loose ground

Retention dams are permitted in low-ice loose ground ground that does not have a tendency to creep. Using appropriate geotechnical stability test methods, it must be shown that the overall stability of the ground is not reduced by the dam beyond the extent permitted. When frozen, loose ground has a reduced angle of friction and markedly higher cohesion.

7.5 Anchoring in permafrost (loose ground or bedrock)

7.5.1 Drilling method

7.5.1.1 Removal of drilled material

In permafrost, difficulty can arise with the removal of drilled material using air flushing. The use of a down the hole hammer leads to the frozen loose ground containing ice being mashed to a thick and sticky mass. When drilling under these conditions, frequent retraction of the drill saddle and flushing of the borehole is necessary. Under very unfavorable conditions (extremely high ice-content of the ground), drilling with the down the hole hammer can prove impossible.

7.5.1.2 Casing drilling

In the (mostly) loose slope debris near the surface, it can be advantageous to drill the top few meters using a casing (casing drilling). Where the stability of the loose ground is insufficient, the use of injection drilled anchors (combined drilled and injection anchors) should be considered.

7.5.1.3 Drilling ability

The ability to drill the ground can be assessed during exploratory drilling (Section 7.2.2) and the necessary measures planned.

7.5.2 Anchor forces in rock

Rock anchors can be installed as given in Section 5.9.7.2. Initial dimensioning can be done as shown in Tab. 11, and final dimensioning according to Section 7.5.4 based on anchor tests.

7.5.3 Anchor forces in ice-deficient loose ground

In slopes with potential avalanche starting zones (inclination > 28°), permafrost that is either dry or contains undersaturated ice, is probable, since slope water may flow off and ice does not accumulate in the ground. Using prior exploratory drillings, the occurrence of ice in the ground can be roughly estimated (Section 7.2.2.7). With dry loose ground, or loose ground undersaturated with ice, the anchors can be predimensioned according to Fig. 42. To assess the ground conditions, the characteristics and compaction density of the ground can be assessed visually as given in Section 5.9.7.4.

7.5.4 Anchor tests

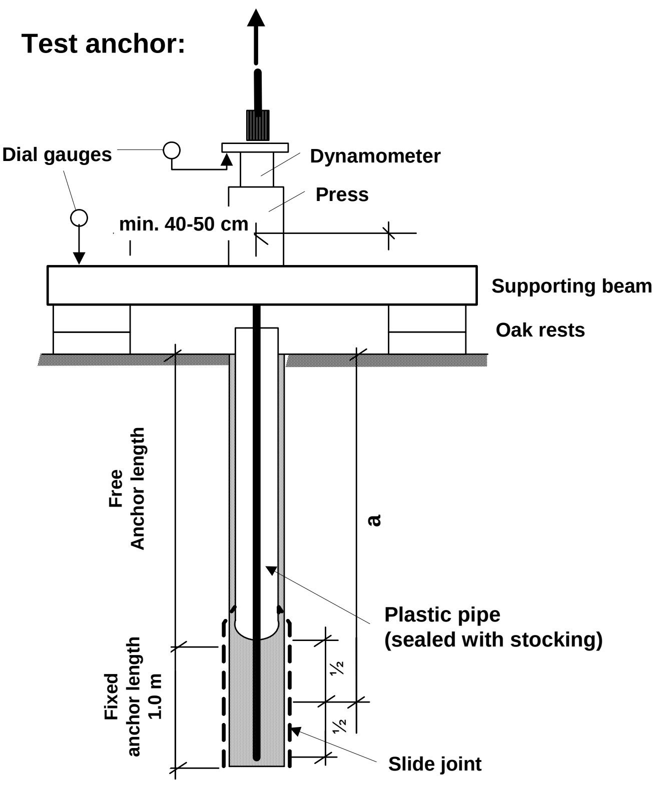
7.5.4.1 General

In the terrain, it is very difficult to distinguish between moderate, poor and very loose ground conditions. To finally determine the anchor lengths, pull-out tests (Fig. 52) must be performed. These tests should preferably be carried out in conjunction with the drillings made to investigate permafrost (Section 7.2.2). A minimum of 3–5 test anchors should be installed per hectare construction site area. Where the ground conditions within the construction site are very inhomogeneous, the number should be increased accordingly.

7.5.4.2 Performance of pull-out tests

The pull-out resistance tests can be performed on the basis of SIA 267/1. The determination of the outer load-bearing capacity of the anchor $R_{a,k}$ (breaking load) is based on SIA 267/1 using the semi-logarithmic creep diagram with a creep index of $k_{krit} = 2.0$ mm. The following formulae are used to determine the necessary anchor lengths (for tension) and pile lengths (for pressure). The upper meter of the anchor is assumed to carry zero load. The pull-out test shown in Fig. 52 gives the specific pull-out resistance $R_{a,k}'$ for a fixed anchor length of 1.0 m.

Fig. 52 > Test arrangement for the performance of pull-out tests.



7.5.4.3 Dimensioning of anchors

The total anchor length necessary to withstand the effective characteristic **anchor forces** (resulting from snow pressure) is calculated for homogeneous ground using the following formula (50):

$$\boxed{L = \sqrt{\frac{F_k \cdot 4 \cdot a}{R_{a,k}'} + 1}} \qquad (50)$$

where:
L: total required anchor length [m]
F_k: characteristic value of the tension or compression force (effective anchor force resulting from snow pressure without load coefficient) [kN]
a: average depth of the anchor (Fig. 52) in anchor test [m]
$R_{a,k}'$: specific pull-out resistance for a fixed anchor length of 1.0 m. (Fig. 52)

7.5.4.4 Dimensioning of micropiles

The load-bearing capacity of a micropile under pressure is 50% higher than under tension (for anchors) (Section 5.9.7.6.5). The total required anchor or pile length necessary to withstand the effective anchor forces resulting from snow pressure is calculated analogously to equation (50) (F_k = characteristic value of the compressive force in [kN], without load coefficient).

7.5.4.5 Stratified ground conditions

For the tension test in stratified ground conditions, the anchors should be placed in single strata. The pull-out resistance is calculated from the sum of the specific pull-out resistances measured in the individual strata.

7.5.4.6 Quality control

Proper injection of the anchors must be monitored by the works supervisor. If during installation it is suspected that the load-bearing capacity of the anchors is insufficient, this may be checked using tension tests. In the tension test, a force equal to 1.35 times the effective anchor force F_k is applied to the anchor. The anchor grout should be removed over a length of 0.5 m below the upper end of the anchor to prevent the transfer of force between the anchor and the abutment structure. The creep index should not exceed the value of k_{adm} = 1.0 mm.

7.6 Use of grout in permafrost

7.6.1 Special grout in permafrost

The quality of the grout must accord with the requirements of Section 6.3. Normal anchor grout is not permissible. Special grouts approved under permafrost conditions (i.e. for ground or rock with a temperature below 0 °C) are recorded by the FOEN in a type approval list (cf. Section 8.9). These can be applied down to a temperature of –4 °C in the ground. In summer, lower permafrost temperatures seldom occur in the region of the anchor. The temperatures in the ground are measured during exploratory drilling (Section 7.2.2).

7.6.2 Preheating and injection

To ensure the onset of setting, these grouts must be heated to 20 °C before filling. The required grout temperature of 20 °C can be achieved simply by heating the water prior to mixing the grout. The following Tab. 14 (assumed mixing ratio water/dry grout = 0.18) shows the necessary water temperatures as a function of the temperature of the dry grout:

Tab. 14 > Required water temperatures as a function of the temperature of the dry grout.

Original temperature of the dry grout prior to mixing	Preheating of the water to
0 °C	44 °C
4 °C	39 °C
8 °C	34 °C
12 °C	29 °C
16 °C	24 °C
20 °C	20 °C

7.6.3 Methods for preheating and temperature measurement

The water may be preheated on site using a gas heater (can contain a built-in thermostat) or, more simply, using a steel vessel in which the water is preheated by gas. The temperature of the mixed grout must be checked during the injection process. The temperatures of the water, the dry grout and mixed grout can be measured using a simple hand temperature appliance. The minimum temperature of the mixed grout is 20 °C. To ensure that the grout does not set too quickly, the maximum temperature should not exceed 30 °C.

7.6.4 Injection

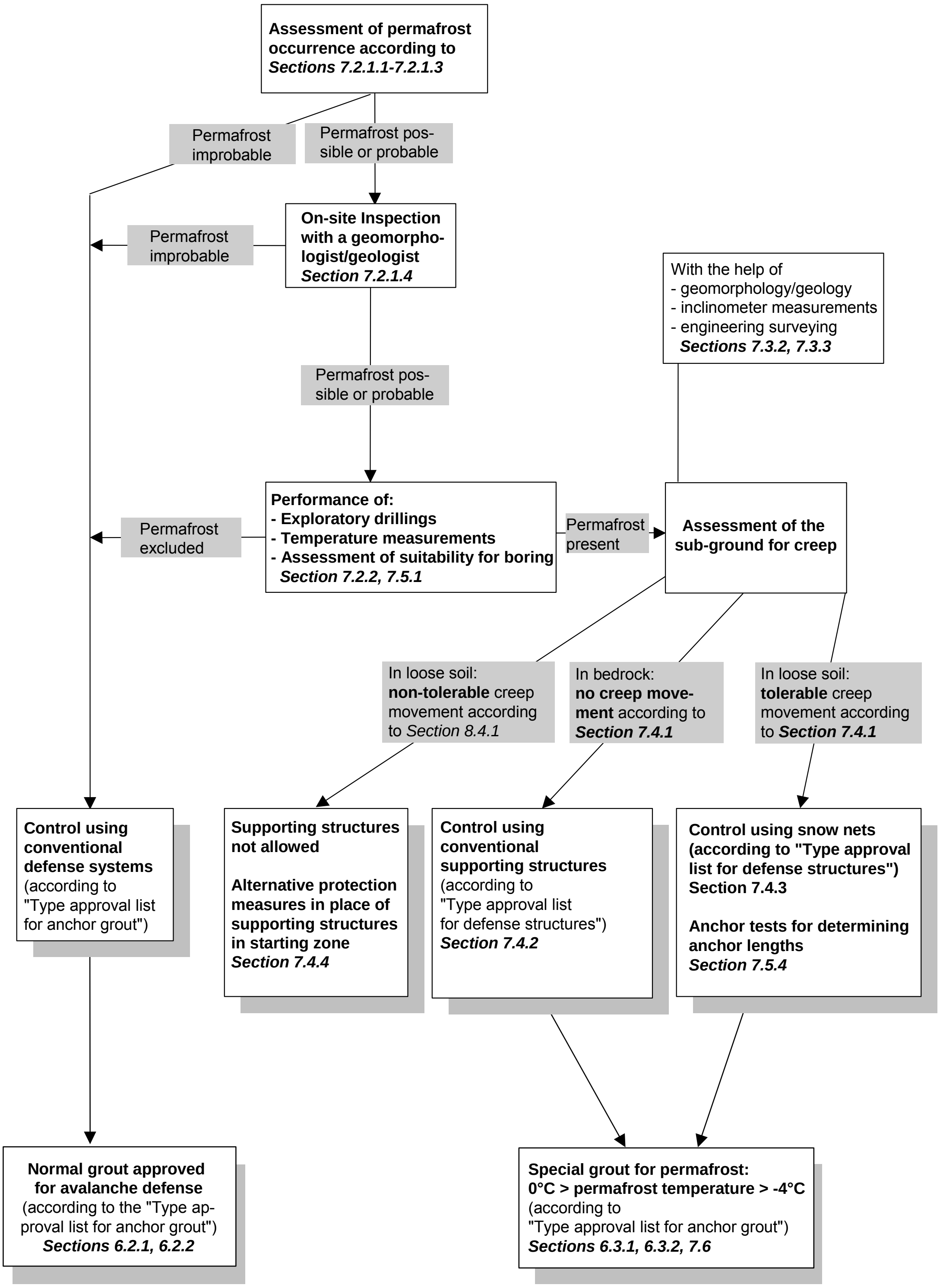
Installation and injection of the anchors and piles should be done as soon as possible following boring, and at the latest on the same day, to avoid blockage of the boreholes through freezing of water entering the borehole.

7.7 Maintenance

Defense structures in permafrost must be inspected as specified in Section 5.4.2. When necessary, snow nets must be readjusted.

7.8 Flow diagram: planning procedure

Fig. 53 > Flow diagram.



8 > Type approval

8.1 Purpose

Where federal subsidies are required for avalanche defense structures according to art. 36 WaG, these must comprise officially tested and approved supporting structures and anchor grouts. This section regulates the procedures for testing and approval. The following general objectives are pursued:

> specification of the type approval procedures
> specification of the requirements for supporting structures, their foundations and anchor grouts
> fulfill the conditions for the subsidization of supporting structures, their foundations, and anchor grouts, as required for avalanche supporting structures subsidized by the Confederation

8.2 Test objects

8.2.1 Supporting structures

To be tested are permanent standard structures and the associated foundations subsidized by the public authorities. Standard structures must be dimensioned under the following site factors:

> slope inclination $\psi = 45°$
> glide factor N = 1.8 or 2.5
> altitude factor $f_c = 1.1$
> effective grate height $D_K = 2.0/2.5/3.0/3.5/4.0/4.5/5.0$ m
> lateral distance between structures A = 2.0 m

Where higher values of the site factors occur within the construction site (e.g. inclination > 45° or N > 2.5), suitably dimensioned supporting structures (special structures) should be used.

The type approval procedure does not apply to temporary supporting structures or to special structures.

8.2.2 Anchor grout

Tests are carried out on anchor grout used in avalanche defense structures subsidized by the Confederation.

8.3 Administrative procedure

Administration of the type approval procedure is performed by the FOEN. The manufacturer submits the application for supporting structures and anchor grouts to the FOEN, which has the tests performed and issues the approval statement. The type approval procedure is carried out by the SLF and the Expert Commission for Avalanches and Rockfall (EKLS). The results and observations made during the tests are recorded by the SLF and presented in a report as follows.

Tab. 15 > Type approval procedures for supporting structures.

Responsible body/participants		Work steps
1	Manufacturer; supplier	Application and documentation to FOEN.
2	FOEN	Registration, confirmation of receipt, cost estimate to applicant.
3	SLF	Inspection of static calculations and planning documents and report to applicant and FOEN.
4	Manufacturer; supplier	Checked and revised planning documentation to FOEN (11 copies).
5	EKLS	Practical utilization test. Report to applicant and FOEN with overall assessment and recommendation on approval (yes/no).
6	FOEN	Approval statement. Entry in the type approval list for avalanche structures.

Item 1: The manufacturer or supplier of a particular structure type sends an application for type approval to the FOEN. At the same time, he/she provides the documentation according to Section 8.6.1.
Item 2: The FOEN records the application and documentation, and informs the manufacturer on further procedure and deadlines in consultation with the SLF.
Item 3: The SLF inspects the static calculations and planning documentation. Any shortcomings identified are recorded in a test report. The corrected documentation is submitted to the SLF for renewed inspection.
Item 5: Following inspection of the planning documentation, the EKLS informs the applicant whether, and in what way, the test should be performed in the terrain. Finally, an overall assessment is made that also contains the recommendation of the FOEN on approval (yes/no).
Item 6: The FOEN decides whether to approve the construction type. The FOEN maintains a type approval list with all approved structure types.

Tab. 16 > Type approval procedure for anchor grout.

Responsible body/participants		Work steps
1	Manufacturer; supplier	Have suitability test done on anchor grout. Application to FOEN with report on suitability test.
2	FOEN	Registration, confirmation of receipt.
3	SLF	Inspection of the report on the suitability test. Test report.
4	EKLS	Practical utilization test. Test report with overall assessment and recommendation on approval (yes/no).
5	FOEN	Approval statement. Entry in the type approval list for anchor grout.

Item 1: For this, the manufacturer or supplier sends 2 sacks of dry grout to the EMPA, or other neutral laboratory, to enable them to perform the suitability test. The sacks of grout must be marked as specified in Section 8.6.2.
Item 2: The FOEN records the application and documentation, and passes the test report on to the SLF.
Item 5: The FOEN decides on approval of the anchor grout. The FOEN holds a type approval list with all approved anchor grouts.

8.4 Requirements

8.4.1 Supporting structures

The dimensioning of the supporting structures must accord with the technical guideline for defense structures in avalanche starting zones, and the corresponding SIA standards and Eurocodes.

8.4.2 Anchor grout

The properties of an anchor grout must be verified in a suitability test. The limiting values to be fulfilled are given in Section 6: Use of anchor grout in avalanche defense.

8.5 Inspections

8.5.1 Supporting structures

8.5.1.1 Inspection of the static calculations

An inspection is made of the assumptions on snow pressure and the analytical load models concerned, the dimensioning and geometry of the supporting surface, the dimensioning of the supporting structure (support, pressure bar, girder, connections), the relevant support forces and the dimensioning of the foundations, and the corrosion protection of the foundations. Further, the suitability from the point of view of snow and avalanche mechanics is inspected. The following points are assessed:

- loads as specified in the technical guideline for defense structures in avalanche starting zones (2006)
- analytical model of the supporting structure/static system (recommendations for use)
- calculation of the internal and reaction forces
- dimensioning of the relevant steel components according to SIA 263: Steel Construction, or Eurocode 3: Steel Construction
- rigid supporting structures: crossbeams, supports, girders, pressure bars, connections (web plates, studs, bolts, load transfer and welded connections), anchors/micropile diameters, ground plates (profiles, load transfer)
- flexible supporting structures: supports, pins (bearing and shear resistance), base of support, net cables, peripheral net cables, guys, connecting cables, edge reinforcing cables, support anchorage, wire rope anchors/anchorage
- details of corrosion protection of the anchorage
- materials used (mechanical properties, quality group; for materials not contained in SIA 263 or Eurocode 3, appropriate materials certificates must be submitted)
- wire ropes used (design, mechanical properties of the wires, minimum breaking strength, existing certificates)

8.5.1.2 Inspection of practical utilization

a) Inspection of the plans:
The following are to be inspected among other things: the numbers and weights of the components, erection of the supporting structures, the type of foundations, the adaptability of the structure to the terrain, the type of materials used, the vulnerability to rockfall, the installation tolerances, the control and repair modes, adaptation to the landscape and economic viability.

b) Inspection in the terrain:
- With new types of supporting structures, test structures are erected. The viability of the supporting structure must be demonstrated during at least 2 winters (or longer with mild winters) at a site specified by the EKLS.
- With structural changes on approved structure types, an installation test is normally required. The positioning and erection of the structure is inspected by the EKLS.
- For minor changes (e.g. of the effective grate height specified in Section 8.2.1), no test is made in the terrain.

8.5.2 Anchor grout

The suitability test for anchor grout is carried out by the EMPA, or other neutral test institute, and must be applied for by the applicant. The test procedure is described in Section 6: Use of anchor grout in avalanche defense. The SLF and the EKLS assess the report on the suitability test based on the limiting values in Section 6.

8.6 Required documentation

8.6.1 Supporting structures

To initiate the type approval procedure, the following documents must be submitted to the FOEN (Address: FOEN, Department of Risk Prevention, 3003 Bern).

a) Basic documentation
Test application with:
- name and address of the applicant
- test object with site factors
- list of the planning documents submitted and the static calculations
- place, date and signature

b) Static calculations
- file containing the static calculations with details of the author, date and test object with site factors
- content of static calculations: all calculations and verification procedures must be clearly presented (formulae, input values, intermediate and final results). Calculations containing simply the final results are not acceptable.

c) General drawing of the assembled structure
- drawing with number, date, type, manufacturer and any changes
- recommended scales 1:25–1:20
- details of any existing foundation variants
- details of maximum foundation forces

d) Design drawings of the individual components (where necessary)
- individual drawings with number, date, type, manufacturer and any changes
- recommended scales 1:2 to 1:20
- specification of the dimensions of the relevant components such as girders, support, connecting elements, fasteners, etc.

e) Material list
- material list of the relevant components with profile specification, dimensions and weight

f) Installation handbook
- details for peg out
- required auxiliary equipment and tools
- stepwise assembly instructions
- check list for final inspection of the installation
- check list of maintenance work

g) Further details (where necessary)
- details of quality management in the company
- existing certificates

h) Documentation to be submitted
- submission of the application forms, the static calculations and the planning documentation (2 copies) to FOEN, Department of Risk Prevention, CH-3003 Bern
- following positive inspection of the static calculations and the planning documentation by the SLF, the revised planning documentation to be submitted to the FOEN, Department of Risk Prevention, CH-3003 Bern (11 copies).

8.6.2 Anchor grout

The report on the suitability test must contain the following assessments:

> properties of fresh grout according to the EMPA method
> compressive strength
> elasticity measurement
> shrinkage
> frost resistance

The following information should be marked on the grout sacks:

> product name
> details of supplier
> non-coded production date
> non-coded expiry date
> production number
> storage conditions

8.7 Costs of inspection

8.7.1 Inspection of the static calculations

The costs for inspection of the static calculations are normally based on work times.

8.7.2 Suitability test for anchor grout

The cost of the suitability test must be borne by the applicant.

8.7.3 Test of practical suitability

The costs of the test structure are borne by the supplying company. The installation and assembly costs are charged to the respective defense project. If the test structure proves to be suitable, the material costs are refunded to the supply company. The total costs of a test structure must not be higher than those of a regular structure in the same defense project.

8.8 Validity of the test

> the validity of the inspection of supporting structures is unlimited
> the suitability test for anchor grout must be renewed every 3 years
> notwithstanding this, should deficiencies arise in the supporting structures or anchor grout following their approval, and/or if they no longer accord with the current status of technology and experience, the test must be repeated.

8.9 Type approval list

The approved supporting structures (excluding those for higher site factors) and anchor grouts are entered in type approval lists by the FOEN. These lists are constantly updated and may be consulted on the FOEN website *(www.umwelt-schweiz.ch/typen-pruefung).*

8.10 Confidentiality and disclosure to third parties

All documentation submitted to the FOEN, the SLF and the EKLS will be treated as confidential. With the exception of the type approval list, the results will only be disclosed to third parties with the written consent of the applicant. The Federal Department of Environment, Transport, Energy and Communications will decide on questions of publication by the EKLS.

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