ABNORMAL LOADING ON STRUCTURES

EXPERIMENTAL AND NUMERICAL MODELLING

EDITED BY K. S. VIRDI, R. S. MATTHEWS,
J. L. CLARKE AND F. K. GARAS

Taylor & Francis
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Experimental and numerical modelling

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Due to unforeseen circumstances the *Abnormal Loading on Structures* conference did not take place. However, it is hoped that this collection of papers will be a useful and authoratitive guide to state-of-the-art research in this key area of structural engineering.

First published 2000 by Taylor & Francis
2 Park Square, Milton Park, Abingdon, Oxon, OX14 4RN

Simultaneously published in the USA and Canada
by Taylor & Francis
270 Madison Ave, New York NY 10016

Taylor & Francis *is an imprint of the Taylor & Francis Group*

Transferred to Digital Printing 2009

Publisher's Note

This book has been prepared from camera-ready copy provided by the authors.

British Library Cataloguing in Publication Data
A catalogue record for this book is available from the British Library

Library of Congress Cataloging in Publication Data

A catalogue record for this book has been requested

ISBN 0-419-25960-0

Publisher's Note
The publisher has gone to great lengths to ensure the quality of this reprint but points out that some imperfections in the original may be apparent.

CONTENTS

INSTITUTION OF STRUCTURAL ENGINEERS' INFORMAL STUDY GROUP 'MODEL ANALYSIS AS A DESIGN TOOL'

The Group, which was formed in February 1977, operates under the auspices of the Institution of Structural Engineers and presently membership stands at over 500 covering some 40 different countries. Members come from a wide range of backgrounds including: research, design, engineering and contracting organisations, universities, government departments, local authorities and utility companies.

The primary objective of the Group is to create opportunities for members of the Institution and the profession to exchange information on the use of testing and model analysis to solve design problems. The scope of the Group encompasses the whole spectrum of structural engineering applications including: conventional structures, bridges, foundations, pressure vessels, offshore, harbour and coastal structures etc. It is intended to cover structures made of a wide range of materials and subjected to different loading conditions.

The Group's activities comprise the publication of a quarterly newsletter, organising international conferences, visits to test centres in the UK and Europe, sponsoring specialist lectures and holding an annual prize award competition for student dissertations on the application of physical modelling and testing in design.

Further information about the Group may be obtained from the Convenor, Prof F K Garas, FIStructE, 6 Amersham Gardens, High Wycombe, HP13 6QP, UK.

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PREFACE

Designing for hazardous and abnormal loads has become an important, even essential, requirement in the design process of most major civil engineering structures. The range of these structures includes tall buildings, bridges, conventional and nuclear power plants, chemical and other processing plants, oil and gas platforms, and harbour and coastal installations.

Hazard identification and risk assessment are of crucial importance when defining the extreme loading which a structure should be able to withstand. They are also vital elements in the establishment of appropriate protection for the user and the population at large in the event of a failure. The complementary requirement to maintain the cost of the structure at an acceptable level is, likewise, served by a clear understanding of its prospective service conditions.

Physical testing of full scale elements and structures combined with analytical modelling has played a significant role, over the years, in the study of their behaviour when subjected to hazardous loads. The experimental and numerical techniques have enabled designers to solve difficult engineering problems and thus improve the standards of design, safety, construction and in-service performance. In dealing with hazardous and abnormal loads, many problems are faced in the design, in undertaking and interpreting relevant experiment resultss, for example, establishing similitude requirements for material properties and behaviour, modelling the effect of time-varying loads and determination of natural frequency and proper boundary conditions.

Thirty four papers are reproduced in this book, whish were presented at the International Conference entitled 'Abnormal Loading on Structures - Experimental and Numerical Modeling', held at City University, London on 17 - 19 April 2000. The aim of the conference was to provide a forum for discussion and exchange of information on the relevant experience in the design and construction of structures subjected to abnormal loading, using physical tests and numerical modelling.

The following subjects are covered in the papers included here: loading on structures, including accidental loading, earthquakes and fire, effects of the methods of construction on structural response to abnormal loading, development of new materials and construction techniques, behaviour of engineering materials, inspection, monitoring, repair, and rehabilitation of structures, non-destructive testing for monitoring and assessment, structural safety and risk analysis, numerical simulation and modelling, and case studies of on-site testing of structures

The papers represent a state-of-the-art examination of structural design for hazardous and abnormal loads.

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ACKNOWLEDGEMENTS

The editors gratefully acknowledge the work of all the authors, and of the members of the Scientific and Organising Committees.

AVALANCHE SCENARIOS FOR ALPINE HIGHWAY BRIDGES
Avalanches on Alpine bridges

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Abstract
After the explanation of some basics of avalanches the calculation of velocities, flow depths and debris zones is shown. An example with a bridge of the St. Gotthard highway demonstrates how hazard scenarios are established and refined following actual events and what actions result from these premises.
Keywords: Avalanche, debris, dense flow, dry snow, friction, hazard scenario, highway bridge, pier, roughness, superstructure.

1 Dealing with avalanches

Avalanches have always threatened inhabitants and travellers in the Alps. Research on avalanches started in Switzerland in 1931 with the establishment of the Federal Committee on Snow and Avalanches in Bern. A further milestone was the foundation of the Swiss Federal Institute for Snow and Avalanche Research on the Weissfluhjoch at Davos in 1942.

Mountaineers fear most avalanches they trigger themselves. They have to judge the local situation, to act responsibly and sometimes to forego an intended tour. Research institutions like the one mentioned above provide a daily avalanche bulletin that judges the regional situation as accurately as possible.

For inhabitants of Alpine regions the situation is different. They are most threatened by avalanches running down to the valley floors. Avalanches may be triggered artificially in order to determine their time of run down and to prevent further accumulation of snow. More effective, however, are preventive measures like growing forests on the steep valley flanks and avoiding housing in endangered zones. The attempt to influence the flow of an avalanche by walls and dams is limited to some

Abnormal Loading on Structures edited by K. S. Virdi, R. S. Matthews, J. L. Clarke and F. K. Garas. Published in 2000 by E & FN Spon, 11 New Fetter Lane, London EC4P 4EE, UK. ISBN 0 419 25960 0

favourable cases. Traffic routes often cannot avoid endangered zones and have to cross them through galleries or on bridges. These structures are designed to withstand the actions from avalanches.

1.1 Types of avalanches

Avalanches can be divided up into two types that behave in a different way and have different properties and characteristics (Table 1). Mixed type avalanches can also be observed.

Dense flow avalanches that consist of dry or wet snow have a granular structure and move similar to sand or gravel downhill. Their velocity depends above all on the inclination and the roughness of the terrain. They follow canyons and valleys and can be modelled as granular fluids.

New snow that has not yet consolidated can have entrained air once getting in motion. It forms a *dry snow avalanche*, like a gravity stream in water or a dispersion stream of a heavy gas. The entrainment of air and the erosion of snow form mechanisms that increase its mass with time. Such processes accelerate the avalanche and even more snow is dragged down. Dry snow avalanches move in the direction of the steepest slope and the friction to the ground matters in the run-out only. They are experimentally modelled as brine or suspensions in water or treated as two-phase flows.

Table 1. Characteristics of dense flow and dry snow avalanches (from [1])

	Unit	Dense flow avalanche	Dry snow avalanche
Typical velocity	[m/s]	30-60	50-100
Flow height	[m]	< 2-5	50-100
Density	[kg/m^3]	100-300	5

1.2 Partition of the avalanche path

Along the track of an avalanche three different parts can be assigned.

The *starting zone* includes all parts with an inclination between about 28 and 50 degrees, where snow packs can become unstable.

The *track* is the path formed by natural or artificial flanks that lead the avalanche downhill. Along the track the underlying snow can be captured thus increasing the volume in motion. Depending on the other relevant conditions the avalanche front speed may increase as well as decrease.

Where the inclination falls under a critical value given by the friction coefficient, the snow mass slows down and the *run-out* or *debris zone* begins.

1.3 Predictions and calculations

It is not the intention to give a complete manual for the calculations of avalanches. It shall be demonstrated, that with some few assumptions and estimations reasonable results can be achieved.

1.3.1 Dense flow avalanches

The model used for dense flow avalanches is based on the works of Voellmy [2] and Salm *et al* [3]. They assumed that the friction of the underlying snow can be described by a dry friction, linearly increasing with weight of the flowing snow layer and a dynamic drag proportional to the square of the avalanche velocity applying Eqn. 1.

$$v_0 = \sqrt{d_0 \xi (\sin\psi - \mu\cos\psi)} \qquad (1)$$

The values needed are the roughness parameter ξ describing turbulent friction, a dry friction parameter μ denominating substratum friction and the angle of inclination ψ. Details, examples and all other formulas are given in [3]. In the meantime, more sophisticated models ([1], [4]) are available that have not greatly influenced practical work until now.

The size of the formation zone can easily be determined. More experience is needed to establish the depth d_0 of the snow pack that loses stability and influences the snow mass being involved. It depends on a basic value d_0^* denominating the possible increment of snow depth within three days and the inclination ψ. Values of d_0^* are available for various regions and return periods of 30, 100 and 300 years. Snowdrifts can increase d_0^* by 0.5 m.

The maximum flow Q can be estimated by multiplying d_0 with the maximum width of the formation zone B_0 and a representative velocity v_0 for a rectangular size (Eqn. 2), or by dividing the total snow volume K by the flow time Δt for any shape of the formation zone (Eqn. 3). In both cases a velocity is needed, that can be calculated, using Eqn. 1.

$$Q = B_0 d_0 v_0 \qquad (2)$$

$$Q = K/\Delta t \qquad \Delta t = l/v_m \qquad (3)$$

Knowing the flow Q, the velocity v can be calculated in all sections along the track, either by estimating the width B of the avalanche for flat slopes or by taking into account a hydraulic radius R for channelled avalanches. Again ξ, μ and ψ are needed. Changes of the inclination ψ cause acceleration or retardation respectively, determining the distance until steady conditions apply again.

The starting point of the run-out can easily be found when the inclination decreases suddenly beneath the critical value $\psi_k = \arctan\mu$ and has to be iterated in other cases. The calculation of the length of the run-out takes into account energy considerations only. Although the proposed values for ξ and μ vary over a range of factor two and depend on numerous parameters, the calculated run-out lengths agree well with observations.

1.3.2 Dry flow avalanches

Dry flow avalanches are more complicated to calculate and the observation of historic events helps to determine the endangered zones. The air pressures produced may be predominant more for facades and roofs of buildings than for engineering structures.

2 Structures and avalanches

In the formation zone appropriate structures are used to prevent snow packs from becoming unstable. They are not treated here.

Run-out zones are calculated with the given rules and by taking into account the local experience of the whole observation period. They are the basics for zones of forbidden or restricted housing, depending on the return period.

Structures forming part of the infrastructure, like roads and railway lines, normally cross the track of the avalanche or the run-out zone. Galleries are built to let avalanches run over them and are supposed to function even when the avalanche is occurring. Bridges should be built high enough to allow dense flow avalanches to pass underneath and withstand dry flow avalanches undamaged. Regarding their height, it is not possible to protect users on the bridge and a well-timed closure is required. Should a bridge always be in operation, it has to be constructed as a tunnel bridge.

3 Actions due to avalanches

The following three types of structures can be identified:

- Large obstacles like averting dams and walls that are loaded by dynamic pressures and friction forces. These are not treated further.
- Supporting structures like galleries that are subject to hydrostatic pressure, dynamic friction and possibly deviating forces during the rundown and carry the remaining snowpack and possible creep forces after the event.
- Small obstacles compared to the cross section of the avalanche like piers and girders of bridges that are subject to dynamic pressure and friction during the run down and may remain covered with debris of snow afterwards.

All actions are normally taken as static loads, neglecting dynamic amplification factors. Large blocks or logs may cause impact loads that exceed static loads by a factor of two. Avalanches with a return period of 30 years are regarded as an ordinary hazard scenario, i. e. the normal load factors apply. Avalanches with a return period of 100 or even 300 years are regarded as accidental events, load factors are reduced to 1.0 and accompanying actions are taken into account with a load factor ψ_{acc} which is equal to zero in most cases [5].

3.1 Actions on avalanche galleries

A guideline, issued by a working group of the Swiss Federal Highway Authority and the Swiss Federal Railways [6], establishes the procedure for determining the actions on avalanche galleries. An avalanche specialist has to specify flow depth, flow width and velocity of the design avalanche in a cross section up to 100 m above the railway or road to be protected. This enables the designer to choose the most favourable position and length of the gallery and to calculate the actions to be taken into account.

3.2 Actions on bridges

For bridges the avalanche specialist normally specifies directly the actions that have to be taken into account by the designer of a new bridge, or by the assessor of an existing bridge. An example is given in Section 5.

4 The St. Gotthard highway

The Northern access ramp of the St. Gotthard highway tunnel crosses the Swiss Canton of Uri over a length of 47 km and rises from 450 m to 1060 m above sea level. Due to the narrow valley the highway is carried by bridges or passes through tunnels over large stretches. The adjacent mountains rise to 3200 m above sea level.

4.1 The rehabilitation project

The highway was commissioned between 1971 and 1980. The first bridges, however, were begun in 1963 to serve as access to the remote construction sites of the various tunnels. In the meantime a growing use of deicing salt, increased traffic and axial loads as well as alpine accidents such as avalanches, rockfalls, mud flows and floods have considerably damaged the highway and especially the bridges. In 1990 a preservation programme was started to remove deficiencies and repair the damage of the first 25 years and to retrofit the structures for at least another 50 years.

The rehabilitation project prepared in 1998 covers the highest part of the highway called Group 4 from the Wassen exit to the entrance of the St. Gotthard tunnel. The knowledge gained until then on frequency, extent and characteristics of avalanches was taken into account, leading to design actions on piers and superstructures of bridges. Check calculations showed that all bridges could withstand the expected actions with a return period of 100 years.

5 Case study Reuss bridges Wattingen

The most interesting situation is now described in more detail based on the technical reports of the avalanche specialist in charge [7], [8].

5.1 The Rorbach avalanche

The Rorbach valley descending West-East produces an avalanche that is well documented since commissioning of the St. Gotthard railway line in 1882. 14 major events are recorded over 115 years, most of them causing damage to the infrastructure. The most severe event was a dry snow avalanche in 1981, that damaged the railway bridge, covered the roads and highways up to 4 m and produced a debris cone 8 m to 30 m deep, 400 m in lateral extent following the river Reuss for another 280 m (Fig. 1).

The starting zone can be divided into two parts. The upper one lies higher than 2000 m a.s.l. and covers an area of about 1 million m^2. Snow masses that begin to move up there must overcome a flat path 700 m in length to reach the valley floor. The lower part of the formation zone is steeper; each heavy snowfall causes snow flows that fill the narrow canyon and reduce the roughness, facilitating subsequent larger avalanches. Table 2 shows estimated and calculated values of two major events.

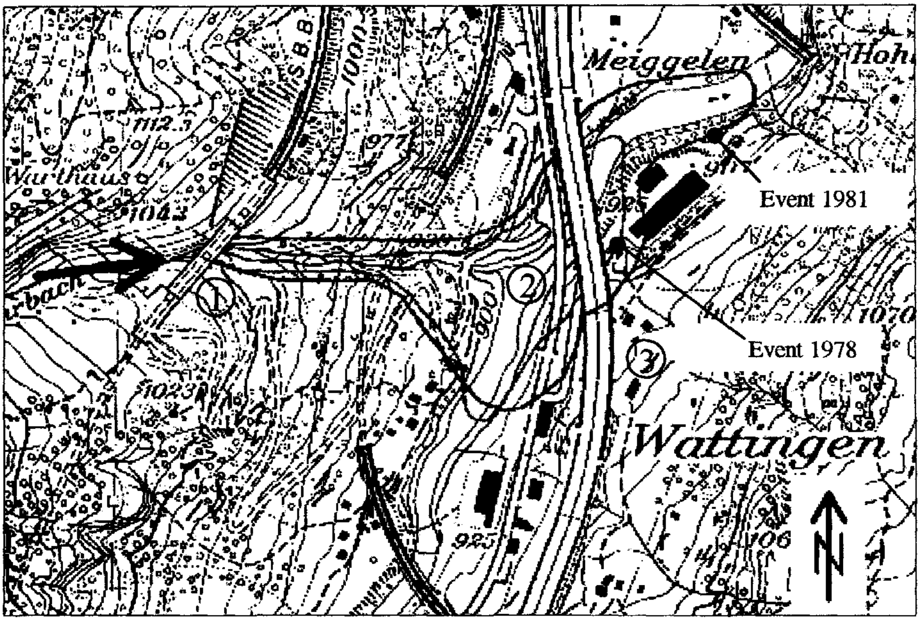

Fig. 1. Debris zones of the Rorbach avalanches of 1978 and 1981 (from [7])

Table 2. Estimated and calculated values of two run downs of the Rorbach avalanche

	Unit	February 2, 1978	January 6, 1981
Estimated debris volume	[m^3]	120,000	220,000
Estimated snow volume in the starting zone	[m^3]	300,000	550,000
Length of avalanche body	[m]	500	700
Duration of event	[s]	20	35

Such avalanches reach the valley floor with high speed and impact the opposite valley flank, losing most of their kinetic energy. The snow then flows down along the river Reuss with reduced speed.

5.2 Structures in the endangered area

5.2.1 The Rorbach bridge of the railway line

The railway line crosses the Rorbach valley on a bridge (marked 1 in Fig. 1) that has been subject to severe damage at every major event. In such critical situations the line had to be closed, because operational safety was not guaranteed, causing delays and distractions. In 1984 the original bridge was replaced by a tunnel bridge designed for a lateral distributed force of 10 kN/m^2, covering the flow pressure of a dry snow avalanche. If the canyon is not completely filled with snow, dense flow avalanches should pass underneath.

5.2.2 The Reuss bridges Wattingen

The St. Gotthard highway A2 as well as the local road cross the river Reuss at the same location, just where the Rorbach avalanche reaches the river (marked 2 and 3 in Fig. 1). Dense flow avalanches act on the piers, dry snow avalanches may blow snow-air mixture high up, producing a snow layer of some meters depth on all bridges.

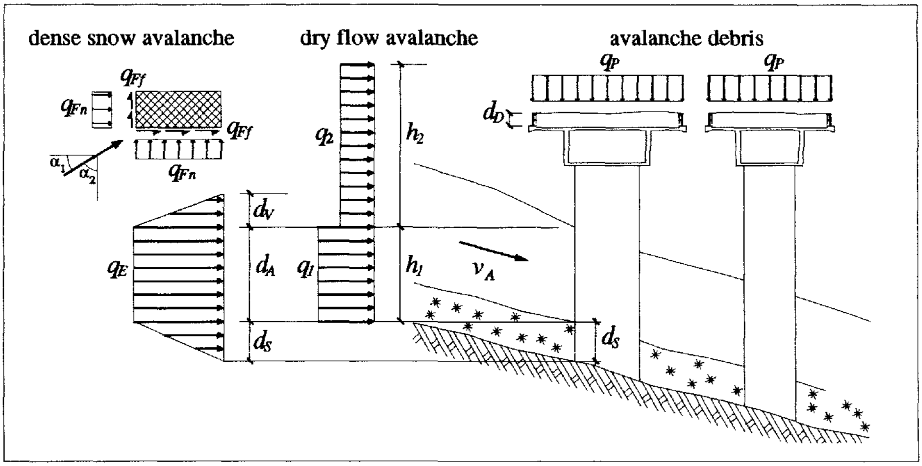

Fig. 2. Actions on piers and girders of the Reuss bridges A2 Wattingen, normal and accidental event

Table 3 Hazard scenarios for the Reuss bridges A2 Wattingen

Hazard scenario	Unit	Normal event	Accidental event
Return period	[years]	30	100
Avalanche debris on bridges			
Maximum depth d_D	[m]	2.5	4.5
Vertical load q_P	[kN/m^2]	7.5	15.8
Dry snow avalanche			
Air pressure q_1, q_2	[kN/m^2]	1.8, 1.3	2.5, 1.7
Altitude above h_1	[m a.s.l.]	920	930
Height h_2	[m]	20	20
Dense flow avalanche			
Avalanche velocity v_A at relevant pier LO2	[m/s]	11	14
Normal component of flow pressure q_{Fn} with most unfavourable angle α	[kN/m^2]	17.3	27.9
Parallel (friction) component of flow pressure q_{Ff} with most unfavourable angle α	[kN/m^2]	5.2	8.4

Dense flow avalanches produce a flow pressure q_F that is taken as constant over the depth of the avalanche d_A and linearly reduced to zero in the underlying snow depth d_S and in the dynamic depth d_V. Since dense snow avalanches can occur combined with a dry snow zone, the associated air pressure has to be taken into account. It is applied to two zones with the heights h_1 and h_2 and the pressures q_1 and q_2, respectively.

For piers the angle α between shaft surface and avalanche motion determines deviating and friction forces. The limited size and a possible favourable shape of the piers are taken into account by reduction factors.

The maximum actions taken into account for the check calculation of the rehabilitation project are given in Fig. 2 and Table 3.

5.3 The situation in February 1999

In the night from February 8 to February 10, 1999 a first dry snow avalanche associated with a snow powder flow ran down. The Rorbach bridge as well as the Reuss bridge Wattingen were covered with snow up to 0.2 m depth.

Since snow fall continued another large avalanche was expected, running on top of the already deposited one and thus being able to hit either the Rorbach bridge or the superstructures of the Reuss bridges Wattingen. Artificial triggering was ruled out due to unforeseeable consequences.

On February 20 the snow pack was finally triggered by natural causes and formed a dense snow avalanche that reached the Reuss bridge of the local road.

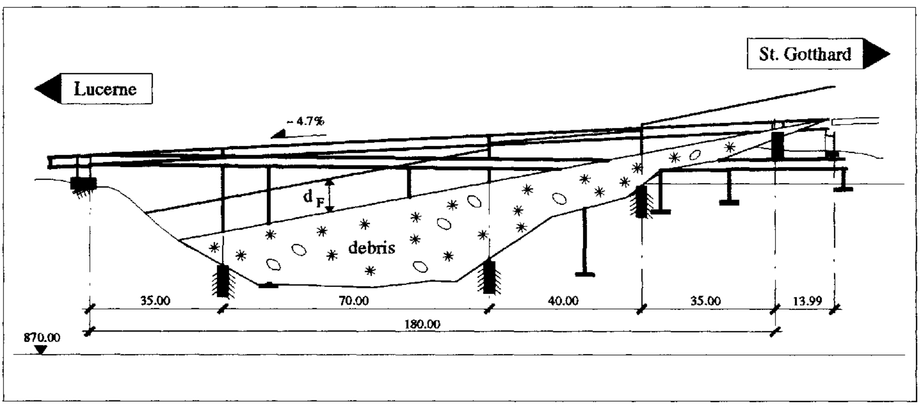

Fig. 3. Longitudinal section through the Reuss bridge A2 Wattingen (uphill track ROMEO)

5.4 Extreme hazard scenario

These experiences initiated the establishment of an extreme hazard scenario with a return period beyond 300 years, namely the filling up of the Rorbach canyon and the formation of a debris cone by a first avalanche and the subsequent release of a second avalanche. The calculation took into account the new topography changed by the debris of the first avalanche, resulting in steeper slopes and smaller depths and changing the direction of the track.

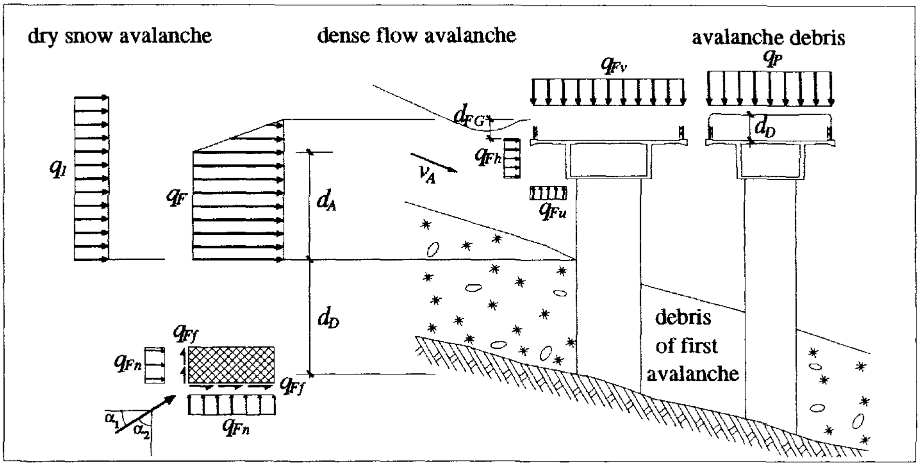

Fig. 4. Actions on piers and girders of the Reuss bridges A2 Wattingen, extreme hazard scenario

Table 4 Extreme hazard scenario for the Reuss bridge A2 Wattingen (uphill track ROMEO)

Extreme hazard scenario	Unit	Range of girder, number of pier			
Avalanche debris on bridges		c	b	a-2	a-1
Maximum depth d_D	[m]	2	3	6	8
Vertical load q_P	[kN/m^2]	5	7.5	21	28
Dry snow avalanche					
Air pressure q_l on total height	[kN/m^2]	2.0	3.0	2.0	
Dense flow avalanche on girder					
Flow depth d_{FG} on girder	[m]	1.5	2	3	4
Vertical load q_{Fv} on girder	[kN/m^2]	2.2	3	9	12
Horizontal pressure q_{Fh} on girder	[kN/m^2]	-	-	85 triangular	100 uniform
Uplift pressure q_{Fu} on cantilevering deck slab	[kN/m^2]	-	-	15	25
Dense flow avalanche on columns		RO4	RO5		RO6
Avalanche velocity v_A	[m/s]	10	16		21
Flow depth d_A on debris	[m/s]	4	8		6
Normal component of flow pressure with most unfavourable angle α	[kN/m^2]	12.5	36.5		74
Parallel (friction) component of flow pressure with most unfavourable angle α	[kN/m^2]	3.8	11		22

Nevertheless, the superstructure of the Reuss bridge A2 Wattingen is subject to either vertical loads caused by the debris of a dry snow avalanche or horizontal flow pressures due to a dense flow avalanche. The cantilever part of the deck slab would be loaded additionally by uplift forces.

All these forces are greatest at the southern end and decrease along the bridge due to the more favourable topography in the northern part (Fig. 3). The bridge carrying the uphill lanes called ROMEO was subject to a new check calculations applying the actions of Figure 4 and Table 4. The calculations showed, that the girder could withstand the vertical loads but would be heavily overstressed in the southern part by carrying the horizontal loads spanning from one abutment to the other.

Which measures will be taken to meet the additional requirements has not been decided yet. The most promising option is the enlargement of pier RO6 to form a stiff intermediate support for horizontal actions.

6 Conclusions

The presented methods of calculating actions due to avalanches are not very sophisticated. At least they have a physical background and fulfil the requirements for accuracy. A direct verification of actions by measurements is difficult, but the extent of the debris zone allows a back calculation of the event that caused them.

The case study shows that avalanche actions can heavily depend on previous events, because the debris accumulates over a whole winter period and can change the topography considerably.

7 Acknowledgements

The author wishes to thank the representative of the owner Heribert Huber, Cantonal Highway Department Uri, for his generous support and the consultants A. Burkard, Brig, R. Vögeli c/o Winkler & Partners Effretikon and D. Meister c/o A. Rotzetter&Partners, Zug for their agreeable co-operation.

8 References

1. Hutter, K. (1992) Lawinen-Dynamik (Dynamics of avalanches), *Schweizer Ingenieur und Architekt*, No. 13/1992, pp. 259-269.
2. Voellmy, A. (1955) Über die Zerstörungskraft von Lawinen (On the destruction force of avalanches), Schweizerische Bauzeitung, Vol. 73, No 12 pp. 159-162, No. 15 pp. 212-217, No. 17 pp. 246-249, No. 19 pp. 280-285.
3. Salm, B., Burkard, A. and Gubler, H. U. (1990) Berechnung von Fliesslawinen – Eine Anleitung für Praktiker mit Beispielen (Calculation of dense flow avalanches – a guideline for practitioners), *Reports of the Swiss Federal Institute for Snow and Avalanche Research*, No. 47, July 1999, 37 pp.

4. Norem, H., Irgens, F., Schieldrop, B. (1987) A continuum model for calculating snow avalanche velocities in *Avalanche Formation, Movement and Effects* (ed. Salm, B. and Gubler, H.), Proceedings of the Davos Symposium, September 1986, IAHS Publication No. 162, pp. 363-378.
5. Swiss Society of Engineers and Architects (1989), *Actions on structures*, SIA-Code 160(1989), Zurich, SIA, 1989, 104 pp.
6. Swiss Federal Department of Traffic and Energy (1994), Richtlinie Einwirkungen auf Lawinenschutzgalerien (Guideline for actions on avalanche galleries), EDMZ, Bern 1994, 15 pp.
7. Burkard A. (1998), Lawineneinwirkungen auf die Wattingerbrücke und den Lehnenviadukt Laubzug (Avalanche actions on the Wattingen bridges and the Laubzug viaduct), Technical Report, Brig, February 19, 1998, 29 pp.
8. Burkard A. (1999), Lawineneinwirkungen auf die Wattingerbrücken A2 – Extrem-Szenario (Avalanche actions on the Wattingen bridges – extreme scenario), Revised Technical Report with appendix, Brig, August 25, 1999, 11 pp/17 pp.

CONSIDERATION OF EXCEPTIONAL SNOW LOADS AS ACCIDENTAL ACTIONS

Exceptional snow loads

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Abstract

The investigation on German snow data show that for some German climatic stations (located in coastal regions of North Germany) the exceptional heavy snow falls can be identified. These snow loads can cause the essential damages of structures. Normally these exceptional values have a very large return period (1,000 or even 10,000 years) but can occur during relative short design working life of structure (50 or 100 years). According to ENV 1991-1 "Basis of Design" these exceptional snow falls can be considered as accidental actions. Based on German data statistical analysis is undertaken and procedure for codified design is considered.

Keywords: Accidental actions, characteristic value, exceptional value, probability distribution, snow loads, statistical analysis.

1 Introduction

Very heavy snow falls were observed in different parts of Europe, particularly in coastal regions. They cause snow loads which are significantly larger than the snow loads which normally occur in these regions.

As an example the city Schleswig (altitude 43 m above sea level) can be considered. It is located in North Germany, not far from the Danish border. It belongs to Snow Load Zone III according to ENV 1991-2-3 "Snow Loads" [1]. According to this standard, the characteristic value (with return period of 50 years) of snow load for this location and altitude, is 1.13 kN/m^2. But the maximum observed value, which occurred on 19th February 1979, was 2.37 kN/m^2. The ratio between these two values is equal to 2.1 and larger as the partial safety factor for snow load which is equal to 1.5. Including those high values together with the more regular snow events into the

Abnormal Loading on Structures edited by K. S. Virdi, R. S. Matthews, J. L. Clarke and F. K. Garas.
Published in 2000 by E & FN Spon, 11 New Fetter Lane, London EC4P 4EE, UK. ISBN 0 419 25960 0

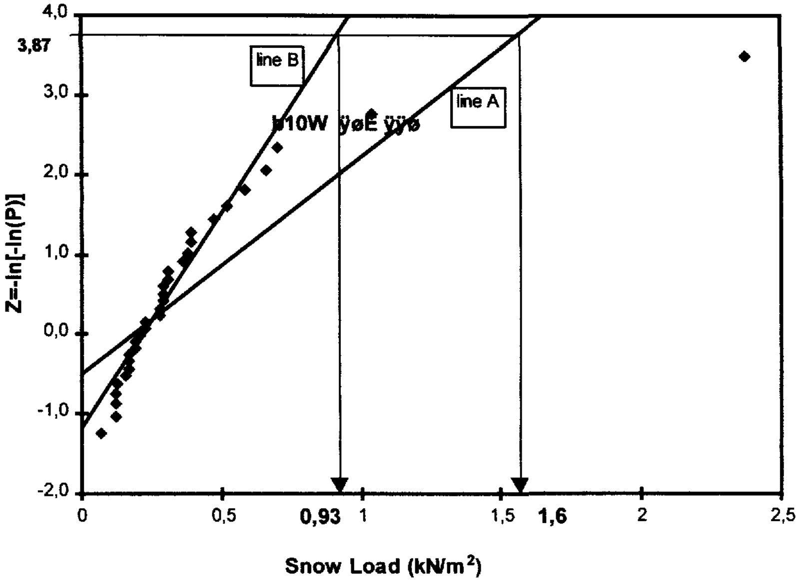

Fig. 1. Station Schleswig, extreme value distribution type I

sample of the annual maxima of snow load disturbs the statistical processing of the snow data. If the extreme value distribution type I for maxima is used for the fitting of data then using of probability paper leads to the plot shown in Fig. 1 (on this plot Z is the axis of reduced variate and P is probability that the corresponding value of snow load is not being exceeded). The parameters and therefore position of fitting line are defined by means of least squares method.

From this plot can be seen that including the exceptional value of 2.37 kN/m^2 in statistical processing (line A) leads to characteristic value of 1.6 kN/m^2 but the line does not fit the data. Excluding the exceptional value (line B) leads to characteristic value of 0.93 kN/m^2 and the line fits the data better (but also not quite well). This problem was also discussed in the background document to ENV 1991-2-3 [2] and in the final report of the "European Snow Loads Research Project" [3]. The main question of discussion is following: if the exceptional value is excluded from the sample considered how should this value be taken into account in codified design ?

2 Investigation on German snow data

During the work on "European Snow Loads Research Project" about 330 German climatic stations were investigated based on observed data of water equivalent (and/or snow depth). The record period of most stations was about 30 years. First of all the annual maxima of snow load were obtained and then the characteristic values (with return period of 50 years) were calculated based on extreme value distribution type I for maxima (but the log-normal and Weibull distributions were also discussed).

12 climatic stations were declared as the stations with the exceptional snow events. List of these stations can be seen from the Table 1.

Table 1. The list of German climatic stations with exceptional snow events

N	Station	Number Of record Years	Number Of years with snow	Exceptional value of snow load Q_{exc} (kN/m^2)	Date of occurrence of exceptional value	Next maximum snow load value after exceptional one (kN/m^2)
1	Norderney	18	15	1.56	22 Feb. 1979	0.70
2	Schleswig	33	32	2.37	19 Feb. 1979	1.04
3	Hamburg	33	32	1.82	15 Feb. 1979	0.67
4	Bederkesa	18	17	1.52	16 Feb. 1979	0.62
5	Bremen	32	30	1.53	18 Feb. 1979	0.58
6	Cuxhaven	31	24	1.48	01 Mar. 1979	0.61
7	Soltau	25	25	1.26	19 Feb. 1979	0.62
8	Hannover	33	31	1.23	24 Feb. 1979	0.56
9	Kiel	33	31	1.78	15 Feb. 1979	0.68
10	Norden	12	12	1.07	19 Feb. 1979	0.31
11	Tostedt	27	26	1.47	16 Feb. 1979	0.74
12	Visselhoevede	27	24	1.54	17 Feb. 1979	0.59

The location of all these stations is the north-west of Germany in the vicinity of North and/or Baltic sea. The date of occurrence of exceptional event for these stations is the second half of February 1979 when a very heavy snow falls were observed in north-west Germany which caused, for example, a damage of a lot of roofs in region of Hamburg.

With the help of extreme value distribution type I for maxima the characteristic (with return period 50 years) values were calculated for the case when the exceptional value is included in the statistical processing and for the case when the exceptional value is excluded from the consideration. The results can be seen in the Table 2. In this table the plot correlation coefficients are also given. These coefficients show how well the line (or other words the chosen distribution) fits the data. In ideal case the plot correlation coefficient will be equal to 1.0. This would mean that all data points lie on the same line.

The values of plot correlation coefficient from Table 2 show that extreme value distribution type I fits the data not well. Even for the case when exceptional value excluded the fitting is not satisfactory, as can be seen for line B on Fig. 1 for Schleswig. The same is observed for the most of other stations. Therefore it can be concluded that extreme value distribution seems to be not the right function for fitting the original data points. Thus other distributions should be considered.

Table 2. Characteristic values and plot correlation coefficients based on the extreme value distribution type I

N	Station	A: Exceptional value is included		B: Exceptional value is excluded		Ratio
		50 years return value S_A (kN/m^2)	Plot correlation coefficient	50 years return value S_B (kN/m^2)	Plot correlation coefficient	$k = Q_{exc} / S_B$
1	Norderney	1.51	0.8601	0.84	0.9097	1.86
2	Schleswig	1.60	0.8224	0.93	0.9673	2.55
3	Hamburg	1.23	0.8284	0.71	0.9759	2.56
4	Bederkesa	1.42	0.8860	0.81	0.9870	1.88
5	Bremen	1.05	0.8237	0.59	0.9811	2.59
6	Cuxhaven	1.11	0.8514	0.66	0.9905	2.24
7	Soltau	1.07	0.9168	0.75	0.9820	1.68
8	Hannover	0.90	0.8928	0.61	0.9946	2.02
9	Kiel	1.22	0.8220	0.71	0.9814	2.51
10	Norden	1.13	0.7704	0.39	0.9538	2.74
11	Tostedt	1.16	0.8769	0.75	0.9706	1.95
12	Visselhoevede	1.17	0.8373	0.66	0.9748	2.35

During the work "European Snow Loads Research Project" [3] it was pointed out that for Germany 3 probability distribution functions are to be considered as possible candidates: extreme value distribution type I, log-normal distribution and Weibull distribution. Which from these functions fits the data better depends on the local climatic conditions. It was found that for lowland of North Germany the log-normal distribution fits the data best. Let us look at the example of station Schleswig presented on the log-normal probability paper at Fig. 2. From this plot it is clear that line B (after excluding the exceptional value) fits the original data very well, essentially better as the line B on the probability paper for extreme distribution type I (compare with Fig. 1). The preference of the log-normal distribution (in comparison with the extreme value distribution type I) for the stations considered can be confirmed with the help of the plot correlation coefficients presented in Table 3.

The comparison of the results from the Table 1, Table 2 and probability plots of the stations (not given here, only station Schleswig is given as example) shows that for all 12 stations the log-normal distribution is the best fitting one if the exceptional value is included in the statistical consideration. But more interesting the case when the exceptional value is excluded. Then for 7 stations the log-normal is again the best fitting distribution. For 3 stations (Bederkesa, Cuxhaven and Hannover) both the distributions, extreme value type I and log-normal, give almost the same value of plot correlation coefficient and both fit the data well. For station Kiel extreme value distribution type I gives higher value of plot correlation coefficient as log-normal one but the both distributions fit the data not satisfactory. In this specific case the best candidate is Weibull distribution which gives the value of plot correlation coefficient equals 0.9849 and 50 year return value of snow load equals 0.64 kN/m^2. Almost the

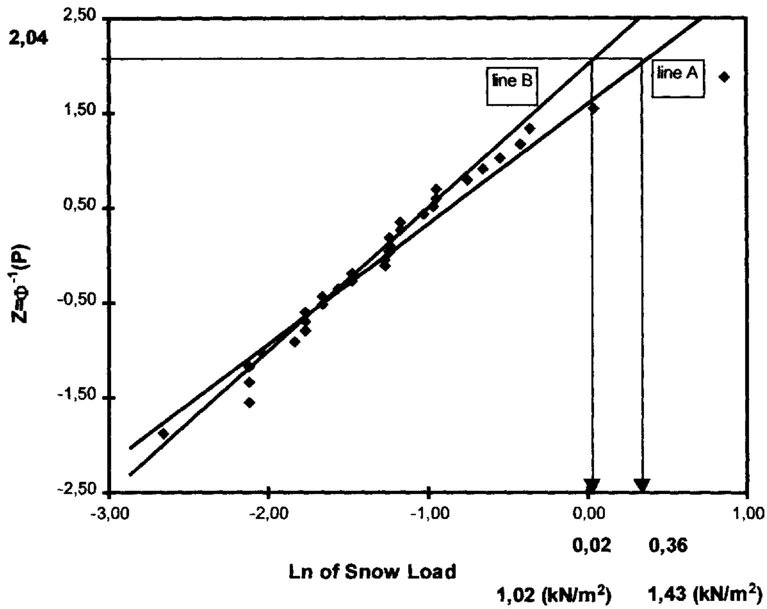

Fig. 2. Station Schleswig, log-normal distribution

Table 3. Characteristic values and plot correlation coefficients based on the log-normal distribution

N	Station	A: Exceptional value is included		B: Exceptional value is excluded		Ratio
		50 years return value S_A (kN/m^2)	Plot correlation coefficient	50 years return value S_B (kN/m^2)	Plot correlation coefficient	$k = Q_{exc} / S_B$
1	Norderney	1.72	0.9473	0.97	0.9567	1.61
2	Schleswig	1.43	0.9724	0.102	0.9924	2.32
3	Hamburg	1.13	0.9719	0.82	0.9883	2.22
4	Bederkesa	1.73	0.9866	1.16	0.9868	1.31
5	Bremen	0.94	0.9677	0.66	0.9878	2.32
6	Cuxhaven	1.06	0.9717	0.77	0.9856	1.92
7	Soltau	1.12	0.9802	0.87	0.9857	1.45
8	Hannover	0.92	0.9865	0.73	0.9925	1.68
9	Kiel	1.18	0.9653	0.89	0.9741	2.00
10	Norden	1.13	0.8814	0.45	0.9401	2.39
11	Tostedt	1.14	0.9744	0.84	0.9875	1.76
12	Visselhoevede	1.13	0.9724	0.78	0.9867	1.97

same situation with station Norden. Again Weibull is the best fitting distribution and gives for snow load the 50 year return value of 0.35 kN/m^2. But in this case the extreme value distribution type I fits the data also well and a little better as log-normal one (but this station has a shortest record period - only 12 years). Snow load value with 50 year return period is equal to 0.39 kN/m^2 and is a little larger as by Weibull distribution.

3 Presentation of exceptional snow load as accidental action

The only country among the CEN members where the problem of exceptional snow load is reflected in the building standard is France. According to the French Code of Practice N84 the snow value should be considered as exceptional one (based on snow depth d) if the following criteria is fulfilled:

$$d_{max} > 1.5\, d_{50} \qquad (1)$$

where: d_{50} the 50 year return period value of snow depth if the maximum value of snow depth is excluded

d_{max} the maximum value of snow depth

The value of 1.5 based on the value of safety factor from French codes. The same value is set for partial safety factor for variable actions in ENV 1991-1 "Basis of Design" [4].

Because in the structural design the effect of snow is considered as loading the criteria similar to Eqn. (1) should be applied to snow load S and can be presented as:

$$S_{max} \geq k\, S_{50} \qquad \text{or} \qquad S_{max} \geq k\, S_k \qquad (2)$$

Here the symbols have the same meaning as in Eqn. (1) but related to the snow load and S_k is called characteristic value. The constant factor k has to be defined. Two approaches can be considered for this purpose.

3.1 Statistical aspect

Firstly the statistical aspect is to be taken into consideration.

Because the extreme value distribution type I for maxima is used for the fitting of snow data almost in all CEN countries (except Denmark) let us consider the P-fractile based on this distribution:

$$X_p = u - \ln(-\ln P)\, /\, c \qquad (3)$$

Parameters u and c can be determined using the method of moments and then fractile can be presented as in Eqn. (4) dependent on the mean value m and the coefficient of variation V of the sample:

$$X_p = m\, \{1 - 0.78\, V\ [\, 0.577 + \ln(-\ln P)\,]\, \} \qquad (4)$$

The European building standards do not define directly a return period for accidental actions. Only the Background Document for ENV1991-1 [5] notes that this return period can be up to 10,000 years. The German Reactor Safety Rules for Nuclear Stations set the same value of 10,000 years. This corresponds to the fractile with the probability of not being exceeded during one year of 0.9999. The characteristic value for variable actions is defined as a value with a return period of 50 years (i.e. $P = 0.98$). Therefore the factor k can be defined as the ratio of these two values and it becomes dependent only on coefficient of variation V:

$$k = X_{0.9999} / X_{0.98} = (1 + 6.73\,V) / (1 + 2.59\,V) \tag{5}$$

As investigations on German snow data show [6], values of V vary with altitude. As mean value of V the value of 0.6 can be set. But if only stations with altitude not higher as 200m above sea level are taken into account (all 12 above considered stations in North Germany belong to this group), then value of 0.6 will be the lower bound for V, because for these stations V varies mainly between 0.6 and 1.4. For comparison a return period (T_{ret}) of 1,000 years is also considered, in this case the probability of not exceeding during one year is equal to 0.999. Using these values of T_{ret} and $V = 0.6$ (as a lower bound) will lead to following values of k:

$k \approx 2.0$ (T_{ret} = 10,000 years); $k = 1.55$ (T_{ret} = 1,000 years)

From the above shown calculations it can be concluded that the load values with $k \geq 2$ have a return period at least of 10,000 years and the load values with $k \geq 1.5$ have a return period at least of 1,000 years.

3.1 Normative aspect

As second possibility a normative procedure from the Eurocodes can be used to define the coefficient k.

The document ENV 1991 - 1 "Basis of Design" [4] defines in Section 4 "Actions and environmental influences" § 4.1 "Principal classifications" (2) snow loads as variable actions. But allowance is made in Clause (4): "Some actions, for example seismic actions and snow loads, can be considered as either accidental and/or variable actions, depending on the site location (see other Parts of ENV 1991)". This permits to consider the exceptional snow events as accidental loads.

Thus, if the snow event is identified as exceptional one, i.e. connected with the characteristic value by Eqn. (2), then the snow load should be taken into account in two design situations: persistent/transient (P/T) situations and accidental (A) situations. The coefficient k needs to be determined by taking into account not only consideration of the actions, but also the influence of the resistance.

According to ENV 1991 - 1 "Basis of Design" [4], Section 9 it shall be verified that:

$$E_d \leq R_d \tag{6}$$

where: E_d design value of the effect of action
R_d corresponding design value of resistance

For simplicity let us consider the case when snow load is the only variable action and there is only one permanent action (e.g. self-weight). Then according to Clause 9.4.2 "Combinations of actions" of ENV 1991-1 [4] there are two cases to be considered:

- persistent and transient design situations for ultimate limit states verification other than those relating to fatigue:

$$E_d = \gamma_G G_k + \gamma_Q Q_k \tag{7}$$

- accidental design situations:

$$E_d = \gamma_{GA} G_k + A_d \tag{8}$$

where:	$\gamma_G = 1.35$	the partial safety factor for permanent action
	G_k	the characteristic value of permanent action
	$\gamma_Q = 1.5$	the partial safety factor for variable action (snow load)
	Q_k	the characteristic value of variable action (snow load)
	$\gamma_{GA} = 1.0$	the partial safety factor for permanent action for accidental design situation
	$A_d = \gamma_A \cdot A_k$	the design value of accidental action
	$\gamma_A = 1.0$	the partial safety factor for accidental action(snow load)
	A_k	the characteristic value of accidental action (snow load)

According to ENV 1992 "Design of Concrete Structures", Part 1-1 "General Rules and Rules for Buildings" [7]:

P/T:	$R_d = R_{k,c} / \gamma_C$	for concrete	(9)
	$R_d = R_{k,s} / \gamma_S$	for steel reinforcement or prestressing tendons	
A:	$R_d = R_{k,c} / \gamma_{CA}$	for concrete	(10)
	$R_d = R_{k,s} / \gamma_{SA}$	for steel reinforcement or prestressing tendons	

where:

$\gamma_C = 1.5$	the partial safety factor for concrete (P/T situations)
$R_{k,c}$	the characteristic value of concrete
$\gamma_S = 1.15$	the partial safety factor for steel reinforcement (P/T situations)
$R_{k,s}$	the characteristic value of steel reinforcement
$\gamma_{CA} = 1.3$	the partial safety factor for concrete (Accidental situations)
$\gamma_{SA} = 1.0$	the partial safety factor for steel reinforcement (Accidental situations)

Using Eqn. (6), (7) and (9) it is possible to write for P/T situations for concrete:

$$1.35\, G_k + 1.5\, Q_k \leq R_{k,c} / 1.5$$

Considering as an unfavourable case the ratio $G_k = 0.5\, Q_k$ and noting that $Q_k = S_k$ one can obtain:

$$3.26\,S_k \leq R_{k,c} \tag{11}$$

Similarly for accidental situations Eqn. (6), (8) and (10) and $A_k = S_{max} = k\,S_k$ from Eqn. (2) will lead to equation:

$$1.0\,G_k + k\,S_k \leq R_{k,c} / 1.3$$

Taking again as unfavourable case the ratio $G_k = 0.5\,Q_k$ it will be obtained:

$$(\,0.65 + 1.3\,k\,)\,S_k \leq R_{k,c} \tag{12}$$

Because the right hand parts in Eqn. (11) and (12) are the same (characteristic value of concrete strength) the left hand parts (design value of action effect) shall be also the same, independently of the design situation. Then k can be calculated as:

$$k = (\,3.26 - 0.65\,) / 1.3 = 2.0 \tag{13}$$

Other ratios of G_k / Q_k can also be taken into consideration. Assuming that $G_k = Q_k$ it will result in $k = 2.29$, and for the case when $G_k = 1.5\,Q_k$, k will be equal to 2.57.

Considering the case $G_k = 0.5\,Q_k$ as unfavourable, exceptional snow events with $k \geq 2.0$ should be fixed as accidental ones.

4 Analysis

The normative approach leads to values of k which are greater or equal to 2.0. The statistical approach leads to values of k which are greater or equal to 2.0 for return period of 10,000 years and values of k which are greater or equal to 1.5 for return period of 1,000 years. Thus snow events with k greater or equal to 1.5 can be considered as accidental ones.

Which from these two design situations (P/T or A) is more unfavourable (and therefore the decisive one) depends on the ratio of the characteristic value of permanent action to the characteristic value of the snow load.

Table 2 shows that if the extreme value distribution type I is used then all 12 considered climatic stations have the factor k greater than 1.5 and 8 station from these 12 ones have the values of k greater than 2.0. If log-normal distribution is used (see Table 3) then 10 stations have the factor k greater than 1.5 and 5 from these ones have the values of k greater than 2.0. Only two stations have the k factor less than 1.5. But for Soltau this factor is equal to 1.45 which is very near to 1.5. Therefore all stations (only Bederkesa is doubtful if the log-normal distribution is used) should be considered as ones with exceptional events and treated according to the procedure for accidental actions.

5 Conclusions

1. Analysis of 12 German climatic stations with exceptional values of snow load shows that the log-normal distribution fit the original snow data better as the extreme value distribution type I for maxima.

2. The exceptional values of snow loads can have the return period of 1,000 or even 10,000 years.

3. The largest snow load value from the sample should be identified as exceptional one if the ratio of this value to the characteristic value (with return period of 50 years) of snow load determined after excluding this largest value from statistical processing is greater or equal to 1.5.

4. The exceptional values of snow loads should be treated as an accidental actions and snow load, in this case, should be taken into account for two design situations, persistent / transient and accidental ones.

6 Acknowledgements

Authors would like to express their appreciation for financial support made by Commission of the European Communities DGIII-D3 to carry out the reported work, which forms the part of the prenormative research for Eurocode 1, Part 2-3 "Snow Loads".

7 References

1. European Committee for Standardisation. (1994) *Basis of Design and Actions on Structures. Part 2-3: Snow Loads.* CEN, Brussels. ENV 1991-2-3.
2. University of Pisa. (1995) *New European Code for Snow Loads. Background Document.* Proceedings of Department Structural Engineering University of Pisa.
3. Commission of the European Communities DGIII-D3. (1998) *Scientific Support Activity in the Field of Structural Stability of Civil Engineering Works. Snow Loads. Final Report.* University of Pisa. Department of Structural Engineering.
4. European Committee for Standardisation. (1994) *Basis of Design and Actions on Structures. Part 1: Basis of Design.* CEN, Brussels. ENV 1991-1.
5. European Convention for Constructional Steelwork. Joint Committee on Structural Safety (JCSS). (1996) *Background Documentation. Eurocode 1 (ENV 1991). Part 1: Basis of Design.* JCSS, Brussels. First Edition.
6. Soukhov, D. (1998) The Probability Distribution Function for Snow Load in Germany. *Leipzig Annual Civil Engineering Report (LACER)*, Vol. 3, pp. 263-274.

EVALUATION OF THE PEAK FORCES ON ROOF TILES UNDER STORMY CONDITIONS

Peak forces on roof tiles

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Abstract

The problem of determining the net forces on tiles and slates in domestic housing has never been resolved in a completely rigorous way, despite considerable safety risks and economic losses per annum due to tile and slate loss during stormy conditions. Instead, empirical rules have been introduced in normative documents [20] and guides of 'good practice' [4,5,6] aimed towards craftsmen. The most particular aspect with respect to the load on the elements resides in the fact that they are not only subject to the external pressure, but also to the internal pressure that builds up just underneath. The internal air pressure depends, among other aspects, on the geometry of the joints between the distinct elements, the global geometry of the roof, the boundary conditions at ridges and roof top, etc. An extra complication is introduced in that the external pressure on the roof is locally altered by the external shape of the elements. The Belgian Building Research Institute (BBRI), together with the Von Kàrmàn Institute (VKI) in Brussels, has initiated a research programme on this subject and this paper describes the scope of the research together with the set-up of the experimental and numerical work and some preliminary results.

Keywords: Full-scale experiments, numerical modelling, peak forces, roof tiles, wind loading, wind-tunnel tests.

1 Introduction: Tiling and slating practice in Belgium

It is common in Belgium to provide an under-roof under the tiles during the construction of a pitched roof. Generally, this under-roof consists of thin wooden boards, but thin plastic sheets are also used. Both are laid with sufficient overlaps to make sure that the under-roof is quite impermeable, at least in comparison with the permeability of the joints between the covering elements.

Abnormal Loading on Structures edited by K. S. Virdi, R. S. Matthews, J. L. Clarke and F. K. Garas.
Published in 2000 by E & FN Spon, 11 New Fetter Lane, London EC4P 4EE, UK. ISBN 0 419 25960 0

The frontal permeability is caused by the gaps in the joints between the tiles. Also, in the thin air layer underneath the tile cladding, air movement in two orthogonal directions is possible, which is characterised by the lateral permeability, that may easily be many times greater than the frontal permeability.

2 Scope and structure of the research programme

It is well known that the net forces induced by the wind on tiles and slates are the result of the (integrated) pressure differences between the external pressure (p_e) and the internal pressure (p_{ia}) that build up just under the elements in the thin air layer. Both external and internal pressures depend on several parameters that we will discuss further in this presentation. Two of the major parameters are the position of the tile on the roof and the wind direction. There are indeed some places locally where the pressure coefficient (Eq. 1) can be much higher than the average value on the roof. This can be explained by the development of 'delta wing vortices' along the roof edges (Fig. 1).

$$C_p = C_{pe} - C_{pia} = \frac{p_e - p_{ia}}{1/2.\rho.V^2} \tag{1}$$

where ρ is density of air (1.225 kg/m³) and V is the wind velocity.

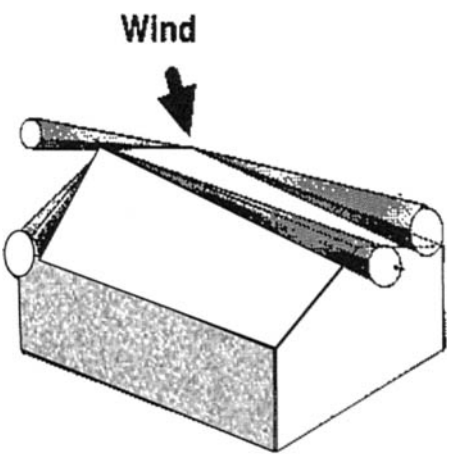

Fig. 1. Delta-wing vortex pairs on duopitch roof [11].

The centres of these vortices are regions of high negative pressure. This is the principal cause of high mean peripheral uplift on roofs, and causes much of the damage reviewed in literature [10].

To this purpose, the Belgian recommendations propose empirical rules for fixing tiles (with nails or other types of fixations) to keep the tiles from taking off. Three classes of fixation are considered: none, one out of every two, one out of every four tiles must be fixed [6]. The extensive damage due to windstorms in Belgium in 1990 has made it evident that the assumptions made for C_{pia} in the existing Belgian Wind Standard NBN B03-002-1 [7], for example, that 2/3 of C_{pe}, in function of the percentage of frontal openings, are not always a correct representation of the reality. It turns out that this approach is a very rough simplification, but until now, there has been no alternative. It was felt that the contribution of the fluctuating internal pressure field might be much more important than thought before. Therefore, the start of an extensive research programme leading to a better knowledge of the internal pressures was estimated necessary.

To perform this research, all the parameters that play a role in determining the net forces on tiles have to be taken into account: 1. structure of the oncoming wind flow: mean values and atmospheric turbulence characteristics, 2. global geometry of the building: plan, shape of the roof, overhangs, parapets, 3. frontal and lateral permeability values of the cladding system: details of roof construction, type of tiles, etc, and 4. local geometry of the cladding: external shape of the tiles, overlap length, etc.

The research programme first concentrates on tiles. Later on, slates, façade elements and external ballast blocks for flat roofs may be treated following the same methodology. The aim is to evaluate the internal pressures underneath the tiles, resulting from a fluctuating, in time and space, external flow field, and given the rest of the above-mentioned parameters. For this purpose a computer program based on finite volumes has been developed. Permeability values are measured in the lab; the influence of the external shape of the elements results from stationary Computational Fluid Dynamics (CFD) calculations. The validation of the numerical method and necessary input data required the set-up of a full-scale experiment, which will be described below. Other external data fields are provided by parametric studies from wind-tunnel tests.

3 Numerical model for the internal pressure

The numerical model used in this study is a fairly simple one, based on the work of CSTB (Centre Scientifique et Technique du Bâtiment, France), expressing the fact that in a small volume under isentropic quasi-stationary conditions, the input in air flow leads to an internal pressure rise and vice versa [1,8,22]. The governing algebraic expressions are (with $\gamma = 1.4$ and Δp the pressure difference between two neighbouring volumes):

$$\frac{dp}{dt} = \frac{\gamma . P_0}{V} . q(t) \tag{2}$$

$$q(t) = q_x(t) + q_y(t) + q_z(t) \tag{3}$$

$$q_x = \Phi_x . S_x . \sqrt{\left(\frac{2.\Delta P}{\rho}\right)} \tag{4}$$

where Φ_x is the permeability coefficient through the surface S_x in the direction x.

In adopting a finite volume representation, these equations can be translated to the following expression:

$$\begin{aligned} \frac{dp_{i,j}}{dt} &= K_{i,j}.\sqrt{P_{ei,j}(t) - p_{i,j}(t)} \\ &+ XK.(\sqrt{p_{i+1,j}(t) - p_{i,j}(t)} + \sqrt{p_{i-1,j}(t) - p_{i,j}(t)}) \\ &+ YK.(\sqrt{p_{i,j+1}(t) - p_{i,j}(t)} + \sqrt{p_{i,j-1}(t) - p_{i,j}(t)}) \end{aligned} \tag{5}$$

in which XK,YK and ZK are factors that incorporate respectively the frontal permeability, the lateral permeability, in the parallel and perpendicular direction to the ridge of the roof.

This relation characterises the variation of pressure for a time-interval dt , in an element, numbered [i,j], of the grid, based of the values of the pressure of the neighbouring volumes. Of course, boundary and initial conditions have to be fed into the program.

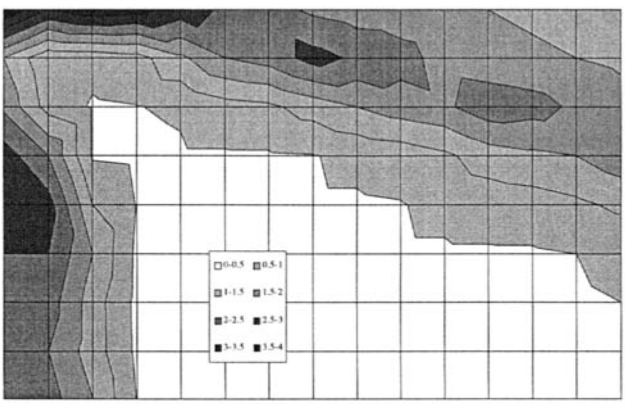

Fig. 2. Results obtained by BBRI using a numerical model to determine the pressure distribution at the upwind corner of the rooftop of a building.