Failures of Roofs under Snow Load: Causes and Reliability Analysis

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ABSTRACT

Collapses of a number of roofs in Europe during the winter 2005/2006 initiated discussions concerning reliability of the roofs exposed to snow loads. Presented overview of extensive investigations of structural failures is focused on causes and consequences of failures. Main observed causes may be subdivided into human errors and insufficient code provisions. Collapses developed from local failures particularly in cases of insufficiently robust structures. Probabilistic reliability analysis reveals that the model for snow loads in the Eurocodes should be modified. Obtained experiences provide valuable background information for future revision of current standards and for forensic assessments of failures of structures exposed to snow loads.

INTRODUCTION

Number of roofs collapsed during the winter period 2005/2006 in European countries such as Austria, Czech Republic, Germany and Poland. Forensic assessments of damaged structures were intended to answer the following questions:

- What were main causes of structural failures?
- Were observed snow loads exceptional?
- Is the reliability of structures designed according to standards sufficient?

Presented overview of extensive investigations of structural failures in the Czech Republic is focused on main causes and consequences of failures. Insufficient code provisions, identified as the cause common for most of damaged structures, are analysed in detail using probabilistic methods. Recommendations for design and assessment of structures exposed to snow loads are proposed.

CAUSES OF STRUCTURAL FAILURES

In total 249 structures mostly in highlands and lowlands in the Czech Republic were investigated, taking into account information provided by Police of the Czech Republic and the Fire Rescue Service of the Czech Republic. The structures were classified as follows:

1. Agricultural structures and other buildings where people usually do not enter (99 damaged structures),
2. Residential houses (68),
3. Industrial buildings (49),
4. Public buildings and other buildings where people gather (33).

Failures of timber roofs were observed in most cases while a lower number of the cases comprised steel roofs. In the latter cases, however, consequences of failures were more severe. In most cases buildings had pre-failure insufficiencies.

Main observed causes of structural damage were subdivided into human errors in design, during execution and use, and insufficient code provisions as indicated in Figure 1. These causes are briefly described in the following. More detailed description is provided in the report by Kucera (2006) and in the special issue of the journal Konstrukce (Structures), Cieslar (2006).

Errors in design

It was observed that gross errors in design such as inconsistencies with code provisions, incorrect loading widths, and numerical errors were less frequent. However, they may have had more severe consequences including the total collapse. Other design errors contributed to a lower extent. These included incorrect models for foundation conditions, local buckling of massive frames braced by insufficiently stiff roofs, torsional buckling, and low resistance of joints.

Errors during execution

Errors during execution (mostly use of low-quality materials for timber structures and inappropriate details) were observed in some cases. It was concluded that such errors had not been identified due to an inadequate quality control of design and execution.

Errors during use

The most frequent errors occurred during use. In particular incompetent interventions into structures (removal or reduction of sectional areas of structural members, new structural members) or installation of new facilities (suspended ceilings, air conditioning etc.) caused unexpected stresses in a structure and contributed to failures. Other errors during use included upgrades of structures without reliability assessment and lack of regards to experts' recommendations for strengthening. In some cases insufficient maintenance together with inappropriate details such as roof parapets with inside drainage yielded accumulation of water on a roof and caused an additional load unexpected in design. Snow load exceeding design assumptions was observed on roofs where snow was required to be removed.

Insufficient code provisions

Insufficient code provisions seem to be the most common cause of structural damage. Use of light-weight roof structures increases significance of snow load and an insufficient reliability level may be obtained by the partial safety factor design as indicated by the probabilistic reliability analysis described below.

In several cases a model for snow loads recommended in standards underestimated actual loads. Use of high-quality materials for heat insulation of roofs protected snow from melting and caused its accumulation (often non-uniform). In several cases a significant load due to the combination of snow and ice on roofs, not considered in design codes, was observed. In few cases snow load was also present on roofs with a great angle of pitch.

In many cases multiple causes such as combination of the errors were observed. The causal network indicating the main identified causes and consequences is shown in Figure 1. Note that available information does not make possible to provide better statistics on the collapsed buildings, more discussion of the causes and their combinations.

CONSEQUENCES OF STRUCTURAL FAILURES

Four levels of damage were distinguished:

- Excessive deflection,
- Failure of a member or few members,
- Partial collapse (up to half of the roof or structure),
- Total collapse (more than half of the roof or structure) – see Figure 2.

In the causal network in Figure 1, the top two levels may represent local failure while the other two collapse. Remarkably, about 60 % of described failures were classified as total collapses and approximately 75 % as the partial or total collapse.

Collapsed structures had mostly insufficient robustness (no tying, low resistance of key members or inappropriate structural detailing). Apparently, robustness is a key property affecting development of collapse from a local failure. That is why the influence of robustness is included in Figure 1. Lack of robustness became important particularly in cases of multiple causes of failure.

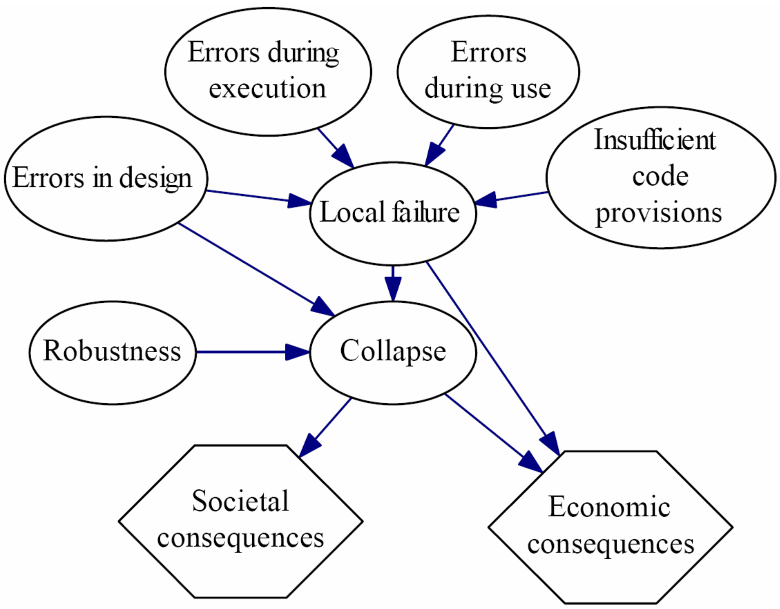

Figure 1. Causes and consequences of structural failures.

Figure 2. Collapse of a stadium.

Recent discussions have indicated that robustness is a complicated concept and is not understood uniformly within engineering society. In accordance with EN 1991-1-7 (2006), robustness is here understood as *the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause*. At present robustness is investigated by researchers from more than 20 European countries within the COST Action TU0601, Faber et al. (2008).

Consequences were qualitatively classified by structural engineers as negligible, low, medium and severe. Two fatalities and one injury were reported. Frequency of the levels of damage for the different types of structures and consequences are indicated in Table 1.

The data in Table 1 may be used as the first estimate of the expected consequences related to a specific type of structure, given local failure/collapse. This may be utilized in risk assessments of structures exposed to snow loads. However, it is emphasized that the information in Table 1 is derived from a limited amount of data and should be considered as indicative only.

DETAILED ANALYSIS OF INSUFFICIENT CODE PROVISIONS

Since the insufficient code provisions were identified as the cause common to most of damaged structures, attempts were made to improve snow load models in design codes. In some European countries including the Czech Republic available measurements of snow loads have been newly evaluated and relevant standards such as National Annexes to EN 1991-1-3 (2003) promptly revised. The following probabilistic reliability analysis is aimed to verify whether the changes in standards guarantee a sufficient reliability of roofs exposed to snow loads.

Statistical evaluation of annual maxima of snow load on the ground

The Czech Republic as well as numerous other regions where roofs collapsed has continental climate. Snow load may increase until late winter or early spring. Drifts of snow may be important.

Table 1. Frequency of the levels of damage for the different types of structures and estimates of consequences.

Class of Structure	Excess. deflect.	Member failure	Partial collapse	Total collapse
1. Agricultural structures	10 %	5 %	20 %	65 %
2. Residential houses	0 %	15 %	20 %	65 %
3. Industrial buildings	10 %	20 %	15 %	55 %
4. Public buildings	5 %	50 %	15 %	30 %

□ negligible consequences, □ low consequences, □ medium consequences, ■ severe consequences.

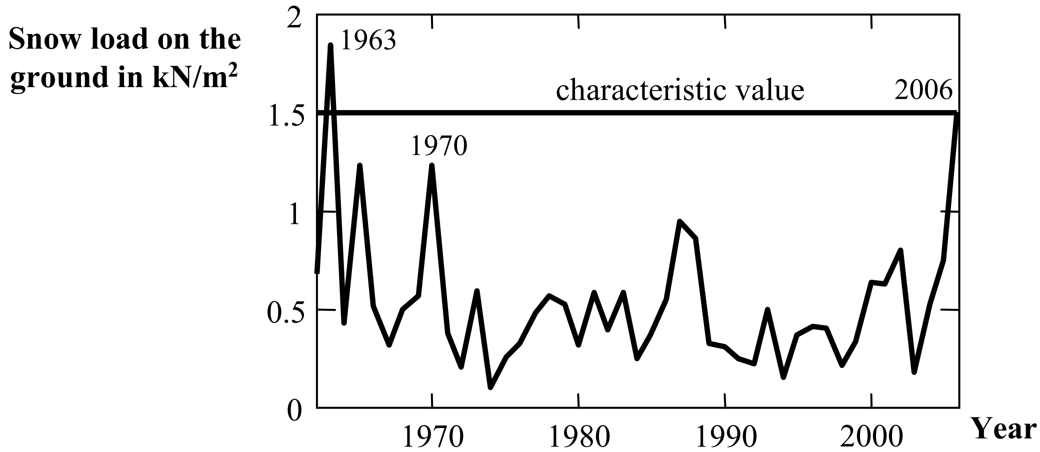

Figure 3. Annual maxima of snow load on the ground in Humpolec.

Annual maxima of snow load on the ground measured by the Czech Hydrometeorological Institute were analysed in detail. Four locations in the Czech Republic were selected. Three of them are in lowlands, close to large cities where a considerable number of new structures is constructed each year. The fourth one is in highlands (altitude 550 m) at the town of Humpolec where the ice-hockey stadium shown in Figure 2 collapsed under snow in 2006. Note that investigations of the stadium collapse were described in detail by Drdacky (2009). The time series of available measurements from Humpolec is indicated in Figure 3.

Point estimates of sample characteristics are estimated by the classical method of moments for which prior information on the type of an underlying distribution is not needed. It follows that means and standard deviations are different for various locations. The coefficient of variation varies within the narrow range from 0.65 to 0.7 while skewness varies between 1.0 and 2.0. However, the sizes of available samples (40-50) may be rather small to estimate credibly skewness.

The sample characteristics indicate that the annual maxima might be described by a two-parameter lognormal distribution having the lower bound at the origin (LN0) or by a more general three-parameter shifted lognormal distribution having the lower bound different from zero (LN). Other possible theoretical models are extreme value distributions: the type II called also the Fréchet distribution (Fre) or type I, a popular Gumbel distribution (Gum). Gamma distribution (Gam) is also considered.

Probability density functions of the considered theoretical models and a sample histogram are shown in Figure 4 for the measurements from Humpolec. To compare goodness of fit of the considered distributions, Kolmogorov-Smirnov and chi-square tests are applied. It appears that the lognormal distribution LN0 is the most suitable model. However, favourable test results are obtained also for the other models.

Appropriate models should be selected on the basis of the statistical tests considering general experience with distributions of annual maxima. It is often assumed that the Gumbel distribution might be a suitable model, ISO 4355 (1994) and Soukhov (1998). Therefore, this distribution is considered hereafter although its constant skewness 1.14 is in most cases lower than the sample skewness.

Theoretical characteristic values of the snow load on the ground are further determined as the 0.98 fractile of the Gumbel distribution with the sample mean and standard deviation. The obtained values are in a good agreement with those provided in the National Annex to EN 1991-1-3 (2003). More specifically, the fractiles of the theoretical model are mostly by about 5 % lower than the characteristic values in the Eurocode.

In two of the considered locations, the snow loads recorded in 2006 slightly exceeded the characteristic values given in EN 1991-1-3 (2003). In view of this, the snow loads observed in 2006 may hardly be considered as “extreme” or “exceptional”.

It should be mentioned that the submitted analysis is based on annual maxima determined from weekly measurements. Several methods have been proposed to estimate daily snow loads from weekly measurements. The detailed analysis conducted by Sadovsky et al. (2008) indicates that the theoretical characteristic value may be increased by about 5 % when daily values are taken into account. Moreover, it appears that trends of snow loads may often be neglected.

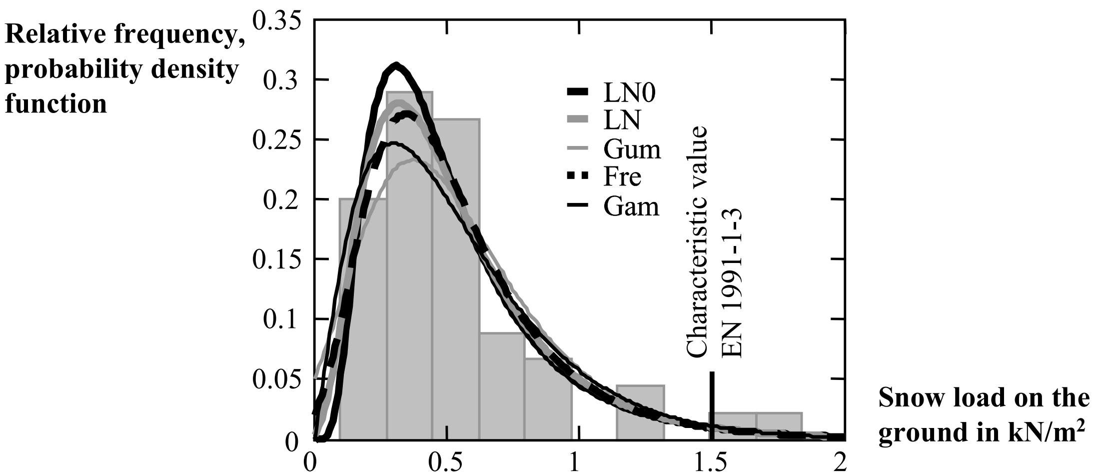

Figure 4. Histogram of the annual maxima and the selected theoretical models.

Reliability analysis

It is further assumed that a considered structure is designed in accordance with the principles of the present suite of the Eurocodes, EN 1990 (2002), EN 1991-1-1 (2003) and EN 1991-1-3 (2003). The characteristic load on the roof is determined as the product of the shape factor μ (for horizontal roofs equal to 0.8); exposure factor C_e; thermal factor C_t and characteristic value of the snow load on the ground s_k given in a snow map. The exposure and thermal factors are usually considered as unity, EN 1991-1-3 (2003), and hence are omitted hereafter.

Steel structural members are further considered only. Design of a member exposed to a permanent load G and snow load S can be based on the partial factor method described in EN 1990 (2002). Using the fundamental load combination (6.10), the design value of a generic resistance R of the member is determined from:

$$r_k / \gamma_{M0} = \gamma_G g_k + \gamma_Q s_{s,k} \tag{1}$$

where r_k is the characteristic value of resistance; γ_{M0} partial factor for resistance of a cross-section; γ_G partial factor for the permanent load; g_k characteristic value of the permanent load (equal to the mean value); γ_Q partial factor for the snow load; and $s_{s,k}$ snow load on the roof.

When structural members are not susceptible to stability phenomena, the partial factor for resistance is considered by the value 1.0 as recommended in EN 1993-1-1 (2005). Assuming unfavourable effects of the actions, the partial factor for the permanent load is 1.35 and for the snow load 1.5, EN 1990 (2002).

In the following effect of a load ratio on the reliability of structural members is analysed. The load ratio is defined as the characteristic value of the snow load on the roof over the total characteristic load:

$$\chi = s_{s,k} / (g_k + s_{s,k}) \tag{2}$$

For roofs, the realistic range of the load ratio may be from 0.4 up to 0.8. For a given load ratio and characteristic snow load on the roof, the characteristic permanent load follows from relationship (3):

$$g_k = s_{s,k} (1 - \chi) / \chi \tag{3}$$

Reliability of generic steel members is further analysed by probabilistic methods. The limit state function is written as follows:

$$\mathrm{g}(\mathbf{X}) = K_R R - K_E (G + \mu S_{50}) \tag{4}$$

Table 2. Models for basic variables.

Variable	Symb.	Dist.	Partial fac.	X_k	μ_X	V_X
Resistance	R	LN0	1.0	Eq. (1)	$r_k e^{2V_R}$	0.08
Permanent load	G	Norm.	1.35	Eq. (3)	g_k	0.10
Shape coefficient	μ	Norm.	-	0.8	0.8	0.15
Snow on ground	S_{50}	Gum	1.5	s_k	$\approx s_k$	0.22
Resistance unc.	K_R	LN0	-	-	1.15	0.05
Load effect unc.	K_E	LN0	-	-	1.0	0.10

where K_R denotes model uncertainties in structural resistance; K_E model uncertainties in load effect and S_{50} are 50-year maxima of the snow load on the ground (corresponding to the design working life 50 years indicated in EN 1990 (2002) for building structures and other common structures). Models for the basic variables accepted from Holicky and Sykora (2008) are described in Table 2.

In the reliability analysis, the probability of failure and corresponding reliability index are determined:

$$P_\mathrm{f} = \mathrm{P}[\mathrm{g}(\mathbf{X}) < 0]; \quad \beta = -\Phi^{-1}(P_\mathrm{f}) \tag{5}$$

where $\mathrm{g}(\mathbf{X}) < 0$ denotes failure; and Φ^{-1} is the inverse cumulative distribution function of the standardised normal distribution. The FORM method proposed by Hasofer and Lind (1974) is applied to estimate the indicators of a reliability level. Results of the reliability analysis are shown in Figure 5.

It follows from Figure 5 that the recommended values of the partial factors lead to a significant variation of the reliability index for all the considered locations. Moreover, for the considered range of the load ratio 0.4 – 0.8 the index decreases below the target value 3.8 recommended in EN 1990 (2002) and the reliability of a structural member seems to be insufficient. Recent study by Holicky and Sykora (2008) indicates that the partial factor for the snow load should be significantly increased.

It is, however, emphasized that generalization of these findings may be rather difficult. The resulting reliability is considerably dependent on the model uncertainties, which may differ for various types of members or structures under consideration. In addition variability of the snow load effect is significantly increased by uncertainties of the shape coefficient. The recent report by CEN/TC250 and JRC (2009) indicates that further research on the shape coefficient is desired.

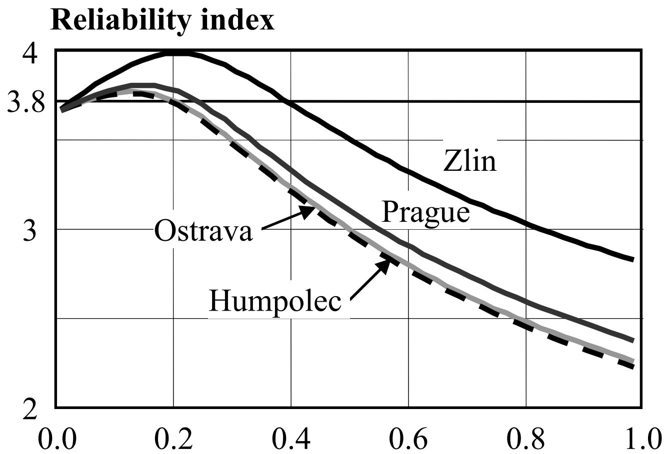

Figure 5. Variation of the reliability index with the load ratio.

CONCLUSIONS AND RECOMMENDATIONS FOR DESIGN

Presented experience gained within investigations of failures of roofs under snow load and analysis of their causes provides valuable background information for future forensic assessments. It is indicated that:

- The main observed causes of structural damage may be subdivided into errors in design, during execution and use, and insufficient code provisions.
- Collapses developed from local failures particularly in cases of insufficiently robust structures.
- In some locations snow loads recorded in 2006 slightly exceeded the characteristic values given in standards and thus may hardly be considered as "exceptional".
- Gumbel distribution is a suitable theoretical model for the annual maxima of snow loads on the ground.
- Reliability of roofs designed in accordance with the present suite of the Eurocodes is lower than the target level recommended in EN 1990 (2002).
- In the Czech Republic, characteristic values of the snow load on the ground well correspond to those derived from the theoretical model. However, the recommended values of the partial safety factor should be differentiated and in case of light-weight roofs increased.

It is emphasized that the presented results are significantly dependent on the assumed models for basic variables and should be considered as informative only.

Note that proper code provisions alone do not prevent collapses. Systematic quality control of design and execution needs to be applied.

In addition robustness aspects should be considered to reduce possible damage due to snow loads. Sufficient robustness may be achieved by an adequate system of ties, increased resistance of key members, secondary protection of key members and by appropriate structural detailing.

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REFERENCES

CEN/TC 250 Structural Eurocodes & JRC (Joint Research Centre) (2009). *The Eurocodes and the construction industry (medium-term strategy 2008 – 2013),* Report, January 2009.

Cieslar, S. (2006). "Mimoradna priloha cisla 2/2006 Havarie strech – nazory odborniku (in Czech - special annex of No. 2/2006 Collapses of roofs – views of experts)", *Konstrukce (Structures),* 2006(2), 1-16.

Drdacky, M. (2009). "Lessons from the failure of a steel-structure ice-hockey hall under a snow load", Forensic Engineering: From failure to understanding, Proc. Fourth Int. Conf. on Forensic Engineering, Thomas Telford, London.

EN 1990 (2002). *EN 1990:2002 Eurocode - Basis of structural design,* CEN.

EN 1991-1-1 (2003). *Eurocode 1: Actions on structures - Part 1-1: General actions; Densities, self-weight, imposed loads for buildings,* CEN.

EN 1991-1-3 (2003). *Eurocode 1: Actions on structures - Part 1-3: General actions; Snow loads,* CEN.

EN 1991-1-7 (2006). *Eurocode 1: Actions on structures – Part 1-7: General actions – Accidental actions,* CEN.

EN 1993-1-1 (2005). *Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings,* CEN.

Faber, M. H. et al. (2008). *Proceedings of the 1st Workshop of the Action TU0601 Robustness of Structures,* Reprozentrale ETH Hönggerberg, Zurich.

Hasofer, A. M. and Lind, N. C. (1974). "An Exact and Invariant First Order Reliability Format", *J Engrg Mech Div (ASCE),* 100(EM1), 111-121.

Holicky, M. and Sykora, M. (2008). "Safety of lightweight steel roofs exposed to snow load", Proc. EUROSTEEL 2008, ECCS Southampton, Brussels.

ISO 4355 (1994). *Basis for design of structures - Determination of snow loads on roofs,* ISO.

Kucera, V. (2006). *Zprava c. 010-019840, Cast I. Analyza pricin havarii stresnich konstrukci v zimnim obdobi 2005 – 2006 (in Czech - Report No. 010-019840, Part I. Analysis of Causes of Roof Failures in the Winter 2005 – 2006),* Ministry of Industry and Trade of the Czech Republic.

Sadovsky, Z. et al. (2008). "Special features of the collection and analysis of snow loads", Proc. ESREL 2008, Taylor & Francis, London.

Soukhov, D. (1998). "The Probability Distribution Function for Snow Load in Germany", *LACER,* 1998(3), 263-274.