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Models for exposure conditions – a review of available data for snow and flooding in the Czech Republic

Milan Holicky
Professor, Klokner Institute, Czech Technical University in Prague, Solinova 7, 16608 Prague 6, Czech Republic, tel.: +420224353842, fax: +420224355232, email: holicky@klok.cvut.cz

and

Miroslav Sykora
Researcher, Klokner Institute, Czech Technical University in Prague, Solinova 7, 16608 Prague 6, Czech Republic, tel.: +420224353850, fax: +420224355232, email: sykora@klok.cvut.cz

Abstract

Extreme events such as extraordinary snow falls or flooding may cause damage to structural members and should be considered in the assessment of structural robustness. Structural failures during the winter period 2005/2006 in the Czech Republic initiated discussions concerning the reliability of roofs exposed to a snow load. Reliability of light-weight roofs is analysed using probabilistic methods and available data and review of causes of structural failures is then provided. Significance of structural robustness was also identified during the flooding in Moravia (1997) and Bohemia (2002). Investigation of structural failures indicated that main causes of structural damage may be subdivided into geotechnical and structural reasons. Evaluation of hydrological data and probabilistic prediction of extreme discharges are then described focusing on differences among results of various statistical techniques.

1. Introduction

Basic European document for structural design (EN 1990, 2002) indicates that sufficient structural reliability can be achieved by suitable measures including ensuring an appropriate degree of robustness (structural integrity). In (EN 1991-1-7, 2006) robustness is defined 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.

However, recent discussions have indicated that robustness is a complicated concept that is not understood uniformly within engineering society. The following two approaches to definition of robustness seem to be useful:

- Definition in a narrow sense providing an indicator of the ability of a structure to perform adequately under accidental situation,
- Definition in a broad sense providing an indicator of the ability of a system containing a structure to perform adequately under accidental situation of the structure.

In the following text robustness is understood rather in the narrow sense than in the broad sense since it seems to be used more often by practical engineers in the Czech Republic. However, it is emphasized that precise definition should be clarified in the beginning of any discussion concerning robustness.

During its life-time a structure may be affected by different exposures that may cause damage to structural members. Such influences to be considered in the assessment of structural robustness may include extreme climatic effects such as snow falls and flooding. These exposures have recently caused unexpected damage to structures having insufficient robustness in the Czech Republic. Structural failures during the winter period 2005/2006 initiated discussions concerning the reliability of roofs exposed to a snow load. Reliability of roofs is thus analysed considering available measurements of snow loads and review of causes of structural failures is provided.

Significance of structural robustness was also identified during the flooding in Moravia (1997) and Bohemia (2002) when severe damage to insufficiently robust structures was observed. Available data for discharge maxima are re-analysed. Results of statistical evaluation of the hydrological data are briefly reviewed and prediction of extreme discharges using different theoretical models and statistical methods is discussed. Overview of investigation of structural failures indicates that main causes of structural damage may be subdivided into geotechnical and structural reasons.

2. Snow load

After the winter period 2005/2006, the reliability of roofs has become an important topic of structural design in the Czech Republic as well as in Europe. In some countries available measurements have been re-evaluated. Newly developed maps of snow loads take into account the principles of the present suite of Eurocodes (EN 1990, 2002; EN 1991-1-1, 2002 and EN 1991-1-3, 2004), specifying the characteristic value of snow load on the ground s_k as the 0.98 fractile of annual maxima. A critical analysis of presently accepted design procedures based on available measurements is further provided.

2.1 Partial factor design

The characteristic snow load on a roof is determined as

$$s_{s,k} = \mu\, C_e\, C_t\, s_k \tag{2.1}$$

where μ denotes the shape factor (for horizontal roofs equal to 0.8). C_e and C_t denote the exposure and thermal factors, respectively, considered usually as unity and omitted further on (EN 1991-1-3, 2004). In the design of a structural member exposed to a permanent load G and snow load S, the value of a generic resistance R is determined using the partial factor method and the fundamental load combination given in (EN 1990, 2002) by expression (6.10)

$$R_k / \gamma_M = \gamma_G\, G_k + \gamma_Q\, s_{s,k} \tag{2.2}$$

Here R_k denotes the characteristic resistance (0.05 fractile), γ_M the resistance partial factor (considered by the generic value 1.15), γ_G the partial factor of permanent load (considered by the recommended value 1.35), G_k the characteristic value of the permanent load (the mean of G) and γ_Q the partial factor of the snow load (considered by the recommended value of 1.5).

2.2 Basic variables

The reliability analysis of a structural member exposed to a permanent load G and snow load S is based on the limit state function g(**X**) given as

$$g(\mathbf{X}) = K_R\, R - K_E\,(G + S_{50}) \tag{2.3}$$

The basic variables **X** are described in Table 1. In accordance with (EN 1990, 2002), the design life time of 50 years is considered and, therefore, the 50 years' extreme S_{50} of the snow load $s_{s,k}$ on the roof (considering the shape factor μ = 0.8) is assumed in equation (2.3). In the following analysis, the load ratio χ is given as the fraction of the characteristic value of the snow load $s_{s,k}$ and the total load $G_k + s_{s,k}$

$$\chi = s_{s,k} /(G_k + s_{s,k}) \tag{2.4}$$

For a given χ and $s_{s,k}$, the characteristic permanent load follows from equation (2.4) as

$$G_k = s_{s,k}\,(1 - \chi) / \chi \tag{2.5}$$

The characteristic value of the snow load on the ground s_k is accepted from the revised map of snow loads in the Czech Republic for the area of Prague (ČSN EN 1991-1-3, 2006). Meteorological data provided by the Czech Hydrometeorological Institute for this area may be considered typical for a number of localities in the Czech Republic. Assuming the Gumbel distribution for annual maxima of the snow load, assessment of available data indicates that the mean of S_{50} is approximately 1.02 times the characteristic value $s_{s,k}$ indicated in (ČSN EN 1991-1-3, 2006) while the coefficient of variation of S_{50} is about 0.22. More details are provided by (Holicky et al., 2006).

Alternatively to the recommendation of (EN 1990, 2002), the partial factor of snow load γ_Q is proposed as a quantity dependent on the load ratio χ

$$\gamma_Q = 1 + \chi \tag{2.6}$$

as similarly suggested in the recent studies (Holicky, 2005 and Holicky and Retief, 2005) for the partial factor of variable actions. The partial factor γ_Q applied to the characteristic snow load $s_{s,k}$ further equals 1.50 or $1 + \chi$. The theoretical models of basic variables indicated in Table 1 are derived taking into account the principles of Eurocodes (EN 1990, 2002; EN 1991-1-1, 2002 and EN 1991-1-3, 2004), following recommendations of the Joint Committee on Structural Safety (JCSS, 2005) and considering results of previous studies (Holicky, 2005; Holicky and Retief, 2005 and Holicky et al., 2006).

2.3 Results of reliability analysis

Results of the reliability analysis are indicated in Figures 1 and 2. Figure 1 shows the variation of the reliability index β with the load ratio χ for the load combination in equation (2.2) and the two different partial factors γ_Q. It follows from Figure 1 that the constant partial factor γ_Q = 1.5 leads to a significant variation of the reliability index β with the load ratio χ. Moreover, for the ratio $\chi > 0.4$ the index decreases below the target value of 3.8 recommended in (EN 1990, 2002) and the reliability of a structural member may be insufficient. The partial factor γ_Q = 1.5 seems to be satisfactory for the load ratio $\chi < 0.4$ only. For the ratio $\chi > 0.4$ the partial factor for snow load γ_Q

should be obviously greater than 1.5. A more uniform reliability level is obtained for the proposed partial factor $\gamma_Q = 1 + \chi$.

Influence of the constant value of the factor γ_Q on β is indicated in Figure 2. It follows that the constant partial factor γ_Q should be increased up to values of 1.7 or 1.9 to achieve a sufficient reliability level for greater χ (the upper bound of 0.8 – lightweight roofs – may be realistic). Similar conclusions follow also from studies for combinations of snow load and wind action in Germany (Schleich et al., 2002 and Sadovsky, 2004).

The presented study indicates that Eurocodes may not guarantee an adequate reliability level in particular for lightweight roofs exposed to a snow load. This may be, therefore, one of the causes of structural failures during the winter 2005/2006 in the Czech Republic as well as in Europe. Extensive investigations of the damaged structures, however, indicated also a number of other causes of structural failures that should be considered in assessment of structural robustness.

2.4 Investigation of structural failures

A special issue of the Czech professional journal (Konstrukce, 2006) provided experts' views of causes of structural failures. It was mostly observed that considerably damaged structures had insufficient robustness (no tying, low resistance of key members or vulnerable structural detailing). Lack of robustness became important particularly in case of multiple causes of failures or failures of joints.

Exposures include:

- Extraordinary snow load: extreme snow load was observed in particular on structures where snow was required to be removed. In other cases significant load was caused by the combination of snow and ice on roofs.
- Additional loadings on structures: unexpected loadings were observed due to incompetent intervention into structures (removal or reduction of sectional areas of load bearing members, addition of new structural members), due to installation of new facilities (suspended ceilings, air conditioning etc.) or due to water on a roof (in case of insufficient maintenance and false details, e.g. roof parapets with inside drainage).

In a number of cases low structural resistance was caused by insufficient code provisions considered at a design stage or by errors in design and defects in execution. Procedures in codes may have guaranteed a rather low reliability in particular in the following cases:

- Use of high-strength materials: use of light-weight roof structures increases the load ratio χ and an insufficient reliability level may be obtained by the partial safety factor design as indicated in the preceding chapter.
- Improved heat insulation of roofs: use of high-quality materials for heat insulation of roofs may cause that snow on a roof is not melting but accumulating, often non-uniformly. In such cases the snow load model considered in design codes may fail.
- Snow load on roofs with a great pitch: in few cases snow load was present also on roofs with a great angle of pitch.

Low structural reliability was also caused by errors in design and by defects in execution:

- Gross errors at a design stage: these errors including design inconsistent with code provisions, incorrect loading widths, numerical errors, upgrading of structures without reliability assessment or no regards to experts' recommendations for strengthening were less frequent, but with more severe consequences.

– Other errors at a design stage: incorrect models for foundation conditions, local buckling of massive frames braced by insufficiently stiff roofs, torsional buckling, fracture of steel exposed to low temperatures and low resistance of joints were also identified as causes of structural failures.
– Inadequate quality control: inadequate quality control of design and construction enabled that errors at a design stage were not found and in some cases (timber structures) low-quality materials were used.

3. Flooding

A number of structures in the Czech Republic was affected by the flooding in July 1997 in Moravia and in August 2002 in Bohemia. In particular damage and destruction caused to insufficiently robust structures was on an unprecedented scale. The main observed causes of structural damage have been subdivided into geotechnical and structural aspects. The geotechnical causes include:
– Insufficient foundation (depth, width),
– Underground transport of sediments and man-made ground (propagation of caverns),
– Increased earth pressure due to elevated underground water.

The major structural causes cover:
– Insufficient structural robustness (no ring beams as indicated in Figure 3),
– Use of inadequate construction materials (unfired masonry units),
– Material property changes caused by moisture (volume, strength).

Consequences of the above-mentioned causes were observed also in Prague that was flooded in 2002 particularly badly. Water levels recorded in the city and its surroundings during the flooding seem to be exceptionally high. However, was the flooding really so exceptional and unpredictable? What was the actual return period corresponding to the measured amount of water?

3.1 Statistical evaluation of annual maximum discharges

Annual maximum discharges Q_i on the Vltava River in Prague measured by the Czech Hydrometeorological Institute since 1827 are further analysed using basic statistical methods (Ang and Tang, 1975) to answer these questions. Statistical characteristics of the annual maximum discharges are initially estimated by the classical method of moments for which prior information on the type of an underlying distribution is not needed.

The characteristics are given in Table 2 for the sample without and with the observation Q_{2002}. It appears that the sample mean, standard deviation and coefficient of variation are influenced by the discharge Q_{2002} rather insignificantly (the enhancing factor varies from 1.02 up to 1.07). However, the coefficient of skewness seems to be considerably affected by Q_{2002} as the enhancing factor is 1.22.

3.2 Probabilistic distributions

The characteristics provided in Table 2 indicate that the annual maxima might be described by a two-parameter lognormal distribution having the lower bound at the origin (LN0) or more universal three-parameter lognormal distribution (LN) having the lower bound (for a positive skewness) generally different from zero. Other possible theoretical models are extreme value distributions: the type II called also the Fréchet distribution (F) or type I, a popular Gumbel distribution (G) with the constant skewness of 1.14.

Probability density functions of the considered theoretical models and a histogram of the analysed measurements are shown in Figure 4. It is observed that the lognormal distribution LN0 describes well the investigated sample. To compare goodness of fit of the considered distributions, Kolmogorov-Smirnov and χ^2- tests are further applied. A hypothesis that a theoretical distribution fits well the sample distribution should be accepted under the condition

$$K_r = K_0 / K_p \leq 1 \ (\chi_r^2 = \chi_0^2 / \chi_p^2 \leq 1) \tag{3.1}$$

Otherwise the hypothesis should be rejected. In equation (3.1) K_0 denotes a test value; K_p critical value and K_r relative test value of the Kolmogorov-Smirnov test. Analogous symbols are used for the chi-square test.

Relative test values are listed in Table 3 for the sample without and with Q_{2002}. It can be seen that all the applied distributions meet the condition (3.1) in accordance with the Kolmogorov-Smirnov test. However, the chi-square test indicates that the measured frequencies significantly differ from the theoretical values for all the considered distributions. It appears that the lognormal distribution LN0 is the most suitable model. Less favourable test results are observed for the three-parameter lognormal LN and Fréchet distribution F and the worst test results are obtained for the Gumbel distribution as the fixed skewness of 1.14 may be rather small. If the discharge Q_{2002} is involved, the tests provide more favourable results for all the distributions, except for the Gumbel distribution.

It is noted that the test results are indicative only. Suitable models should not be selected on the basis of the statistical tests only, but also taking into account experience with discharges measured at other localities. Experience of the Czech Hydrometeorological Institute indicates that the lognormal distribution LN0 could be a suitable model. Therefore, this distribution is further considered in estimation of extreme discharges.

3.3 Parameter estimation

The method of moments applied in Chapter 3.1 to estimate the sample characteristics is often assumed to be not very efficient. Assuming that the underlying distribution of the sample is the lognormal distribution LN0, the sample characteristics can be improved by the maximum-likelihood method, which is considered as the most efficient method for parameter estimation, particularly for large samples. The maximum-likelihood estimators $\hat{\mathbf{q}}$ of unknown parameters θ of the distribution (here mean m and standard deviation s) are obtained maximizing the logarithm of a likelihood function

$$\max_{\mathbf{q}} \ln\left[\mathrm{L}\left(\mathbf{q}|\mathbf{Q}\right)\right] \rightarrow \hat{\mathbf{q}} \tag{3.2}$$

where $\mathbf{Q} = (Q_1,\ldots,Q_n)$ is the sample, $\mathbf{q}$ realization of the vector of the parameters θ and $\mathrm{L}(\mathbf{q}|\mathbf{Q})$ is the likelihood function

$$\mathrm{L}\left(\mathbf{q}|\mathbf{Q}\right) = \prod_{i}^{n} \mathrm{f}\left(Q_i|\mathbf{q}\right) \tag{3.3}$$

where f(•) is the probability density function of the underlying distribution. Here the non-linear conjugate-gradient method implemented in the software package Mathcad® is applied.

Comparison of distribution parameters estimated by the method of moments and the maximum-likelihood method is provided in Table 4. It appears that the estimates of the mean are nearly independent of the applied method (differences about 1 %). However, the standard deviations estimated by the maximum-likelihood method are systematically greater than those obtained by the method of moments (differences about 10 %).

3.3 Estimation of extreme values

Upper fractiles Q_p of the lognormal distribution LN0 are further estimated using the classical coverage method for the given confidence level γ, see e.g. ISO 12491 (1997)

$$P(Q_{p,\mathrm{cov}} > Q_p) = \gamma \tag{3.4}$$

In accordance with EN 1990 (2002), the characteristic value Q_k is obtained as the 0.98 fractile of annual maxima while the design value Q_d is the fractile of the life-time maxima corresponding to the probability

$$p_\mathrm{d} = 1 - \Phi(\alpha_E \times \beta) = 1 - \Phi(-0.7 \times 3.8) = 1 - 0.0039 \tag{3.5}$$

where Φ denotes the cumulative distribution function of the standardised normal distribution, α_E is the FORM sensitivity factor (considering the recommended value of -0.7 for the leading action) and β is the reliability index equal to 3.8 for the 50-years reference period. Partial safety factors γ_Q for unfavourable effects of variable actions are consequently obtained as the ratio $Q_\mathrm{d} / Q_\mathrm{k}$.

Estimated extreme discharges and partial factors are summarized in Table 5. It is indicated that the extreme values predicted from available data including the discharge Q_{2002} are greater than those estimated without this discharge (by about 9 % for the method of moments and 4 % for the maximum-likelihood method). It also appears that the extreme discharges predicted by the maximum-likelihood method are greater than those obtained by the method of moments (by about 6-9 % for the characteristic value and 12-19 % for the design value). Furthermore, the upper fractiles estimated considering the commonly accepted 0.75 confidence level are greater than the expected upper fractiles (by about 6 % for the characteristic values and 11 % for the design values). The partial safety factors $\gamma_Q \approx 3.0$ derived from the data seem to be significantly greater than the recommended value of 1.5.

3.4 Return period of the discharge Q_{2002}

Expected return periods T corresponding to the discharge Q_{2002} are further derived using the relationship

$$T = 1 / [1 - F(Q_{2002})] \tag{3.6}$$

where F(•) denotes the cumulative distribution function of the underlying distribution.

Expected return periods are listed in Table 6. It appears that the return period is considerably influenced by the fact whether the discharge Q_{2002} is taken into account or not. In addition the estimates based on the maximum-likelihood method are significantly lower than those obtained using the method of moments. Considering the data without Q_{2002} and the maximum-likelihood method, the observed discharge Q_{2002} = 5250 m^3/s corresponds to the exceptionally long return period of 210 years. Obviously the discharge Q_{2002} could have been hardly expected. Note that estimates of the return period may enormously vary with a type of the applied distribution as indicated by Holicky and Sykora (2004).

4. Conclusions

During its life-time a structure may be affected by different exposures such as extreme snow load or flooding that should be considered in the assessment of structural robustness. Results of reliability analysis of roofs and investigations of structural failures during the winter period 2005/2006 lead to the following conclusions:

- For the load ratio χ > 0.4 (typically light-weight roofs) the reliability index β of structures designed in accordance with Eurocodes is less than 3.8; the partial factor for snow load γ_Q should be in these cases greater than 1.5.
- Damaged structures had often insufficient robustness, which seemed to be particularly important in case of multiple causes of failures or failures of joints; these structures had no tying, low resistance of key members or vulnerable structural detailing.
- Exposures include extraordinary snow load and additional loadings on structures due to incompetent intervention into structures or due to water on a roof.
- Low structural resistance was caused by insufficient code provisions, by errors in design and by defects in execution.

Robustness is also an important aspect of flooded structures. Investigation of structural failures due to flooding in Moravia (1997) and Bohemia (2002) indicated that main causes of structural damage may be subdivided into geotechnical and structural reasons. Statistical analysis of available data for annual discharge maxima further shows that:

- Discharges may be well described by a two-parameter lognormal distribution LN0.
- The characteristic and design values for discharges predicted using data including the discharge in 2002 are greater than those estimated without considering the discharge in 2002 (by about 5 %).
- Extreme discharges predicted by the maximum-likelihood method are greater than those by the method of moments (by about 10 %).
- The recommended value of the partial safety factor γ_Q = 1.5 is considerably lower than the value derived from the available data ($\gamma_Q \approx 3.0$).
- The discharge observed in 2002 corresponds to an exceptionally long return period and, therefore, could have been hardly expected.

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Table 1. Models of basic variables.

Variable	Symb. X	Distr.	Partial factor γ	Char. val. X_k	The mean μ_X	CoV V_X
Resistance	R	LN	1.15	From (2.2)	$1.25R_k$	0.10
Permanent load	G	N	1.35	From (2.5)	$1.0G_k$	0.10
Snow, 50-years max.	S_{50}	GU	1.5 or 1 + χ	$s_{s,k}$	$1.02s_{s,k}$	0.22
Resistance uncertainty	K_R	N	-	-	1.0	0.05
Load effect uncertainty	K_E	N	-	-	1.0	0.10
"N" - normal distribution, "LN" - lognormal distribution with the lower bound at the origin, and "GU" - Gumbel distribution of maximum values.						

Table 2. Sample characteristics of the annual maxima in m^3/s (sample size n = 165 or 166).

Sample characteristic	Formulae used in the analysis	Without Q_{2002}	With Q_{2002}	Enhancing factor
Mean	$m = \frac{1}{n}\sum_{i=1}^{n} Q_\mathrm{i}$	1197	1221	1.02
Standard deviation	$s = \sqrt{\frac{1}{n-1}\sum_{i=1}^{n}(Q_\mathrm{i}-m)^2}$	787	846	1.07
CoV	$v = \frac{s}{m}$	0.66	0.69	1.05
Coefficient of skewness	$w = \frac{n}{(n-1)(n-2)s^3}\sum_{i=1}^{n}(Q_\mathrm{i}-m)^3$	1.43	1.74	1.22

Table 3. Results of the Kolmogorov-Smirnov and chi-square tests.

Probabilistic distribution	Without Q_{2002}		With Q_{2002}	
	K_r	χ_r^2	K_r	χ_r^2
Lognormal distribution LN0	0.53	1.11	0.49	1.01
Lognormal distribution LN	0.73	1.26	0.65	1.16
Fréchet distribution F	0.82	1.35	0.73	1.30
Gumbel distribution G	0.85	1.43	0.86	1.63

Table 4. Estimated parameters.

Method	Without Q_{2002}		With Q_{2002}	
	m	s	m	s
Moments	1200	790	1220	850
Maximum-likelihood	1210	870	1230	910

Table 5. Estimated extreme discharges in m^3/s.

Method	Characteristic value (p_k)				Design value (p_d)				Partial factor γ_Q
	Without Q_{2002}		With Q_{2002}		Without Q_{2002}		With Q_{2002}		
	Expect.	$\gamma =$ 0.75	Expect.	$\gamma =$ 0.75	Expect.	$\gamma =$ 0.75	Expect.	$\gamma =$ 0.75	
Moments	3430	3640	3630	3860	9650	10640	10700	11840	2.82 – 3.07
Maximum-likelihood	3710	3950	3830	4090	11360	12620	11960	13310	3.07 – 3.26

Table 6. Expected return periods in years.

Method	Without Q_{2002}	With Q_{2002}
Moments	350	240
Maximum-likelihood	210	180

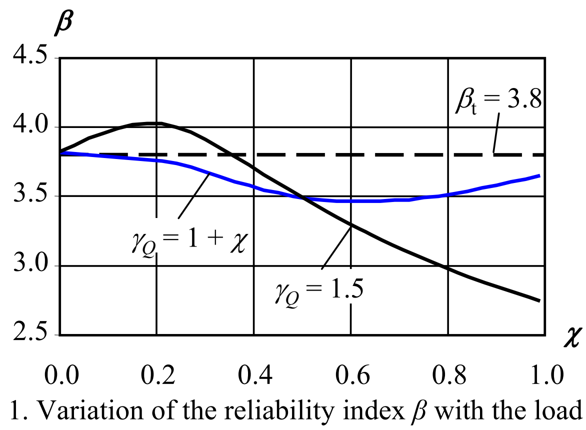

Figure 1. Variation of the reliability index β with the load ratio χ.

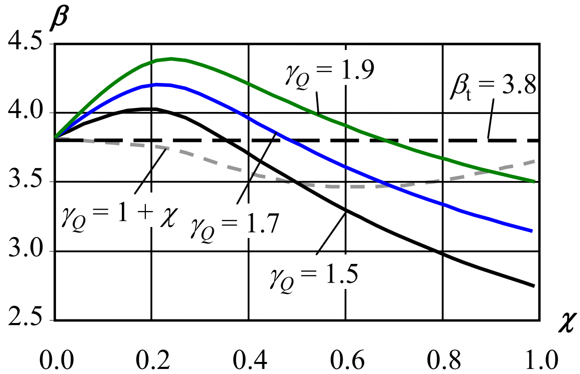

Figure 2. Influence of the factor γ_Q on the reliability index β.

Figure 3. Failure of a structure without ring beams.

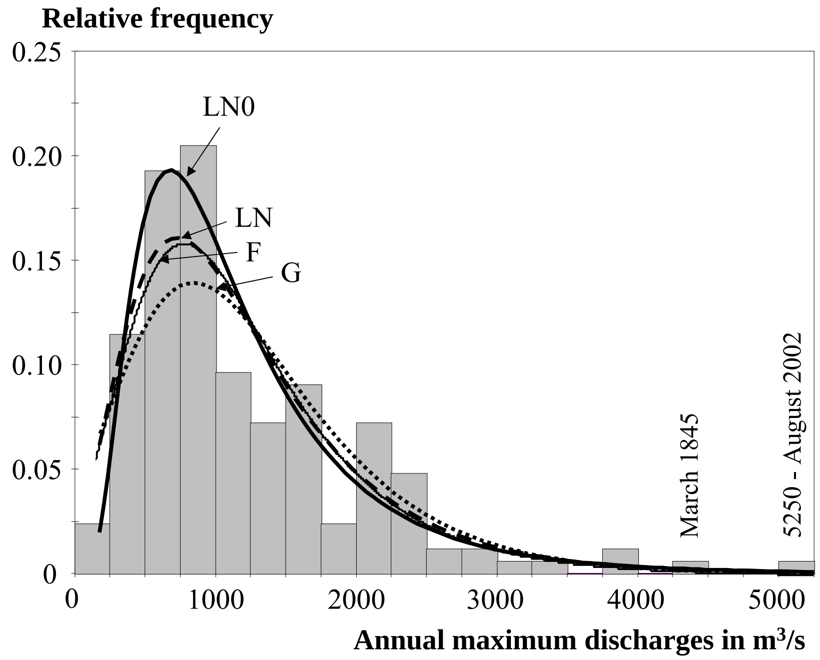

Figure 4. Histogram of the annual maxima and the selected probabilistic distributions.