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Universität für Bodenkultur
Department für Bautechnik + Naturgefahren
Institut für Konstruktiven Ingenieurbau

3rd Probabilistic Workshop Technische Systeme Naturgefahren

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PREFACE

Modern engineering structures should ensure an economic design, construction and operation of structures in compliance with the required safety for persons and the environment. In order to achieve this aim, all contingencies and associated consequences that may possibly occur throughout the life cycle of the considered structure have to be taken into account. Today, the development is often based on decision theory, methods of structural reliability and the modeling of consequences. Failure consequences are one of the significant issues that determine optimal structural reliability. In particular, consequences associated with the failure of structures are of interest, as they may lead to significant indirect consequences, also called follow-up consequences.

However, apart from determining safety levels based on failure consequences, it is also crucially important to have effective models for stress forces and maintenance planning.

The present contributions of this proceeding covers both issues of maintenance and assessment of structures from a traditional engineering perspective and issues of natural hazards. Models developed in recent years are discussed, including algorithms for assessing uncertainty in the domains of stress forces and resistance for maintenance- and risk planning.

In traditional engineering, the identification of cost efficient maintenance strategies for structures usually has its basis in condition assessments achieved through inspections, tests and monitoring. It is well recognized that such condition assessments are subject to significant uncertainties and, in general, at best provide indications rather than observations about the condition of the structure. Probabilistic frameworks for the quantification of inspection, of predictable future degradation, of estimated remaining service life and the expected service life costs of the structure are some of the topics of the contributions.

One possibility to mitigate natural hazards is the the implementation of technical protection measures. In this case the same considerations as above about safety assessment and associated uncertainties are valid. For the risk analysis of natural hazards such as gravitational mass movements important steps include the determination of the magnitude and frequency of hazardous events, the delineation of the endangered area (often equal to the runout zone), and the estimation of relevant intensity parameters such as impact pressure.

This volume includes contributions which discuss uncertainties related to the design of technical protection structures against natural hazards. Other contributions deal with uncertainties which are involved in the hazard assessment and risk analysis of processes such as floods, landslides, debris flows, and snow avalanches.

The Authors

Vienna, November 2005

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SAFETY ISSUES IN CIVIL ENGINEERING

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1. INTRODUCTION AND TERMS

1.1 INTRODUCTION

Structures have to be safe. This is one of the fundamental demands to the technical product structure. In general the public does not notice the question of sufficient reliability of structures compared to other technical products. Safety is used unlike as for products as cars and airplanes, where it is an common advertising point. Obviously the safety of structures is appreciated as sufficient by the public. Therefore no need for a rise of the safety requirements exists. This means that, the safety requirements for technical products for civil engineering are fulfilled in an excellent manner during the last decades in the developed countries.

Certainly structures exposed to extreme actions like earthquakes are exceptions. Nevertheless the public accepts a certain degree of failure of the claim for safety for such actions, even if considerable improvements were made during the last decades.

1.2 SAFETY

The term of safety characterizes the degree of conservation or achievement of a specified state. The claim for safety by beings or creatures in general includes perpetuation of the vital functions. The safety of people, namely the maintenance of vital physical and mental function of a being, complies with fundamental right of man.

For civil engineering safety is defined as the qualitative ability of a structure to withstand an action (DIN 1055-100[3], DIN ISO 8930, appendix 1.2 [12]). Of course a structure is not able to withstand all theoretical actions, but it has to withstand most actions to a sufficient degree (DIN 1055-9 5.2 (2) [4]). The decision if the structure is safe or not has to be adduced in a quantifiable degree. This is interpreted by the actual design codes as probability (et al. DIN ISO 8930, 1.1 & 1.2 [12]). Thus a conclusion, if a structure is safe or not, is possible by comparing probabilities. Other safety concepts are objects of research, like fuzzy-probability safety concepts (Möller et al. [[17]]). A possible scale of safety concepts can be found in figure 1.

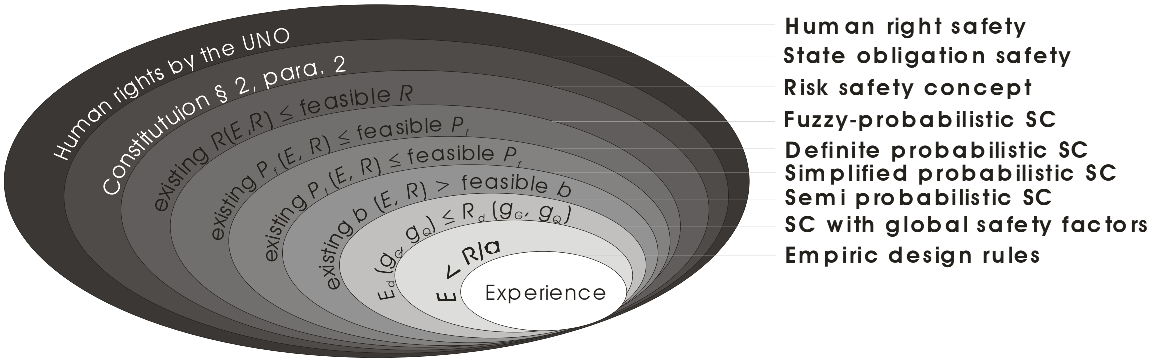

Fig. 1: Definition of failure probability

Safety is considered as adduced, when the existing risk is not exceeding a risk of a similar situation, which is accepted by the society (DIN 1055-9, Abs. 5.1 (3) [4], Eurocode 1 [6]).

1.2 RISK

In general the term risk describes the possibility of the failing of an operation in consideration of the consequences. The term judges therefore a possible state, which is connected with an impairment or loss. Thus the term is always connected with a valuation of events and it will be difficult to describe it only by objective and measurable characteristics. It considers most often as well subjective judgments [13], [18].

Mathematical risk is defined in a classical sense as product of frequency of occurrence H alternatively probability of a damage causing event and the damage or consequence K, which is caused by the event (DIN IEC 56 410, VDI 4006, ISO/IEC Guide 73):

$$R = H \cdot K$$

The technical definition of risk in civil engineering coincides with this mathematical definition (e.g. CEB [2]). Nevertheless numerous additional definitions can be found in different specific fields (e.g. ISO 10006, IEEE Standard P 1540, NASA, EPA ANSI), which differ only in the consideration and definition of damage [11]. The definitions of the term risk can be classified according to [14]:

1. Risk as probability of damage
2. Risk as degree of damage
3. Risk as function of probability and degree of damage
4. Risk as variance of the probability distribution of all consequences of a decision
5. Risk as semi-variance of the probability distribution of all consequences of a decision
6. Risk as weighted linear combination of the variance of the probabilities expectation of all consequences of a decision

1.2 HAZARD

Colloquial hazard is a special form of risk. Thus hazard is recognized as the impairment of health or the loss of life. Indeed both terms, hazard and risk, are prognostic, but colloquially the probability of occurrence is bigger for hazards.

Technical regulations use slightly different definitions. As hazards are a potential source of damage according to DIN EN 61508-4 [5]. DIN VDE 31000 part 2 [7] relates hazards to circumstances, when the risk is bigger than its limitation.

In the field of risk research an additional definition exists, which emphasizes a more defined difference between risk and hazard. While hazard, for the purpose of this definition, is based on external, dispositional perspectives of negative consequences, risk includes internal, subjective perspectives. Hazards exist therefore as e.g. latent negative environmental property. Thunder includes the danger of lightning strokes. No preventive action can be carried out against the possibility of a lightning stroke. But risk resulting from the hazard of a lightning stroke like fire can be decreased e.g. by installing a lightning conductor. As per description the term of risk includes a certain scope of decision. Therefore the term risk is the basis for the valuation and choice of actions [16].

2. SAFETY OF STRUCTURES

Civil engineering structures are highly reliable systems, which provide no representative amount of failures to derivate a failure probability. Their failure is a rare incident and therefore the failure probability is very low (Spaethe [19]). In addition every structure is a prototype without data of failure on hand for a statistic evaluation of damage. Nevertheless structures consist of a set of components, which are characterized by their properties. Considering, that these properties are no determinants, but are subject to a certain scattering, allows assessing the probability of failure for a structure. Basic principles for such evaluations have to be:

1. Sufficient statistic data must be available for the basic variables.
2. Clarity of the mechanical model describing the interaction between the inner resistance and the outer action to the structure must be given.
3. A limit state function must be given to refer the probability of failure
4. An accepted level for the probability of failure

Consequently an objective assessment of the probability of failure requires an evaluation by comparable models. Eichinger [8] defines in this context the operative probability of failure: “The operative probability of failure is that theoretical probability, which can serve the engineer as value of comparison and decision support for the quantification of predictions about the safety and reliability of structures.” Design codes within the European standards follow semi-probabilistic partial safety factor concepts based on these considerations. These standards proof the criteria of the operative probability of failure in a diminished form by the use of characteristic values and partial safety factors.

The assessment of the probability of failure may be conceived as special form of risk study, which examines as damage event the failure of structural parts or structures.

2.1 HAZARDS OF STRUCTURES

Most of the actions on structures are sufficiently defined and covered by design codes. In developed countries only very rare failures of civil engineering structures will be caused by dead load of even predicted traffic load. Regarding these actions reliability of a structure will be questioned mostly because of asserted or assumed degradation or damage to the structure and exposure to changed actions. In addition the use of new materials or designs can require detailed reliability analysis, when they are not covered by existing codes.

The biggest hazard potential comes from extreme actions, which a structure has to withstand most certainly never during its lifetime. The following list major hazards most notably because of their potential connected with civil engineering structures:

Extreme weather conditions:
Early August 8th, 1975 broke the Shimantan barrage during an extreme Taifun in the Henan-province, China. The flood wave hit half an hour later the bigger Banqiao barrage and broke it. In the following 60 more barrages broke by the flood wave. Within 24 h app. 86.000 people died and more than one million people were stuck in a devastated and flooded area. Additional 145.000 people died within the following weeks because of consequent epidemic [15].

Earthquakes:
At 5:46 January 17th, 1995 an earthquake of a strength of 7,2 according Richter scale hit the city of Kobe, Japan. Wide areas of the city have been destroyed and 6.433 people died. The Hanshin Expressway, supposed to withstand earthquakes of this kind and one of the main traffic routes of the city, broke at several sections. One of the biggest earthquake (8.2 Richter scale) occured in Tangshan (China) on July, 28 in 1976, where more than 250.000 people died.

Landslides:
At 22:39 October 9th, 1963 one mountainside of mount Toc slide into the reservoir of Vajont, Italy. 260 million cubic meter rushing into the water forced 50 million cubic meter of water to a flood wave, destroying the surrounding of the reservoir and washing over the crest into the valley. The barrage remained unbroken, but 2.100 people died by the flood wave.

Fire:
March 1999, 24th a truck passing the Mont-Blanc-Tunnel from France to Italy catches fire. The truck driver stops his vehicle in the middle of the tunnel and absconds on foot. As well several other vehicles, driven against the truck, catch fire. Finally it took 53 h to get the fire under control. 39 people died and it took almost 3 years to reopen this important bottle neck crossing the Alps. Another big accident occurred at the Kitzsteinhorn, where in a rope driven rail cabine 155 people died in a smoke and fire environment.

Explosion:
September 21st, 2001 a plant for fertilizers in Toulouse, France, storing 400 t of ammonium nitrate, explodes. Within a radius of 350 m of the plant 30 people died and a total of 2.240 people were injured. The economic total loss was estimated to 2,3 Billion Euro [10].

Impact:
At 7:38 Mai 9th, 1980 the freighter Summit Venture hits the pier n° 2 of the Sunshine Sky Bridge crossing the Tampa bay, USA. Almost the entire middle part of the cantilever bridge fall 50 m down into the sea. With it several vehicles including one Greyhound bus fall and 35 people died.

3. MODELING OF CONSEQUENCES

At first, consequences occur as an amount of lost, damaged or destroyed assets such as goods, services and lives. The amount of such assets also defines the intensity of the consequence. When assessing risks related to the failure of a building, the amount of the assets mentioned above can be known precisely or with a negligible variability when the structure fails completely. Such amount may be summarized in the random vector $\mathbf{a}= (a_1, a_2, \ldots, a_n)^T$, where n is the number of considered consequences. For some variables a_i it may be sufficiently accurate to use expected values rather than describing the individual variables by their distribution type and the corresponding parameters. This approach is especially useful when an inventory list is already available. Furthermore, a damage factor d_i may be associated to each element $\boldsymbol{\varepsilon}_i$ of the inventory list, which is represented by the vector $\boldsymbol{\varepsilon}$. For specific hazards it is possible to interrelate the damage factor with hazard specific measures.

In a risk analysis all consequences which are relevant and meaningful for the underlying decision making have to be identified. If a priori the relevance of a consequence type can not be estimated, it should be considered in a first approach. A sensitivity study will then reveal its relevance by showing its influence on the optimum decision.

All consequences should be expressed in monetary units. This is easy to achieve for all losses. When fatalities are considered the Life Quality Index and the Societal Life Saving Costs may be used as a basis for assessing the corresponding monetary loss to society.

4. CONCLUDING

The massive enlarged distribution of information in the public caused a more intensive notice of hazards and risks by all social levels of the public. More critical judgment of possible hazards by introducing new technical practices results in higher demands of safety, which are connected to additional cost. These constrain lead to the question of effectiveness of protective measures. But the effectiveness may be valuated only, when the possible damages are considered and shall be limited or totally prevented by them. The consideration of damages leads to the term of risk and therefore to risk assessment. The application of risk assessment as a degree for valuation of efficiency of protective measures can be observed in many social areas independent of the specific field. Already long-time in medicine, sociology and nuclear engineering, it has been worked with risk parameters as a decision support for the selection of protective measures.

The selection of protective measures may be not limited to a certain field. Rather all protective measures within the society have to compete with each other. Therefore the risk assessments, which have to valuate the protective measures, have to be multidisciplinary as well. This awareness became widely accepted. Numerous studies, like the KATANOS/KATARISK-study in Switzerland [1], the BMBF-joint project of the German research network natural hazards (DFNK) in Germany [9], the graduated course of lecture “Natural Hazards” or the DFG project InterRisk or examples.

The development of multidisciplinary holistic theories and risk survey and statement is not finished now. Nevertheless risk parameters are an important tool for the valuation of hazards and protective measures. Because civil engineering structures have extensive impact on public safety, the civil engineer has to deal with their risks and dangers and consider them for the design and the construction, as well as for the maintenance and use of these structures. Therefore risk analysis is of special interest for civil engineering structures. Many tools, like the Modelcode of the JCSS, are available nowadays for the evaluation of operative failure probability of a structure as a part of the risk formulation.

The application of risk assessment is a modern tool for the valuation of safety and hazard potential for civil engineering structures. They proof explicit the high responsibility, which is taken by the civil engineer in the society.

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LIFE-CYCLE ANALYSIS AND OPTIMIZATION OF CIVIL INFRASTRUCTURE UNDER UNCERTAINTY

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ABSTRACT

A life-cycle analysis facilitates generation of cost-effective competitive solutions for designing and managing civil infrastructure from a long-term economical point-of-view. With this approach, one can appropriately balance the competing objectives of reducing life-cycle cost and enhancing lifetime performance. In this paper, the recent advances made in life-cycle analysis for design and maintenance of civil infrastructure are reviewed. Relevant methods for representing the various sources of uncertainty associated with time-dependent structural capacity and load demand are compared. Simultaneous consideration of multiple and conflicting objectives is emphasized. Optimization techniques for solution improvement are provided. This paper is ended by stressing the importance of integrating health monitoring technologies into life-cycle management of civil infrastructure.

1. INTRODUCTION

Civil infrastructure usually has a service life of several decades or even longer. It is constantly subject to hazards of various types, including natural and manmade disasters. Lifetime safety must be assured through cost-effective risk-mitigation strategies. At the same time, safety and condition of civil infrastructure have been undergoing creeping deterioration due to combined effects of material aging, harsh environmental stressors, and ever-increasing live loads. This long-term performance deterioration may have significant social, economic, and political consequences as a result of in-service functional impairment under normal operation and catastrophic failure under extreme loads. The associated consequences can be enormous.

Life-cycle analysis (LCA) provides a unifying approach to evaluating the effects of various time-dependent factors on the lifetime performance and on the relevant consequences that are possibly accumulated over the entire or remaining lifespan of a system. For civil structures, LCA may involve a variety of activities and interventions including construction, operation, inspection, monitoring, maintenance, repair, and replacement. Therefore, one is able to compare alternative design or preservation solutions from a long-term prospective. This is achieved by desirably balancing conflicting objectives of reducing initial/lifetime investment and improving overall performance (i.e., reducing consequences). In addition, by demonstrating the long-term cost-effectiveness over traditional materials and systems, the LCA-based methodology encourages application of novel methods, materials and systems to designing and preserving civil structures.

LCA of civil infrastructure includes a series of important steps, such as (i) modeling of impacts of short-term extreme loads and long-term deterioration on lifetime structural capacity, and (ii) prediction of structural safety and performance evolution. The complex and uncertain structure deterioration problem is further affected by uncertainties associated with inspection and maintenance interventions. Therefore, probabilistic treatment becomes necessary. Various sources of uncertainty have to be considered in modeling time-dependent variation of structural capacity and demand and thus in predicting lifetime structural safety and performance. In general, there are two types of uncertainty: aleatory and epistemic (Wen et al. 2003). The aleatory uncertainty results from intrinsic randomness of nature and the epistemic uncertainty results from lack of sufficient knowledge and/or perfect mathematical models.

This paper reviews recent accomplishments and necessary techniques for conducting LCA of civil infrastructure. In this process, the lifetime safety and performance are assessed and predicted probabilistically. Analytical modeling, numerical simulation, and Bayesian updating are all important ingredients in this endeavor. Necessary techniques of time-varying structural capacity and load demand are presented. Applications of LCA and optimization techniques to generating cost-effective solutions for maintaining satisfactory lifetime performance civil infrastructure are presented. Limitations of the current research and practice in LCA are pointed out and future research needs to meet these challenges are outlined.

2. LIFETIME DESCRIPTION OF STRUCTURAL CAPACITY AND LOAD DEMANDS

Prediction of long-term performance deterioration necessitates the identification of major deterioration mechanisms and simulation of the associated structural deterioration. Concrete and steel are two of the most common materials for civil construction. The structural components deteriorate progressively when affected by harsh environmental stressors. For steel corrosion in concrete, chloride and carbonation contamination are most frequently observed, which are followed by cracking and spalling-induced debonding of rebars due to internal pressures. These lead to the formation of rusts and hence loss in rebar cross section. Fatigue is another deterioration mechanism for structures subject to fluctuating loads, leading to progressive damage of concrete and steel. Various mathematical models have been developed to simulate the environmentally induced structural deterioration (e.g., Bažant 1979, Ellingwood and Mori 1997). For example, Fick’s second law of diffusion is often used to simulate chloride-induced concrete deterioration, based on which the corrosion initiation time can be obtained. After that, different deterioration models are proposed to predict the percentage of corrosion in steel. Relevant parameters may be treated as random variables to account for various sources of uncertainty.

3. PREDICTION OF PERFORMANCE DETERIORATION

Significant uncertainties are present in predicting deterioration and its propagation over time. There are a variety of prediction models. The failure rate function approach uses a lifetime distribution to represent the uncertainty in the time to failure of a component or structure. A structure can be in a range of states depending on its degrading condition. A serious disadvantage of failure rates is that they cannot be measured. The Markov deterioration model assumes that the component condition can be described by a limited number of condition states. Therefore, this model is flexible to be adapted to visual inspection data and is used in most bridge management systems. Markov chain models with stationary transition matrices are used to depict the deterioration of structure components over time. Another alternative is the stochastic process model, which may model the impacts of different levels of inspections realistically (Kallen and van Noortwijk 2003).

In contrast, the advanced structural reliability method provides a more rigorous and systematic approach to quantifying the evolution of bridge safety (e.g., Melchers 1999; Frangopol 1999; Val et al. 2000) provided probabilistic information on structural resistance, traffic loads, environmental stressors, and other necessary contributing factors is readily available. Experience gained in different countries shows that the major part of the work on existing bridges depends on the load carrying capacity (or structural reliability) of the bridge system rather than the condition states of the bridge elements alone (Frangopol and Das, 1999). Consequently, bridge management systems have to consider bridge reliability deterioration. The reliability profile is defined as the variation of the reliability index with time (e.g., Thoft-Christensen 1996; Estes and Frangopol 1996; Nowak et al. 1998). An advantage of reliability-based maintenance/management is that the reliability is explicitly taken into account. A disadvantage is that the effects of maintenance (e.g., lifetime extension) on the reliability index are difficult to estimate.

4. INDICATORS FOR QUANTIFYING LIFETIME PERFORMANCE

The life-cycle cost is mostly used measure to generate cost-effective design and maintenance management solutions of civil infrastructure. In particular, life-cycle cost minimization criterion is also widely adopted in maintenance optimization of civil infrastructure (e.g., Frangopol et al. 1997; Hawk 2003). Multiple and conflicting objective functions need to be considered simultaneously in order to obtain a well-balanced solution (Liu et al. 1997; Miyamoto et al. 2000; Furuta et al. 2004; Liu and Frangopol 2005a,b). For example, bridge management decisions should be made by improving the overall bridge network performance and reducing various long-term costs (e.g., agency cost, user cost) while ensuring satisfactory safety and condition levels of individual bridges in the highway network (Adey et al. 2003; Liu and Frangopol 2005c,d).

5. UPDATING PERFORMANCE PREDICTION

Due to uncertainty propagation over time, deterioration prediction using the numerical models alone cannot produce very accurate results. In order to resolve this problem, the structural condition and structural performance need to be monitored on a regular basis and the structural defects as symptoms of deterioration can be detected and corrected if necessary. If new information on structural performance is available, for example, by inspection and monitoring, one can update the performance assessment and prediction using Bayesian techniques. The present bridge condition systems are mainly based on visual inspections augmented by in-depth inspection. Risk and reliability based inspection programs have been recently developed to reduce the possible economic consequences due to structural failure under normal operational conditions (e.g., Faber and Sørensen 2002; Onoufriou and Frangopol 2002).

With the rapid development of information and sensing technologies, the structural health monitoring (SHM) has received extensive research in various disciplines such as civil, mechanical, and aerospace engineering. In civil engineering, SHM has been mainly applied to essential infrastructure systems such as highway bridges due to their immediate importance to the society. SHM is used to automatically detect, locate, quantify, and assess the level of structural damages and/or deterioration, based on changes in salient response features, as measured by deployed sensor arrays, due to extreme loads and/or progressive long-term deterioration. The potential successful civil application of SHM technologies, however, is contingent upon a thorough knowledge of their cost-effectiveness as opposed to traditional inspection methods in terms of unit cost, applicability, operation, resolution, capability, and long-term durability, among other relevant concerns. In particular, the ability of SHM to monitor corrosion and crack-induced debonding of steel reinforcement, which are among major causes for condition deterioration of reinforced concrete bridges, is of special interest. For example, the distributed piezoelectric actuator/fiber Bragg gratings sensor network is found useful to monitor and assess such deterioration (e.g., LANL 2003). Bayes theorem provides a rational method for incorporating the prior information or judgment into prediction of future outcomes. Probabilistic models for the structural components and load demands can be updated using Bayesian theory; the structural system reliability can be then be recalculated accordingly (Enright and Frangopol 1999, Estes and Frangopol 2004). Prediction of deterioration of bridges develops a baseline deterioration rate that can be updated as monitored data become available.

6. MAINTENANCE OF DETERIORATING CIVIL INFRASTRUCTURE

Deterioration of civil structures can exert serious, widespread, and prolonged impacts on various societal sectors. In order to ensure satisfactory long-term safety and performance, both preventive (i.e., proactive) and corrective (i.e., reactive) maintenance interventions need to be carried out in a timely and adequate manner in order to mitigate progressive deterioration and for correcting major structural defects. The available maintenance resources, however, have far been outpaced by the maintenance demands. To resolve this situation, decision support tools for maintenance management are developed to cost-effectively allocate maintenance resources to deteriorating civil structures. Most existing bridge management systems (BMSs) utilize visual inspection-based condition state to quantify and predict bridge performance. The minimum life-cycle cost criterion is commonly used to determine a single optimum maintenance management solution (Pontis 2001; Hawk 2003). Recent studies show that cost minimization alone may not necessarily lead to long-term bridge performance levels adequate to meet bridge managers' specific requirements (Shepard et al. 2004). Therefore, more rational bridge management decisions should be made by considering the following aspects (e.g., Liu and Frangopol 2005a-d): (i) the actual structural capacity under deterioration should be accurately modeled; (ii) improving long-term structure performance and decreasing life-cycle expenditures need to be simultaneously considered; and (iii) maintenance management should be conducted from an overall system perspective while ensuring satisfactory performance of individual components within the system.

7. CHALLENGES AND FUTURE RESEARCH NEEDS

One research challenge is to investigate the effect of long-term deterioration on structural performance in extreme environments. Considerable research has been conducted towards understanding of structural performance subject to extreme loads and development of improved design and reactive retrofit standards in order to mitigate such risks (e.g., Ghosn et al. 2003). Although the negative effects of long-term gradual deterioration on structural performance in extreme environments have long been recognized (e.g. Chang and Shinozuka 1996), systematic investigation of such effects and relevant methodologies for enhancing structural capacity against extreme loads are lacking. Rather, the structural capacity is usually assumed invariant between occurrences of extreme events when reactive retrofit is carried out to partially or fully repair the induced damages. This assumption does not reflect the realistic structural capacity deterioration over time and therefore the damages and consequences caused by such loads are estimated on the unconservative side. This would unfortunately lead to erroneous risk assessment and preparedness decisions.

Therefore, it is important to evaluate and predict structural performance with full consideration of unavoidable and detrimental capacity deterioration and subsequently to enhance life-cycle structural capacity through well planned proactive maintenance actions. LCA is needed to assess the impacts of gradual deterioration on capacity of civil structures, based on the assumption that realistic level of safety under extreme loads can be reliably predicted (Ellingwood and Song 1996). This requires accurate modeling of component deterioration that is responsible for the decrease of structural capacity against the type of extreme load under consideration. In addition to reactive retrofit immediately after extreme events occur, proactive maintenance interventions are important to retain a satisfactory capacity level over the specified time horizon. The reliable decision on cost-effective allocation of maintenance resources relies on the time-dependent prediction of structural capacity with and without maintenance.

Another interesting research topic is the long-term cost-effectiveness of smart materials and systems for civil structure applications. Facing the possible tremendous repair and maintenance expenditure and the associated consequences over the life-cycle, civil engineers have recently been seeking alternative approaches to ensuring satisfactory lifetime performance. Smart materials (e.g., optical fibers, piezoelectric materials, shape memory alloys, cementitious/polymeric materials) that have sensing, actuating, and self-repair capabilities have been investigated for potential civil structure applications (Rogers et al. 2000). Besides, there has been considerable research on integrating novel energy dissipation and control technologies into structural system in order to lessen the excessive vibrations and ensuing damages. It is envisioned that these intelligent materials and systems will have significant impacts on the next generation of design and preservation practice of civil structures.

Despite the promising advantages of smart materials and systems, their long-term benefit as opposed to their conventional counterparts must be established before these technologies can gain widespread industry acceptance. For example, the efficacy of smart materials and systems has so far been mostly exhibited in tightly controlled experimental settings. Their long-term performance durability and functional reliability in, for example, detecting damages and actuating proper forces for real-world civil structures, especially under extreme loads, is still uncertain and thus requires much investigation. In addition, the relatively high initial installation and in-service operation expenses apparently discourage civil engineers to adopt most of these advanced technologies for preserving a majority of large-scale civil structures. Another important issue is how to meaningfully interpret the sensed information and use it for decision-making support of managing deteriorating civil structures. These concerns can only be appropriately addressed through sound LCA that, in a unified manner, assesses long-term economic and safety implications of smart materials and systems to lifetime performance of civil structures.

8. CONCLUSIONS

Concepts and ingredients of life-cycle analysis (LCA) and its significant role in design and management of civil infrastructure from a long-term economical perspective are discussed. Relevant computational techniques for performing LCA-based assessment and prediction of lifetime structural performance under uncertainty are reviewed. The importance of using advanced inspection/monitoring for performance reassessment and updating is emphasized. Application of LCA to maintenance management and performance-based design of civil infrastructure subject to gradual and/or extreme loads are reviewed. LCA is very useful in supporting design and management decision-making for enhanced cost-effectiveness over the specified time horizon. Finally, future research needs and challenges are pointed out (i) to consider structural deterioration on lifetime risk assessment under extreme loads and (ii) to develop durable and reliable smart materials and systems for satisfactory lifetime performance prediction and enhancement of real-world civil infrastructure.

ACKNOWLEDGMENTS

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ROBUSTNESS ANALYSIS IN STRUCTURAL OPTIMIZATION

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ABSTRACT

Structural optimization typically aims at high performance levels for a clearly specified set of conditions. Unfortunately, this goal can usually be achieved only by sacrificing robustness of the design. This implies a high sensitivity with respect to unforeseen stochastic situations or unavoidable random manufacturing tolerances. In order to prevent structural failure due to loss of robustness it is therefore desirable to incorporate a suitable measure of robustness into the optimization process. This can be achieved by introducing additional constraint conditions or appropriate modifications of the objective function. An example for such a design concept is *reliability-based optimization* based on the notion of the failure probability. This is most appropriate for high-risk structures such as e.g. power-generating facilites. Alternatively, simpler stochastic measures such as variances or standard deviations might be more appropriate for the design of low-risk structural elements which are frequently found e.g. in the automotive industry. The paper discusses the basic requirement for robust optimization and attempts to outline pros and cons of different approaches to the solution of this problem.

1 INTRODUCTION

Uncertainties in the optimization process can be attributed to three major sources as shown in Fig. 1 These sources of uncertainties or stochastic scatter are

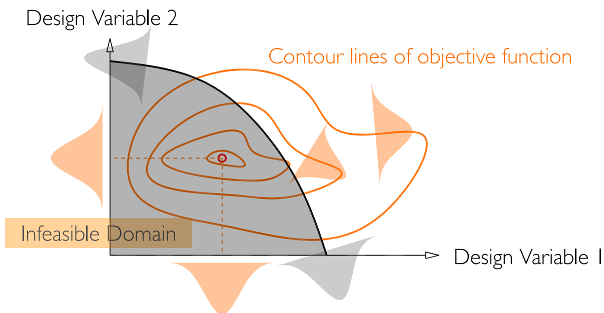


Figure 1: Sources of uncertainty in optimization

- Uncertainty of design variables. This means that the manufacturing process is unable to achieve the design precisely. The magnitude of such uncertainty depends to a large extent on the quality control of the manufacturing process.
- Uncertainty in the objective function. This means that some parameters affecting the structural performance are beyond the control of the designer. These uncertainties may be reduced by a stringent specification of operating conditions. This may be possible for mechanical

structures, but is typically not feasible for civil structures subjected to environmental loading such as earthquakes or severe storms which cannot be controlled.

- Uncertainty of the feasible domain. This means that the admissibility of a particular design (such as its safety or serviceability) cannot be determined deterministically. Such problems are at the core of probability-based design of structures.

2 STOCHASTIC MODELING

2.1 Basic Definitions

Probability in the mathematical sense is defined as a positive measure (between 0 and 1) associated with an event in probability space. For most physical phenomena this event is suitably defined by the occurrence of a real-valued random value X which is smaller than a prescribed, deterministic value x. The probability associated with this event is called *probability distribution function* (or, equivalently *cumulative distribution function*, cdf):

$$F_X(x) = P[X < x] \tag{1}$$

Differentiation of $F_X(x)$ with respect to x yields the so-called *probability density function* (pdf):

$$f_X(x) = \frac{d}{dx} F_X(x) \tag{2}$$

A qualitative representation of these functions is given in Fig. 2.

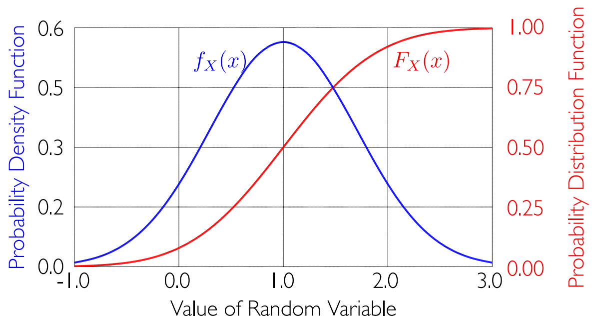


Figure 2: Schematic sketch of probability distribution and probability density functions.

In many cases it is convenient to characterize random variables in terms of expected values rather than probability density functions. Special cases of expected values are the *mean value* $\bar{X}$:

$$\bar{X} = \mathbf{E}[X] = \int_{-\infty}^{\infty} x f_X(x) dx \tag{3}$$

and the *variance* σ_X^2 of a random variable:

$$\sigma_X^2 = \mathbf{E}[(X - \bar{X})^2] = \int_{-\infty}^{\infty} (x - \bar{X})^2 f_X(x) dx \tag{4}$$

The positive square root of the variance σ_X is called *standard deviation*. For variables with non-zero mean value ($\bar{X} \neq 0$) it is useful to define the dimension-less coefficient of variation

$$V_X = \frac{\sigma_X}{\bar{X}} \tag{5}$$

A description of random variables in terms of mean value and standard deviation is sometimes called "second moment representation". Note that the mathematical expectations as defined here are so-called *ensemble averages*, i.e. averages over all possible realizations.

2.2 Two Types of Distributions

Due to its simplicity, the so-called Gaussian or normal distribution is frequently used. A random variable X is *normally distributed*, if its probability density function is:

$$f_X(x) = \frac{1}{\sqrt{2\pi}\sigma_X} \exp\left[-\frac{(x-\bar{X})^2}{2\sigma_X^2}\right]; \quad -\infty < x < \infty \tag{6}$$

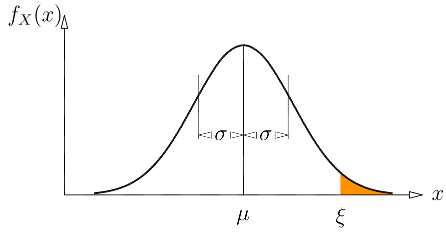
Here $\bar{X}$ is the mean value, and σ_X is the standard deviation. The distribution function $F_X(x)$ is described by the normal integral $\Phi(.)$:

$$F_X(x) = \Phi(\frac{x-\bar{X}}{\sigma_X}) \tag{7}$$

in which

$$\Phi(z) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{z} \exp(-\frac{u^2}{2}) du \tag{8}$$

This integral is not solvable in closed form, however tables and convenient numerical approximations exist. The use of the Gaussian distribution is frequently motivated by the central limit theorem which states that an additive superposition of independent random effects tends asymptotically to the Gaussian distribution.



$P_\xi = P[X \geq \xi]$

ξ	μ	$\mu+\sigma$	$\mu+2\sigma$	$\mu+3\sigma$	$\mu+4\sigma$
P_ξ	$5\cdot 10^{-1}$	$1.6\cdot 10^{-1}$	$2.3\cdot 10^{-2}$	$1.4\cdot 10^{-3}$	$3.2\cdot 10^{-5}$

Figure 3: Gaussian (normal) probability density function and probabilites of exceeding threshold values ξ

A random variable X is *log-normally distributed*, if its pdf is:

$$f_X(x) = \frac{1}{x\sqrt{2\pi}s} \exp\left[-\frac{(\log\frac{x}{\mu})^2)}{2s^2}\right]; \quad 0 \leq x < \infty \tag{9}$$

and its distribution function is given by

$$F_X(x) = \Phi(\frac{\log \frac{x}{\mu}}{s}) \tag{10}$$

In these equations, the parameters μ and s are related to the mean value and the standard deviation as follows:

$$\mu = \bar{X} \exp(-\frac{s^2}{2}); \quad s = \sqrt{\ln(\frac{\sigma_X^2}{\bar{X}^2} + 1)} \tag{11}$$

Two random variables with $\bar{X} = 1.0$ and $\sigma_X = 0.5$ having different distribution types are shown in Fig. 4. It is clearly seen that the log-normal density function is non-symmetric and does not allow negative values. Another important difference lies in the fact that the probability of exceeding

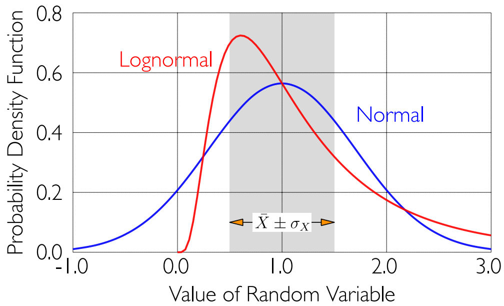


Figure 4: Normal and log-normal probability density functions.

certain threshold levels ξ is significantly influenced by the type of probability distribution. For a normal distribution, the probability of exceeding a level $\xi = 3$ corresponding to the mean value plus 4 standard deviations is $3.2 \cdot 10^{-5}$ while in the case of a lognormal distribution the same threshold has an exceedance probability of 0.083. In order to achieve the same exceedance probability as in the Gaussian case, the threshold level must be set to $\xi = 7.39$, which is the mean value plus 12 standard deviations.

2.3 Random Vectors

In many applications a large number of random variables occur together. It is conceptually helpful to assemble all these random variables $X_k;\ k = 1 \dots n$ into a *random vector* $\mathbf{X}$:

$$\mathbf{X} = [X_1, X_2, \dots X_n]^T \tag{12}$$

For this vector, expected values can be defined in terms of expected values for all of its components:

Mean value vector

$$\bar{\mathbf{X}} = \mathbf{E}[(\mathbf{X}] = [\bar{X}_1, \bar{X}_2, \dots \bar{X}_n]^T \tag{13}$$

Covariance matrix

$$\mathbf{E}[(\mathbf{X} - \bar{\mathbf{X}})(\mathbf{X} - \bar{\mathbf{X}})^T] = \mathbf{C_{XX}} \tag{14}$$

The dimensionless quantity

$$\rho_{ik} = \frac{\mathbf{E}[(X_i - \bar{X}_i)(X_k - \bar{X}_k)]}{\sigma_{X_i}\sigma_{X_k}} \tag{15}$$

is called *coefficient of correlation*. Its value is bounded in the interval $[-1,\ 1]$.

3 ANALYSIS OF RESPONSE VARIABILITY

3.1 General Remarks

In order to obtain meaningful correlations between the input and output variables it is essential to precisely capture the input correlations in the simulated values. Monte-Carlo based methods use digital generation of pseudo-random numbers to produce artificial sample values for the input variables. The quality of these numbers can be measured in terms of their statistical properties. For the case of two random variables X_1 and X_2, Monte Carlo methods produce sequences of numbers $X_1^k, X_2^k, k = 1 \ldots N$ in such a way that the prescribed statistics as estimated from these samples match the prescribed statistics as closely as possible. Typically, plain Mont-Carlo methods are fairly well able to represent individual statistics of the random variables. At small sample sizes N, however, the prescribed correlation structure may be rather heavily distorted. Significant improvement can be made by utilizing the Latin Hypercube sampling method (Florian 1992). Comparing Plain Monte-Carlo (PMC) with Latin Hypercube Sampling (LHS) it is easily seen that LHS covers the space of random variables in a significantly superior way. In particular, PMC introduces unwanted correlation into the samples which becomes very pronounced if the number of samples is small. This is readily seen from Fig. 5, where a positive correlation of $\rho = 0.28$ appears in the samples on the left hand side. Unfortunately, many real-world structural problem are so large that only a small number of samples can be accepted.

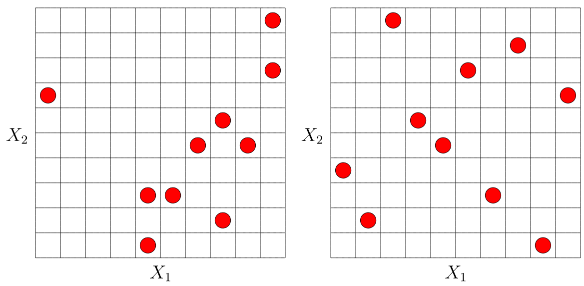


Figure 5: 10 samples of two uniformly distributed independent random variables. Left: Plain Monte Carlo, Right: Latin Hypercube Sampling

3.2 Correlation Statistics

Assume that we want to estimate a matrix of correlation coefficients of m variables from N samples. This matrix has $M = m \cdot (m-1)/2$ different entries in addition to m unit elements on the main diagonal. The confidence intervals for the estimated coefficients of correlation ρ_{ij} are computed based on the Fisher's z-transformation. The interval for a significance level of α (i.e. a confidence level of $1-\alpha$) is given by

$$[\tanh(z_{ij} - \frac{z_c}{\sqrt{N-3}}),\ \tanh(z_{ij} + \frac{z_c}{\sqrt{N-3}})] \tag{16}$$

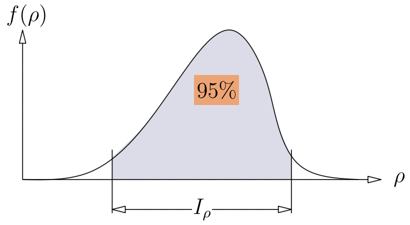


Figure 6: Confidence interval for estimated coefficients of correlation ρ

In this equation, N ist he number of samples used for the estimation of ρ_{ij}. The critical value z_c is computed by using the Bonferroni-corrected value for the significance level $\alpha' = \alpha/M$ with M being the number of correlation coefficients to be estimated (see above). The transformed variable z is computed from

$$z_{ij} = \frac{1}{2} \log \frac{1+\rho_{ij}}{1-\rho_{ij}} \tag{17}$$

and the critical value z_c is given by

$$z_c = \Phi^{-1}(1-\alpha'/2) \tag{18}$$

where $\Phi^{-1}(.)$ is the inverse cumulative Gaussian distribution function.

3.3 Effect of Latin Hypercube sampling

In order to study the effect of LHS on the reduction of statistical uncertainty, a numerical study performing a comparison of the estimation errors (standard deviations) of the correlation coefficients is carried out. The following table shows confidence interval for a confidence level of 95% as a function of the correlation coefficient ρ and the number of samples N used for one estimation. The statistical analysis is repeated 1000 times. In summary, it turns out that the net effect of LHS is an effective reduction of the sample size by a factor of more than 10. For example, as seen from Tables 1 and 2, it is possible to estimate a coefficient of correlation of $\rho = 0.3$ using 1000 samples of MCS with a 95%-confidence interval of 0.11, while the same confidence interval (actually 0.1) is achieved with only 100 samples using LHS. On the other hand, 1000 LHS samples would reduce the respective 95%-confidence interval to 0.03, which is an enormous improvement.

Table 1: 95% confidence interval of correlation coefficient, Plain Monte Carlo

	ρ				
N	0	0.3	0.5	0.7	0.9
10	1.261	1.231	1.054	0.757	0.299
30	0.712	0.682	0.557	0.381	0.149
100	0.409	0.374	0.306	0.199	0.079
300	0.230	0.209	0.170	0.116	0.045
1000	0.124	0.115	0.093	0.062	0.023

Table 2: 95% confidence interval of correlation coefficient, Latin Hypercube Sampling

N	ρ				
	0	0.3	0.5	0.7	0.9
10	0.420	0.382	0.260	0.158	0.035
30	0.197	0.194	0.139	0.073	0.018
100	0.111	0.101	0.071	0.042	0.009
300	0.065	0.057	0.042	0.024	0.006
1000	0.038	0.033	0.025	0.014	0.003

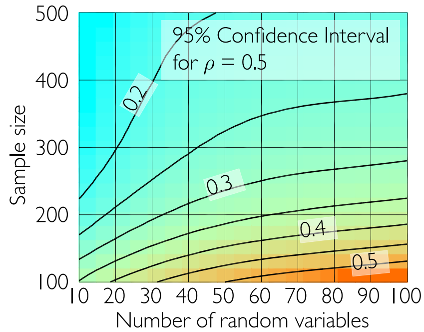


Figure 7: Confidence intervals for coefficients of correlation

4 RELIABILITY ANALYSIS

4.1 Definition

Generally, failure (i.e. an undesired or unsafe state of the structure) is defined in terms of a limit state function $g(.)$ defining the set $\mathcal{F} = \{\mathbf{X} : g(\mathbf{X}) \leq 0\}$. Frequently, $Z = g(\mathbf{X})$ is called *safety margin*. The failure probability is defined as the probability of the occurrence of $\mathcal{F}$:

$$P(\mathcal{F}) = P[\{\mathbf{X} : g(\mathbf{X}) \leq 0\}] \tag{19}$$

4.2 FORM - First Order Reliability Method

The FORM-Concept is based on a description of the reliability problem in standard Gaussian space (Rackwitz and Fiessler 1978). Hence transformations from correlated non-Gaussian variables $\mathbf{X}$ to uncorrelated Gaussian variables $\mathbf{U}$ with zero mean and unit variance are required. This concept is especially useful in conjunction with the Nataf-model for the joint pdf of $\mathbf{X}$ (Liu and DerKiureghian 1986). Eventually, this leads to a representation of the limit state function $g(.)$ in terms of the standardized Gaussian variables U_i:

$$g(\mathbf{X}) = g(X_1, X_2, \dots X_n) = g[X_1(U_1, \dots U_n) \dots X_n(U_1, \dots U_n)] \tag{20}$$

This function is linearized with respect to the components in the expansion point $\mathbf{u}^*$. This point is chosen to minimize the distance from the origin in Gaussian space. From this geometrical interpretation it becomes quite clear that the calculation of the design point can be reduced to an optimization problem:

$$\mathbf{u}^* : \mathbf{u}^T\mathbf{u} \rightarrow \text{Min.}; \quad \text{subject to: } g[\mathbf{x}(\mathbf{u})] = 0 \tag{21}$$

Standard optimization procedures can be utilized to solve for the location of $\mathbf{u}^*$ (Shinozuka 1983). In the next step, the exact limit state function $g(\mathbf{u})$ is replaced by a linear approximation $\bar{g}(\mathbf{u})$ as shown in Fig. 8. From this, the probability of failure is easily determined to be

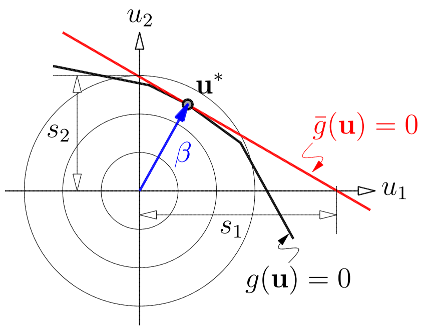


Figure 8: Linearization required for first order reliability method

$$P(\mathcal{F}) = \Phi(-\beta) \tag{22}$$

5 APPLICATION EXAMPLE

As an example, consider the weight optimization of a simple beam subjected to a dynamic loading (cf. Fig. 9). For this beam with a rectangular cross section (w, h) subjected to a harmonic loading $F(t)$ the mass should be minimized considering the constraints that the center deflection due to the loading should be smaller than 10 mm. Large deflections are considered to be serviceability failures. The design variables are bounded in the range $0 < w, h < 1$.

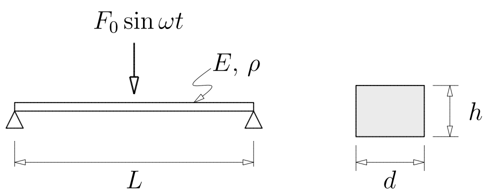


Figure 9: Beam with rectangular cross section

First, the problem is formulated in terms of deterministic parameters. It turns out that the feasible domain is not simply connected (cf. Fig. 10). Such problems typically exhibit multiple local minima. It is then highly recommendable to apply a non-local search strategy for the optimization. For numerical values of $F_0 = 20\,kN$, $\omega = 60\,rad/s$, $E = 3 \cdot 10^{10}\,N/m^2$, $\rho = 2500\,kg/m^3$, $L = 10\,m$ and $g = 9.81\,m/s^2$ the objective function (i.e. the cross sectional area) and the feasible domain are shown in Fig. 10. It can be seen that there are two separate parts of the feasible domain. It is well known that gradient-based optimization techniques have difficulties crossing domain boundaries and localizing the global minimum. The global minimum is located at $w = 0.06$ and $h = 1.00$ as indicated in Fig. 10.
In the next step, the loading amplitude F_0 and the excitation frequency ω are assumed to be Gaussian random variables. The mean values are assumed to be the nominal values as given above, and the coefficients of variation are assumed to be of the order of 10%. This implies that the constraints

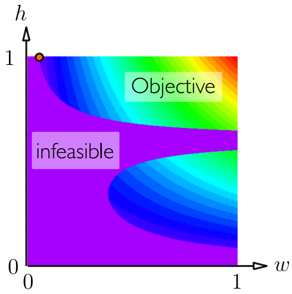


Figure 10: Objective function and feasible domain, deterministic situation

can be satisfied only with a certain probability < 1. In addition to that, stochastic uncertainty in the design variables w and h is considered as well. For this, it is assumed that the optimization controls the mean values of $\bar{w}$ and $\bar{h}$, and that the actual structural dimensions are log-normally distributed random variables with at coefficient of 10%. Fig. 11 shows the probability $P(\mathcal{F}|\bar{w}, \bar{h})$of violating the constraint as a function of the design variables $\bar{w}$ and $\bar{h}$.

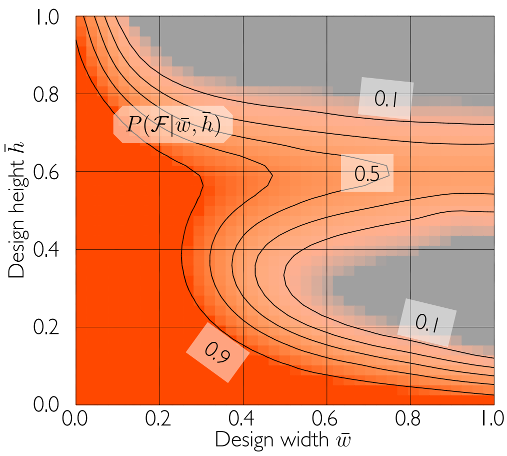


Figure 11: Conditional failure probability $P(\mathcal{F}|\bar{w}, \bar{h})$ depending on $\bar{w}$ und $\bar{h}$

For the following optimization the constraint is formulated as the condition that the probability of violating the prescribed displacement threshold of 10 mm should be smaller than 1%. In the context of a genetic optimization algorithm, constraints are frequently formulated in terms of a penalty function added to the objective function. The magnitude of the penalty term is chosen to depend on the magnitude of the failure probability. The total objective function then becomes

$$L = \bar{h} \cdot \bar{w} + S \cdot H[P(\mathcal{F}) - 0.01] \cdot [P(\mathcal{F}) - 0.01] \tag{23}$$

In this equation, $H[.]$ is the Heaviside function which yields zero penalty for feasible designs. The penalty scale is assumed to have the value of $S = 100$. An optimization run with 10 generation, each having 100 individuals yielded the best individual at $w = 0.888$ and $h = 0.289$. The probability of failure was $P(\mathcal{F}) = 0.0098$, i.e. close to the acceptable limit of 0.01. The cross sectional area was 0.26 which is considerably larger than in the value of 0.06 obtained in the deterministic case.

Fig. 12 shows the progress of genetic optimization. The first, third, fifth, and 10-th generations are shown. The concentration of the populations in the region of acceptable probability of failure is easily seen.
One important outcome of this example is the fact that the locations of the deterministic optimization and the probability-based robust optimization are entirely different. This emphasizes the necessity of incorporation robustness and reliability analysis into the optimization process.

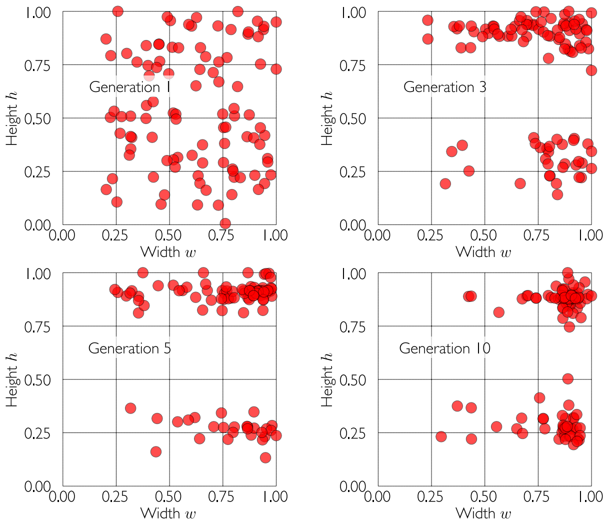


Figure 12: Progress in genetic optimization

6 CONCLUDING REMARKS

Structural optimization tends to lead to highly specialized designs which, unfortunately, very likely lack robustness of performance with respect to unforeseen situations. A prominent cause for such situation lies in the inherent randomness of either design parameters or constraint conditions. One possible way to overcome this dilemma lies in the application of robustness-based optimization. This allows to take into account random variability in the problem formulation thus leading to optimal designs which are automatically robust. It appears that this concept should be applicable to a large number of structural optimization problems. However, the numerical effort to carry out the analysis is quite substantial. Further research into the reduction of effort is therefore required.

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CONCEPTS FOR HEALTH JUDGEMENT OF STRUCTURES

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ABSTRACT

Engineering structures are designed for special requirements and a defined and limited service life. With regard to the assumed timeframe of utilization, the existing inventory of structures will result in enormous maintenance and rehabilitation costs in future. To a continuously growing degree, we observe an increasing shift from investments in the construction of new infrastructures to the maintenance and lifetime extension of the existing ones. In front of this background it is necessary to identify the key parameters and procedures to verify and update the knowledge about the present condition of a structure with respect to a number of aspects.

1. INTRODUCTION

The life cycle reliability assessment is needed by administration of transport infrastructure network operators. The decision-making tools should intercept the degradation and retrofitting process in order to support the maintenance of engineering structures (Frangopol, 2000), see Fig. 1.

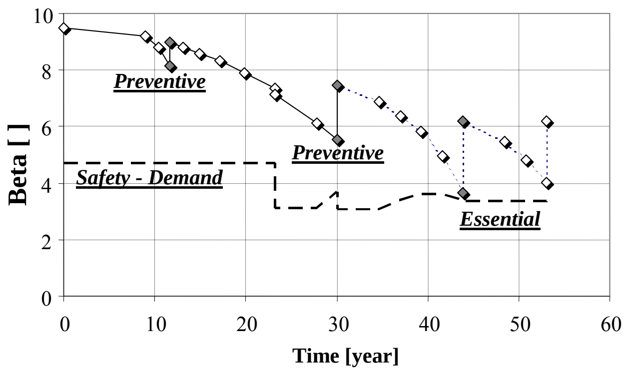


Fig. 1: Maintenance – Retrofitting processes

Numerous approaches like the Baysian Updating (Oh et al., 2004), the Monte Carlo Simulation (Petcherchoo et al., 2004) and the Asset Management using multinational Genetic Algorithm (Furuta et al. 2004) have been used for Life Cycle Cost analysis. Since the stochastic models behind these procedures are rather demanding and time consuming, these approaches are commonly based on simplified mechanical models or formulas. More realistic reliability analysis can be achieved using nonlinear FEM modeling. The life-cycle analysis is a complex task and requires an interdisciplinary approach. It should combine modeling of

-) nonlinearities in material
-) uncertainties and
-) degradation phenomena.

The particular methodologies for the use of probabilistic based assessment are available and have been proven to work in practice (eg. Furuta et al. 2004, fib 2003, Teplý et al. 2003).
The reliability calculation of structures from the stochastically obtained structural resistance and expected load distribution is a transparent and easily understandable concept. The stochastic response requires repeated analyses of the structure with random input parameters to reflect randomness and uncertainties in the input values. A nonlinear computer simulation should be utilized for realistic prediction of structural response and its resistance. As the nonlinear structural analysis is computationally very demanding, a suitable technique of statistical sampling should be used to allow relatively small number of simulations. A special attention should be paid to modeling of degradation phenomena, like carbonation of concrete, corrosion of reinforcement, chloride attack, etc.

The main expected results are estimation of structural reliability using reliability index and/or theoretical failure probability during the degradation/retrofitting processes. In order to perform a complete life-cycle analysis, a wide spectrum of methods should be used and combined. It must include nonlinear FEM modelling, statistical and reliability techniques and degradation phenomena modelling. The problem is rather complex and requires an interdisciplinary approach and should be accomplished by a health monitoring system. The approach presented in this contribution differs from the previous mentioned approaches mainly by the use of nonlinear models describing the real structure. The goal is a more realistic modelling of the structural behaviour, and consequently of the health index (eg. efficient realistic nonlinear modelling of structures). The method permits a direct link between nonlinear degradation models and nonlinear material behaviour at the “mesoscale”, see Fig. 2.

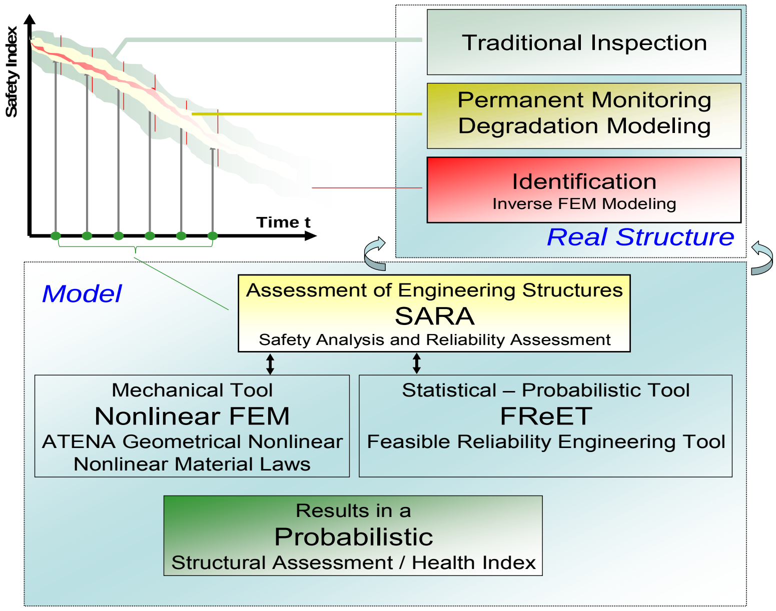


Fig. 2: Approach for a realistic nonlinear modelling and reliability analysis

Due to the fact that nonlinear structural analysis is computationally very demanding, a suitable technique of statistical sampling has to be used, to allow a relatively small number of simulations. Final results include: statistical characteristics of response (stresses, deflections, crack width etc.), information on dominating and non-dominating variables (sensitivity analysis) and estimation of reliability and theoretical failure probability. New and/or updated (significantly improved) theoretical statistical and reliability methods, which had to be developed, verified and implemented, are itemized as follows:

- Small-sample simulation (Monte Carlo type) of Latin hypercube sampling for both random variables and random fields
- Statistical correlation using the simulated annealing approach (Vorechovský and Novák, 2003)
- Small number of random variables to represent random fields based on the spectral decomposition of covariance matrix
- Sensitivity analysis based on nonparametric rank-order statistical correlation

The multipurpose probability-based software for statistical sensitivity and reliability analysis of engineering problems FREET (Novák et al., 2002, 2003) is based on the techniques described above.

As mentioned before, the nonlinear analysis is an essential mechanical tool for the realistic description of structures. It represents a well established methodology for failure analysis of civil engineering structures in the deterministic sense. The response of the structure under loading and environmental actions can be traced until structural failure. It enables to calculate the structural behaviour under service load (serviceability limit states, SLS) as well as the load carrying capacity of the structure (ultimate limit states, ULS). The behaviour of reinforced concrete structures should be analyzed by means of correspondingly advanced technology, taking into account all the important material properties and features: tensile cracking, compressive confinement, reinforcement, including its bond to concrete etc.
One of the most appropriate methodologies for a realistic failure analysis of complex heavy reinforced and pre-stressed concrete structures like bridges seems to be the smeared damage mechanics approach. The main features of the smeared crack modelling of reinforced concrete can be outlined as follows:

- damage mechanics, nonlinear fracture mechanics, enhanced plasticity
- softening in both tension and compression
- smeared crack approach, crack band method
- discrete and smeared reinforcement, bond-slip relationship.

This approach is implemented in a nonlinear finite element software ATENA, which is a tool widely used in practice for realistic computer simulations and predictions of damage and failure of concrete and reinforced concrete structures (Cervenka 2000, 2002).
A further essential tool within the realistic reliability assessment of structures are prognoses elements. Deterioration can be described through a variety of analytical models, whose main difference is their complexity and the number of input parameters necessary. Models describing the carbonation process of concrete as well as chloride ingress and corrosion of mild and pre-stressed steels are used.

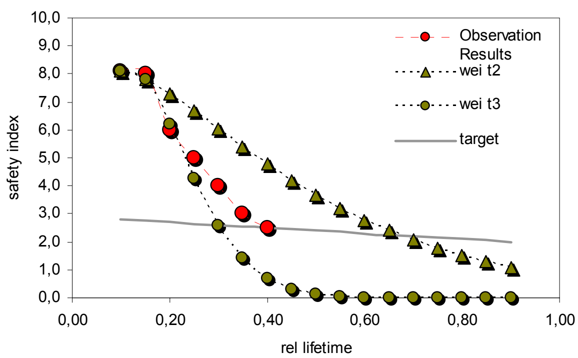


Fig. 3: Weibull fitting

In a first approximation degradation processes can be based on simplified models referenced in literature. The exponential distribution is often used as a model of time-to-failure measurements for a structure or parts of an infrastructure, when the failure (hazard) rate is constant over time. When the failure probability varies over time, the Weibull distribution is appropriate. Thus, the Weibull distribution is often used in reliability testing (e.g., of electronic relays, ball bearings etc.; see Hahn and Shapiro, 1967). The Weibull distribution is defined as: f(x) = c/b*(x/b)(c-1) * e[-(x/b)^c], for $0 \leq x < \infty$, $b > 0$, $c > 0$; where b is the scale parameter, c the shape parameter and e the base of the natural logarithm, sometimes called Euler's e (2,71). For engineering structures f(x) can be seen as the health index β(t) varying over the time t. Selective results β(t1)… β(tn) from simulations regarding inspection knowledge permit to adapt the parameters b and c. These parameters can be continually updated by further simulation and actual inspection results, see Fig. 3. In recent studies detailed models regarding carbonation, chloride ingress etc. are tested and included in the FReET environment by Teply 2005. This models provides the possibility for complex degradation research on engineering structures.

The before mentioned analysis elements are basics for the realistic description of structures. Nevertheless a further element has to be coupled to the existing cluster to include material-, geometrical-, model uncertainties etc. Considering these the demand for a probabilistic approach is given. The main objective of the probabilistic analysis here discussed is to combine nonlinear FEM models taking into account reliability techniques and degradation phenomena, see Fig. 1. The whole solution procedure can be itemized as follows:

1. The deterministic model of the structure is prepared and checked within ATENA.
2. Uncertainties and randomness of the input parameters are modelled as random variables described by their probability density functions (PDF). The result of this step creates the sets of input parameters for ATENA's computational model random variables described by mean value, variance and other statistical parameters (generally by PDF).
3. The random input parameters are generated according to their PDF using LHS sampling. Statistical correlation among the parameters is imposed by using simulated annealing.
4. Generated samples of random parameters are used as input sets for ATENA computational model. The complex nonlinear solution is evaluated and selected results (structural response) are saved.
5. The previous two steps are repeated for all samples.
6. The resulting sets of structural responses from the whole simulation process is statistically evaluated, resulting in: histogram, mean value, variance, coefficient of skew ness, empirical cumulative probability density function of the structural response, sensitivity of the structural response to input parameters, reliability index assessment.

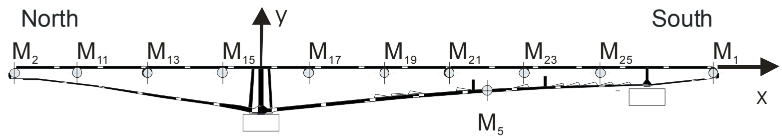
The probabilistic/stochastic analysis serves for the instantaneous record of the health condition of a structure. Based on monitoring data, the stochastic analysis will deliver *explicit results* regarding the time-depending health condition of the structure. If several health conditions at a well-determined time are analyzed, the Weibull fitting can be used for the examination of the remaining lifetime and the degradation process.

By combining the stochastic analysis with analytical deterioration models, an *implicit approach* is given. Starting from at least one calculated health condition an extrapolation of the degradation behaviour is available.
Both the *explicit* and the *implicit* approach are used for retrofitting measurements and consequently lifetime planning.
In the case of retrofitting, the probabilistic/stochastic analyses are focused on the point after measurement and the implicit or explicit approach should give information for the remaining time. The explicit approach seems to be easier to handle, because of the lower number of less

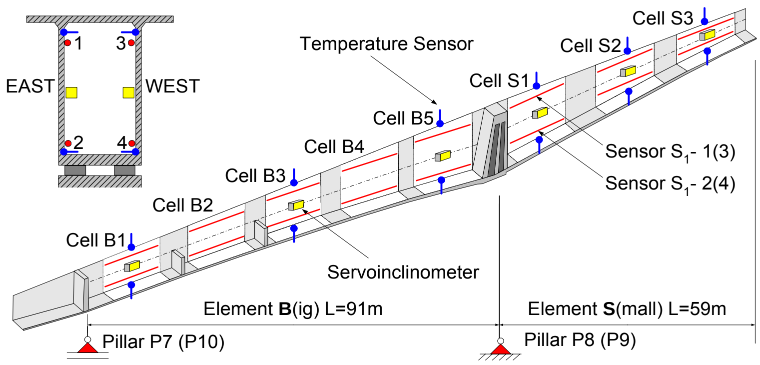
necessary input quantities. SARA, together with the implicit and explicit approach, provides a dynamic tool for health evaluation and lifetime planning.

2. STRUCTURAL ANALYSIS WITH INTIGRATED MONITORING

The Colle Isarco Viaduct is a cantilever beam bridge in Italy with a total length of 1,000 m. Built in 1969, it is a fully post-tensioned box-girder. The measurements of selected values – monitoring points placed on the structure – allow the verification of the service ability level and the ultimate limit level of the structure, see Fig. 5. The monitoring points should be located along the structure in agreement with sensors already mounted on the structure. (Santa, 2004). During the simulation process, for each incremental increase of the line load, the monitoring points recorded the structural response - mainly the vertical deflection.



Monitoring points of the numerical simulation



Sensors mounted on the structure

Fig. 4: Monitoring points + Instrumentation scheme of the box girder for evaluation of the structural behaviour

The data achieved by the monitoring, as shown in Fig. 5, serves mainly for the customization of the input quantities of the nonlinear simulation. The customization of the input quantities is made by comparison of the deflections or inclinations. The customization is carried out in most cases heuristically and therefore includes many uncertainties. The reliability index derived from the probabilistic considerations is charged therefore also with spreads. For preservation planning and lifetime planning an automated identification algorithm has to be used for the compensation of the differences between the monitored and simulated quantities. This identification algorithm delivers the input data for the simulation which agree with the real conditions at the structure.

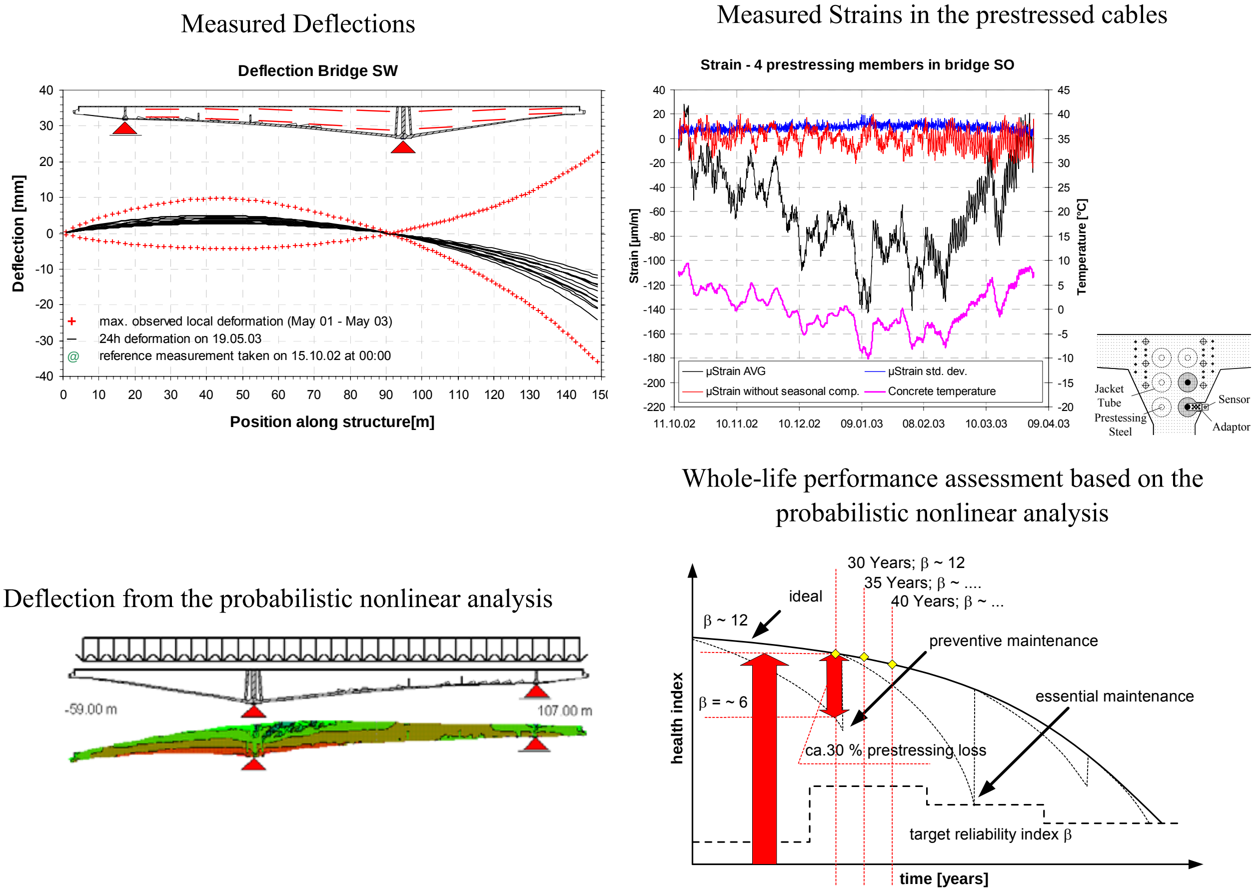


Fig. 5: Monitoring points + Instrumentation scheme of the box girder for evaluation of the structural behaviour

3. IDENTIFICATION

Health monitoring and probabilistic monitoring as described above provides statistical information (e.g. deflections). An inverse FEM problem can be considered: what statistical parameters should be used to reproduce monitored random responses? A primary key problem in practical utilization is an identification of material model parameters. Parameters like fracture energy, tensile strength etc. cannot be always determined by experiments. Based on these ideas the following two approaches are worked out to use monitoring data for the evaluation of material properties and consequently degradation processes in existing structures.

Sensitivity based approach

The approaches for identification can be based on statistics updating using sensitivity factors α_i , achieved by probabilistic calculations, see Eq. 1, evaluated for each random variable.

$$\vec{\alpha} = \alpha_{M_i,k} \qquad k = 1...n; \quad k = Ec, ft, fc... \tag{1}$$

$$\Delta m_{M_i} = m^{Y}_{M_i} - m^{X}_{M_i} \tag{2}$$

By this approach the sensitivity factors of the parameter included in the random vector X (X is the vector of input parameters of the model) are calculated in each monitoring point. In the first step the algorithm determines the differences of the mean values of the output concerning the simulation obtained from vector X and the observation Y, see Fig.6 and Eq.2.

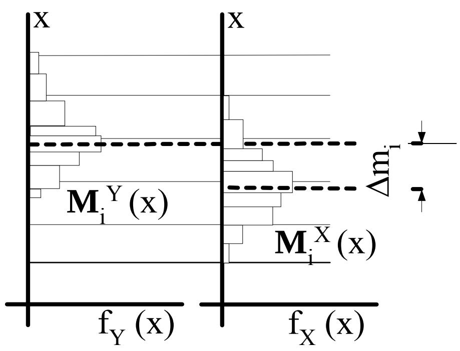


Fig. 6: Differences of the mean values of the output at a monitoring point

The procedure changes the input data of the simulation in order that the simulated output data agree with the observed data. The utilization of the sensitivity factors α_i in the correction process occurs as an efficient way. The difference of the mean values of the output data are divided according to the sensitivity factors. This formulation allows the utilization of the dominancy of the sensitivity factors in the updating process of the input (Eq. 3-5). Sensitivity factors describe the influence of the input data on the simulation output.

$$\Delta\alpha_{M\,i,k} = \Delta m_{Mi} \cdot \alpha_{Mi,k} \tag{3}$$

$$\Delta X^{mean}{}_k = -1 \cdot E[\,\Delta\alpha_{Mi}\,]_k \cdot F \tag{4}$$

$$_{new}X^{mean}{}_k = X^{mean}{}_k + \Delta X^{mean}{}_k \tag{5}$$

The calibration factor *F* is responsible for the step length of the algorithm. The range of *F* can be chosen heuristically between [0;2] according to the nonlinearity of the defined problem. After repeated iteration repetition the distances of the simulated mean values to the monitored mean values become smaller (Eq. 5) (first phase of the algorithm, taking into account the mean values). The convergence criterion for the iteration process can be established by controlling the error of observed monitoring and simulation responses. In the second phase of the identification process the standard deviations are adjusted. The process is similar as in the case of the mean values (Eq. 6-9).

$$\Delta std_{M_i} = std^{Y}_{M_i} - std^{X}_{M_i} \tag{6}$$

$$\Delta\alpha_{M\,i,k} = \Delta std_{Mi} \cdot \alpha_{Mi,k} \tag{7}$$

$$\Delta X^{std}{}_k = -1 \cdot E[\,\Delta\alpha_{Mi}\,]_k \cdot F \tag{8}$$

$$_{new}X^{std}{}_k = X^{std}{}_k + \Delta X^{std}{}_k \tag{9}$$

In general, at the initial iteration steps the differences of the mean values are reduced and the differences of the standard deviations are increased. This seems to be a weak point of the identification of the standard deviation. But after capturing the mean value within an accepted error, however the differences of the standard deviations start to reduce too. The process described above will be carried out in iteration steps. From several examples we can draw the conclusion that the input quantities of the simulations can capture the stochastic models of the real material parameters after several iterations.

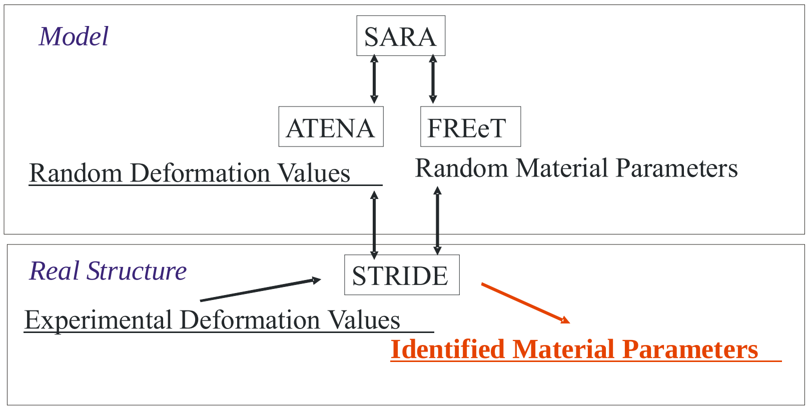


Fig. 7: Interface between the probabilistic modules and the identification module

As shown in Fig. 7 and Fig. 8 the identification algorithm STRIDE is assigned to the existing core of the probabilistischen calculation modules
The algorithm bases on a MATLAB code and has an open interface, so that the communication can also performed with other programs. STRIDE therefore offers an essential support of the reliability assessment and the degradation elements.

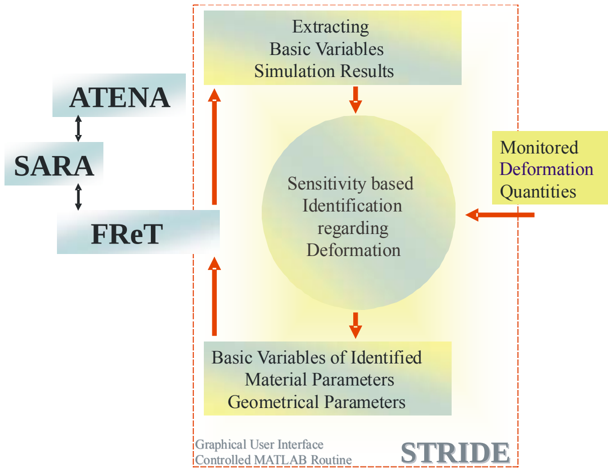


Fig. 8: Structure of the identification algorithm

3.1. Neural Network based Identification

A sophisticated identification technique using artificial neural networks in combination with stochastic training of the networks can be used as an alternative to the previous approach, see Fig. 9. The main ideas of the approach are described in (Novák, 2003). From generated basic random variables and the corresponding random response the first two statistical moments of can be obtained. This set of data can then be used for training of a suitable type of neural network, see Fig. 9.

Once the network is trained it represents an approximation, which can be utilized in an opposite way: For given experimental displacements in monitoring points, which are random as well, to provide the best possible set of material model parameters.

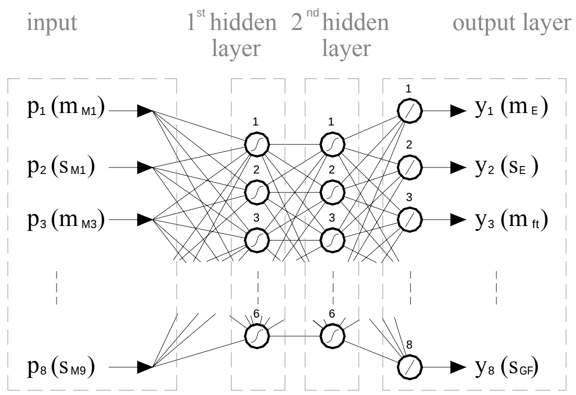


Fig. 9: Scheme of neural network used

4. CONCLUSION

A complex methodology including nonlinearities in material, uncertainties and degradation phenomena is proposed for advanced life-cycle reliability analysis of reinforced concrete structures. The described methodology is compiled into the software tool SARA for instantaneous practical application. It represents an innovative decision-making tool for the maintenance of structures, which can be very powerful especially in combination with an existing health monitoring system.

Based on selective inspection data, permanent monitoring and degradation models, the phenomenon of degradation and the decrease of the reliability index of bridges in time can be modelled. The concept is powerful to support inverse material detection algorithm to deviate time-dependent material conditions from monitored deflection lines.

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SMALL-SAMPLE SIMULATION METHODS FOR STATISTICAL, SENSITIVITY AND RELIABILITY ANALYSES

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ABSTRACT

The objective of the paper is to present methods for efficient statistical, sensitivity and reliability assessment. The attention is given to the techniques which are developed for an analysis of computationally intensive problems which is typical for a nonlinear FEM analysis. The paper shows the possibility of "randomization" of computationally intensive problems in the sense of the Monte Carlo type simulation. Latin hypercube sampling is used, in order to keep the number of required simulations at an acceptable level. The technique is used for both random variables' and random fields' levels. Sensitivity analysis is based on nonparametric rank-order correlation coefficients. Statistical correlation is imposed by the stochastic optimization technique – the simulated annealing. The multipurpose software FReET is briefly described.

1. INTRODUCTION

A large number of efficient stochastic analysis methods have been developed during last years. In spite of many theoretical achievements the acceptability and a routine application in industry is still rare. Two main categories of stochastic approaches can be distinguished: Approaches focused on the calculation of statistical moments of response quantities, like estimation of means, variances etc. and approaches aiming at the calculation of estimation of theoretical probability of failure. There are many different methods developed by reliability researchers covering both the approaches. The common feature of all the methods is the fact that they require a repetitive evaluation (simulations) of the response or limit state function. The development of reliability methods is from the historical perspective a struggle to decrease an excessive number of simulations. Some small-sample simulation methods utilized by authors and implemented in probabilistic software FReET are described:

- Small-sample simulation of Monte Carlo type Latin hypercube sampling for both random variables and random fields
- Imposing statistical correlation using the simulated annealing approach
- Small number of random variables to represent random fields based on spectral decomposition of covariance matrix
- Sensitivity analysis based on nonparametric rank-order statistical correlation

The methods were integrated within the complex software system SARA (Pukl et al. 2003ab, Novák at al. 2002, Bergmeister at al. 2004). The system represents a combination of statistical simulation package FReET (Novák et al. 2003, 2005) and nonlinear mechanics software ATENA (Červenka and Pukl 2005, Červenka 2003). The most interesting applications are referenced.

2. SMALL-SAMPLE SIMULATION OF MONTE CARLO TYPE – LATIN HYPERCUBE SAMPLING

For time-intensive calculations, the small-sample simulation techniques based on stratified sampling of Monte Carlo type represent a rational compromise between feasibility and accuracy. Therefore Latin hypercube sampling (LHS) was selected as a key fundamental technique.

The method belongs to the category of stratified simulation methods (e.g. Mc Kay and Conover 1979, Novák et. al 1998). It is a special type of the Monte Carlo simulation which uses the stratification of the theoretical probability distribution function of input random variables. It requires a relatively small number of simulations to estimate statistics of response – repetitive calculations of the structural response (tens or hundreds).

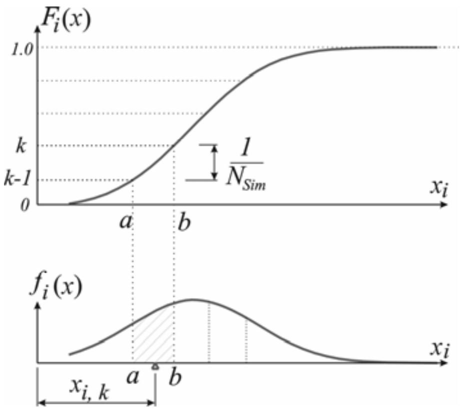

Figure 1. Illustration of LHS.

The basic feature of LHS is that the probability distribution functions for all random variables are divided into N_{Sim} equivalent intervals (N_{Sim} is a number of simulations); the values from the intervals are then used in the simulation process (random selection, middle of interval or mean value). This means that the range of the probability distribution function of each random variable is divided into intervals of equal probability. The samples are chosen directly from the distribution function based on an inverse transformation of distribution function. The representative parameters of variables are selected randomly, being based on random permutations of integers $1, 2, ..., j, N_{Sim}$. Every interval of each variable must be used only once during the simulation. Being based on this precondition, a table of random permutations can be used conveniently, each row of such a table belongs to a specific simulation and the column corresponds to one of the input random variables.

It has been proved that best LHS strategy, which simulates the means and variances very well, is the approach suggested by Keramat and Kielbasa (1997) and Huntington and Lyrintzis (1998). The mean of each interval should be chosen as (Fig. 1):

$$x_{i,k} = \frac{\int_{y_{i,k-1}}^{y_{i,k}} x \cdot f_i(x)\,dx}{\int_{y_{i,k-1}}^{y_{i,k}} f_i(x)\,dx} = N_{Sim} \cdot \int_{y_{i,k-1}}^{y_{i,k}} x \cdot f_i(x)\,dx \qquad (1)$$

where f_i is the probability density function of variable X_i, and the integration limits are:

$$y_{i,k} = F_i^{-1}\left(\frac{k}{N_{Sim}}\right) \tag{2}$$

The estimated mean value is achieved accurately and the variance of the sample set is much closer to the target one. For some probability density functions (inclusive e.g. Gaussian, Exponential, Laplace, Rayleigh, Logistic, Pareto, etc.) the integral (1) can be solved analytically Vořechovský and Novák (2003).

3. IMPOSING STATISTICAL CORRELATION

Once samples are generated, the correlation structure according to the target correlation matrix must be taken into account. There are generally two problems related to the statistical correlation: First, during sampling an undesired correlation can occur between the random variables. For example, instead of the correlation coefficient zero for the uncorrelated random variables, i.e. an undesired correlation, can be generated. It can happen especially in a case of a very small number of simulations (tens), where the number of interval combination is rather limited. The second task is to introduce the prescribed statistical correlation between the random variables defined by the correlation matrix. The columns in LHS simulation plan should be rearranged in such a way that they may fulfill the following two requirements: to diminish the undesired random correlation and to introduce the prescribed correlation. It can be done by using different techniques published in literature on LHS (e.g. Huntington and Lyrintzis 1998, Iman and Conover, 1982) but we found some serious limitations while using them.

A robust technique to impose statistical correlation based on the stochastic method of optimization called simulated annealing has been proposed recently by Vořechovský and Novák (2003). The imposition of the prescribed correlation matrix into the sampling scheme can be understood as an optimization problem: The difference between the prescribed **K** and the generated **S** correlation matrices should be as small as possible. A suitable measure of quality of the overall statistical properties can be introduced:

$$E_{overall} = \sqrt{\sum_{i=1}^{N_V - 1} \sum_{j=i+1}^{N_V} \left(S_{i,j} - K_{i,j}\right)^2} \tag{3}$$

The norm E has to be minimized from the point of view of the definition of the optimization problem using simulated annealing optimization approach, N_v random variables realizations are related to the ordering in the sampling scheme.

4. SIMULATION OF RANDOM FIELDS

A higher level of uncertainties modeling may be in the consideration of the spatial variability of mechanical and geometrical properties of a system and intensity of load. Such quantities should be represented by means of random fields. Because of the discrete nature of the finite element formulation, the random field must also be discretized into random variables. This process is commonly known as random field discretization. The computational effort in reliability problem generally increases with the number of random variables. Therefore it is desirable to use small number of random variables to represent a random field. To achieve this goal, the transformation of the original random variables into a set of uncorrelated random variables can be performed through a well-known eigenvalue orthogonalization procedure. A few of these uncorrelated variables with largest eigenvalues are sufficient for the accurate representation of the field.

Let us consider the fluctuating components of the homogenous random field, which is assumed to model the material property variation around its expected value. Correlation characteristics can be specified in terms of the covariance matrix C_{aa} constructed by discretization using autocorrelation function and geometry of FEM mesh. An eigenvalue orthogonalization procedure will transform variables into uncorrelated space:

$$\mathbf{C}_{\mathrm{XX}} = \mathbf{\Phi\Lambda\Phi}^{\mathrm{T}} \tag{4}$$

The covariance matrix in the uncorrelated space **Y** is a diagonal matrix $\mathbf{\Lambda}=\mathbf{C}_{\mathbf{yy}}$. The vector of uncorrelated Gaussian random variables **Y** can be simulated in the traditional way (Monte Carlo simulation). The transformation back into correlated space yields the vector **X** (discretized random field) using eigenvectors **Φ**:

$$\mathbf{X} = \mathbf{\Phi Y} \tag{5}$$

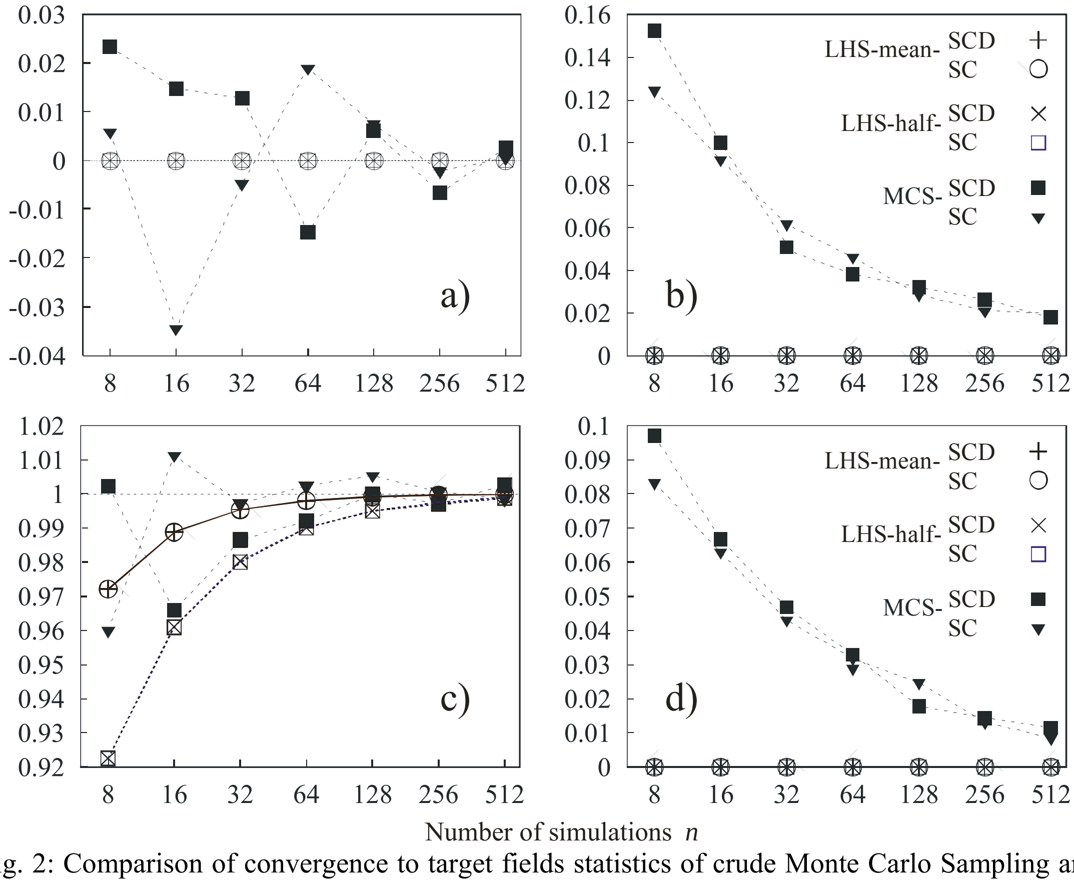

Fig. 2: Comparison of convergence to target fields statistics of crude Monte Carlo Sampling and Latin Hypercube Sampling with number of simulations: a) average, b) dispersion of mean value estimation, c) sample standard deviation, d) dispersion of sample standard deviation.

The utilization of LHS method for simulation of Gaussian uncorrelated variables is the new simple idea of improvement of random field simulation using orthogonal transformation of covariance matrix suggested e.g. by Novák et al. (2000). The superiority of this stratified technique remains here also for accurate representation of random field, thus leading to the decrease of number of simulations needed. This was proved numerically by Vořechovský and Novák (2005). In particular, it has been shown that the ability to simulate *mean value* of random field is excellent in case of LHS, see Figs. 2a) and b). This ability is rather poor in case of MCS, average value of mean fluctuates and standard deviation of mean is high in comparison to LHS. With regard to the second statistical moment, the ability to simulate standard deviation of random field is documented in figures 2c) and d). Again, capturing of this statistics is "random" in case of MCS, standard deviation of sample standard deviation is high in comparison to LHS. In the same study, it has been shown the impact of having the random vector **Y** perfectly uncorrelated. If an attention is paid to spurious correlation between marginals of **Y** (this correlation diminished by a suitable technique) the resulting estimated autocorrelation structure of the field after orthogonal transformation matches perfectly the desired one. Note that the algorithm described briefly in section 3 has proved itself to be very efficient in this regard.

5. SENSITIVITY AND RELIABILITY ANALYSES

An important task in the structural reliability analysis is to determine the significance of random variables. With respect to the small-sample simulation techniques described above the straightforward and simple approach uses the non-parametric rank-order statistical correlation between the basic random variables and the structural response variable (Iman and Conover 1980, Novák et al. 2004). The sensitivity analysis is obtained as an additional result of LHS, and no additional computational effort is necessary.

The relative effect of each basic variable on the structural response can be measured using the partial correlation coefficient between each basic input variable and the response variable. The method is based on the assumption that the random variable which influences the response variable most considerably (either in a positive or negative sense) will have a higher correlation coefficient than the other variables. Because the model for the structural response is generally nonlinear, a non-parametric rank-order correlation is used by means of the Spearman correlation coefficient or Kendall tau.

In cases when we are constrained by small number of simulations (tens, hundreds) it can be difficult to estimate the failure probability. The following approaches are therefore utilized here; they are approximately ordered from elementary (extremely small number of simulations, inaccurate) to more advanced techniques:

- Cornell´s reliability index - the calculation of reliability index from the estimation of the statistical characteristics of the safety margin
- The curve fitting approach - based on the selection of the most suitable probability distribution of the safety margin.
- FORM approximation (Hasofer-Lind´s index)
- Importance sampling techniques
- Response surface methods

These approaches are well known in reliability literature and also providing all details is beyond the aim of this paper. In spite of the fact that the calculation of the failure probability (or/and reliability index) using some of these techniques does not always belong to the category of very accurate reliability techniques (first three in the list), they represent a feasible alternative in many practical cases.

6. SOFTWARE FREET

The multipurpose probabilistic software for statistical, sensitivity and reliability analysis of engineering problems FREET (Novák, et al., 2003, Novák, et al., 2005) is based on efficient reliability techniques described above. There are three basic parts in present version:

The window "Random Variables" (Fig. 3) allows the user-friendly input of basic random variables of analyzed problem. Uncertainties are modeled as random variables described by their probability density functions (PDF). The user can choose from the set of selected theoretical models like normal, lognormal, Weibull, rectangular, etc. Random variables are described by statistical characteristics (statistical moments): Mean value, standard deviation (or coefficient of variation) and coefficient of skewness, respectively.

The window "Statistical Correlation" serves for the input of correlation matrix, Fig. 4. The user can work at the level of a subset of correlation matrices (each related to a group of random variables) or at the global level (all random variables resulting to a large correlation matrix). The level of correlation during interactive input is highlighted, the positive definiteness is checked. Note, that the Simulated Annealing applied consequently does not require this strong requirement.

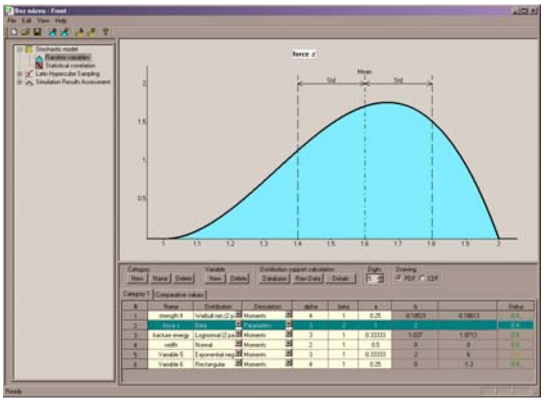

Fig. 3: Window "Random variables".

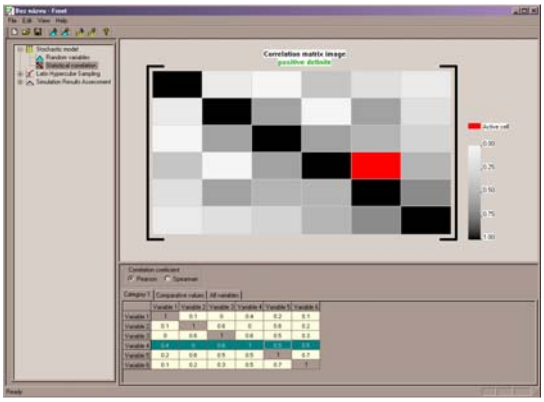

Fig. 4: Window "Statistical correlation".

Random input parameters are generated according to their PDF using LHS sampling. Samples are reordered by Simulated Annealing approach in order to match required correlation matrix as close as possible, Fig. 5. Generated realizations of random parameters are used as inputs for analyzed function (computational model). The solution is performed N times and results (structural response) are saved.

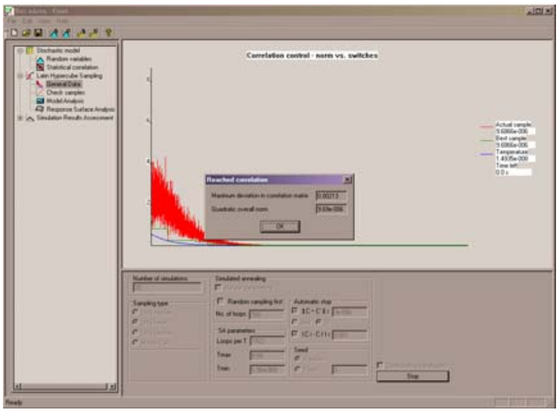

Fig. 5: Window showing the progress of imposing the statistical correlation by Simulated Annealing algorithm.

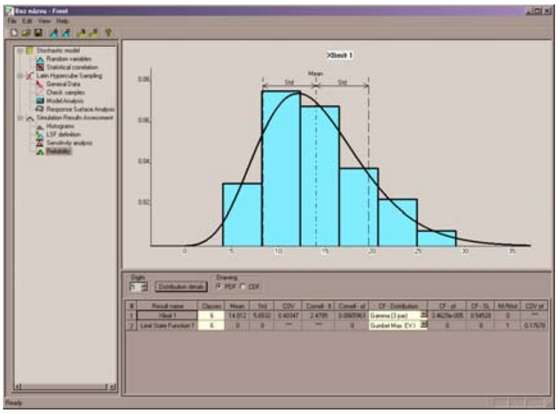

Fig. 6: Window “Reliability” with empirical histogram, Curve fitting, Cornell safety index and Monte Carlo sampling estimation.

At the end of the whole simulation process the resulting set of structural responses is statistically evaluated. The results are: estimations of the mean value, variance, coefficient of skewness and kurtosis, empirical cumulative probability density function estimated by empirical histogram structural response. This basic *statistical assessment* is visualized through the window Histograms. Such a basic statistical analysis is followed by *reliability analysis* based on several approximation techniques: (i) basic estimation of reliability by the Cornell safety index, (ii) curve fitting approach applied to the computed empirical histogram of response variable and (iii) simple estimation of probability of failure based on the ratio of failed trials over the total number of simulations, see Fig. 6.

Additional information to the problem solved is the *sensitivity analysis* of each response function based on its rank-order correlation coefficient. Even though this a actually a by product of the simulation not requiring special additional effort, it provides very useful information in many cases. If the correlation coefficient between a certain input and output variables is close to zero, we can conclude that the input variable has (in its simulated range) a small or even negligible effect on the output. This can sometimes help to decrease the probabilistic dimension of the problem because such an input can be considered deterministic.

7. LIST OF SELECTED TYPES OF APPLICATIONS

The applications of software FReET within the framework of complex system SARA belong to the most successful and interesting ones. Dominating topics with published results are listed as follows:

Probabilistic analyses of concrete structures
The presented approach has been used for statistical and probabilistic nonlinear analysis of concrete structures. The main interest is focused on probabilistic bridge assessments, including degradation and retrofitting modeling. References: Pukl and Bergmeister (2005), Bergmeister et al. (2005), Pukl et al. (2005), Bergmeister et al. (2004), Pukl et al. (2003ab), Novák et al. (2002).

Statistical size effect studies
The probabilistic simulation approach was used to capture the statistical size effect obtained from experiments. The probabilistic treatment of nonlinear fracture mechanics in the sense of extreme value statistics has been recently applied for two crack initiation problems which exhibits the Weibull-type statistical size effect. References: Bažant et al. (2005), Vořechovský et al. (2004, 2005), Bažant at al. (2004), Novák et al. (2003), Lehký and Novák (2002).

Identification of computational model parameters
The recently proposed inverse analysis is based on a coupling of the stochastic nonlinear fracture mechanics analysis and the artificial neural network. Such inverse analysis utilizes SARA package. References: Novák and Lehký (2005), Lehký and Novák (2005), Červenka et al. (2005), Strauss et al. (2004ab), Lehký and Novák (2004), Novák and Lehký (2004).

8. CONCLUSIONS

The paper briefly describes the small-sample simulation techniques for statistical, sensitivity and reliability analyses of computationally intensive problems implemented in FREET software. Efficient techniques of stochastic simulation methods were combined in order to offer an advanced tool for the probabilistic assessment of the complex problems, like those of nonlinear fracture mechanics modeling (SARA, ATENA). A wide range of applicability both practical and theoretical gives an opportunity for further intensive development – bridging first theory and praxis, and second, reliability and nonlinear computation.

ACKNOWLEDGMENTS

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SAFETY AND RELIABILITY ASSESSMENT OF CONCRETE STRUCTURES AND PRACTICAL APPLICATIONS

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Abstract

The presented concept for safety and reliability assessment of concrete structures integrates nonlinear finite element analysis with stochastic and reliability technology into an advanced engineering tool. The feasibility of the developed complex software system is documented on numerical example of statistical failure simulation and reliability evaluation of existing concrete bridge structure. The presented approach is going beyond the boundaries of design codes and can lead to considerable cost saving as the reliability requirements can be targeted more precisely.

1. INTRODUCTION

Safety and reliability assessment of aged structures is becoming more and more important issue for civil infrastructure management systems. The increasing demand on the load carrying capacity combined with limited budgets for rehabilitation e.g. concrete bridges is a common problem worldwide. The general approach for safety evaluation of existing structures is based on codes and different specific regulations. It has been found that reliability assessment which is going beyond the boundaries of codes can bring significant money saving and provide a new insight into administration of structures and decision-making process (Enevoldsen 2001, Bergmeister et al. 2005). The principal methodologies for use of probabilistic based assessment are available and have been proven to work in practice (Casas et al. 2002, fib 2003). Efficient techniques of both nonlinear numerical analysis of engineering structures and stochastic methods are combined here to offer an advanced tool for assessment of realistic behavior of concrete structures from the safety and reliability point of view.

2. STRUCTURAL ANALYSIS AND RELIABILITY ASSESSMENT

The reliability of structures can be calculated from the stochastically obtained structural resistance and expected load distribution. The stochastic response requires repeated analyses of the structure with stochastic input parameters, which reflects randomness and uncertainties in the input values. The presented software system SARA – Structural Analysis and Reliability Assessment – employs the nonlinear computer simulation for realistic prediction of structural response and its resistance. As the nonlinear structural analysis is computationally very intensive, a suitable technique of statistical sampling is utilized, which allows relatively small number of simulations. As the final result, safety and reliability of the analyzed structure can be estimated.

The SARA system consists of four major parts:

- the interactive graphical shell SARA STUDIO for data management and program control
- the nonlinear finite element simulation ATENA

- the statistical and reliability package FREET
- the integrated DATABASE with stochastic parameters of material properties

The presented software package has been successfully used for probabilistic nonlinear analysis of concrete structures (Bergmeister et al. 2002, Pukl et al. 2003). The SARA application is documented in this paper on example of statistical failure simulation and reliability assessment of existing bridge structure, Fig. 1.

Fig. 1: Highway bridge - Colle d'Isarco, Brennero, Italy

3. NONLINEAR FINITE ELEMENT SIMULATION

The nonlinear finite element software ATENA is well established for realistic computer simulation of damage and failure of concrete and reinforced concrete structures in deterministic way (Červenka 2000, 2002, Bergmeister 2005). The constitutive relation in a material point (constitutive model) plays the most crucial role in the finite element analysis and decides how the structural model represents reality, Fig. 2.

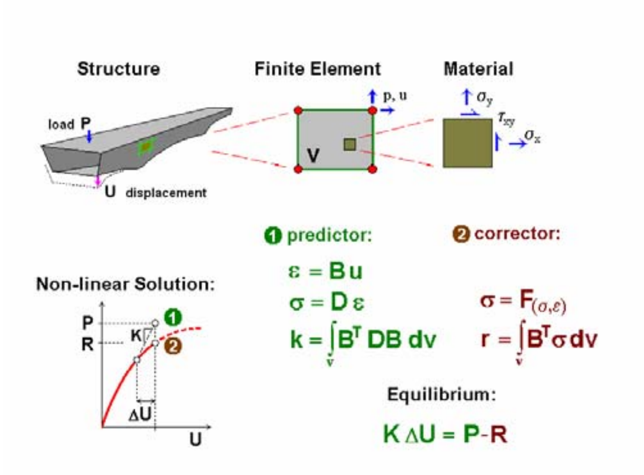

Fig. 2: Scheme of the nonlinear finite element method

Since concrete is a complex material with strongly nonlinear response even under service load conditions, special constitutive models for the finite element analysis of concrete structures are employed (Červenka and Bergmeister 1999, Červenka 2003, Pukl et al. 2005).

Tensile behavior of concrete is modelled by nonlinear fracture mechanics combined with the crack band method and smeared crack concept, Fig. 3. Main material parameters are tensile strength, fracture energy and shape of the stress-crack opening curve.

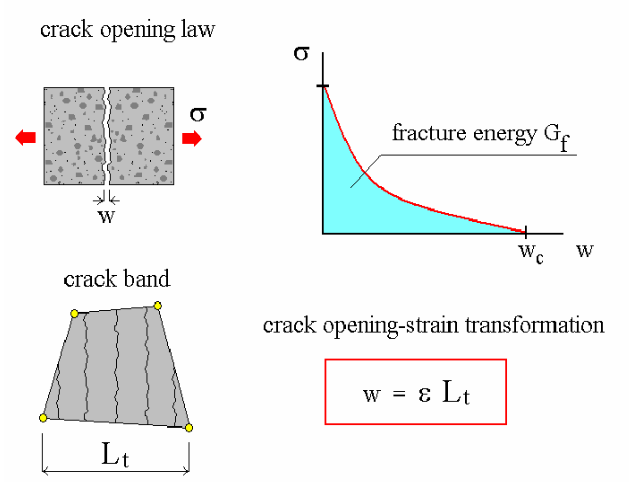

Fig. 3: Smeared crack model for tensile behavior of concrete

A real discrete crack is simulated by a band of localized strains, Fig. 4. The crack strain is related to the element size. Consequently, the softening law in terms of strains for the smeared model is calculated for each element individually, while the crack-opening law is preserved. This model is objective due to the energy formulation and its dependency on the finite element mesh size is neglectable, which was confirmed by numerous studies (e.g. Červenka and Pukl 1995).

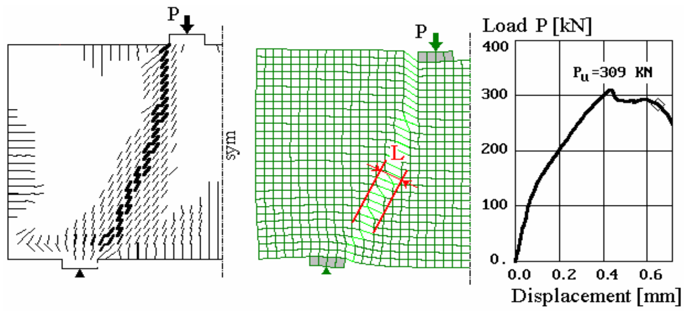

Fig. 4: Crack band in a shear wall analysis

Concrete under multi-axial compression exhibits confinement effect, i.e. increase of the compressive strength due to lateral stresses. This behavior is covered in ATENA by theory of plasticity with a non-associated flow rule, used in the fracture-plastic constitutive model for cementitious materials, Fig. 5.

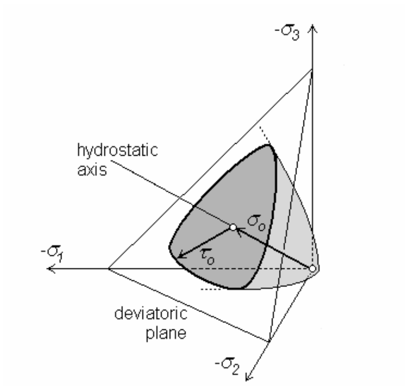

Fig. 5: Concrete failure surface in 3D-stress state

Variety of other material models is implemented in ATENA for support of modelling all specifics in simulation of reinforced concrete structures.

An efficient solution of engineering problems based on the described material models is supported by user-friendly graphical environment (ATENA GUE), which supports the user during pre- and post-processing and enables real-time graphical tracing and control during the analysis. The pre-processing includes an automatic finite element meshing procedure. Reinforcement can be treated in form of reinforcing bars, pre-stressing cables or as smeared reinforcement given by reinforcement ratio and direction. The discrete reinforcement is fully independent on the finite element mesh. The structure can be loaded with various actions: body forces, nodal or linear forces, supports, prescribed deformations, temperature, shrinkage, pre-stressing. These loadings are combined into load steps, which are solved utilizing advanced solution methods. The interactive solution control window enables graphical as well as numerical monitoring of the actual task and supports user interventions during the analysis (user interrupt, restart). The graphical post-processing can show cracks in concrete with their thickness, shear and residual normal stresses. User-defined crack filter is available for obtaining realistic crack patterns. Other important values (strains, stresses, deflections, forces, reactions etc.) can be represented graphically as rendered areas, iso-areas, iso-lines, in form of vector or tensor arrow plots. All values can be also obtained in well-arranged numerical form.

4. PROBABILISTIC-BASED ASSESSMENT

The stochastic engine of the SARA system is the probabilistic program FREET - Feasible Reliability Engineering Efficient Tool. This probabilistic software for statistical, sensitivity and reliability analysis of engineering problems was designed with focus especially on the computationally intensive problems, which do not allow performing thousands of samples (Novák et al. 2003, 2005).

A special type of numerical probabilistic simulation called Latin Hypercube Sampling (LHS) makes it possible to use only a small number of Monte Carlo simulations for a good estimation of the first and second moments of the limit state function. LHS uses the stratification of the theoretical cumulative probability distribution function (CPDF) of input random variables.

Statistical correlation among input random variables can be considered. Stochastic optimization technique called Simulated Annealing (Vořechovský & Novák 2003) is utilized to adjust random samples in such a way that the resulting correlation matrix is as close as possible to the target (user-defined) correlation matrix.

An important task in the structural reliability analysis is to determine the significance of random variables, i.e. how they influence a response function of a specific problem. Sensitivity analysis approach based on nonparametric rank-order statistical correlation with Spearman correlation coefficient or Kendall’s tau is employed. Parallel coordinates representation gives an insight into statistical structure of relationship between random input variables and response output variables. Cornell´s reliability index can be calculated from the limit state function under assumption of normal probability distribution for both structural resistance and acting load. Reliability index is estimated from mean value and standard deviation of the limit state function.

More details to the background of FREET stochastic and reliability methodologies are given in (Novák 2005).

5. ILLUSTRATIVE EXAMPLE

The randomization, analysis and evaluation procedure in SARA is documented on a practical example of stochastic failure simulation and reliability assessment of existing bridge structure: cantilever beam bridge on the Brennero highway in Italy with a length of 167.5 m, Figs. 1 and 6. A permanent monitoring system is installed on this bridge in order to collect data of strains and displacements under traffic load. Evaluation of the measured data in combination with the stochastic nonlinear analysis should be utilized for efficient bridge maintenance (Strauss et al. 2003, Bergmeister 2003, Bergmeister et al. 2005).

The fully post-tensioned box-girder bridge built in 1969 is cast-in-place balanced cantilever beam with varying girder depth. The mid-span has a length of 91 m, the cantilever beams have a length of 59 m and 17.5 m, the total length of the analyzed bridge structure is 167.5 m. The height of the box girder varies from 10.80 m over the middle support to 2.85 m in the mid-span. The bridge is cast from concrete B500 and is reinforced with mild steel BST 500. The post-tension tendons system consists of 211 strands of St 1350/1500.

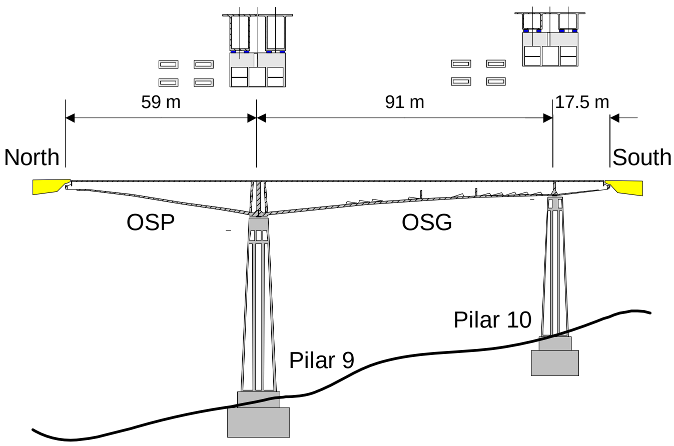

Fig. 6: Colle d'Isarco. Bridge scheme

6. DETERMINISTIC MODEL

First, a deterministic finite element model of the structure is prepared and checked within ATENA. Geometry of the structure is defined; material parameters are given or generated according to the material properties. Boundary conditions and loading history are prescribed. Variables desired for evaluation are selected as monitored values (forces, reactions, deflections, stresses or strains at specified locations etc.). The expected structural response under prescribed loading conditions should be preliminary evaluated in the deterministic nonlinear analysis. The bridge geometry for preparing the finite element model of the example bridge structure is shown in Fig. 7. Mean values of concrete properties for nonlinear analysis were generated from the cubic compressive strength for concrete B500 using ATENA defaults, i.e. built-in recommendations by CEB, fib, RILEM etc.

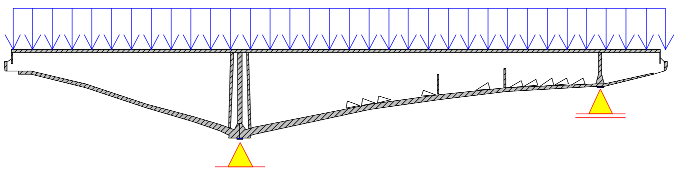

Fig. 7: Bridge geometry for the finite element model

7. STOCHASTIC MODELING

Uncertainties and randomness of the input parameters are modelled as random variables described by their probability density functions (PDF). The user defines in FREET statistical parameters (variance, distribution type) for desired variables. The input parameters from ATENA deterministic model are assumed to be mean values. SARA Studio offers a support for selection of appropriate statistical parameters from the integrated database. Statistical properties of the random variables in the example

(Table 1) originated mostly from the integrated database and from JCSS recommendations (2001).

Table 1. Statistical description of random variables

Variable*	Units	Mean	CoV	Distribution type
E_c	GPa	37.0	0.15	Lognormal
f_t	MPa	3.26	0.18	Weibull
f_c	MPa	42.5	0.10	Lognormal
G_f	N/m	120.0	0.20	Weibull
ρ	MN/m^3	0.023	0.10	Normal
E_s	GPa	210.0	0.03	Lognormal
f_{ys}	MPa	500.0	0.05	Lognormal
f_{yp}	MPa	1350.0	0.20	Lognormal

* Notation of random variables:

Concrete: E_c = Young's modulus of elasticity, f_t = tensile strength, f_c = compressive strength, G_f = specific fracture energy, ρ = specific material weight.

Reinforcement: E_s = Young's modulus of elasticity for both mild steel and pre-stressing tendons, f_{ys} = yield stress of mild steel, f_{yp} = yield stress of pre-stressing tendons.

Statistical correlation among the input variables can be introduced in FREET by a user-defined correlation matrix. Using the Simulated Annealing technique, the desired correlation is imposed, and the undesired correlation due to the random permutation of samples is avoided. In the presented example, the correlation between material parameters was prescribed according to the correlation matrix shown in the upper triangle of Table 2. The lower triangle of Table 2 shows the correlation matrix generated by Simulating Annealing for 30 samples.

Table 2. Correlation of random variables

Variable*	E_c	f_t	f_c	G_f
E_c	1	0.7	0.9	0.5
f_t	0.698	1	0.8	0.9
f_c	0.896	0.798	1	0.6
G_f	0.500	0.892	0.601	1

* Notation of random variables see Notes to Table 1

Number of samples is selected according to complexity of the problem to be solved and required quality of expected results. Already several samples could give a reasonable estimation of stochastic parameters of the structural response (first and second moments) and an acceptable prediction of the reliability index. Random input parameters are generated in FREET according to their PDF using LHS sampling with Simulated Annealing optimization. SARA Studio prepares input data for the multiple nonlinear analyses. In the presented example, stochastic simulations with 8 and 30 samples were performed.

The generated samples are consequently solved in ATENA under SARA Studio control; the complex nonlinear solution is repeatedly performed. The solution procedure for each sample can be traced in the ATENA run-time GUE, the overall survey of the stochastic solution is shown in the SARA Studio shell as histogram of results. Selected monitored values from the structural response (ultimate load, deflection, maximum crack width etc.) are collected.
The relationship between the applied line load and the vertical displacement at selected points has been monitored in the example simulations. The ultimate loads and deflections at the bridge failure were obtained.

The obtained results are transferred to FREET and evaluated in form of histograms of structural response and sensitivity plots. The available results for a monitored value are: histogram, mean, variance, coefficient of skewness, empirical cumulative probability density function of structural response, sensitivity of the structural response to input parameters. Example of a histogram of the displacement at mid-span is shown in Fig. 8, estimated statistical characteristics of the ultimate load (i.e. resistance of the structure) for 8 and 30 samples are compared in Table 3.

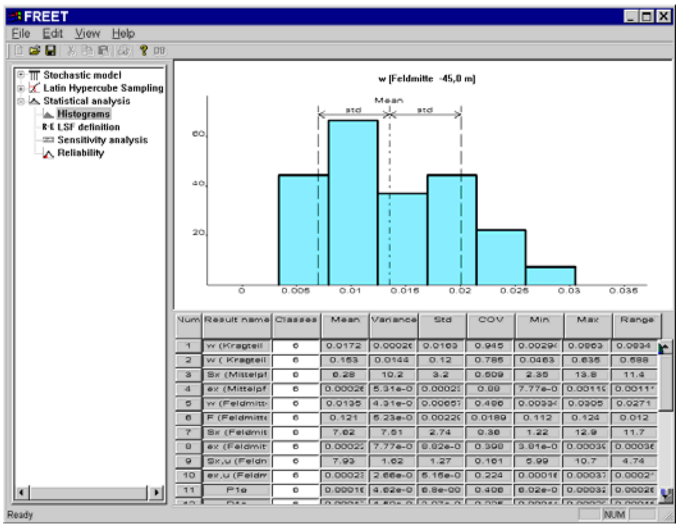

Fig. 8: Histogram of displacement at mid-span

Table 3. Estimation of statistical parameters of the ultimate line load

Number of samples	Mean value	Variance	Standard deviation	Coefficient of variation
	kN/m	$(kN/m)^2$	kN/m	-
8	234.3	388	19.69	0.084
30	235.0	324	18.00	0.077

8. STRUCTURAL SAFETY EVALUATION

Reliability index can be calculated in FREET from the mean value and standard deviation of the structural resistance and acting load, based on the defined limit state function. For the reliability assessment of the Colle d'Isarco bridge the resistance with mean value of 235 kN/m and standard deviation of 18 kN/m was assumed (see Table 3 – case with 30 samples). The reliability index as a function of the mean line load is plotted in Fig. 9. The horizontal line represents the target reliability index 4.7 as specified by Eurocode (2001) for 1 year.

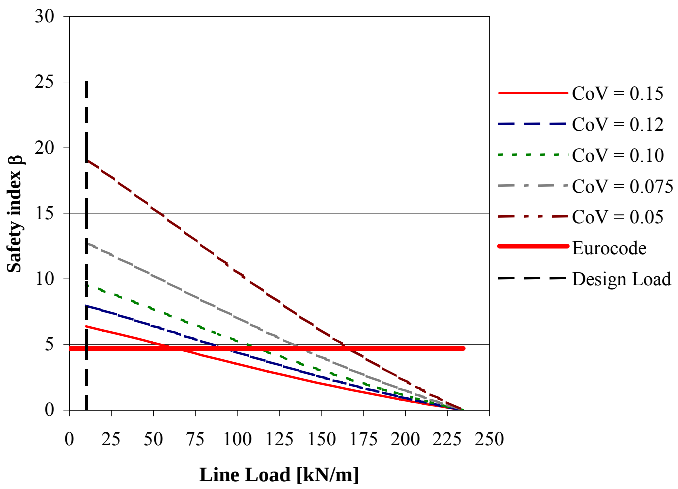

Fig. 9: Reliability index assessment

Reliability index plots for alternative coefficients of variation are compared in Fig. 9. The basic value of 0.15 corresponds to a relatively uniform structure of the traffic. Reduction of the coefficient of variation represents an improvement of information quality regarding the acting load and documents the corresponding increase of the structural reliability.

9. CONCLUSIONS

The presented software system for safety and reliability assessment of concrete structures is ready to use in engineering practise. It integrates the nonlinear finite element modelling with advanced stochastic and reliability technology into a powerful tool, which can support the decision-making process in maintenance of structures and can lead to considerably higher efficiency and cost savings.

ACKNOWLEDGMENT

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PROBABILISTIC AGEING MODEL FOR INFRASTRUCTURE BUILDINGS

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Abstract

The maintenance of structural equipment along the Austrian road network is supported by a database system named BAUT holding various types of structures like bridges, tunnels, noise barriers, retain walls and so on. With the help of this database system the maintainer should retrieve a comprehensive overview of general and technical parameter for decision making. This paper presents a practical approach for prediction of degradation and necessary investments for infrastructure buildings. On basis of simple technical parameters and inspection results collected during the past 8 years valuable results can be presented.

1. INTRODUCTION

During the past 8 years a database system BAUT (2005) was developed. The name stands for **B**uilding database **AU**s**T**ria and hosts nearly all types of structures on the Austrian road network, i.e. bridges, tunnels, retain walls, tunnels, noise barriers etc. BAUT started as a small bridge database and has grown up to a system holding a wide range of information relevant for operating a road network. Beside crude technical aspects also planned maintenance measures, actual construction sites with a lot of information about traffic impact or done rehabilitation works are input into the database.

The software is owned and developed by the road administration ASFINAG responsible for the highway net in Austria. It is used also by all road administrations in the nine counties and by some communities. About 200 users are working with the software.

BAUT offers a tool for engineers to support daily work on structural objects as well as for long term management of the building stock along the road network. Information about administrative and technical data are stored into a hierarchical data structure which offers a great flexibility for further development. The system BAUT offers capabilities for time invariant and variant information like inventory data or inspection results. The history of each input is stored to fulfil today's requirements of e-government. Further on, BAUT supports also typical work flow scenarios like registration of shortcomings, decision of repair action and finally input of information regarding undertaken work or costs.

Each administration in Austria runs its own database system with up to 50 users. Data relevant for ASFINAG are transmitted on-line just in time to a backbone database employing modern techniques like web services. Also peer-to-peer technologies are used to test connectivity between each other and exchange documents.

BAUT contains a description of inventory and time dependent data mainly of those structures, which are critical from a safety point of view. The tool can be applied from either on project level, i.e. retrieving detail data, pictures, inspection reports, as well as on management level by means of summary tables and reports or even OLAP methods.

The basis of the software is a hierarchical data model which provides a flexible solution for description of different kind of structures. It is possible to define for example a very simple small bridge as well as a long, technical complicated bridge within the same schema. The strength of BAUT is a compromise between amount of information stored in the database and completeness of data in it considering the necessary resources keeping a database alive.

2. COLLECTION OF INSPECTION DATA

In Austria an inspection for every type of structure is regulated in codes abbreviated as RVS. There the amount of investigations and the time span between such inspection events are described. The code also defines a condition rating system which follows a scale from one up to five. The first level one stands for newly built and perfect condition and the last level five expresses that the operational condition is never more fulfilled and immediate action has to take place.

During regular inspections every 2 respectively 6 years in average, shortcomings are recognized and reported. This results in a classification of the whole structure as well as of each element on the condition rating scale. Further on, each element itself is subdivided into sub-elements critical for the construction. These check lists are implemented in BAUT. There is the possibility to define dynamically a set of such lists and tailor it to the special needs of each administration.

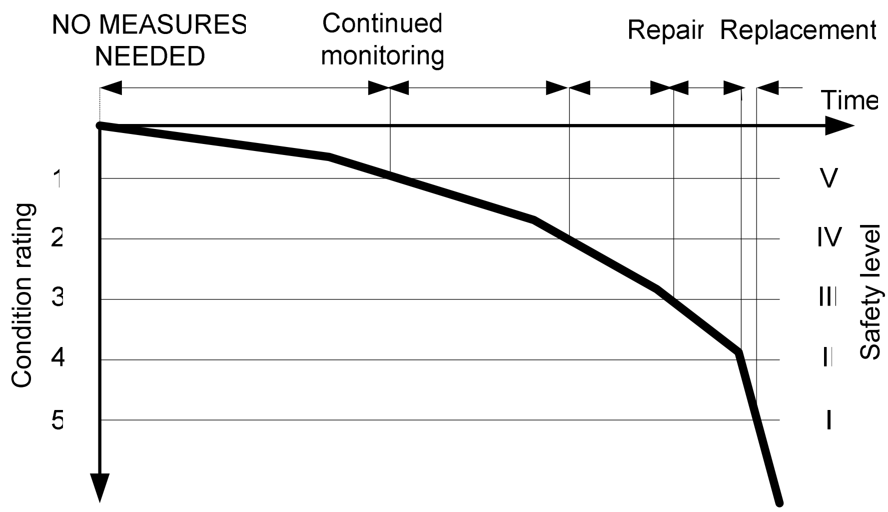

Fig. 1: Relation between condition rating and safety level

The purpose for conducting condition ratings is to determine the remaining life time till rehabilitation works have to be done. In case, data over nearly the life time of the bridges would be available, the quality of predictions would be very good. Actually, only data over a period of 8 years are stored in BAUT and can therefore be used for analysis. This sounds at first bad, but with modern analysis methods good extrapolations can be done. It would be unrealistic to await a completed dataset. There are the costs and time constrains, the man power for inspections is limited and the growing network increases the number of bridges. Also practical considerations regarding impact on traffic or weather conditions limits the time where inspections can take place.

It would be vice, to make selective decisions on what part of the bridge stock to force the power of further investigations. This policy is done in Austria, where for example newly built bridges are inspected for the first time after ten years, or constructions without moveable parts, like expansion joints or bearings, also follow a raster of ten years. On the other hand, bridges in bad condition or simply when there are doubts about their condition, are inspected in shorter time steps.

3. MODELLING BRIDGE DETERIORATION

In order to optimize the reliability of the bridge stock and need to minimize the maintenance costs, it is necessary to develop deterioration models. With such models it is possible to predict future situations within the system. Knowing the deterioration tendency of the bridges rehabilitation and repair measures can be better planned to guarantee a high quality of the road network. Further, prediction of future expenditures can be improved.

Data collection is done inside BAUT by means the special software modules. There all relevant data like construction year, system, material, area and condition assessments are collected and stored in different abstraction layers. By use of data mining technologies correlations between attributes are discovered. This is important in order to define similar types of constructions, whereby this paper focuses only on bridges. Basic drawbacks from this analysis are normally in correspondence with the experience of practical engineers.

The data analysis starts in formulation of bridge population with similar behaviour. Thereby, age and condition are the most important parameters. Due to the wide range of deterioration causes and repair options, it seems reasonable that deterioration rate and selection of repairs methods or even replacement are random processes.

Central for the further analysis is to establish a warning level. The normal approach is to treat bridges with condition level four. The deterioration model must be able to predict how long it will take the bridge to progress to level five. The experience shows a time span from 3 to 6 years. Determining the remaining useful life time decision about repair measure can be taken. With the possibility to fix rehabilitation requirements in time, a long-term maintenance and financial planning program can be established.

In principle the modelling of bridge ageing and deterioration is probabilistic in nature. Therefore, when observed data must be interpreted and used as a basis for system predictions, probabilistic techniques are employed. Two approaches are available, the Markov Chain and the Cohort Survival model. The first method can be characterized shortly as very data intensive when setting up transition probabilities. The accuracy of these transition probabilities are very sensitive to the prediction abilities of the model. As mentioned before, inspection data from the last 6 up to 8 years are available in a consistent manner. This limited history for each bridge makes the Cohort Survival approach the ideal candidate from the authors point of view.

4. BASIC STATISTICS

In principal the ageing is described by a hazard function h(t), which describes the transition of a structure from one condition level to the next worse. The hazard function is defined by

$$\lambda(t) = \frac{f(t)}{R(t)} \tag{1}$$

where f(t) is the probability density function of bridge ages and R(t) is the survival probability, see (Stuart and Ord, 1994). In terms of a bridge, R(t) defines how long it operates at a certain condition level. The hazard function h(t) defines a time dependent failure rate. Within the cohort survival model time and age are synonyms.

In engineering the Weibull model is one of the most widely used distributions when fitting failure rate data, see (Lewis, 1987). It is very flexible especially when using a set of Weibull distributions, normally used to describe the so-called *bathtub* hazard curve. Another approach used in this paper is a distribution developed by (Herz, 1994), who defined a suitable service life density function for the purposes mentioned herein.

$$f(t) = \frac{(a+1)b\exp[b(x-c)]}{(a+\exp[b(x-c)])^2} \tag{2}$$

$$R(t) = \frac{a+1}{a+\exp[b(x-c)]} \tag{3}$$

where *a* is the ageing parameter, *b* is the shape parameter and *c* is the resistance time until transition is possible at first time. Based on available data for bridges in BAUT parameters are estimated for each condition level. In Fig. 1 the resulting probability density functions and hazard curves are plotted. The hazard curves have to be interpreted as transition probabilities from one condition level to the next. The whole life time of a bridge, from year of construction, during time of operation until repair action is described by transition rates from the first ageing curve (green) up to the last one (red). The average life time is a result of simultion.

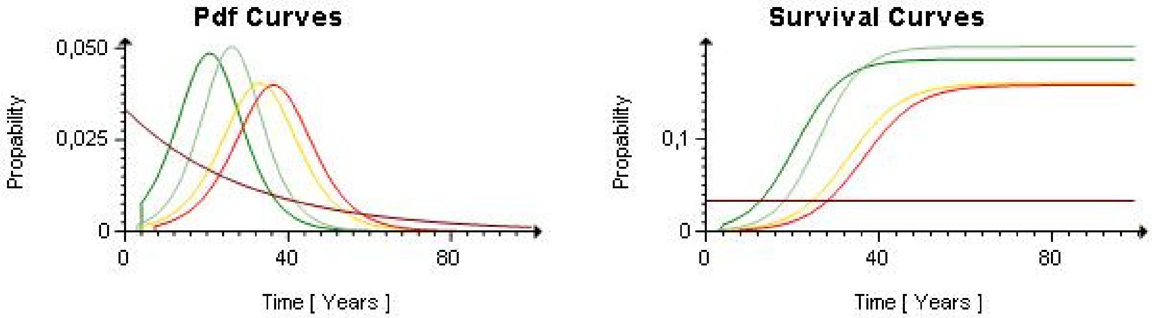

Fig. 2: Density functions and hazard rates for different condition levels

5. COHORT SURVIVAL METHOD

The Cohort Survival Method (CSM) is applied to make prediction of further population numbers. Originally used by demographers using mortality and fertility data to analyse development of human population. Thereby a cohort corresponds to a group of individuals that are born during a certain time period. The demographer use age and gender specific death rates to calculate the number of survivors within each of the cohort age groups.

The key idea is adopted to infrastructure, particularly to bridges in this paper. The life cycle process can be thought similar to that of humans. When a bridge is built and taken into operation the process of ageing and deterioration starts.

In case of a failure of a bridge this is equivalent to the death in the human cohort model. Deaths in the human population are replenished by the natural reproduction of its members. In the bridge cohort model objects are replaced by new constructions, or they are repaired or strengthened. These actions need monetary input. As with social planning that occurs with human population prediction, the road administration needs a tool for planning the monetary investment amount in future. The net present value is archived by correcting the evaluated monetary input by following function:

$$D(r, y) = (1+r)^{y+Y_0} \tag{4}$$

where r is the constant discounting rate, y is the actual year of prediction and Y_0 is the base year of analysis.

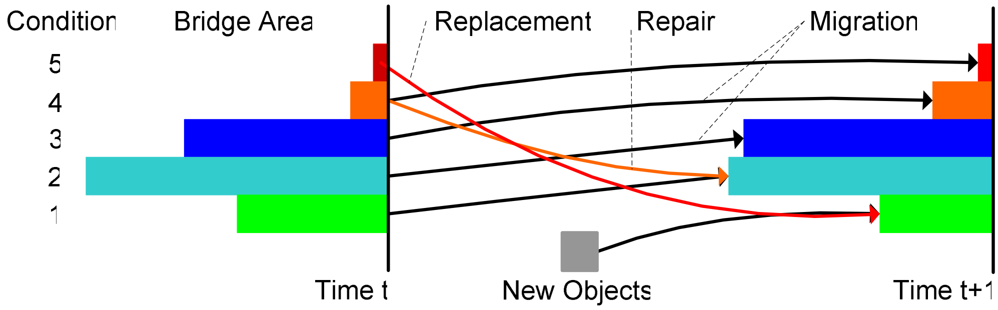

Fig. 3: Principle of Cohort Survival Method

The time span a bridges resists in one condition class or cohort in terms of this model is a random quantity. The distribution of ages in each condition class, the point in time of transition into the next worse class, and the transition rate are related to each other.

The model in Fig. 3 is applicable to all road pavements as well. In principle it is based on a system parameter control or damage accumulation. In the particular case the global condition rating describes the actual state. A multidimensional state description would consider the rating of the elements like sub- and superstructure, bearuings, expansion joints, equipment and so on.

6. MAINTENANCE STRATEGIES

Minimum condition level is defined as a fixed percentage of bridges in class four compared to the total bridge area. It is normally assumed that bridges with such a classification will need rehabilitation work during the next 3 up to 6 years. From that point of view the normal strategy is defined. A fraction of about 1/3 of the bridge area with condition rating four should be repaired within one year. A repaired bridge will not be perfect as a new one. In the cohort survival model it will therefore come back into condition level 1 or 2. On the other hand when a bridge will be replaced at least the super-structure, than it will move down into condition level 1 and start with a new building year.

Another strategy may be to change the fraction of replacements compared to repair over the time. This preventive strategy may significantly reduce the transition rate into condition level 4. Normally, ideal maintenance is assumed. In terms of the bridges cohort model a repaired bridge will move into condition level 2. The probabilistic approach may introduce some disturbances to model not perfect conditions after repair caused by low quality of work.

Now a days, constraints are the maximum number of construction sites on the road network. A road administration like ASFINAG must offer a high quality of the roads for the users. This results also in the need to have only moderate disturbances and elongation in travel time during building season compared to perfect situations with no constructions sites. The quality of service will be measured by such indicators.

The most convincing constraint is the limited budget. Therefore forward-looking maintenance strategies supports decision makers in their financial planning. The aim is to optimize investments over time in order to reach a high condition level. This is done by using different strategies and varying the input parameters like average unit costs and discounting rate.

7. ANALYSIS

On base of the probabilistic code VaP 2.1 the Cohort Survival Simulation was implemented, see (PSP, 2004). The survival functions are estimated from available data whereby two basic distribution types are offered, i.e. Weibull and Herz. The parameters are fitted by a Maximum Likelihood approach considering the censored information. The strategies and financial limits are changed easily during runtime to figure out sensitivities.

The Analysis is done by simulation according to the principle in figure 3. For each time step the equilibrium is calculated and the results like total bridge area inside one cohort and accumulated costs are stored. The result for a typical bridge population in Austria is presented in figure 4. The actual forecast is done for 60 years with a discounting rate of 0%.

The average costs are taken from projects during the past 5 years. Two types of maintenance actions are distinguished, repair and replacement. Due to the growing net also newly built bridges are considered in the simulation but they contribute not very much. Projection of costs in the future are done by discounting the monetary values, see equation (4).

Figure 4 shows at the left the development of the predicted fraction of bridge area for each cohort in the future. The cohorts or bridge areas at a specific condition level are drawn from bottom up to top by different colours. The lowest area (green) shows condition level one, the upper most visible area is condition level four. Five is very small and is hidden by this resolution.

It can be seen, that till 2020 an continuous increase of condition class four happens, the upper most area. This is caused by the existing age distribution of other condition classes. From them a higher rate of bridges will move in future into this condition class four. The growing amount of repair/replacement actions could be damped by adopting another strategy. On possibility would be increase the amount of repair in the next years in order to have a moderate increase during the following years.

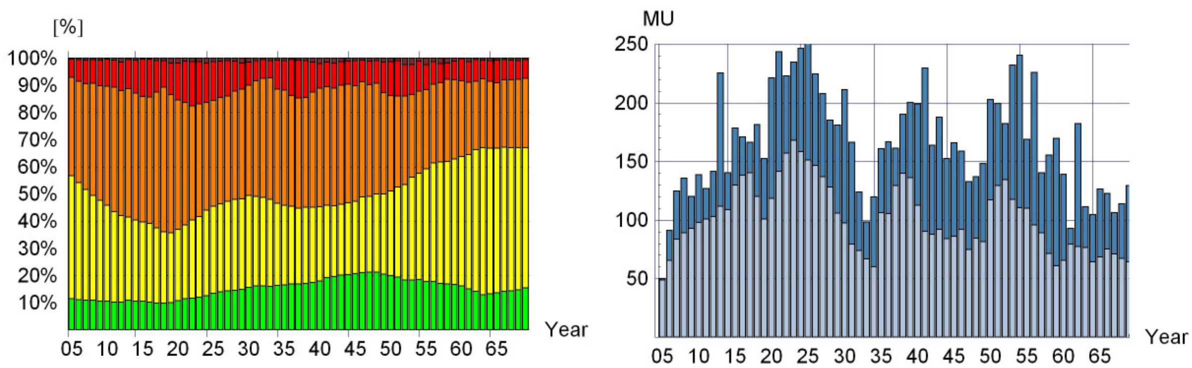

Fig. 4: Results from CSM simulation for whole bridge population.

Further on, Fig. 4 shows at the right the yearly costs, caused by the repair (light) and replacement actions (dark), see also Fig. 3 where the principle is given. Investment costs for newly built bridges are very small and disappear by this kind of presentation. Each column in the curve stands for the yearly amount of investment. Additional costs for minor repair are ignored herein for they belong to another budget.

The reinforced and prestessed bridge populations are plotted in Fig. 5 and 6. Reinforced bridges form the biggest group in terms of number, whereby prestressed bridges give in sum the largest area. It can be seen that prestressed bridges contribute mainly to the necessary increase of investments. They are in average 25 years old and must be rehabilitated due to low quality standard during the building boom in the 70th and 80th. The investments are not equally distributed or at least constant over some period. But this is, what road administrations are looking for.

The simulation tool can now be used to change the strategy considering a fixed budget. This will lead to a different distribution of condition ratings. The optimization can now be done in variing the maximum budget and/or combining strategies.

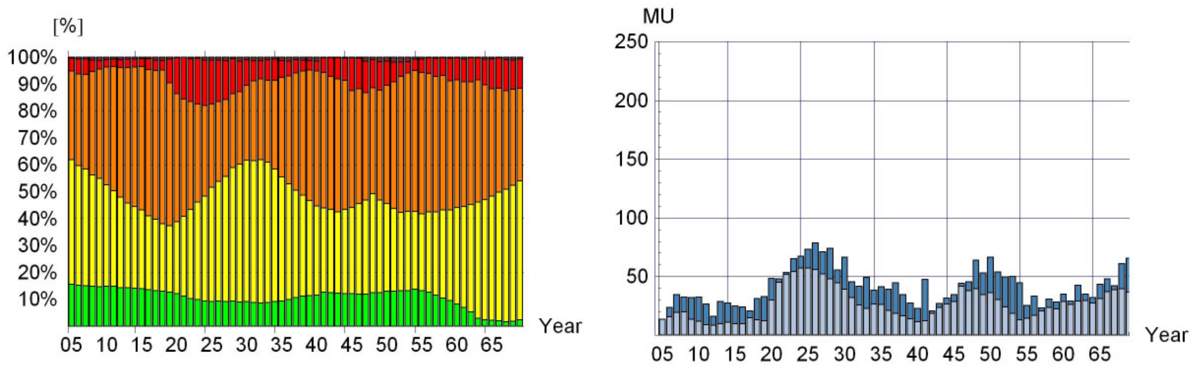

Fig. 5: Results from CSM simulation for reinforced bridges.

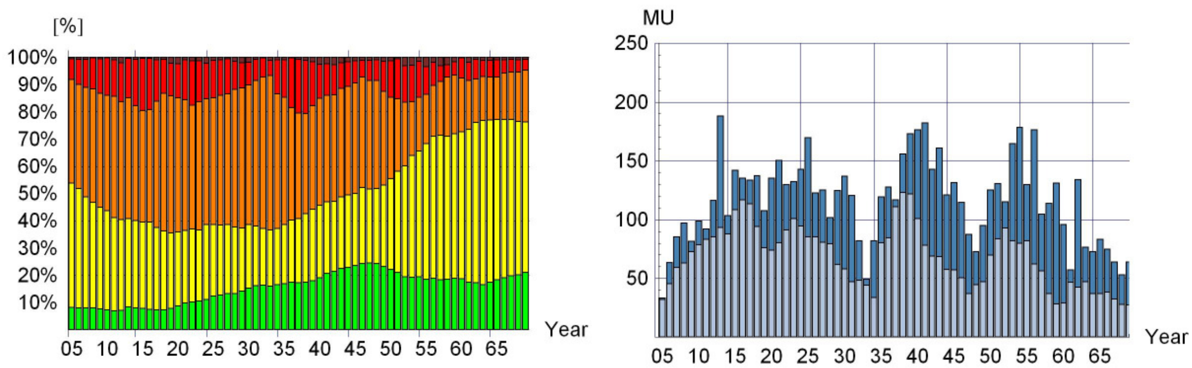

Fig. 6: Results from CSM simulation for prestressed bridges.

8. CONCLUSIONS

Maintaining the serviceability of bridges along the road network and preventing failures involves considerable repair and investment costs. The financing of it can lead to an unforeseeable increase of the budget. This is caused by the rehabilitation backlog of older parts in the system which overlaps with rehabilitation requirements of more recent built bridges caused by increased traffic load and environmental impacts. In such a situation modern software tools have to be employed, which support indication of weak points, interactive development of strategies and optimization of financial planning.

It can be shown, that the condition level of bridges can be improved in the short-term and secured in the long-term view without an extraordinary increase of the budget. The presented software BAUT shows a tool used in the daily work at ASFINAG. The information hold in the database is sufficient employing a probabilistic forecasting tool.

Asset management can be defined as a systematic process of operating, maintaining, and upgrading assets in a cost effective manner. Today, inspection based asset management is usually done. The measures are determined by actual inspection results. One step towards a more sophisticated approach is to use the cohort survival model. This predictive asset management optimizes infrastructure performance and reliability at the lowest possible price. This will be the most desirable approach to asset management because it encourages better prediction of failures, planning of repairs and/or replacements, and resource allocation.

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BERÜCKSICHTIGUNG VON ÜBERWACHUNGSMASSNAHMEN IM RAHMEN DER ZUVERLÄSSIGKEITSANALYSE VON BETONBAUTEILEN

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ABSTRACT

Die Qualität einer baulichen Anlage spiegelt sich in ihrer Zuverlässigkeit wider. Bei qualitätsorientierter Herstellung sind geringere Streumaße eingehender Parameter wie bspw. der Abmessungstoleranzen oder Materialfestigkeiten vorhanden. In einer probabilistischen Analyse werden diese Zufallsvariablen mit unterschiedlichen Streumaßen für verschiedene Überwachungsintensitäten auf der Beanspruchungs- und Widerstandsseite berücksichtigt. Dadurch wird eine Aussage über den Einfluss einer Überwachungsintensität auf die operative Versagenswahrscheinlichkeit und daraus resultierende normative Sicherheitselemente möglich.

1. EINLEITUNG

Die Zuverlässigkeit von Betontragwerken wird zum einen durch verschiedenartige Einwirkungen und zum anderen durch den Tragwiderstand, den das Bauteil oder Tragwerk der einwirkenden Größe entgegenbringt, beeinflusst. Die statistische Beschreibung der Einwirkungen unterliegt oftmals relativ großen Ungenauigkeiten, so dass sie in normativen Festlegungen für die Tragwerksbemessung in der Regel als deterministische Lastgröße definiert wird. Der Tragwiderstand von Betonbauteilen ist einerseits von der Betonfestigkeit, die in hohem Maße von den Herstellungsbedingungen abhängt, anderseits aber auch von dem untersuchten Versagensfall abhängig. Somit bestehen bei einem Betonbauteil unter gleicher Belastung unterschiedliche Zuverlässigkeiten für z. B. Normalkraft-, Biege- oder Querkraftversagen.

Bei einer besseren Ausführungsqualität ist eine höhere Tragwerkszuverlässigkeit zu erwarten. In der nationalen Norm [1045-1/01] wird dieser Behauptung durch unterschiedliche Teilsicherheitsbeiwerte für Fertigteile und andere Bauteile aus Beton Rechnung getragen. Die angesetzten ca. 10%-igen Unterschiede sind zwar nicht wissenschaftlich belegt, erscheinen jedoch durchaus begründet, da zwischen der Betonfertigung im Fertigteilwerk, im Lieferbetonwerk oder auf Baustellen nicht zu vernachlässigende Unterschiede bestehen. So sind bei der Betonherstellung auf der Baustelle die Unsicherheiten und Streuungen der zu bestimmenden statistischen Kennwerte sehr groß. Die Wahrscheinlichkeit des Eintritts von systematischen Fehlern, zufälligen Fehlern und groben Fehlern ist wesentlich höher als bei der automatisierten

Fertigung. Aus der unterschiedlichen Güte von Überwachungsmaßnahmen bei der Herstellung von Stahlbetonbauteilen resultieren Unterschiede in der Bauteilzuverlässigkeit. Diese sollen nachstehend dokumentiert werden.

2. QUALITÄTSANFORDERUNGEN UND ÜBERWACHUNGSINTENSITÄT

Die statistische Auswertung der Festigkeitskennwerte von Beton ist nicht neu. Durch die Anwendung hochwertigerer Baustoffe und Herstellungsverfahren können jedoch die heutigen Ergebnisse von den bisherigen Erkenntnissen abweichen.

So wurde bspw. für die Streuung der Betondruckfestigkeit in [Rüsch et al.-69] festgestellt, dass diese mit einer fast konstanten mittleren Standardabweichung von rund 5,0 N/mm^2 näherungsweise unabhängig von der mittleren Druckfestigkeit ist. In [JCSS-00b] wird jedoch für verschiedene Betonfestigkeitsklassen unter Differenzierung in Baustellenbeton, Transportbeton und Beton für Fertigteile eine Unterscheidung hinsichtlich der vorhandenen Überwachungsintensitäten durchgeführt.

Auch die Festlegung der für probabilistische Untersuchungen wichtigen Verteilungsart muss hinterfragt werden. Für eine Grundgesamtheit von Betonwürfeldruckfestigkeiten wird, wie für Massengüter zu erwarten ist, in der Literatur die Normalverteilung als beste Beschreibung der empirischen Verteilungsfunktion angegeben. Um unsinnige negative Festigkeitswerte zu vermeiden, wird diese zumeist noch logarithmiert. Andere Verteilungen sind trotz z. T. besserer Anpassungsgüte an vorhandene Messergebnisse aufgrund eines unangemessen hohen mathematischen Aufwandes nicht praktikabel. Bei der in [Hansen-04] durchgeführten statistischen Auswertung von 10.634 Würfelprüfungen aus der Eigenüberwachung verschiedener Transportbetonwerke an fünf Standorten in Deutschland wurde jedoch festgestellt, dass die Messwerte in vielen Fällen nicht der zu Grunde gelegten Normalverteilung entsprachen.

Neben der angesetzten Verteilungsart und Streuung der Festigkeitswerte sind auch verarbeitungsspezifische Unterschiede zu beachten. Durch die Messung an Prüfkörpern entstehen systematische und zufällige Abweichungen von den Festigkeiten im Bauwerk. Diese sind bedingt durch die Übertragung der Laborergebnisse auf die Verhältnisse im Bauwerk, die Streuungsunterschiede der Baustoffeigenschaften im Bauwerk und im Labor und zeitabhängige Baustoffeigenschaften, vgl. Bild 1.

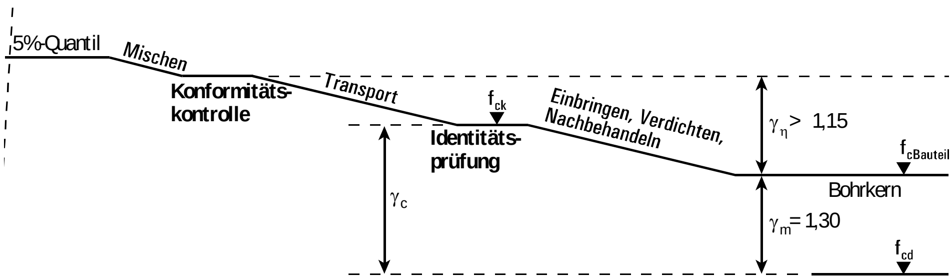

Bild 1 Einflüsse auf die Betondruckfestigkeit und die durchzuführenden Prüfungen

In verschiedenen Untersuchungen wurden die durch eine sog. „Übertragungsvariable" zu berücksichtigenden Unterschiede aus dem Transportvorgang des Betons vom Lieferwerk zur Baustelle erfasst und das Verhältnis zwischen charakteristischer Bauwerksfestigkeit $f_{ck,BW}$ und Zylinderdruckfestigkeit $f_{ck,cyl}$ bestimmt. In der Fachliteratur hat sich für die Übertragungsvariable der Quotient 0,85 bzw. als Teilsicherheitsbeiwert die Inverse ≈1,15 durchgesetzt.

In den nachstehenden Ausführungen werden drei Überwachungsintensitäten definiert, die sich hinsichtlich der bei der Betonherstellung und Bauausführung vorhandenen Streuungen der eingehenden Variablen unterscheiden.

3. MODELLPARAMETER DER PROBABILISTISCHEN RECHNUNG

3.1 Qualitätsbeeinflusste Basisvariablen

Durch eine verstärkte Überwachung der Materialherstellung und Bauausführung werden einzelne Basisvariablen beeinflusst. So kann durch eine Differenzierung der Beton- und Bauteilherstellung nach Tabelle 1 der Einfluss einer veränderten Überwachungsintensität probabilistisch erfasst werden. Die einzelnen Kennwerte sind dabei auf Grundlage einer umfangreichen Literaturrecherche ausgewählt worden, vgl. [Hansen-04].

Tabelle 1 Wesentliche Unterschiede infolge verschiedener Überwachungsintensitäten für die probabilisitische Berechnung

Überwachungsintensität		**1**	**2**	**3**
Bezeichnung (allgemein)		Fertigteil	Transportbeton	Baustellenbeton
Bauteilherstellung		Werk	Baustelle	Baustelle
Betonherstellung		Werk	Werk	Baustelle
Betondruckfestigkeit f_c	v [%]	0,150	0,225	0,300
Statische Nutzhöhe d	v_d [%]	1,0	2,5	
Bauteilbreite b	v_b [%]	0,8	2,0	
Streckgrenze f_y	v [%]	5,4		
Längsbewehrung A_{sl}	v [%]	2,5		
Querkraftbewehrung a_{sw}	v [%]	2,5		
Ständige Einwirkung	v_G [%]	0,024	0,060	

3.2 Untersuchte Grenzzustände

Die untersuchte Versagensbedingung für eine Bauteiltragfähigkeit wirkt sich zum einen durch die Definition des Grenzzustandes und zum anderen durch die angesetzten Modellunsicherheiten auf der Widerstandsseite unmittelbar auf die Tragwerkszuverlässigkeit aus. Um qualitative und quantitative Aussagen über den Einfluss einer unterschiedlichen Überwachungsintensität nach Tabelle 1 zu verdeutlichen, werden daher verschiedene Grenzzustände untersucht. Diese werden durch die Versagensbedingungen nach [1045/1-01] definiert.

Innerhalb einer probabilistischen Analyse wird die Grenzzustandsgleichung für die Normalkrafttragfähigkeit N mit Gl. (1) beschrieben. Der Modellfaktor für das Widerstandsmodell M_R wird in Abs. 3.3 erläutert.

$$N = M_R \cdot \left(b \cdot h \cdot f_c + A_s \cdot f_y \right) \qquad (1)$$

Das Widerstandsmodell „Biegung“ (ohne Normalkraft) kann nach Gl. (2) beschrieben werden.

$$M = M_R \cdot \left[A_s \cdot f_y \cdot d \cdot \left(1 - k \cdot \frac{A_s \cdot f_y}{b \cdot d \cdot f_c} \right) \right] \text{ mit } k = \frac{k_a}{\alpha_R} \tag{2}$$

Dabei werden der Lagebeiwert der Betondruckkraft k_a und der Völligkeitsbeiwert α_R nach Gl. (3) berücksichtigt.

$$k_a = 0{,}493 - f_{ck} \cdot \frac{0{,}076}{60} \qquad \alpha_R = 0{,}87945 - 0{,}21921 \cdot \sqrt{\frac{f_{ck}}{60}} \tag{3}$$

Für den probabilistischen Nachweis der Querkrafttragfähigkeit werden Gl. (4) bis (6) verwendet.

$$V_{ct} = M_R \cdot \left[0{,}10 \cdot \left(1 + \sqrt{\frac{200}{d}} \right) \cdot \left(100 \cdot \frac{A_{sl}}{b_w \cdot d} \cdot f_c \right)^{1/3} \right] \cdot b_w \cdot d \tag{4}$$

$$V_{\max} = M_R \cdot \left(\frac{b_w \cdot 0{,}9d \cdot 0{,}75 \cdot f_c}{\cot\theta + \tan\theta} \right) \tag{5}$$

$$V_{sy} = M_R \cdot \left(\frac{A_{sw}}{s_w} \cdot f_y \cdot 0{,}9d \cdot \cot\theta \right) \tag{6}$$

Durch die Auflösung des Druckstrebenwinkels θ nach Gl. (7) sind Gl. (5) und (6) auch von der Einwirkungsseite abhängig.

$$\cot\theta = \frac{1{,}2}{1 - \dfrac{0{,}216 \cdot f_c^{1/3} \cdot b_w \cdot d}{M_E \cdot (G+Q) \cdot \left(1{,}25 \cdot \dfrac{l}{2} - 0{,}5 \cdot b_{Aufl} - d \right)}} \tag{7}$$

Analog zum probabilistischen Nachweis der Querkrafttragfähigkeit nach Gl. (4) wird der Nachweis des Tragwiderstandes gegen Durchstanzen mit Gl. (8) geführt.

$$v_{ct}^R = M_R \cdot \left[0{,}10 \cdot \left(1 + \sqrt{\frac{200}{d}} \right) \cdot \left(100 \cdot \frac{a_{sl}}{d} \cdot f_c \right)^{1/3} \right] \cdot d \tag{8}$$

3.3 Modellunsicherheiten

Da die realen Tragfähigkeiten durch Versagensmodelle nur ungenau abgebildet werden können, sind für probabilistische Berechnungen auch Angaben über die zu berücksichtigenden Modellunsicherheiten erforderlich. In [1990-02] wird ein Verfahren zur versuchsgestützten Bestimmung von Modellunsicherheiten angegeben. Dabei wird für ein mechanisches oder empirisches Bemessungsmodell über den Vergleich der experimentellen Werte mit den rechnerischen Werten eine Modellstreuung ermittelt. Der Variationskoeffizient der gesamten Bemessungsfunktion v_r kann dann nach Gl. (9) aus den Variationskoeffizienten der Modellunsicherheiten v_δ und der Material- und Bauteilstreuungen v_{rt} bestimmt werden.

$$v_r^2 = \left(v_\delta^2 + 1\right) \cdot \left(v_{rt}^2 + 1\right) - 1 \approx v_\delta^2 + v_{rt}^2 \tag{9}$$

Mit diesem Vorgehen werden in [Hansen-04] Versuchsreihen ausgewertet, deren Ergebnisse in Tabelle 2 angegebenen sind.

Tabelle 2 Statistische Kennwerte der Widerstandsmodelle

Widerstandsmodell		Variationskoeffizient
Normalkraft	N_{Rd}	0,050
Biegung	M_{Rd}	0,070
Querkraft	$V_{Rd,i}$*[1]	0,155
Durchstanzen	$v_{Rd,ct}$	0,170

*[1] Die Modellunsicherheit ist für alle Querkraftwiderstände gleich.

4. VERSAGENSWAHRSCHEINLICHKEITEN UND SICHERHEITSELEMENTE

4.1 Probabilistische Verfahren

Im Rahmen der durchgeführten Untersuchungen werden verschiedene Methoden der Zuverlässigkeitsanalyse eingesetzt, die sich hinsichtlich ihres Genauigkeitsgrades unterscheiden (Level of Sophistication). Als Methoden der Stufe II werden *FORM* und *SORM* genutzt. Genauere Ergebnisse werden mit den Methoden der Stufe III, der *Numerischen Integration (NI)* und der *Monte-Carlo-Methode mit Importance-Sampling (MCMIS)* erzielt. Für die vergleichenden Untersuchungen werden die Ergebnisse der FORM-Berechnung verwendet.

4.2 Versagenswahrscheinlichkeiten und Wichtungsfaktoren

Mit den Ergebnissen einer FORM-Berechnung wird ein Vergleich der Zuverlässigkeitsindizes und Wichtungsfaktoren bei Ansatz unterschiedlicher Eingangsgrößen möglich. Eine Berechnung mit FORM liefert im Vergleich zu den anderen Verfahren in der Regel ungünstigere Ergebnisse. Für das Widerstandsmodell Normalkraft beträgt der Unterschied zwischen den Zuverlässigkeitsindizes nach FORM- und SORM-Berechnung bis zu 7%. Für das Widerstandsmodell Biegung nach Gl. (2) sind vergleichbare Aussagen möglich, beim Widerstandsmodell Querkraft nach Gl. (6) sind diese Unterschiede geringer.

Mit Hilfe einer probabilistischen Rechnung werden die Einflüsse einer Ausführungsüberwachung auf die Zuverlässigkeit untersucht. Dafür wird der Beitrag einzelner Basisvariablen anhand der globalen Wichtungsfaktoren α_E und α_R nach Gl. (10) analysiert.

$$\alpha_E = \frac{\sigma_E}{\sqrt{\sigma_R^2 + \sigma_E^2}} \qquad \alpha_R = \frac{\sigma_R}{\sqrt{\sigma_R^2 + \sigma_E^2}} \tag{10}$$

Mittels der Wichtungsfaktoren kann der zunehmende Einfluss des Modellfaktors für das Widerstandsmodell M_R verdeutlicht werden. So ist bspw. bei der Nachweisgleichung für Biegung nach Bild 2 eine deutliche Reduzierung der Wichtung der veränderlichen Einwirkung Q zu erkennen, die weiteren Einflussgrößen bleiben annähernd konstant.

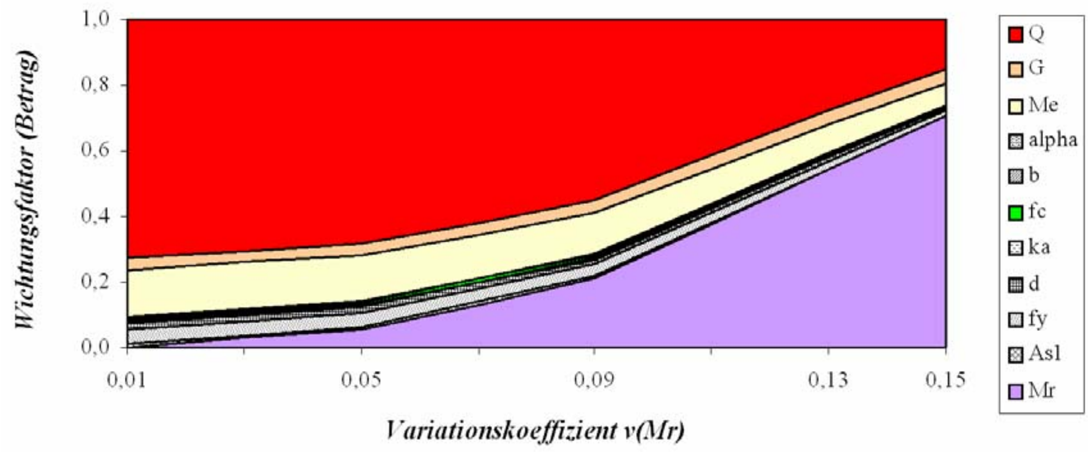

*Bild 2 Wichtungsfaktoren in Abhängigkeit des Variationskoeffizienten des Modellfaktors der Widerstandsgrößen v(M_R) für das Widerstandsmodell **Biegung** bei Baustellenfertigung mit Baustellenbeton (**Überwachungsintensität 3**)*

Bei dem Widerstandsmodell Querkraft ist eine gleichartige, wenn auch nicht ganz so ausgeprägte Wechselbeziehung zu beobachten. Die Auswirkung einer verbesserten Überwachung macht sich bei diesen beiden Widerstandsmodellen hauptsächlich durch den reduzierten Einfluss der ständigen Einwirkung G und der Bauteilgeometrien bemerkbar.

Auch bei dem Widerstandsmodell Normalkraft nach Bild 3 ist der Anstieg des Einflusses des Modellfaktors M_R mit der Reduzierung der Wichtung der veränderlichen Einwirkung Q verbunden. Zudem wirkt sich die zunehmende Streuung von M_R auch auf den Einfluss der Betondruckfestigkeit f_c aus.

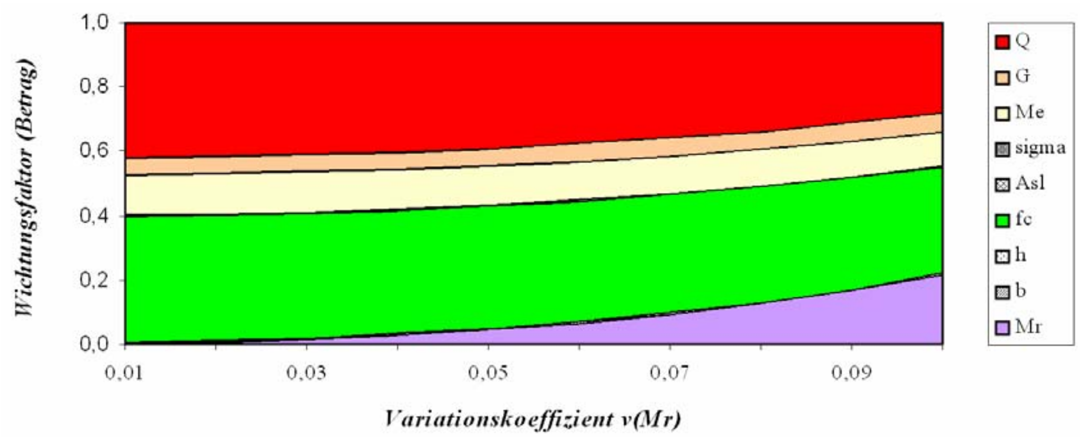

*Bild 3 Wichtungsfaktoren in Abhängigkeit des Variationskoeffizienten des Modellfaktors der Widerstandsgrößen v(M_R) für das Widerstandsmodell **Normalkraft** bei Baustellenfertigung mit Baustellenbeton (**Überwachungsintensität 3**)*

Bei einer verbesserten Überwachung nach Bild 4 nehmen die Streuungen der Betondruckfestigkeit f_c, der Abmessungen b und h sowie der damit festgelegten ständigen Einwirkung G stark ab. Dadurch verringert sich auch die Wichtung dieser Variablen zugunsten der verbleibenden Größen.

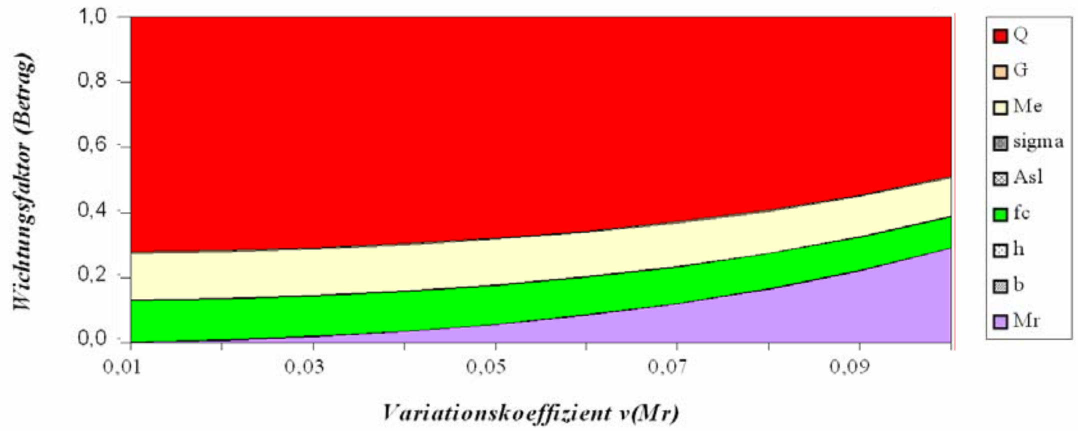

Bild 4 Wichtungsfaktoren in Abhängigkeit des Variationskoeffizienten des Modellfaktors der Widerstandsgrößen v(M_R) für das Widerstandsmodell ***Normalkraft*** *bei Fertigteilen (****Überwachungsintensität 1****)*

4.3 Teilsicherheitsbeiwerte

Der Teilsicherheitsbeiwert γ_E für die Einwirkungsgrößen kann nach Gl. (11) und γ_R für die Widerstandsgrößen nach Gl. (12) für normal- bzw. logarithmisch normalverteilte Basisvariable formuliert werden.

$$\gamma_E = \frac{1+\beta \cdot \alpha_E \cdot v_E}{1+k_E \cdot v_E} \tag{11}$$

$$\gamma_R = \frac{1-k_R \cdot v_R}{1-\beta \cdot \alpha_R \cdot v_R} \quad \text{bzw.} \quad \gamma_R = \frac{R_k}{R_d} = \exp[(\alpha_R \cdot \beta + \Phi^{-1}(p)) \cdot v_R] \tag{12}$$

Die Bemessungswerte für Beanspruchungen und Widerstände werden in der Regel für einen vorgegebenen Zuverlässigkeitsindex β festgelegt. Die globalen Wichtungsfaktoren sind abhängig vom anvisierten Zuverlässigkeitsindex. Für die praktische Anwendung z. B. in [1055/100-01] werden feste Werte $\alpha_R = 0{,}8$ und $\alpha_E = -0{,}7$ derart definiert, dass der anvisierte Zuverlässigkeitsindex β nicht wesentlich unterschritten wird. Durch diese getrennte Behandlung der Einwirkungen und Widerstände vereinfacht sich die Untersuchung eines Grenzzustandes wesentlich!

Im Stahlbetonbau ist in erster Linie der logarithmisch normalverteilte Ansatz nach Gl. (12) für die Materialien Beton und Bewehrungsstahl von Interesse. Generell sind für diese beiden Baustoffe die gleichen Einflussfaktoren nach Gl. (13) zu berücksichtigen.

$$\gamma_R = \gamma_M \cdot \gamma_\eta = (\gamma_m \cdot \gamma_{Rd}) \cdot \gamma_\eta = (\gamma_m \cdot \gamma_{St} \cdot \gamma_a) \cdot \gamma_\eta \tag{13}$$

M	= Grundwert	m	= Festigkeit
Rd	= Modellfaktor	St	= Widerstandsmodell
η	= Übertragung (auch „conv“)	a	= Geometrie

Der Grundwert γ_M beinhaltet statistische Festlegungen für die Materialfestigkeit „m“, das Widerstandsmodell „St“ und die geometrischen Abweichungen „a“. Der Modellfaktor „Rd“ resultiert aus den Anteilen des Widerstandsmodells „St“ und den geometrischen Abweichungen „a“. Weiterhin geht ein Übertragungsfaktor in die Berechnung ein, der aus einem Vergleich der unter Laborbedingungen bestimmten Prüfwerte mit den feststellbaren Ergebnissen im eingebauten Bauteil ermittelt wird, vgl. Bild 1.

Die Teilsicherheitsbeiwerte γ_E und γ_R sind abhängig von dem Zuverlässigkeitsindex β und den Wichtungsfaktoren der Beanspruchungsgrößen α_E bzw. der Widerstandsgrößen α_R. Sie werden ermittelt, indem der Wichtungsfaktor α_E aus der Quadratsumme aller positiven und α_R analog aus der Quadratsumme aller negativen Wichtungsfaktoren nach Gl. (14) bestimmt wird.

$$\alpha_E = -\sqrt{\sum \alpha_{E,i}^2} \quad ; \quad \alpha_R = \sqrt{\sum \alpha_{R,i}^2} \quad mit \quad \alpha_E^2 + \alpha_R^2 = 1 \qquad (14)$$

Eine Variation des Modellfaktors des Widerstandsmodells M_R wirkt sich auf den Teilsicherheitsbeiwert der veränderlichen Einwirkungen γ_Q sehr stark aus. In Bild 5 wird dies besonders an der Darstellung der berechneten Teilsicherheitsbeiwerte mit β und α_E = variabel deutlich. Für einen Variationskoeffizienten $v(M_R) = 0{,}01$ ist im Extremfall bei $v(Q) \approx 0{,}60$ ein Teilsicherheitsbeiwert der Nutzlast $\gamma_Q \approx 4{,}0$ vorhanden, für $v(M_R) = 0{,}15$ beträgt der Wert hingegen $\gamma_Q \approx 0{,}90$. Auch für den Fall, dass für den Zuverlässigkeitsindex eine konstante Größe $\beta = 3{,}8$ angesetzt wird, beträgt der Unterschied zwischen einem Teilsicherheitsbeiwert γ_Q für $v(M_R) = 0{,}01$ und 0,15 etwa 2,0 / 0,9 ≈ 200%. Daher ist eine fundierte Festlegung der statistischen Kenngrößen für die Modellunsicherheit wichtig!

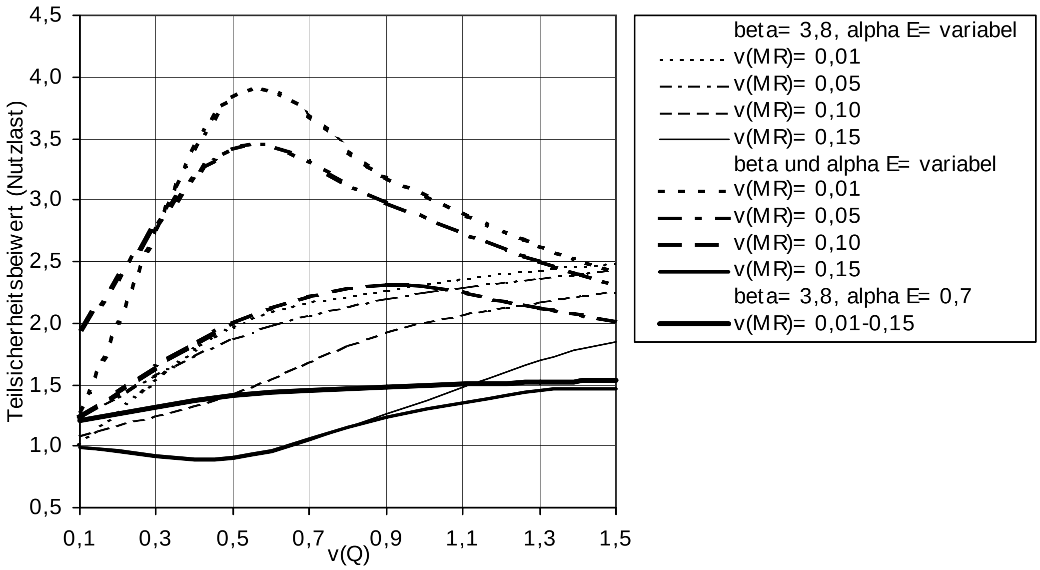

Bild 5 Teilsicherheitsbeiwerte der veränderlichen Einwirkung (Nutzlast) γ_N in Abhängigkeit des Variationskoeffizienten der Nutzlast v(Q) für das Widerstandsmodell Biegung und verschiedenen Streumaßen für den Modellfaktor M_R des Widerstandsmodells

Obwohl für die Widerstandsmodelle Querkraft und Normalkraft andere Differenzen bei unterschiedlichen Variationskoeffizienten $v(M_E)$ bzw. $v(M_R)$ vorliegen, lassen sich die Abhängigkeiten der Teilsicherheitsbeiwerte der Nutzlast von den Modellfaktoren für diese beiden Widerstandsmodelle generell ähnlich beschreiben.

4.4 Globaler Sicherheitsbeiwert

Eine Interpretation der Ergebnisse wird durch die Wechselbeziehungen zwischen Einwirkungs- und Widerstandsgrößen erschwert. Daher wird nachstehend ein „globaler Sicherheitsbeiwert“ nach Gl. (15) als Produkt der Teilsicherheitsbeiwerte der Einwirkungs- und Widerstandsgrößen betrachtet.

$$\gamma_{global} = \gamma_E \cdot \gamma_R = \gamma_Q \cdot \gamma_C \qquad (15)$$

Als maßgebender Wert der Einwirkungsgrößen geht der Teilsicherheitsbeiwert der Nutzlast γ_Q in diese Betrachtung ein. Auf der Seite der Widerstandsgrößen sollte der für das Versagen maßgebende Materialwiderstand berücksichtigt werden. Bei biegebeanspruchten Bauteilen mit üblichen Ausnutzungsgraden wäre dies in der Regel die Bewehrung, lediglich bei stark bewehrten Biegeträgern oder druckbeanspruchten Bauteilen tritt ein Betonversagen auf. Die überwachungsbedingten Qualitätsunterschiede der Betonbauteile sind jedoch maßgeblich durch die statistischen Kenngrößen des Betons gekennzeichnet. Daher wird nachstehend als Teilsicherheitsbeiwert für den Widerstand γ_C eingesetzt. Für das Widerstandsmodell Biegung wird damit ein zu großer globaler Sicherheitsbeiwert ermittelt, für das Widerstandsmodell Normalkraft sind die Werte annähernd realistisch.

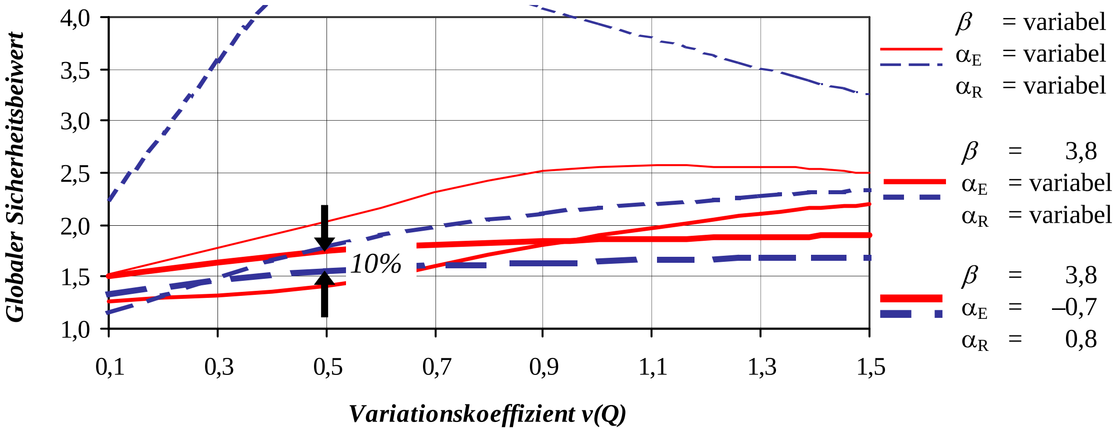

*Bild 6 Globaler Sicherheitsbeiwert in Abhängigkeit des Variationskoeffizienten der Nutzlast v(Q) für das Widerstandsmodell **Normalkraft** bei **Überwachungsintensität 1** (gestrichelt) und **Überwachungsintensität 3** (durchgezogen)*

Mit den normativen Ansätzen ($\beta = 3{,}8$, $\alpha_E = -0{,}7$ und $\alpha_R = 0{,}8$) befindet sich der globale Sicherheitsbeiwert nach Bild 6 für das Widerstandsmodell Normalkraft in dem betrachteten Bereich v(Q) = 0,1 bis 1,5 etwa zwischen $\gamma_{global} = 1{,}3$ bis 1,9. Unter den für Fertigteile angesetzten guten Herstellungsbedingungen (gestrichelt dargestellt), liegt der globale Sicherheitsbeiwert etwa 10% geringer als bei Fertigung auf der Baustelle mit Baustellenbeton (durchgezogen).

Falls die in der probabilistischen Berechnung bestimmten Größen für β und α_E bzw. α_R angesetzt werden, sind insbesondere beim Widerstandsmodell Normalkraft N_{Rd} deutliche Unterschiede zu erkennen. Der globale Sicherheitsbeiwert nimmt bei guten Herstellungsbedingungen (Bild 6, gestrichelt) einen maximalen Wert $\gamma_{global} \approx 4{,}5$ bei $v(Q) \approx 0{,}60$ an. Unter mäßigen Herstellungsbedingungen (Bild 6, durchgezogen) ist der maximale globale Sicherheitsbeiwert mit $\gamma_{global} \approx 2{,}6$ bei $v(Q) \approx 1{,}0$ deutlich geringer.

Anhand der durchgeführten probabilistischen Analysen lässt sich belegen, dass auch mit den normativen Festlegungen der Wichtungsfaktoren und des Zuverlässigkeitsindexes Differenzen infolge überwachungsbedingter Qualitätsunterschiede der Betonbauteile von etwa 10% bestehen. Sofern die zuverlässigkeitstheoretischen Kenngrößen β, α_E und α_R der Berechung entnommen werden, sind wesentlich größere Differenzen zu erkennen.

5. ZUSAMMENFASSUNG

Die qualitativen Unterschiede zwischen Betonbauteilen aus der Fertigteil- oder Baustellenfertigung werden in den probabilistischen Berechnungen durch unterschiedliche Variationskoeffizienten der Basisvariablen berücksichtigt. Eine probabilistische Untersuchung ermöglicht mit Hilfe von Wichtungsfaktoren in anschaulicher Weise die Bestimmung wesentlicher Einflussgrößen auf die Zuverlässigkeit eines Tragwerks. Aus diesen Analysen wird deutlich, dass insbesondere die Unsicherheiten der Widerstandsmodelle und der veränderlichen Einwirkungen einen großen Einfluss auf das Endergebnis nehmen. Die Wichtungsfaktoren werden gemeinsam mit einem Zuverlässigkeitsindex als Kenngröße der Versagenswahrscheinlichkeit in die Bestimmungsgleichungen der Sicherheitselemente einbezogen. Mit diesen Werten wird der erforderliche Sicherheitsbeiwert festgelegt. Eine Differenzierung nach der Überwachungsintensität führt zu dem Ergebnis, dass die Zuverlässigkeit eines Betonbauteils nur dann von der Überwachungsgüte beeinflusst wird, falls eine starke Abhängigkeit der Grenzzustandsfunktion von der Betondruckfestigkeit besteht. Für die Normalkrafttragfähigkeit wird z. B. der Unterschied mit etwa 10%, für das Widerstandsmodell der Druckstrebentragfähigkeit im Querkraftnachweis $V_{Rd,max}$ mit bis zu 14% bestimmt. Bei den übrigen Widerstandsmodellen sind diese Unterschiede mit max. 3% vernachlässigbar. Eine Umsetzung der Berechnungsergebnisse in Sicherheitselemente verdeutlicht, dass die Einwirkungsgrößen mit den normativen Ansätzen unterschätzt und die Widerstandsgrößen mit zu großen Werten definiert werden. Nur im Zusammenspiel der Einwirkungs- und Widerstandsgrößen ist ein relativ gleichbleibendes Zuverlässigkeitsniveau gewährleistet. Daraus resultiert, dass die Einwirkungsgrößen bei geringeren Unsicherheiten der Widerstandsmodelle einen größeren Einfluss erhalten.

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INTERDISCIPLINARY QUALITY-OF-LIFE PARAMETERS AS A UNIVERSAL RISK MEASURE

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1. INTRODUCTION

One major aim of human activity is based on building and cultivating social structures. The efficiency of such structures forms the basis for the exponential growing of human society, which has been going on for two centuries now.

Despite their vastness, eye to eye contact is still today the main criterion for successful social structures. The enormous economic strains which have been used to widen the range of communicative means within the last years are proof of this development. The beginning of personal contact, regardless which means of communication is used is marked by a formalized gesture or an `empty phrase`, which is known as greeting. The form of welcome, in almost all cultures on earth also includes the question concerning the well-being of the counterpart. The answer to the question requires an evaluation of the personal life situation. Basis for such qualitative evaluation processes are usually quantitative parameters, which enable us to compare actual values and target values. The question for evaluating the personal life situation can be answered by a quality-of-life parameter.

2. QUALITY-OF-LIFE PARAMETERS

The building and developing of social structures is closely connected with the term quality-of-life. Although the famous social critics of the 19th century did not know the term quality-of-life yet, they already regarded the improvement of the circumstances of living as the main motivating force of human development. One of the first definitions of quality-of-life is therefore taken from the area of social charity science:

"Measure of the congruence between the conditions of a certain objective life standard and the subjective evaluation of the thereby marked group of population."

It was the economist Cecil Pigou who coined the term "Quality-of-life" at the beginning of the 20th century and who brought the term into the academic discussion as a target figure for social actions and as measure of individual well-being. The development and usage of the term quality-of-life has since then not only touched economy and social science, but it has also been introduced to many other areas, e.g. medicine. In 1947, the World Health Organization termed `Health` as a condition of absolute physical, mental, and social well-being. Over fifty years ago, this definition widened aims and criteria for actions of physicians. Not only somatic aspects of health and illness, but also psychological and social aspects, the patient's well-being and his capacity to act are part of the physician's duty.

In the field of medicine new target figures were introduced via quality-of-life parameters, which among physicians are also known as quality-of-life measure instruments; the aim of this was to make quality-of-life measurable and therefore testable. Quality-of-life parameters are used today in preventive medical check-ups, in therapy research, quality security and health economy. Over 20,000 scientific publications deal with the topic of medical life quality in the German speaking countries [4], whereas probably 2,000 new publications are added per year [11]. These scientific strains led to the development of over 800 life quality parameters in the field of medicine [1], [11], [21]. Exemplary, some medical quality-of-life parameters are mentioned in table 1. The trodden path of metrical representation of the life quality required at first a definition of the term "quality-of-life". In the following, a definition from the field of medicine is quoted:

"By health-related quality-of-life is meant a psychological construct, which describes the physical, psychological, mental, social and functional aspects of the well-being and the function capacity of the patients from their view." [4]

As the short description illustrates, the term quality-of-life goes far beyond pure medical questions, so as to other areas other definitions were formed:

"Quality-of-life is the individual perception of the personal living situation in the context of the respective culture and the respective value system in relation to personal aims, expectations, judgment scales and interests." [25]

"Quality-of-life is the result of an individual, multi-dimensional evaluation process of interaction between an individual and its environment. As evaluation criteria can be used social norms as well as individual value judgments and affective factors." [11]

Numerous other definitions are to be found in Proske [20].

According to the respective definition, quality-of-life depends on a number of introductory figures, which are partly hard to seize numerically. In table 2 possible introductory figures are collected tabular. Based on the high multi-dimensionality, the insecurity in choosing and evaluating introductory figures and the very specific question in many fields, special quality-of-life parameters were developed in a wide range, which limit the number of introductory figures and which seize the quality-of-life in a certain situation, comparable to the development in medicine. The quality-of-life parameters differ tremendously in the assembly. Even the assembly of various quality-of-life parameters for identical questions reveals difficulties in the fixing of decisive introductory figures in the functional connection. Table 3 gives proof of this statement by the example of the number of introductory figures for various quality-of-life parameters for psychiatric patients.

Tab. 1. health-related quality-of-life parameters

Illness-comprehensive Quality-of-life parameters	illness-specific quality-of-life parameters
• Nottingham Health Profile • Sickness Impact Profile • SF-36 (SF-12) • WHOQoL • EuroQol • McMaster Health Index Questionnaire • MIMIC-Index • Visick-Skala • Karnofsky-Index • Activities-of-Daily-Living Index • Health-Status-Index • Index-of-Well-being • Rosser-Matrix • Rosser & Kind Index • Quality of Well Being Scale	• Quality of Life Index – Cardia Version III (QLI) • Seattle Angina Questionnaire (SAQ) • Angina Pectoris Quality of Life Questionnaire (APQLQ) • Minnesota Living with Heart Failure Questionnaire • Asthma Quality of Life Questionnaire (AQLQ) • Fragebogen zur Lebensqualität bei Asthma (FLA) • Fragebogen für Asthmapatienten (FAP) • Asthma Questionnaire (AQ20/AQ30) • Osteoporosis Quality of Life Questionnaire (OQLQ) • Quality of Life Questionnaire for Osteoporosis (OPTQol) • Osteoporosis Assessment Questionnaire (OPAQ) • QOL Questionnaire of the European Foundation for Osteoporosis (QualEFFO) • Juvenile Arthritis QOL-Questionnaire (JAQQ) • Schmerzempfindlichkeitsskala (SES) • Pain Disability Index (PDI)

Tab. 2. possible introduction variables for quality-of-life parameters [14]

predominant objective variable	predominant subjective variable	societal variables
- living conditions - family - social conditions - participation in social life - life standard - income - health - education and work	- life contentment - happiness - carefree ness - subjective class membership - optimism pessimism about future developments - judgment of the personal living conditions	- social conflicts - trust in other people - security, freedom, justice - social integrity

In some of the publications, the possibility of a metrical description of quality-of-life is categorically excluded [19]. There, a rather coarse fixing of quality-of-life is considered to be possible, as is schematically represented in figure 1.

The topic quality-of-life and the problems connected to this topic are fortunately not only subject of discussion in academic journals, such as the "Journal of Social Indicators" or at academic conferences, such as the "International Society for Quality-of-life Studies", a conference held in Frankfurt/ Main in 2003. An advertisement from the `Aktion Mensch` in figure 2 is to serve as an example for the question of defining of quality-of-life. The media reported about the quality-of-life atlas for Germany [18] or the Human Development Index of the UNO [23].

Tab. 3. Quality-of-life measure instruments for psychiatric patients

Quality-of-life measure instruments	number of parameters
Social Interview Schedule (SIS)	48
Community Adjustment Form (CAF)	140
Satisfaction of Life Domain Scale (SLDS)	15
Oregon Quality of Life Questionnaire (OQoLQ)	246
Quality of Life Interview (QoLI)	143
Client Quality of Life Interview (CQLI)	65
California Well-Being Project Client Interview (CWBPCI)	304
Quality of Life Questionnaire (QoLQ)	63
Lancashire Quality of Life Profile (LQoLP)	100
Quality of Life Index for Mental Health (QLI-MH)	113
Berliner Lebensqualitätsprofil (BeLP)	66
Quality of Life in Depression Scale (QLDS)	35
Smith-Kline Beecham Quality of Life Scale (SBQoL)	28
Quality of Life Enjoyment and Satisfaction Questionnaire (Q-LES-Q)	93

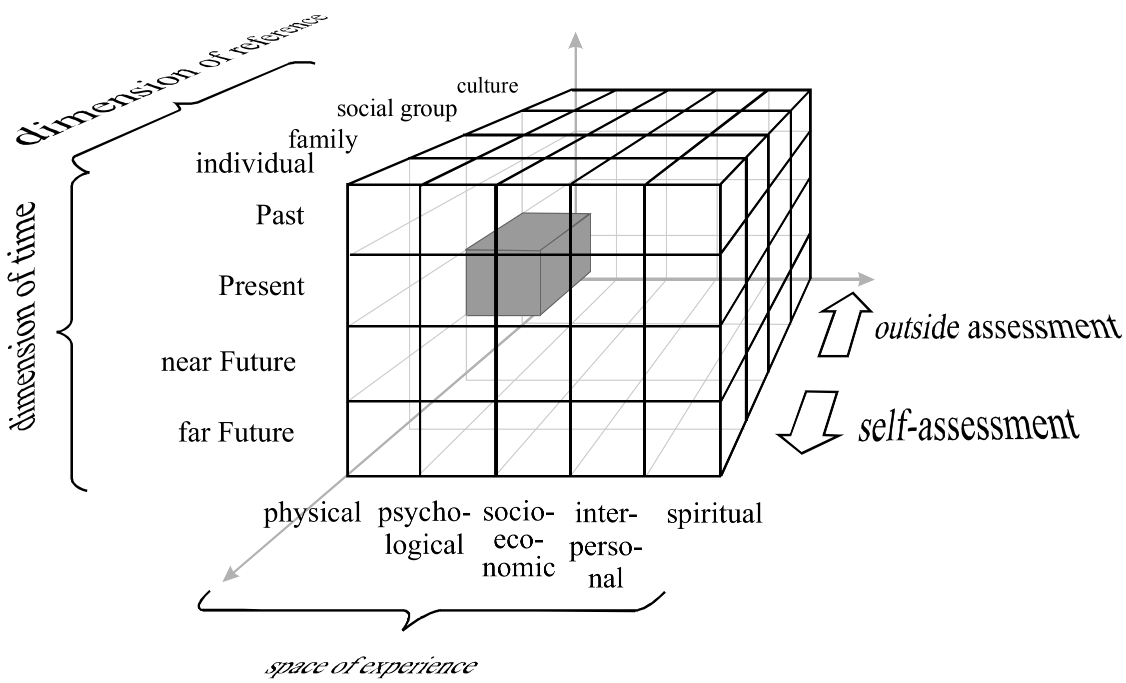

Fig. 1. dimension of quality-of-life according to [19].

Just as in those parameters, it is assumed that a metrical representation of quality-of-life is possible in general. In the following, it will be illustrated that quality-of-life parameters are always risk parameters. Subsequently, various hierarchical risk parameters are commented upon.

Fig. 2. Advertisement of the `Aktion Mensch` (translation: “What means or contains the term quality-of-life?”).

3. RISK PARAMETERS

3.1 FREQUENCY OF DEATH

The risk parameter of frequency of death, probability of death respectively corresponds with the classical definition of risk, as it can be found for instance in norms (DIN VDE 31 000). Here, risk is defined as a product of the frequency of occurrence of an incident with damage and the extent of the damage. Various other varieties of the term risk will be mentioned for reasons of completeness.

Frequency of death and probability of death can be regarded as special cases of risk. In this case, the damage is the loss of human lives. Based on necrologies, which were introduced to Australia in the 18th century and to England and Wales in 1837, it is possible to determine the frequency of death for human beings [13].

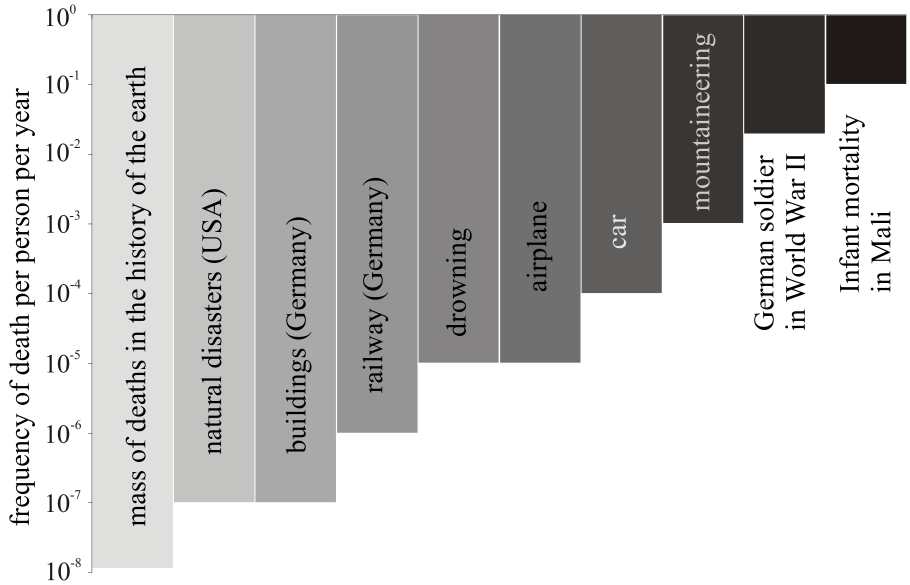

Fig. 3. Examples of frequency of death for people for various actions.

Figure 3 lists some of the frequencies of death for people in various actions. In Proske [20], a collection of over 125 values for frequency of death in various situations or actions can be found. When comparing such frequencies of death, one has to consider the basic totality and the date of gathering. In figure 3 for example, the first two causes are related to partly basic totality of the population. The infant mortality is related only to the group of infants, and the frequency of death of the soldiers is only related to the basic totality of soldiers. Besides these very high values, the relative frequency of death can be represented in very small values, such as 10^{-8} per year for death caused by mass deaths in the history of the earth. As target value for workers, in many countries 10^{-4} is used; as target value for members of the uninvolved public 10^{-6} is used.

The parameter of the frequency of death is usually considered for one year. It doesn`t give evidence of how frequent or for how long the person was exposed to a certain action in one year. In order to improve the quality of the risk comparison, a calibration of time is necessary.

The representation of a mortality risk in relation to a defined reference time of 10^8 or 10^3 hours is called Fatal Accident Rate. Examples for FAR`s can be found among others in Proske [20]. Target values for the area of the oil industry are at 15, for member of the uninvolved public at about 0.1.

The frequency of death doesn`t necessarily have to be related to a certain span of time. The frequency of death can also be related to distances, number of actions or amounts of substances. The unit-risk-value for air pollution quantities is an example for such a parameter. A unit-risk-value for a certain substance specifies the additionally assumed risk of cancer for humans when inhaling polluted air for 70 years with an amount of air pollution of 1 µg per m^3. The real risk can be calculated by multiplication with the actual time of exposure of the human to the pollution.

3.2 FAMILY OF THE *F-N*-DIAGRAMS

The frequency of death and the fatal accident rates do not consider the extent of a certain incident of damage. The figures will be the same for an accident with one casualty which occurs one thousand times, and an accident with one thousand casualties which occurs only once. Experience has shown that people differentiate between those two cases very much in regards to the subjective judgment of security. If the risk parameter is to be used successfully, it has to reflect the security perception of the population to a sufficient extend.

For the improvement of the subjective risk evaluation, which is also called risk aversion, so-called *F-N*-Diagrams were developed. The first of these diagrams were developed by Farmer in 1967 [9]. The risk research received huge impulses by the building of nuclear power stations. *F-N*-Diagrams became very famous in the so-called Rasmussen-report in the beginning of the 70`s of the last century.

F-N-Diagrams are double-logarithmic diagrams, which show on the *x*-axis the number of casualties and on the *y*-axis the frequency of accidents with equal or larger numbers of casualties. By this definition, we receive graphs which go down from left to right. We have to consider that there are also so-called *f-N*-graphs, which on the *y*-axis show the frequency of accidents with *N* casualties. In this case, a rising graph can be the result.

Since the introduction of the first diagrams, a huge variety of diagrams has been developed, so that it can be called the family of the *F-N*-Diagrams. The basic principle of the diagrams is the same, but the single units on the *x*-axis can differ. Therefore, there are diagrams, which on the *x*-axis show the costs of damage, a damage parameter which consists of various other parameters, the number of persons concerned (PAR), the time for removing the damage, the energy which has been used for removing the damage or the radioactive radiation, as has originally been intended. A summary of various representations of *F-N*-Diagrams can be found in [17].

For the development of such diagrams, data of the accidents from the past with specification of the damage and the number of persons concerned are necessary or calculations which provide such data. The data, number of casualties and the frequency of accidents have to be sized according to rising numbers of casualties. After that, the data will be cumulated, which means the frequency of accidents with a number of casualties which equals *N* or is higher than *N* will be determined and sized. These data pairs will be represented graphically in the *F-N*-Diagram. Examples for the development of classical *F-N*-Diagrams can be found among others in [3], [20].

The proof of sufficient security will be done graphically in a diagram. Comparison lines, which were developed for a variety of incidents, will be used as a proof. In [3] and [20], about 20 target graphs are collected. The target graphs divide the area of the diagram in two sub-areas: in an acceptable and an unacceptable area (figure 4). Some of the target graphs have an additional area, which is under certain circumstances acceptable. This area is known as the ALARP-area (as low as reasonably possible). If the determined graph is inserted into the diagram, one will see whether the graph is in the acceptable area. The average number of casualties or the average costs of damage expected in case of an accident are called the Potential Loss of Life.

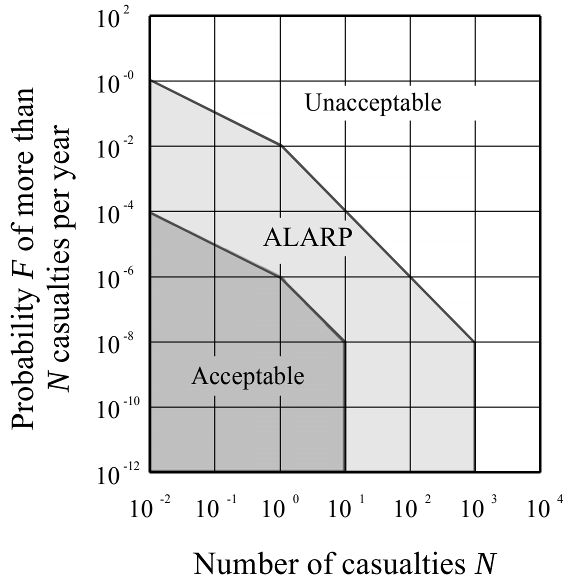

Fig. 4. Example of a proving graph (Groningen-graph 1978).

3.3 LOST LIFE YEARS

The family of the *F*-*N*-Diagrams is excellently suitable for the representation of technical and natural risks, since in the cases of those risks high numbers of casualties are possible. In cases of health risks, there is always only one person concerned. Additionally, the age of the person concerned is considered when risks are judged subjectively. The death of a person of 90 years of age, caused by an illness, will be judged subjectively different, than the death of a young person. The risk parameter of the lost life years or lost life days can consider this effect. The parameter is defined as the difference between average life expectation without the analyzed risk and the average life expectation with the analyzed risk. The parameter is widely used in representations of cancer diseases in Germany, but it is also used for other illnesses. Cohen collected lost life days for various diseases and various social circumstances (figure 5) [6]. Further data can be found in [15], [20].

In addition to the loss of life years, it is possible to collect health reductions during life and to calculate the loss of life time. Here, for instance so-called Quality Adjusted Life Years (*QALY*), Disability Adjusted Life Years (*DALY*) or Health Years Equivalent (*HYE*) are mentioned. Figure 6 is used to clarify those terms. One example for the calculation of figures, which represent risks in environmental pollution in the Netherlands, can be found in [13]. In developed countries, the share of the *DALY* amounts to about 10 % of the life time, in India about 30 %, and in some of the African countries it is almost 50 % [10].

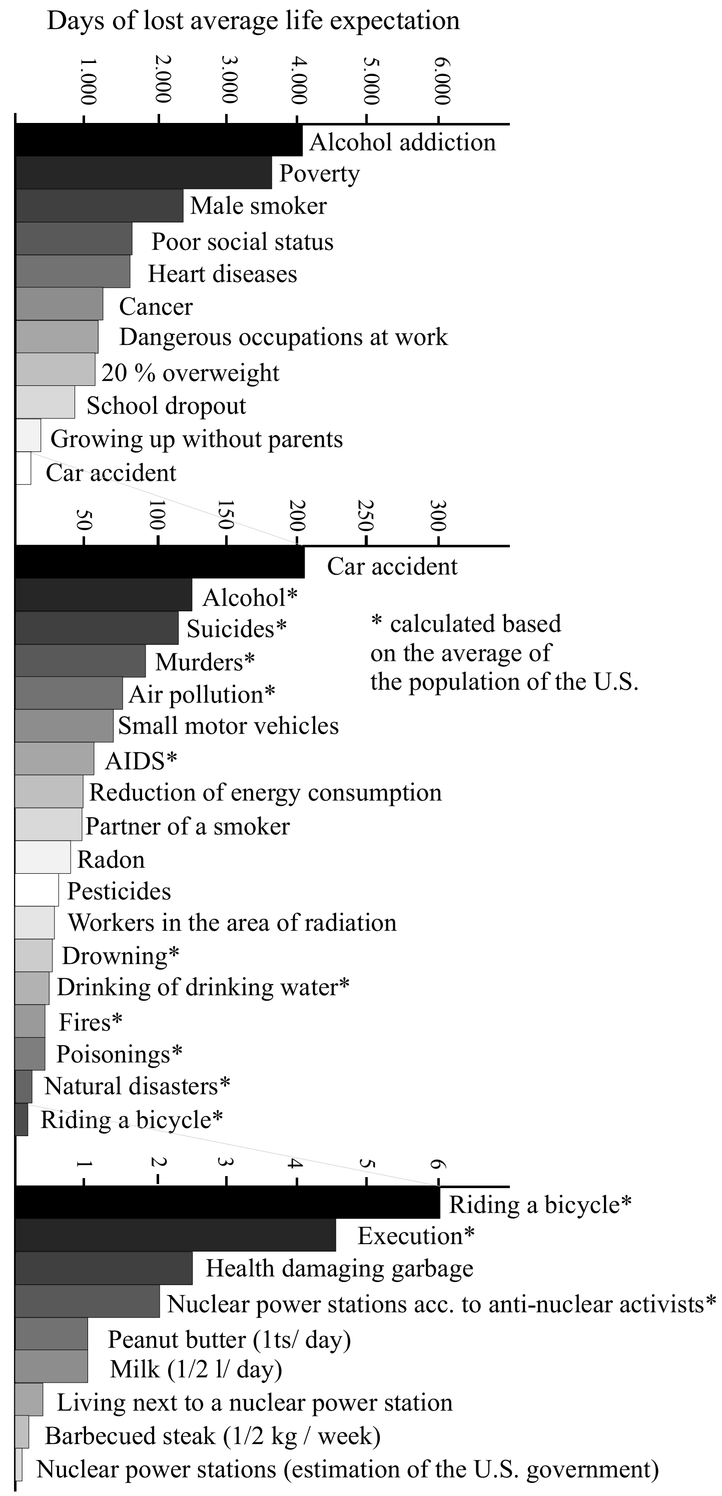

Fig. 5. Lost Life Days in various situations according to Cohen [6].

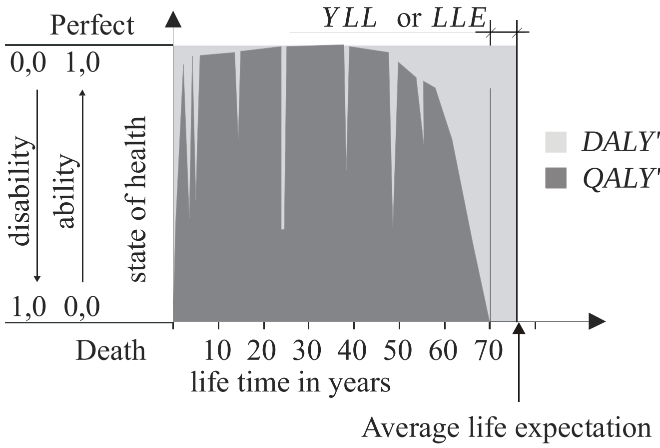

Fig. 6. Representation of the concept of lost life years.

3.4 QUALITY-OF-LIFE PARAMETERS

The figures for the *DALY*`s clearly show that people who live in regions under poor geopolitical circumstances, are exposed to high risks. This fact also accounts for groups of people who live in industrial countries, in which the conditions of living are equal to those in developing countries. Poverty and poor social status, in industrialized countries also produce the highest losses of life expectation, as figure 5 shows. The connection between poverty and average life expectation on the level of countries is represented more clearly in figure 7. Based on the assumption that poverty is an essential introductory figure for life quality, one can draw the conclusion that life quality and life risk are closely connected with each other. Quality-of-life parameters are universal risk parameters, since, according to their structure they can include minor disturbances of life quality as damages.

Life quality parameters as risk parameters can be used to judge the efficiency of risk lowering measures. Risk lowering measures are all safety and security measures, which are installed by human society, such as social security systems, police, jurisdiction, hospitals, technical safety systems or safety systems which are used in order to prevent natural disasters etc.

It is common knowledge that legal standards for safety systems are imbalanced [16], [22], [24]. While in some cases, the realization of governmental safety standards is not very successful when at the same time large amounts of financial resources are connected to it; in other cases laws and norms can be realized with very little pecuniary means and show great success in securing humans. Quality-of-life parameters enable us to judge those relations, from the view point of the society as a whole as well as for special cases.

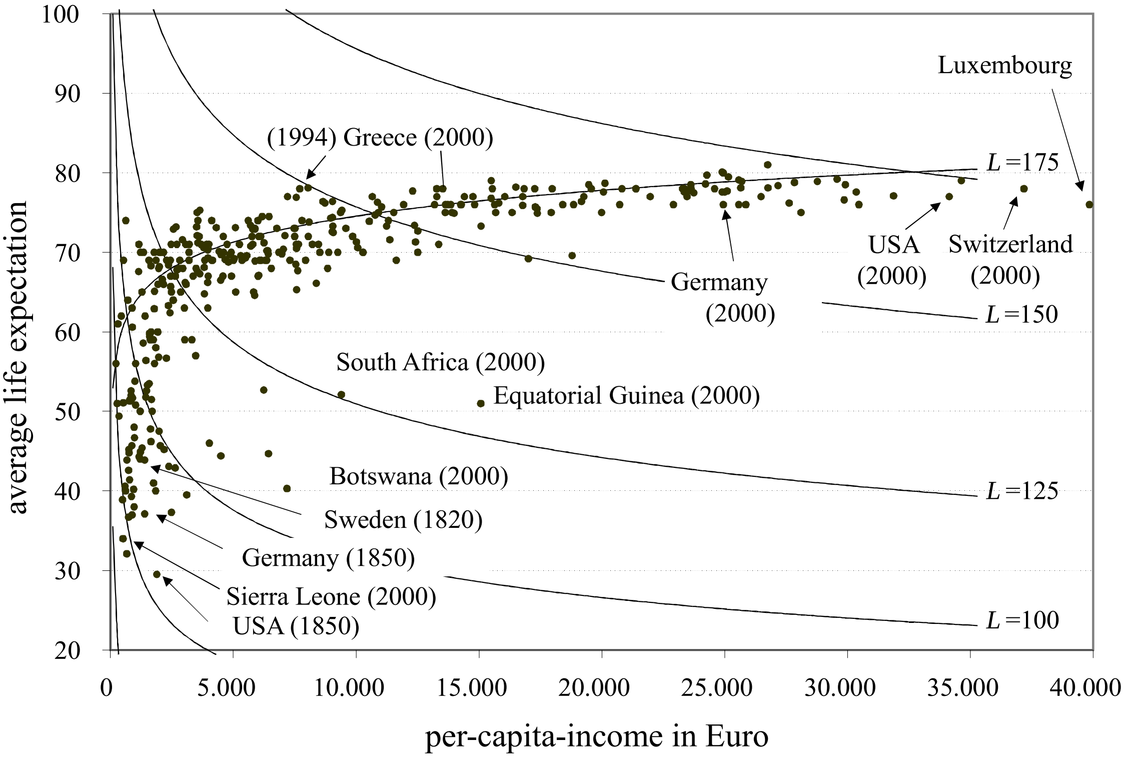

Fig. 7. per-capita-income, average life expectation and life quality index for 170 countries [20].

Evaluations of adjuvant therapies in oncology are an example for the evaluation of efficiency. Adjuvant theories include measures for preventing the growth of tumors after complete removal of the primary melanoma. Therefore, for patients with high risk melanomas (size of the tumor ≥ 1.5 mm) after the surgical removal of the primary malignant melanoma the question has to be raised, whether or not the treatment is useful. It can not be predetermined, which of the patients will develop new tumors. Also, the success of the therapy can be determined only with some probability.

As analyses of the patient`s quality-of-life (SF-36) showed, the quality-of-life was lowered considerably over a period of several months up to two years for patients who were treated with the adjuvant therapy as a result of the side effects (figure 8). If one considers that for the therapy success can only be determined with some probability and if we also consider that there is a large amount of side effects, the usage of quality-of-life parameters can provide an answer to the question whether or not a therapy should be done.

In Proske [20], one example from structural engineering is presented. There, it is analyzed with the help of a quality-of-life parameter, which strengthening for bridges is most efficient.

In the sense of actual quality-of-life, the adjuvant theory or the strengthening of bridges should not be analyzed singularly, but in connection to each other. The whole range of medical, technical, and social preventive measures should be comparable. This would in part mean that measures would have to be used parallel to each other; in part it would also mean that competitive measures would have to be used. Quality-of-life parameters must be developed, which function independent of the field of research and which enable us to convert specific quality-of-life parameters. The authors consider the so-called dynamic profile parameters which are reunited with an index parameter a possible solution.

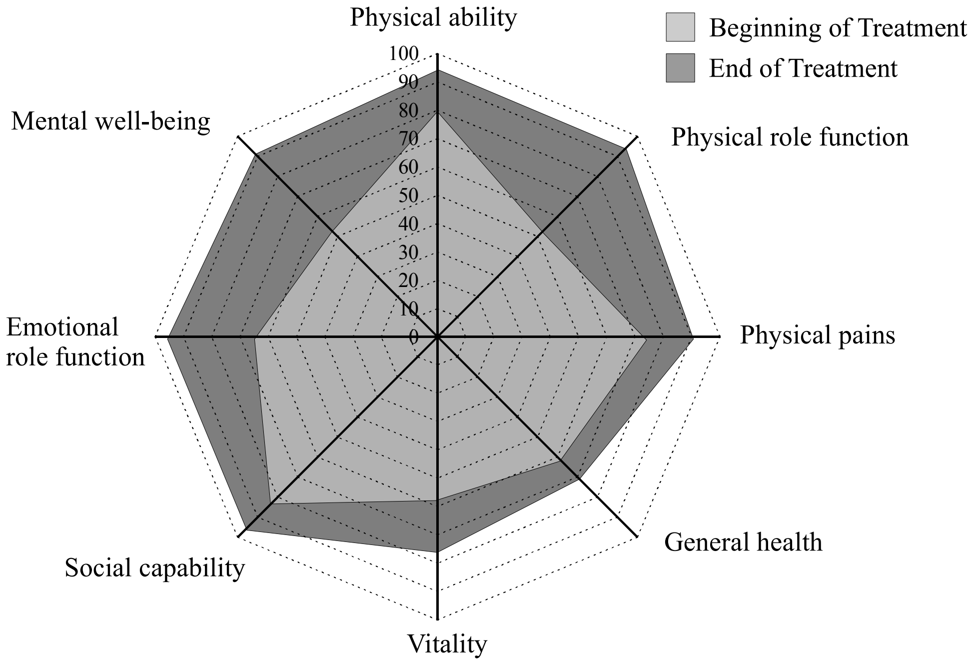

Fig. 8. Representation of the SF-36 parameters for the participants of the adjuvant therapy.

In general, one has to differentiate between profile- and index- quality-of-life parameters. Profile parameters include various introductory figures or groups of introductory figures which can not be summed up, whereas index parameters include all introductory figures into one indicator. Examples for those profile parameters from the field of medicine are the SF-36, the Sickness Impact Profile (SIP) and the Nottingham Health Profile (NHP). Examples for index parameters, also from the field of medicine, are the Karnofsky Index, the EuroQol and the Quality-of-well-being scale. [12]

Dynamic profile parameters are based on groups of introductory figures, which can be exchanged according to the issue in question. This modular construction enables the user to develop the system by and by, so that new issues can be treated with those parameters. In order to make those various profile parameters comparable, one has to reunite the dynamic profile parameters into one general index parameter. Higher-ranged index quality-of-life parameters can then be developed out of several of those subject-specific index parameters (figure 9). Depending on the issue in question, a more or less subject-specific quality-of-life parameter can be selected from a pedigree of quality-of-life parameters.

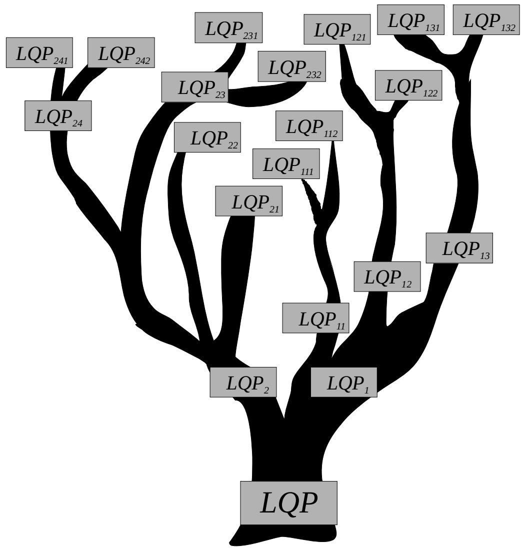

Fig. 9. graduation of quality-of-life parameters (QLP).

This development is in part visible in the field of social science, where efforts are made to develop summarized and standardized quality-of-life parameters. Diener`s Value-Based Index of National Quality-of-Life [7], Estes`Index of Social Progress [8], the American Demographics Index of Well-Being [8] or the Fordham Index of Social Health should be mentioned here as examples. The Human Development Index of the UNO or the International Well-Being-Index belong to this group as well. In other areas this development is also brought forward. The quality-of-life index by Nathwani, Pandey and Lind, which takes into account the conversion of life spans while also considering reductions of life quality, is able to integrate quality-of-life parameters from medicine; thus a standardized quality-of-life parameter can be received.

The aim of research should be to bundle up the developments regarding quality-of-life in the individual research fields and to develop comprehensive concepts. Up to now, this has not been done. The development of metrical and interdisciplinary quality-of-life parameters as universal measure for risk is according to the authors of major importance for the society, in order to enable it to evaluate objectively the capability of preventive measures within itself.

Apart from the development of metrical quality-of-life parameters, a graphic representation of the individual components is helpful, in order to check the quality-of-life parameters. As already said, the relatively high number of introductory figures makes visualization in a diagram quite difficult. Parallel to the developing of computer-based techniques, within the last years a number of ways of visibly representing high dimensional data were developed. The so-called Chernoff-faces [2], [5] can be regarded as one form of representation. In figure 10, the data of the SF-36 from figure 8 is represented again.

The usage of human facial characteristics in the visualization of high-dimensional amounts of information is not only grounded on the high differentiable quality of the visualizing of faces and facial expressions, but it also relies on the representation of facial expressions based on a large number of degrees of anatomical freedom and high fine motor capabilities. The visualization of introductory and result figures of quality of life parameters by ways of the human face is after all, what human beings do anyway: to transfer information about their state of health through their facial expressions. The facial expression of human beings enables us extraordinarily to give an answer to the question of the well-being of humans dealt with in the beginning and consequently also to the question of quality-of-life.

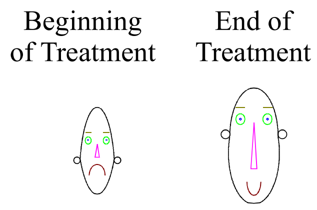

Fig. 10. Representation of the SF-36 parameter for the participants of the adjuvant therapy according to the Chernoff-faces.

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ZUSTANDSANALYSE UND INSTANDHALTUNG VON SCHUTZBAUWERKEN DER WILDBACHVERBAUUNG

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ZUSAMMENFASSUNG

Die Instandhaltung von Schutzbauwerken stellt eine der wichtigsten Aufgaben der Wildbachverbauung dar. Die Risikofaktoren, die zur Beschädigung/ zum Versagen von Bauwerken führen können, ergeben sich nicht nur aus Schäden und Mängeln am Bauwerk selbst, sondern auch aus Einflüssen in der Umgebung des Bauwerkes (einwirkende Prozesse, Untergrund, hydraulische Beanspruchung, Witterung). Die Erfassung und Analyse an Schutzbauwerken der Wildbachverbauung beschränkt sich daher nicht nur auf das Einzelbauwerk, sondern schließt auch die Wirkungsanalyse des gesamten Schutzsystems mit ein. Die Planung von Instandhaltungsmaßnahmen erfordert einen Überblick über den Zustand aller Verbauungen in einem Einzugsgebiet (in einer Region), eine Information, die in vielen Fällen nicht verfügbar ist. Ein zukünftiger Forschungsschwerpunkt soll daher in der Entwicklung einer Methodik für die Durchführung eines effizienten „Zustandsmonitorings“ für Schutzmaßnahmen in Wildbacheinzugsgebieten liegen.

ABSTRACT

The maintenance of torrent control works is a major task of natural hazard management. The determinant risks, which can lead to the damage or destruction of torrent control works, are not only related to defects or flaws at the construction itself, but also to influences from the environment (processes, bed-rock, hydraulic stress, weather). The recording and analysis of the condition of torrent control works is not reduced to the individual construction but also includes the analysis of the protection effect of the total torrent control system. The planning of maintenance measures requires the survey of the condition of all protection works in a watershed area (region). This information in many cases is not available, so that a future scientific task will be the development of methods for the operation of efficient monitoring system concerning the condition and needs for restoration of torrent control works.

KEY-WORDS: Wildbachverbauung, Instandhaltung, Bestandesrisiko, Zustandsmonitoring, Lebensdauer, Funktionalität, Standsicherheit.

1. EINLEITUNG

Seit 1884 werden in Österreich systematisch Schutzbauten gegen Wildbachgefahren errichtet. Die jährlichen Investitionen in Maßnahmen zum Schutz vor alpinen Naturgefahren haben heute ein Ausmaß von ca. 120 Mio. € erreicht, eine wesentlicher Rückgang des zu erfüllenden Schutzbedarfs ist zurzeit allerdings noch nicht absehbar. Durch die laufende Maßnahmensetzung hat der Bestand an Schutzbauten in Wildbacheinzugsgebieten in Österreich bereits einen erheblichen Umfang erreicht.[1]
Die Verpflichtung zur **Instandhaltung von Maßnahmen der Wildbachverbauung** liegt im Sinne des Wasserrechtsgesetzes (WRG 1959) und der einschlägigen Förderungsbestimmungen (Wasserbautenförderungsgesetz 1985) bei den Interessenten[2]. Die Finanzierung von notwendigen Instandhaltungs- und Sanierungsmaßnahmen erfolgt in Österreich über einen beim Forsttechnischen Dienst für Wildbach- und Lawinenverbauung (WLV) eingerichteten Betreuungsdienst.[3] Die Beurteilung des Wirkungsgrades und des Bauwerkszustandes der Wildbachschutzanlagen sowie die Einleitung erforderlicher Instandsetzungsmaßnahmen fallen im in den Zuständigkeitsbereich dieser Dienststelle. [4]

Aufgrund der ständig steigenden Zahl an Schutzbauten ist in Österreich – wie auch in anderen Alpenländern – mittelfristig mit einer starken Zunahme des Aufwandes für Instandhaltungsmaßnahmen zu rechnen.[5] Es besteht daher ein dringender Bedarf an effizienten Methoden zur dauerhaften Erfassung und Beobachtung des Zustandes der Schutzbauwerke („**Zustandsmonitoring**"), um Instandhaltungsprogramme zeitgerecht und effektiv planen zu können.

2. BEGRIFFSBESTIMMUNG, ZIEL UND GRUNDSÄTZE DER INSTANDHALTUNG

Im Schutzwasserbau wird grundsätzlich zwischen den Begriffen "**Instandhaltung**"[6] und "**Instandsetzung**" unterschieden.
Während die Instandhaltung alle jenen Arbeiten umfasst, die zum Erhalt der Funktionsfähigkeit einer Schutzanlage beitragen, beinhaltet die Instandsetzung alle jene Maßnahmen, die diese Funktionsfähigkeit im Falle der Beeinträchtigung oder Einschränkung, beispielsweise durch Schäden oder Baumängel, wieder herstellen. (BRETSCHNEIDER ET AL., 1982). Die Instandhaltung umfasst nicht nur Bauleistunge sondern auch die Aufgabe der Zustandskontrolle und des „Betriebes" der Schutzanlagen[7] sowie Wartungsarbeiten. Die Instandhaltung der Schutzbauwerke steht in unmittelbarem Zusammenhang mit der

[1] Exakte Angaben über die Anzahl der Schutzbauten in Österreich sind nicht verfügbar.
[2] In der Regel Gemeinden oder Wassergenossenschaften.
[3] Die Finanzierung von Instandhaltungsmaßnahmen erfolgt im Sinne des Wasserbautenförderungsgesetzes 1985 zu je einem Drittel durch den Bund, das Land und die Interessenten.
[4] Rechtsgrundlage: Forstgesetz 1975.
[5] Bereits jetzt hat der Aufwand für die Instandhaltung von Schutzbauten einen erheblichen Anteil an den Gesamtbaukosten für die Maßnahmen der Wildbachverbauung erreicht; die Schätzungen schwanken zwischen 5 (10) % (LÄNGER, 1999) und 40 % (ROMANG ET AL., 2004).
[6] In der Schweiz ist der Begriff „Unterhalt" in Verwendung (BÖLL, 1999).
[7] Schutzanlagen mit erhöhtem Betriebsrisiko (z.B. Hochwasserrückhaltebecken) bedürfen einer laufenden Kontrolle und Wartung. Dafür sind entsprechende organisatorische Vorkehrungen zu treffen, die Zuständigkeiten festzulegen und dies in einer Betriebsordnung (Beckenbuch) zu regeln.

Gewässerinstandhaltung, da bei einer Veränderung des Gefahrenpotenzials im Einzugsgebiet auch das Bestands-/Wirkungsrisiko für die Schutzanlagen signifikant steigen kann.[8]

Ziel der Instandhaltungs-/Instandsetzungsmaßnahmen ist, die **Sicherheit** und **Funktionalität** der Schutzanlagen in Wildbacheinzugsgebieten aufrecht zu erhalten. In der Wildbachverbauung steht nicht die Bestandssicherheit des Einzelbauwerks im Vordergrund, wie dies in der Regel im Hochbau der Fall ist, das maßgebliche Sicherheitsinteresse liegt vielmehr in der Nachhaltigkeit der Wirkung des gesamten Schutzsystems. Das bedeutet, dass das aktuelle Ausmaß sowie die Entwicklung des Risikos in dem durch Verbauungsmaßnahmen (Schutzmaßnahmen) gesicherten Zonen den primären Referenzwert darstellen. Daraus leitet sich für das Einzelbauwerk in einem System von Verbauungsmaßnahmen in Abhängigkeit der angestrebten Schutzwirkung ein unterschiedlicher Sicherheitsanspruch ab („Schlüsselbauwerke“, Bauwerke mit untergeordneter Schutzwirkung)[9].

Grundsätzlich umfasst ein Zustandsmonitoring für Schutzbauten der Wildbachverbauung die Erhebung, die Dokumentation sowie die Bewertung (Analyse) des Bauwerkszustandes und der Wirkung (des Schutzerfüllungsgrades). Insbesondere müssen dabei die Tragsicherheit (Standsicherheit), die Gebrauchstauglichkeit und die Dauerhaftigkeit beurteilt werden. Aus den erhobenen Daten kann auf den Schutzerfüllungsgrad des gesamten Verbauungssystems geschlossen werden. Ein institutionalisiertes **Monitoringsystem** ermöglicht es, Schäden und Wirkungsmängel rechtzeitig zu erkennen, Instandsetzungsprioritäten zu entwickeln und dadurch wirtschaftliche Maßnahmenprogramme (Instandsetzung) zu planen. Mittelfristig kann ein Zustandsmonitoring auch geeignet sein, die Wirkung bestimmter Maßnahmentypen (z.B. Funktionstypen von Wildbachsperren) systematisch zu erfassen und dadurch die Planung und Ausführung zukünftiger Maßnahmen zu optimieren. Ein derartiges System wurde in Österreich bisher nur im regionalen Bereich umgesetzt (z.B. Gebietsbauleitung Osttirol der WLV).
Außerdem gibt es für den Bereich der Wildbachverbauung keine weiterführenden Richtlinien oder Normen, wie diese beispielsweise für den Brückenbau vorliegen (RVS 13.71 „Überwachung, Kontrolle und Prüfung von Kunstbauten, Straßenbrücken“), welche die Art und den Umfang von Zustandsprüfungen für Schutzbauwerke regeln.

Die Wirksamkeit von Maßnahmen der Wildbachverbauung bemisst sich in erster Linie an der erreichten oder erreichbaren Risikoverminderung. Damit hat sich die **Wirtschaftlichkeit von Instandhaltungsmaßnahmen** in Wildbacheinzugsgebieten primär an jenen Effekten zu orientieren, die Sanierungsmaßnahmen für die Sicherheit und Funktionalität des gesamten Schutzsystems haben. Eine auf Wirtschaftlichkeit basierende Prioritätenplanung für die Instandsetzung von Schutzbauten ist daher als komplexe, auf die Schutzwirkung des gesamten Verbauungssystems ausgerichtete Aufgabe.
Aus ökologischer Sicht ist vor allem die Frage des Bauverfahrens („harte“ oder naturnahe Baumaßnahmen) von Bedeutung, die in engem Zusammenhang mit der Lebensdauer, der zeitlichen Entwicklung des Wirkungsgrades und dem Versagensrisiko der Verbauung steht. Die Kosten für Instandhaltungsmaßnahmen an unterschiedlichen Bautypen spielen daher

[8] Beispielsweise kann eine Akkumulation von Holz im Hochwasserabflussgebiet zu einer wesentlichen Erhöhung des Wildholzrisikos führen, das in der Folge die Schutzwirkung der Verbauungsanlagen beeinträchtigen kann.
[9] In Verbauungssystemen ist zwischen „Schlüsselbauwerken“ mit einer zentralen Schutzfunktion und Bauwerken mit einer lokal begrenzten Schutzwirkung zu unterscheiden; dem entsprechend sind bei der Zustandskontrolle auch unterschiedliche Sicherheitsanforderungen zu stellen.

(neben den Errichtungskosten) eine entscheidende Rolle bei Wirtschaftlichkeitsüberlegungen (Variantenvergleich) im naturnahen Schutzwasserbau.

3. KONZEPTE ZUR BEURTEILUNG DEr SICHERHEIT UND FUNKTIONALITÄT VON SCHUTZBAUTEN

Gefahrenbilder und Gefahrenszenarien stehen im Mittelpunkt der Betrachtung über die Tragsicherheit und Gebrauchstauglichkeit von Schutzanlagen (ROMANG ET AL., 2004). Diese ergeben sich aus der Art und dem Zustand des Schutzbauwerkes einerseits und den natürlichen Rahmenbedingungen andererseits. Sie können sowohl auf außergewöhnliche Ereignisse (z.B. Hochwasser) als auch auf kontinuierliche Veränderungen (z.B. Veränderung der Lage der Gerinnesohle durch Auflandung oder Eintiefung, Hangbewegungen) zurückzuführen sein. Bekannte Gefährdungsbilder (siehe Abschnitt 5.) müssen mit anderen, die sich aus der fortschreitenden Lebensdauer des Schutzbauwerks ergeben, kombiniert werden.

Die Zustandserfassung von Schutzbauwerken stellte in der Praxis bisher auf Einzelbauwerke ab. In der Literatur sind zahlreiche Methoden zur Bauwerkszustandsprüfung beschrieben. (ZELLER, RÖTHLISBERGER, 1987; GRABNER, 1989; STREIT, 1992) Bisher kaum entwickelt wurden Verfahren, um die Schutzfunktionalität ganzer Verbauungssysteme zur erfassen, zu bewerten und über längere Zeiträume zu beobachten, um daraus die Entwicklung des Naturgefahrenrisikos in den gesicherten Zonen darzustellen (Gefahrenzonenplanung).

4. DER VORGANG DES ZUSTANDSMONITORINGS FÜR MASSNAHMEN DER WILDBACHVERBAUUNG

Ähnlich wie bei der Überwachung der Sicherheit und Funktionalität anderer technischer Systeme umfasst das „Zustandsmonitoring" von Schutzanlagen der Wildbachverbauung grundsätzlich die Elemente der **Zustandserfassung**, der **Zustandbewertung** (in bautechnischer und funktionaler Hinsicht) und die zeitabhängige **Prognose der Entwicklung von Sicherheit und Funktionalität** unter Berücksichtigung der Instandhaltungsmaßnahmen.

Die Zustandserfassung beinhaltet die Aufnahme des Bauwerkszustandes, aufgetretener Baumängel und Schäden am Bauwerk selbst, Schäden in der unmittelbaren Umgebung des Schutzbauwerkes und der Beeinträchtigung der Bauwerksfunktion. Ebenso werden Prozesse (z.B. Muren, Hangbewegungen, Erosion) oder naturräumliche Faktoren im Einzugsgebiet erfasst und deren Auswirkungen auf die Sicherheit oder Funktionalität beurteilt. Bei der Zustandserfassung an den Bauwerken selbst und in der Umgebung ist mit offensichtlichen und versteckten Gefahren-/Schadensfaktoren zu rechnen.
Die Bewertung des Zustandes ist zunächst eine bauphysikalische Aufgabenstellung und zielt auf die Überprüfung der äußeren und inneren Standsicherheit des Schutzbauwerkes ab. Daran anknüpfend ist die Auswirkung von Mängeln und Schäden auf die Funktionalität (Gebrauchstauglichkeit) des Bauwerks abzuleiten.[10] In gleicher Weise wird die Auswirkung von ablaufenden Prozessen und naturräumlichen Einflussfaktoren in der Bauwerksumgebung, die Einfluss auf Sicherheit und Funktion haben, analysiert.
In einem nächsten Schritt wird von der aktuellen Standsicherheit und Funktionalität des Einzelbauwerks auf den Schutzerfüllungsgrad des gesamten Verbauungssystems (oder Teilen

[10] Mängel in der Gebrauchstauglichkeit von Schutzbauten können aber auch zur Reduktion der Tragfähigkeit (Standsicherheit) führen.

davon) geschlossen, um die generelle Risikoentwicklung für die geschützten Zonen ermitteln und den Wirkungsgrad von Instandsetzungsmaßnahmen insgesamt bewerten zu können.

Die **operative Durchführung eines Zustandsmonitorings** für Schutzbauwerke der Wildbachverbauung erfordert folgende Maßnahmen:
- Erstmalige Zustandserfassung (Bauwerkskataster, Zustandsdatenbank)
- Entwicklung eines Schutzfunktionsmodells (Sicherheitsanforderungen, Funktionsrisiko) für das gesamte Wildbachverbauungssystem
- Laufende Überwachung
- Regelmäßige Zustandskontrolle
- Zustandkontrolle nach außergewöhnlichen Ereignissen (Hochwasser, Hangrutschung)
- Wiederkehrende Funktions- und Zustands-(Standsicherheitsprüfung)
- Planung der Instandhaltung und Instandsetzung (Prioritätenplanung)

Methodisch steht heute für die Zustandskontrolle je nach Bauweise eine Reihe von einfachen und anspruchsvollen Verfahren zur Verfügung, die in Abhängigkeit des Schadensumfangs und der für das Bauwerk unterstellten Sicherheitsanspüche eingesetzt werden:
- Visuelle Zustandsprüfung
- Einfache Probeverfahren am Bauwerk (Bewegungsmessung, Rissbreiten, Klangprobe)
- Probeentnahme (z. B. Kernbohrung, Probewürfel) für Laborprüfung
- Statischer Standsicherheitsnachweis
- (zerstörungsfreie Prüfverfahren: z.B. Resonanzfrequenzmessung)

5. SCHADENSRELEVANTE GEFAHRENBILDER UND SCHADENSRISIKEN FÜR WILDBACHSCHUTZBAUTEN

Die Schadenserhebung an der Bausubstanz gibt nur über einen Teil der Risikofaktoren Aufschluss, die die Tragfähigkeit (Standsicherheit) und Gebrauchstauglichkeit von Schutzbauwerken beeinflussen. Weitere relevante Faktoren für die Beurteilung des Zustandes und der Funktionalität sind:
- Umwelteinflüsse
- Prozesse und Lastwirkungen
- Konstruktive oder funktionale Schwachstellen
- Restrisiko durch Überlastfall (Versagen durch Belastungen über der Bemessungsgrenze)

Wildbachschutzbauwerke können grundsätzlich Lasteinwirkungen durch Wasserdruck, Erddruck, Auftrieb, seitlichen Druck der Talflanken und Stoßimpulse infolge eines Murganges ausgesetzt sein. Die Lasteinwirkungen treten häufig in charakteristischen Kombinationen auf, aus denen sich für Sperrenbauwerke typische Gefahrenszenarien für das Bauwerk ergeben, die bei Planung und Bemessung zu berücksichtigen sind (BÖLL, 1997; BERGMEISTER ET. AL., 2005):
- Hydrostatischer Wasserdruck auf die Sperre bei unverlandetem/nicht hinterfülltem Stauraum
- Verkleinerter Wasserdruck durch Sickerströmung im Verlandungskörper der Sperre (nach allmählicher Hinterfüllung)
- Stoßbelastung infolge eines Murganges im nicht verfüllten/teilverfüllten Stauraum
- Aktiver Erddruck nach vollständiger Hinterfüllung des Bauwerks und Abdichtung (Kolmatierung) der Sohle, Murstoß auf die Sperrenflügel

- Hydraulischer Grundbruch bei nicht hinterfüllter, voll eingestauter Sperre[11]
- Wasserdruck auf die Sperrenflügel, aktiver Erddruck auf das Bauwerk; die Talflanken sind luftseitig nach Auskolkung des Vorfeldes abgerutscht („Katastrophenfall“: häufigste Ursache für das Versagen einer Wildbachsperre)

<table>
<tr><th></th><th colspan="4">Hauptbaustoffe der Wildbachverbauung</th></tr>
<tr><th>Schutzbauwerk</th><th>Schüttmaterial, Erde</th><th>Stein, Mauerwerk</th><th>Beton, Stahlbeton</th><th>Holz, Stein/Holz</th></tr>
<tr><td>Hochwasserdämme</td><td rowspan="2">Setzungen
Erosion
Ausschwemmung
Grundbruch
Durchwurzelung
Tierbauten</td><td></td><td rowspan="3"></td><td rowspan="4"></td></tr>
<tr><td>Leitdämme für Muren</td><td rowspan="3">Abwitterung
Rissbildung
Abplatzungen
Ausbrechen des Fugenmörtels
Lockerung/Heraus-brechen von Steinen
Bewuchs</td></tr>
<tr><td>Steinschlichtung</td><td rowspan="6"></td></tr>
<tr><td>Ufermauern und Regulierungen</td><td rowspan="5">Rissbildung[12]
Ausblühungen
Betonabplatzungen
Betonabrieb durch Geschiebetransport (Verschleiß-beanspruchung)
Betonkorrosion durch Karbonatisierung, durch Chloride[13] oder Huminsäuren
Freilegung der Bewehrung
Korrosion an der Bewehrung
Verformung der Bewehrung</td></tr>
<tr><td>Grundschwellen und Sohlgurten</td><td rowspan="4"></td><td rowspan="2">Durchnässung
Pilzbefall
Verformung des Sperrenkörpers
Korrosion
Bruch von Hölzern
Vermorschung und Fäulnis (besonders im Bereich der Krone und der Einbindung)</td></tr>
<tr><td>Konsolidierungs-sperren</td></tr>
<tr><td>Geschiebe-Dosiersperren Sortiersperren</td><td rowspan="2"></td></tr>
<tr><td>Murbrecher</td></tr>
</table>

Tab. 1: Bauwerksspezifische Risikofaktoren betreffend die Tragfähigkeit (Standsicherheit)[14]

Für die korrekte Bewertung von Bauschäden ist fallweise eine bautypenspezifische Interpretation erforderlich. Beispielsweise haben Vertikalrisse in einer Betonsperre, die in den Talflanken eingespannt ist, eine andere Bedeutung (Ursache), als in einer Schwergewichtsmauer.

[11] Für gestaffelte Konsolidierungssperren ist zu beachten, dass die Zerstörung einer einzelnen Sperre negative Folgen für die nächst obere Sperre haben kann. Je nach Art der Schäden und der Dauer des Hochwasserereignisses ist daher auch mit einer Zerstörung dieser oberhalb befindlichen Sperren zu rechnen. (ZELLER, RÖTHLISBERGER, 1984)

[12] Da in Stahlbetonbauteilen eine Rissbildung infolge Beanspruchung durch Biegung, Zug, Querkräfte und Torsion meist nicht vermeidlich ist, verlangt die ÖNORM B4700 einen Nachweis der Rissbreitenbeschränkung für den Grenzzustand der Gebrauchstauglichkeit (für Bauwerke der Wildbachverbauung auf 0,3 mm).

[13] Eine Zusammenstellung der relevanten Expositionsklassen für den Angriff von Beton gemäß ÖNORM B 4710-1 in BERGMEISTER ET AL. (2005)

[14] Weitere Bauweisen („Baustoffe“) in der Wildbachverbauung sind lebende Pflanzen (ingenieurbiologische Maßnahmen) und Drahtschotterkörbe (heute kaum noch in Verwendung).

Schutzbauwerk	Funktions- und umgebungsspezifische Risikofaktoren				
	Baumängel Funktion	**Fundierung Gründung**	**Hydraulische Prozesse**	**Hangprozesse**	**Witterung Vegetation**
Hochwasser-dämme	Überlastung durch Hochwasser Mangelhafte Verdichtung	Grundbruch Unterströmung	Überströmung Seitliche Erosion Tiefenerosion	Hangdruck Hangbewegung	Bewuchs Bestockung Wühltätigkeit von Tieren Viehtritt
Leitdämme für Muren	Vorverfüllung des Ablagerungs-raums	-	Überlastung durch hydrodyn. Druck	Steinschlag Felssturz	-
Steinschlichtung	Übersteile Bauweise Schlechte Verzahnung mit Untergrund	Sohleintiefung Erosion der Fundamente Ausschwemmung	Hydrodynam. Beanspruchung Sogwirkung Geschiebetrieb	Hangdruck Hangbewegung	Durchwurzelung
Ufermauern und Regulierungen		Sohleintiefung Fugenerosion an der Grenze Boden-Beton	Überlastung durch Hochwasser Auflandung Geschiebeabrieb Wildholztrift Wechselsprung	Hangdruck Hangbewegung	Eisstoß Frostsprengung
Grundschwellen und Sohlgurten	Unterdimen-sionierte Abflusssektion	Sohleintiefung Grundbruch Unterströmung Fugenerosion an der Grenze Boden-Beton	Überlastung durch Hochwasser Auflandung Geschiebeabrieb Wildholztrift	Hangdruck Hangbewegung Abrutschende Talflanken Seitenerosion	
Konsolidierungs-sperren	Sperrenbruch unterdimen-sionierte Abflusssektion Kippen der Sperre Arbeitsfugen Kein Flügelanzug	Unterkolkung Sohleintiefung Grundbruch Unterströmung Umgehung des Bauwerks Freilegen der Fundierung nach Bruch der Sperre unterhalb	Überlastung durch Hochwasser Kolkbildung Auftrieb Murstoß Abscheren der Flügel und Krone	Hangdruck Hangbewegung Abrutschende Talflanken Seitenerosion Verschiebung der Flügel Einseitiges Senken der Sperre	
Geschiebe-Dosiersperren Sortiersperren	Verlandung des Stauraums Verklausung der Dolen/ Öffnungen Kippen der Sperre Arbeitsfugen	Unterkolkung Sohleintiefung Grundbruch Unterströmung Umgehung des Bauwerks		Hangdruck Hangbewegung Abrutschende Talflanken	
Murbrecher	Vorverfüllung des Ablagerungs-raumes				

Tab. 2: Funktions- und umgebungsspezifische Risikofaktoren betreffend die Tragfähigkeit (Standsicherheit) und Gebrauchstauglichkeit (Funktionalität) von Schutzbauwerken in Wildbächen

Die Wirkung der angeführten Risikofaktoren ist oft mehrschichtig und führt erst in Kombination mehrerer Faktoren zum Bauwerksversagen. Beispielsweise kann eine hochwasserbedingte Auskolkung des Sperrenvorfeldes zum Abgleiten der Einhänge und zur Unterschwemmung, fallweise sogar zur Umgehung des Bauwerkes durch den Wildbach führen. Durch den Verlust des Erdwiderstandes und der Sohlreibung in den Talfanken ist schließlich die äußere und/oder innere Standsicherheit der Sperre nicht mehr erfüllt und es kann zum Kippen oder zum Bruch des Bauwerks kommen.

Untersuchungen von KRONENFELLNER-KRAUS (1962) ergaben, dass von 8 Sperrenbrüchen nur 4 auf eine zu geringe Dimensionierung der Sperre, 4 Brüche jedoch auf mangelhafte Ausführung oder vernachlässigte Instandhaltung zurückzuführen waren.

Nachfolgend wurde aufgrund von Praxiserfahrung und Expertenbefragung eine Bewertung der einzelnen Risikofaktoren für die wichtigen Bautypen der Wildbachverbauung versucht.

	Risikofaktoren										
Schutzbauwerk	**Materialschäden Bauwerksschäden**	**Statisch unterdimensioniert**	**Bewehrung zu Schwach**	**Funktionale Schwachstellen**	**Sohlauskolkung, Tiefenerosion**	**Hangdruck, Hangbewegung**	**Setzungen, Grundbruch**	**Murstoß Sturzprozesse**	**Wildholz, Verklausung**	**Mangelhafte Wartung, Kotrolle**	**Überlastfall**
Hochwasser-dämme	hoch	hoch	-	gering	mittel	gering	hoch	gering	hoch	hoch	hoch
Hochwasser-Rückhaltebecken	hoch	hoch	hoch	hoch	gering	gering	hoch	gering	hoch	hoch	hoch
Leitdämme für Muren	gering	hoch	hoch	hoch	mittel	gering	mittel	hoch	hoch	gering	mittel
Steinschlichtung Steinrampen	mittel	gering	-	gering	hoch	hoch	hoch	-	mittel	gering	hoch
Ufermauern und Regulierungen	mittel	mittel	mittel	gering	hoch	hoch	hoch	gering	hoch	gering	mittel
Grundschwellen und Sohlgurten in Stein/Beton	mittel	gering	gering	gering	hoch	mittel	hoch	gering	hoch	gering	gering
Konsolidierungs-sperren in Holz	hoch	hoch	-	gering	hoch	hoch	mittel	hoch	mittel	gering	gering
Konsolidierungs-sperren in Beton	mittel	hoch	hoch	gering	mittel	hoch	hoch	hoch	mittel	gering	mittel
Geschiebe-Dosiersperren Sortiersperren	mittel	hoch	hoch	hoch	gering	mittel	mittel	hoch	hoch	mittel	mittel
Murbrecher Bremsbauwerke	mittel	hoch	hoch	mittel	mittel	mittel	mittel	gering	gering	gering	mittel

Tab. 3: Bewertung der Empfindlichkeit für schadensrelevante Risikofaktoren betreffend die Tragfähigkeit (Standsicherheit) und Gebrauchstauglichkeit (Funktionalität) von Schutzbauwerken in Wildbächen

6. LEBENSDAUER UND PROGNOSE DER ZUSTANDSENTWICKLUNG

Schäden an Schutzbauwerken, der Verlust der Schutzwirkung oder in manchen Fällen das Bauwerksversagen können die Folge der Alterung und damit verbunden der vernachlässigten Instandhaltung bzw. nicht zeitgerecht durchgeführter Instandsetzungsmaßnahmen sein.

In der Literatur wird folgende Lebensdauer für Schutzanlagen angegeben:

Bauwerkstyp/Baustoff	Literaturangabe	Lebensdauer
Holzsperren (Steinkastensperren)	ZELLER, RÖTHLISBERGER, 1987 LÄNGER, 1999 BÖLL ET AL., 1999 NÖTZLI ET AL., 2002	20 - 50 Jahre max. 60 Jahre[15]
Steinsperren Gemauerte Sperren	LÄNGER, 1999 ROMANG ET AL., 2004	60 – 80 Jahre max.. 100 Jahre
Sperren in Beton und Stahlbeton	ZELLER, RÖTHLISBERGER, 1987 ROMANG ET AL., 2004	ca. 100 Jahre (bis 150 Jahre) alte Bauwerke: 50 - 80 Jahre

Tab. 4: Literaturangaben zur mittleren (maximalen Lebensdauer) von Schutzmaßnahmen der Wildbachverbauung.

[15] Unter den Bedingungen der Beschattung und ständiger Befeuchtung sowie bei konstruktiv einwandfreier Ausführung ist im Einzelfall eine Lebensdauer bis zu 80 Jahren möglich.

Nach BÖLL ET. AL. (1999) sind für die tatsächliche Lebensdauer eines Bauwerks neben den objekts- und umgebungsbedingten Faktoren vor allem die Häufigkeit und Intensität von Extremereignissen im Gerinne und das zeitabhänige Stabilitätsverhalten der Hänge von zentraler Bedeutung.
Die unter 6. angeführten Schadensfaktoren bewirken im Eintrittsfall eine wesentliche Reduktion der „normalen“ Lebensdauer eines Wildbachbauwerkes. Von zentraler Bedeutung ist auch der Standort des Schutzbauwerkes: Für Holzsperren kann sich die Lebensdauer beispielsweise bei stabilen Boden und Hangverhältniss, Beschattung und laufender Befeuchtung der Sperrenkörpers um ca. 10 Jahre verlängern (bis zu 60 Jahren). Im Gegensatz dazu bedürfen Holzsperren auf Untergrund mit starken Kriech- und Gleitbewegungen häufig schon nach 30 Jahren einer Sanierung. Stark reduzierend auf die Lebensdauer (Rückgang bis auf die Hälfte) wirkt sich eine starke Besonnung oder ein häufiges Trockenfallen der Sperre aus (die Vermorschung wird stark beschleunigt). (ZELLER, RÖTHLISBERGER, 1984)

Die richtige Einschätzung der tatsächlichen Lebensdauer ist von zentraler Bedeutung für die Planung und Priorisierung von Instandhaltungsprogrammen und Instandsetzungsmaßnahmen.

7. SCHLUSSFOLGERUNGEN UND AUSBLICK

In der Regel ist bei Instandhaltungskonzepten in Wildbacheinzugsgebieten aufgrund der großen Zahl an sanierungsbedürftigen Schutzbauwerken die Erstellung einer Dringlichkeitsreihung für die Sanierungsmaßnahmen erforderlich. Die Dringlichkeitsreihung sollte einerseits den aktuellen Zustand des Bauwerkes (Standsicherheit, Gebrauchstauglichkeit), aber auch die Beschädigung (teilweiser Funktionsverlust) oder Zerstörung (völliger Funktionsverlust) sowie deren Folgen berücksichtigen. (GRABNER, 1989)
Damit ist es erforderlich, die Auswirkung von Bauwerksversagen auf den Schutzerfüllungsgrad des damit gesamten Verbauungssystems realistisch einzuschätzen.
Bisher stehen kaum Instrumente zur Verfügung, um die komplexe Fragestellung für ein gesamtes Wildbacheinzugsgebiet zu lösen und die Risikoentwicklung in den gesicherten Zonen darzustellen. Daraus ergibt sich ein dringender Forschungsbedarf zur Entwicklung neuer Methoden, die es ermöglichen, ein umfassendes Zustandsmonitoring und effektive Instandhaltungsprogramme für gegliederte Wildbacheinzugsgebiete mit umfangreichen Schutzmaßnahmen zu entwickeln. Derartige Modelle sind auch von zentraler Bedeutung für die Gefahrenzonenplanung und die darauf aufbauende Raum- und Sicherheitsplanung.

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STRUCTURAL MITIGATION MEASURES

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Abstract
Structural mitigation structures are an integral part of countermeasures against floods and debris flows and act as water and sediment controlling components. Within the last centuries different types were developed, originating mainly on empirical approaches. These structures have to fulfil different functions within a protection concept and to withstand high impact forces. The historical development of structural mitigation measures is summarized and an overview of designs applied nowadays in torrential catchments is mentioned. The effectiveness of function types is shown by an Austrian sample and impact forces are listed. Considerations about failure mechanisms lead to suggestions strengthening the structures.

1. INTRODUCTION

An essential aspect of risk management is the design of mitigation measures which reduce the existing risk to an accepted level of residual risk. Two types of mitigation measures can be distinguished (Zollinger, 1985): active measures, and passive measures.
Active measures focus on the hazard, while passive measures focus on the potential damage. It is of fundamental importance to risk management to clearly define the spatial and temporal objectives of the desired degree of protection, with an understanding of acceptable residual risk.
Active mitigation measures may affect the initiation, transport or deposition of floods or debris flows and can therefore change their magnitude and frequency characteristics. This can be achieved either by changing the probability of occurrence (disposition management), or by manipulating the flow itself (event management).
Passive mitigation measures are used to reduce the potential loss by, for example, altering the spatial and temporal character of either the damage produced by flows or the associated vulnerability. Vulnerability of a disaster can be changed either with land use planning like hazard mapping, or through immediate disaster response.

Active mitigation measures

Objective	Task	Measure
Disposition management		
Decrease runoff	Decrease peak discharge	Mainly non-structural mitigation measures
Decrease erosion	Decrease surficial erosion due to overland flow	Mainly non-structural mitigation measures
	Increase slope stability	Non-structural and structural mitigation measures
	Decrease vertical and lateral erosion in the channelbed	Mainly structural mitigation measures
	Decrease water discharge at high erodible channel-reach	Structural mitigation measures

Event management		
Discharge control	Decrease peak discharge to prevent damage	Structural mitigation measures
Sediment control	Transformation process	Structural mitigation measures
	Deposition debris under controlled conditions	Structural mitigation measures
	Debris flow deflection to adjacent areas	Structural mitigation measures
	Organic debris filtration	Structural mitigation measures

Figure 1: Active countermeasures – Overview (HÜBL et al., 2005 modified)

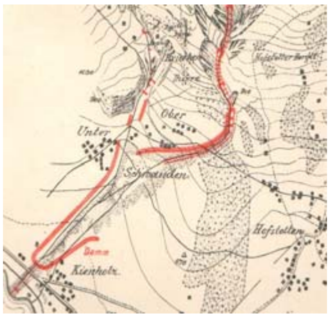
2. DESIGN OF STRUCTURAL MITIGATION MEASURES

Active countermeasures are mainly of two types: structural and non-structural. Structural mitigation measures are numerous and their design up to now has essentially been the result of empirical approaches.
The main task of the first mitigation structures was to protect private properties (houses and goods as well as agricultural areas) and important traffic routes (STRELE, 1938). In Tyrol structural mitigation measures like

- longitudinal structures (walls)
- deflection and redirecting structures
- transverse structures (barriers for sediment control)

date back to the 15th century.

Walls



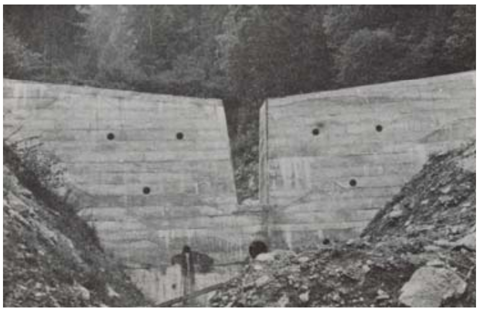
Realignment

Sediment control barriers

Figure 2: Examples of structural mitigation measures in the last centuries

Due to increased settlement on fans and riversides on one hand and decreasing effectivity of check dams because of filled up deposition areas on the other hand, new methods like temporary storage of sediments by control structures with slots and inclined rakes were developed. At the end of the 19th Century the first “forest-technical service” was founded in France, with the intention to establish an integral watershed management by the combination of forestal and technical measures. The protection concepts were extended to the whole catchment area. After Second World War construction methods became cheaper and their installation less time-consuming. Thus new defence concepts had been developed. One of these new methods was the design of sediment control barriers and slit barriers with the purpose of sediment retention and temporary storage. Since 1980´s one of the main aspects of torrent control was to increase the knowledge of debris flow process. The objective is not any longer sediment retention but energy dissipation by debris flow breakers with additional sediment sorting and dosing barriers.

Slot barrier, 19th Century (Lünitzbach, Kärnten-Austria)



Slit barrier, 19th Century (Reißgraben, Kärnten-Austria)

Sectional beam barrier (Niedernsiller Mühlbach, Salzburg-Austria)

Sectional barrier with fins (Gemmersdorferbach, Kärnten-Austria)

Figure 3: Development of barrier design in torrential catchments

The definition and design of structural mitigation measures (levees, check dams, slit dams, retention basins…) requires some knowledge on two aspects: the stresses applied to the structure by the flow and the "hydraulic" action of the structure (deviation of the flow, retention of sediments…). Consequently, structural mitigation measures require some preliminary knowledge of the flow that is likely to occur and hit the protection structures or buildings. Furthermore, the presence of protection structures modifies the flow and consequently the hazardous areas.

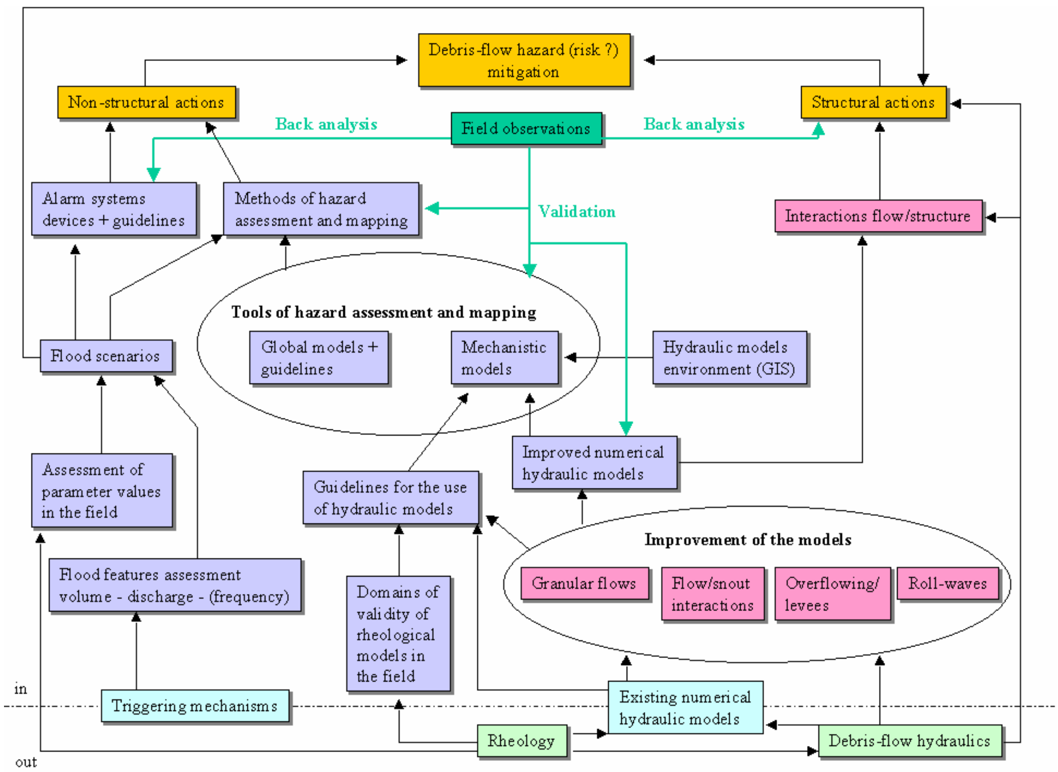


Figure 4: Assessment domains for the design of mitigation measures (LAIGLE, 2003)

The barrier typology is based on the division between **Solid Body Barriers** and **Open Barriers**. Barriers featuring no functional openings in the barrier body are called Solid Body Barriers. Open Barriers include barrier types with openings to allow parts of the water and/or sediments to pass through.

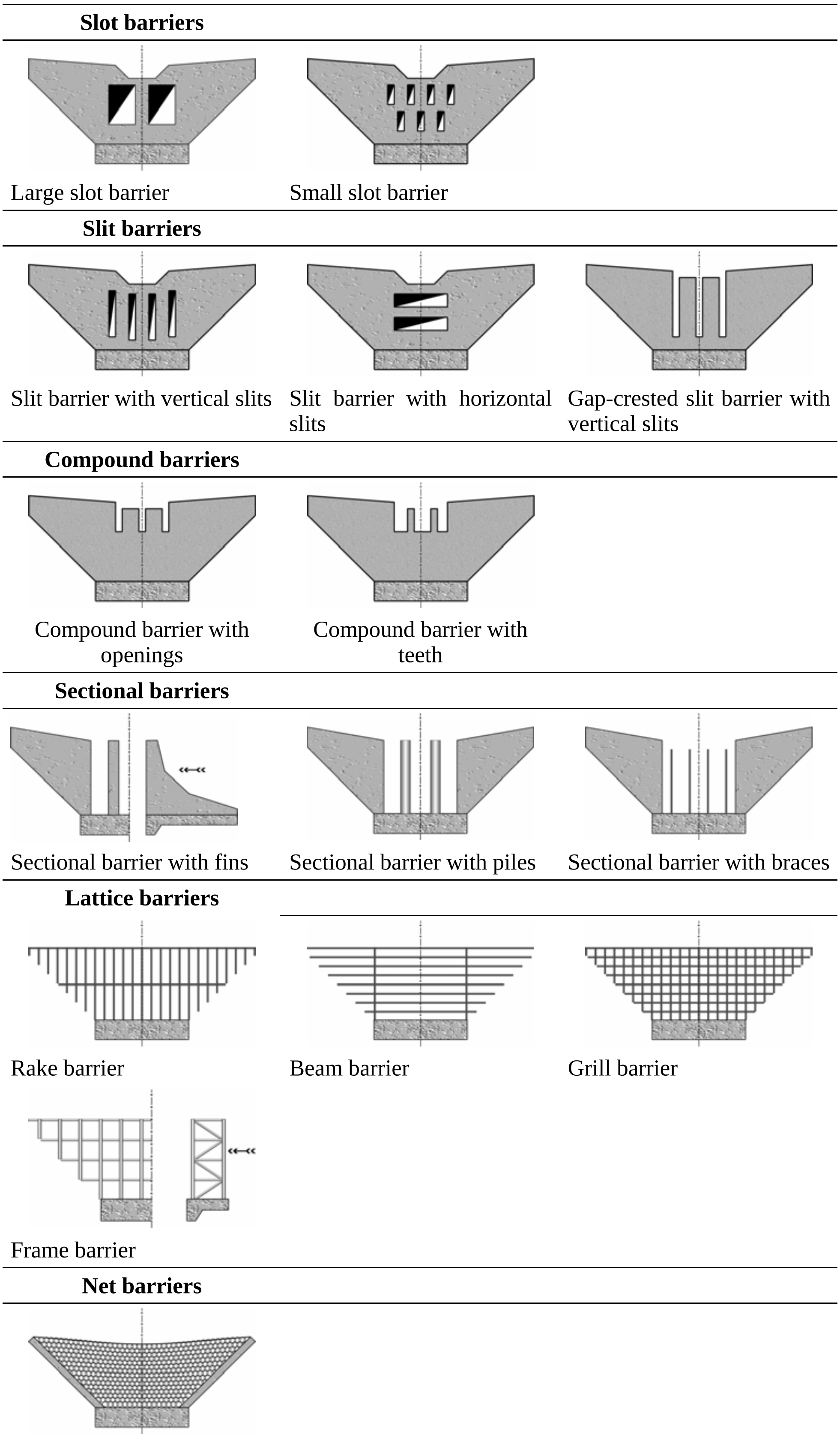


Figure 5: Construction types of open barriers (HÜBL et al., 2003)

Construction types within modern torrent control are specified according to their functions. The main functions within a protection concept can be summarized as follows (FIEBIGER, 1997):

Consolidation and stabilisation:	Fixation of the longitudinal profile of a torrent bed at a distinct elevation to stop depth erosion and/or lateral slides
Retention:	Storage of water and/or deposition of bedload during an event
Sorting:	Filtration and deposition of specific bedload components during an event
Dosing:	Temporary retaining of water/sediment
Debris flow breaking:	Declining the high energy level of a debris flow to a lower level (dissipation)
Woody debris retention:	Filtration of woody debris during an event

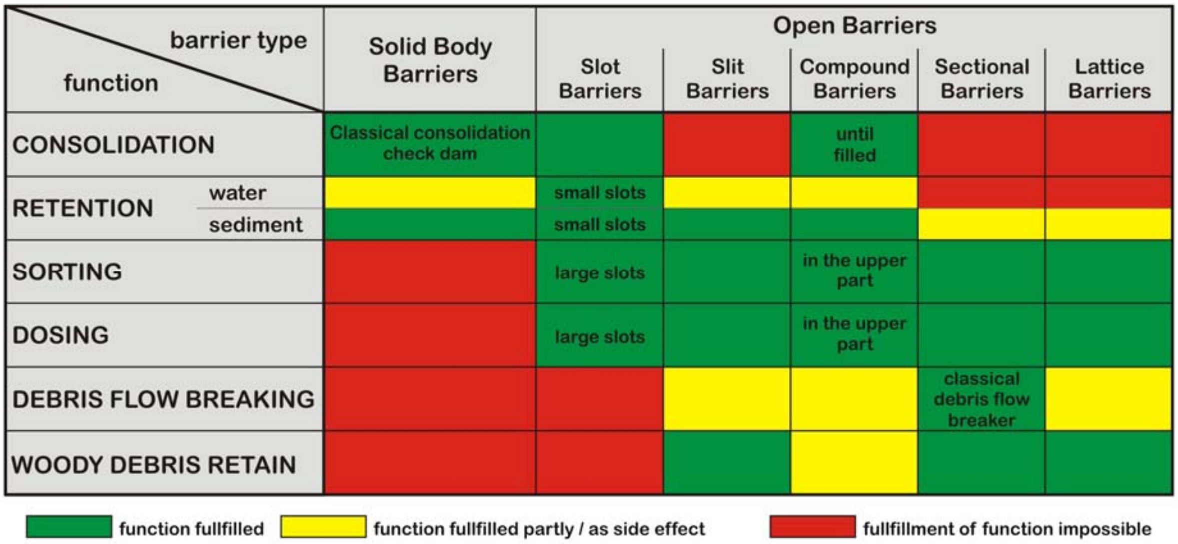


Figure 6: Functions and types of structural mitigation measures (HÜBL et al., 2003)

The same function can be fulfilled by different construction types while on the other hand one construction type can be used for different tasks within the protection concept.

3. EFFECTIVENESS

In the context of an examination in Austria, 131 structures for bed load control were investigated in detail in geometry, function and mode of operation. The degree of function fulfilment is acquired by “debit – is” scenarios by means of analysis of debris input, deposition at the structure as well as debris output of every single barrier. The debit initial setting arises from the assumed design event with a return period of 100 to 150 years. Documented impacts of the last 20-30 years are compared to this to be able to show possible differences in the function fulfilment. The function fulfilment degree is judged both into regard on quality (correlation of assumed transport process to the actual one) and on quantity (correlation of mobilized and deposited debris volume).
The analysis of data resulted, that function fulfilment was reached completely at 59 %, partly at 36 % and not at 5 % of all taken events and structures, without consideration of the function type.

Reasons for insufficient function fulfilment – considering self-acting emptying of retention basins too– are found in one for the structures unsuitable function, in the woody debris difficulties, in the geological qualities of the debris as well as in an inadequate discharge. At increasing fine substance quota or increasing cohesion, the trend to self-acting emptying of deposition rooms by mean discharges decreases rapidly. The indispensable consequence is machine clearing to maintain the protection effect of the barrier.

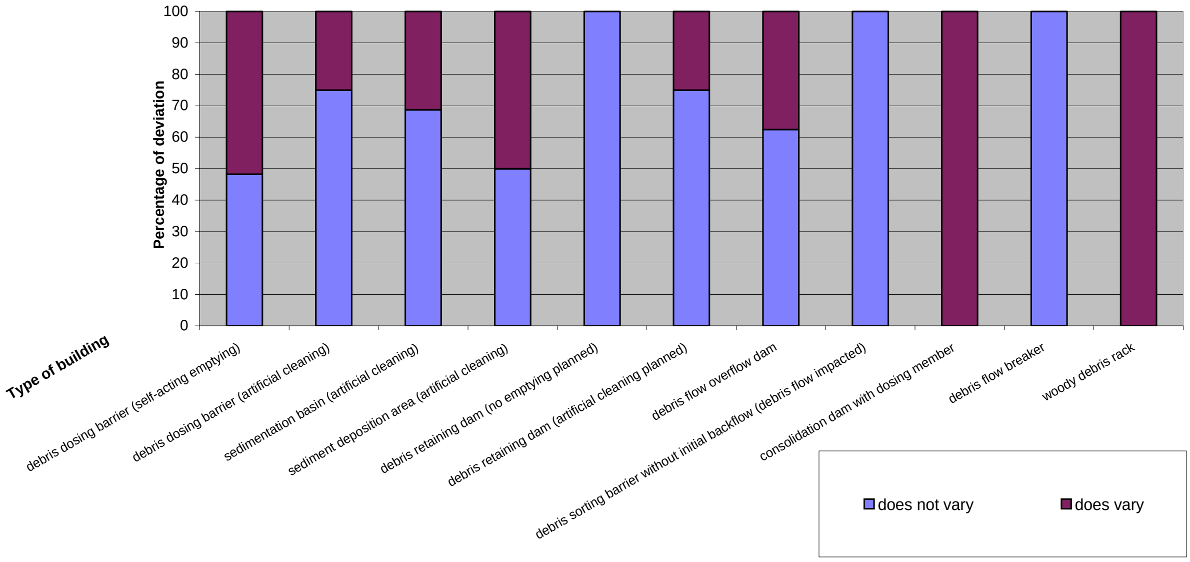


Figure 7: Qualitative deviation of the function type

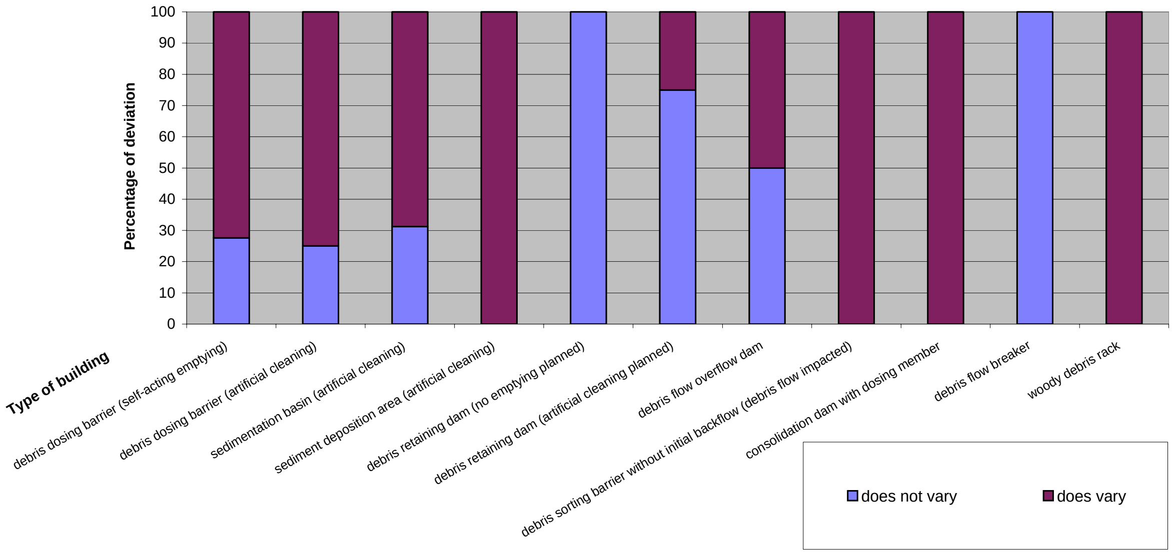


Figure 8: Quantitative difference of the function type

This examination showed that partitioning of functions to several constructions provides better results than one single measure. Inclined rakes tend less to woody debris clogging than vertical ones. Vertical elements force self-active emptying more than horizontal ones, which tend to clogging very fast, even if the woody debris consist of small pieces.

4. IMPACT FORCES

Debris flows, unlike floods, exert enormous impact forces on obstacles in their path, such as bridge piles, structural mitigation measures, buildings and so on. The estimation of the range of impact force is necessary for reasonable planning of structures against debris flows.

The impact force of debris flows consists of two parts: the dynamic pressure of fluid and the collision force of single boulders. The latter often causes damages to engineering structures. Several devices have been developed by different researchers to measure the impact force of debris flows. Some of them only record the maximum impact force, such as pressure mark gages. In other experiences strain gages and recorders have been used to measure impact forces. Automatic systems have been equipped with piezoelectrical sensors connected to a recording microcomputer. These studies report to have collected more than 70 impact force graphs, finding fluid dynamic pressures as great as 500 t/m^2 and impulse forces of individual boulders as large as 318 tons.

Torrent	Location	Velocity [m/s]	Depth [m]	Density [kg/m³]	Estimated pressure [kN/m²]	Reference
Rio Reventado	Costa Rica	2,9 - 10,0	8,0 - 12,0	1130 - 1980	582 - 1136	Waldron 1967
Hunshui Gully	China	10,0 - 13,0	3,0 - 5,0	2000 - 2300	396 - 764	Li & Luo 1981
Bullock Greek	New Zealand	2,5 - 5,0	1,0	1950 - 2130	92 - 127	Pierson 1981
Pine Creek	USA	10,0 - 31,1	0,1 - 1,5	1970 - 2030	89 - 957	FINK et al. 1981
Wrightwood Canyon (1969)	USA	0,6 - 3,8	1,0	1620 - 2130	149 - 110	Morton & Campell 1974
Wrightwood Canyon (1941)	USA	1,2 - 4,4	1,2	2400	178 - 152	Sharp & Nobles 1953
Lesser Almatinka River	USSR	4,3 - 11,1	2,0 - 10,4	2000	193 - 1034	Niyazow & Degovets 1975
Nojiri River	Japan	12,7 - 13,0	2,3 - 2,4	1810 - 1950	381 - 429	Watanabe & Ikeya 1981
Mayflower Gulch	USA	2,5	1,5	2530	183	CURRY 1966
Dragon Creek	USA	7,0	5,8	2000	555	COOLEY et al. 1977

Figure 9: List of surveyed or observed debris flow surges and estimated pressure (COSTA, 1984)

5. CONSIDERATIONS OF FAILURE MECHANISMS

Although thousands of structural mitigation measures have been constructed worldwide, only a few critical damages are reported. In principle two different failure mechanisms have to be considered:

- Functional failure
- Structural failure

Functional failure means, that the construction did not work in the way as designed within the protection concept. This failure may lead either to a subsequent structural failure or the structure itself stays intact.

Underestimation of sediment volume

Lateral bypassing after clogging by woody debris

Figure 10: Examples of functional failures

Structural failures can be attributed to misinterpreted design estimates or faults in the building material. As a matter of fact these failures lead consequently to functional failures too.

Underestimation of impact forces

Scouring and/or inadequate design

Figure 11: Examples of structural failures

Structures are damaged or destroyed mainly by:

- Application of inadequate building material
- Alteration of material properties
- Underestimation of impact forces and loads
- Lateral pressure induced by slope processes
- The scouring of the lateral abutment
- Scouring downstream of the structure
- Lateral bypassing, caused either by the absence of a sloping wing wall top or when discharge is blocked by bedload, as a consequence of sediment accumulation downstream of the dam
- Impact of a debris flow, when the body of the structure is not adequately supported

6. STRENGTHENING OF BARRIERS

From visual inspection and conceptual static analyses it could result that the internal and outer stability of the system is not given anymore for specified action events. This situation demands a strengthening measure. Which kind of strengthening measure results in a higher effectivity could be verified by parameter studies as shown in the following.

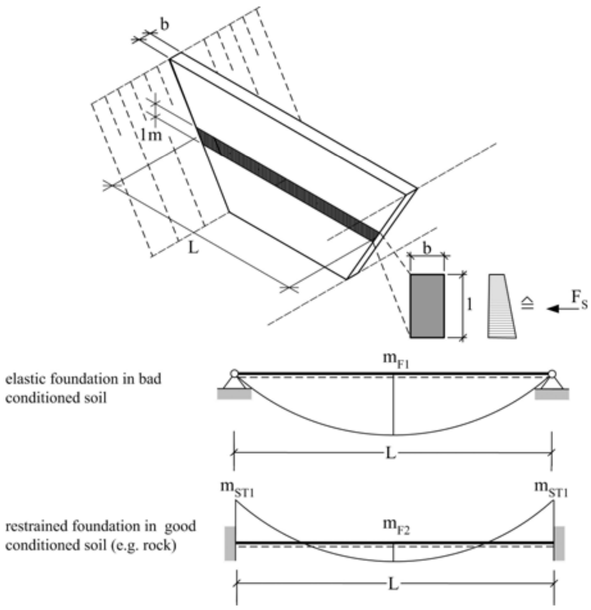


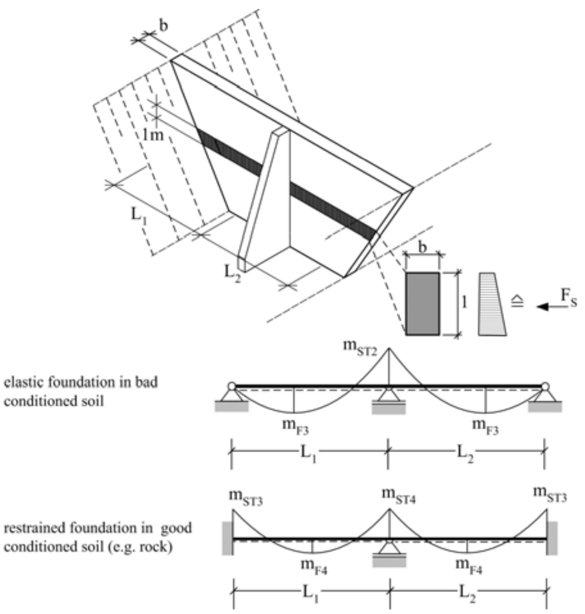
Figure 12: System 1: Slab barrier (single solid body barrier) – elastic foundation and restrained foundation

6.1 SLAB BARRIER - SIMPLY SUPPORTED OR FIXED SUPPORTED

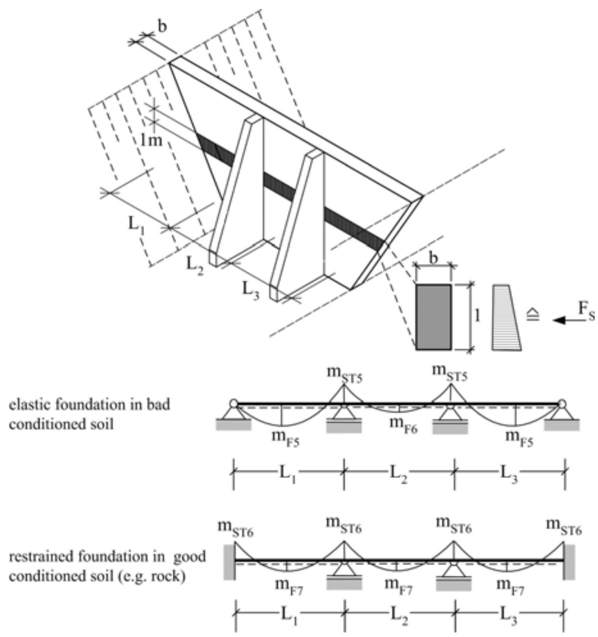
In the first case a full plate barrier has been considered regarding the static system, see Figure 12 to Figure 14. The flank foundation conditions and slabs supporting the plate are varied to get an impression about the additional activatable resistance compared to the original plate system. The achievable load increasing factors are given in Figure 13. While the Lane demonstrates the original systems and the columns show the increasing factors due to the change to a strengthened system.

System	1 – simply supported	1 – fixed supported	2 - simply supported	2 - fixed supported	3 - simply supported	3 - fixed supported
1 - simply	1					
1 - fixed	1,5	1				
2 - simply	4,1	2,3	1			
2 - fixed	12,1	3,7	1,4	1		
3 - simply	13,8	4,5	1,9	1,1	1	
3 - fixed	29,6	7,8	2,8	1,8	1,0	1

Figure 13: Activatable resistance: Load increasing factors



System 2



System 3

Figure 14: Slab barrier (single solid body barrier) – elastic foundation and restrained foundation and additional slabs

6.2 SLIT BARRIER – SIMPLY SUPORTED OR FIXED SUPPORTED

In a similar way as for the full plate barrier the considerations are performed for the half space of a slit barrier, see Figure 15 to Figure 19. The achievable load increasing factors are given in Figure 16.

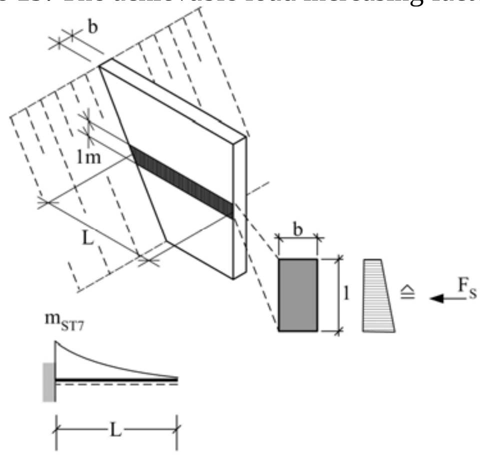
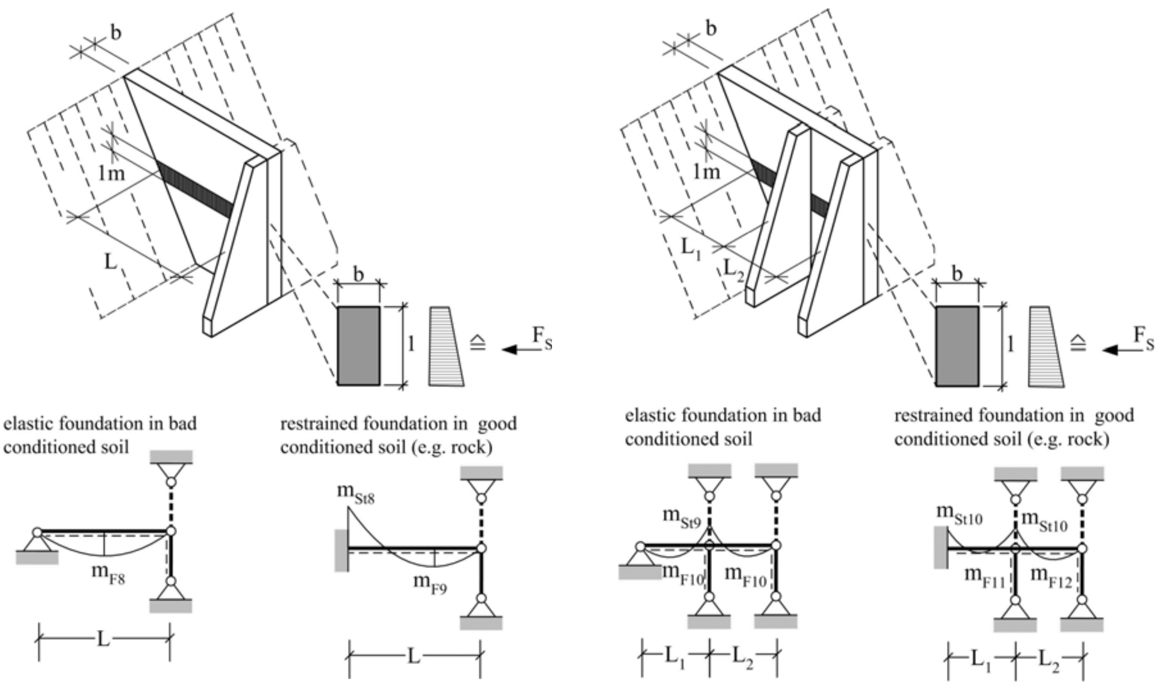


Figure 15: System 1: Slit barrier restrained supported

System	1 – fixed supported	2 – simply supported	2 - fixed supported	3 - simply supported	3 - fixed supported	4 - simply supported	5 - simply supported
1 - fixed	1						
2 - simply	1,6	1					
2 - fixed	1,5		1				
3 - simply	7,7	6,2	3,5	1			
3 - fixed	10,2	5,5	3,1	0,9	1		
4 - simply	0,9	4,7	0,6	0,1	0,1	1	
5 - simply	0,9		0,6	0,1	0,1	1,0	1

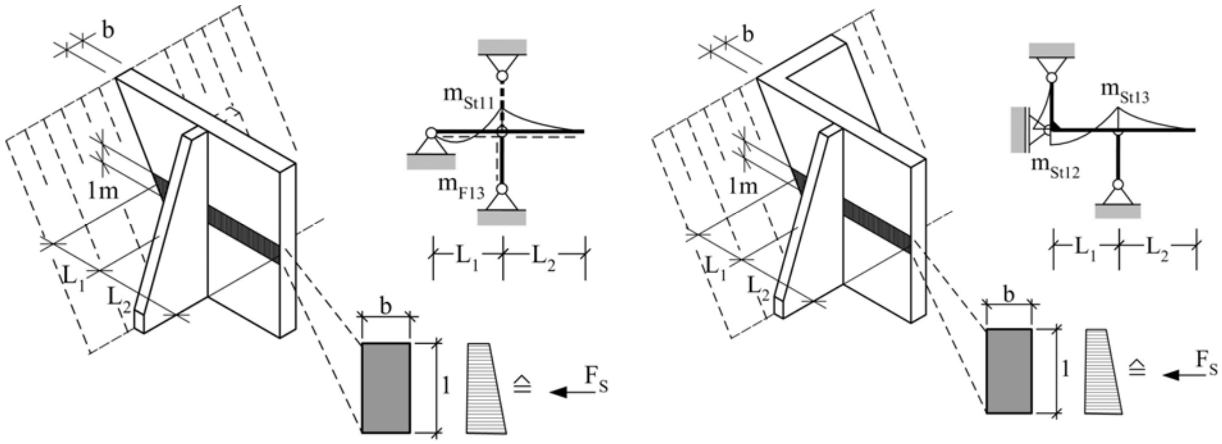
Figure 16: Activatable resistance: Load increasing factors



System 2

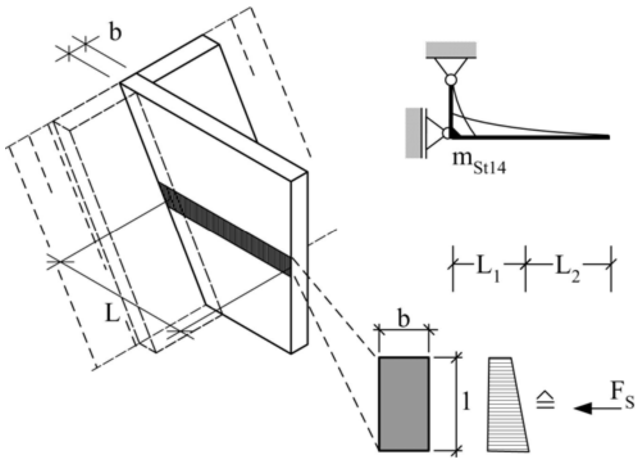
System 3

Figure 17: Slit barrier – elastic foundation and restrained foundation and slab systems



System 4 System 5

Figure 18: Slit barrier – elastic foundation and restrained foundation and slab systems



System 6

Figure 19: Slit barrier - Structural measures to achieve restrained supported conditions

For a group of barriers these considerations can provide beside the functional aspects a guideline during the process of strengthening planning. The load increasing factors gives a qualitative inside in the effect of the measures.

7. CONCLUSIONS

Quite a huge variety of structural mitigation measure designs are applied in torrent control works. For selecting the best adjusted arrangement great importance to the knowledge of all ongoing geomorphic processes and their possible interaction with the mitigation measures is needed. This means a multidisciplinary approach has to be applied, including specialised skills in Applied Geomorphology, Fluid dynamics, Forestry and Structural Engineering.

Although there is a large pool of experience gained by practitioners working in this field of activity, a lot of scientific gaps still exist. The rare occurrence of design events obliges the engineers to derive special solutions, taken into account the functionality, effectivity and the stability of the structure. Therefore it is most important to collect and exchange the experience of existing mitigation measures worldwide.

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ASSESSMENT OF EXISTING BARRIER STRUCTURES

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Abstract
The dominance of natural hazards in our environment is very high. Only when natural events occur we realize their power and consequences. The risk potential of natural hazards and their possible consequences have prompted scientific studies of torrent events and barrier structures. This paper presents a method that allows the assessment of uncertain stress forces and resisting elements. The method is based on a probabilistic approach and includes uncertain parameters regarding acting and resistance into a risk analysis. This way of modeling also makes it possible to take into account in time changes of the barrier resistance.

1 INTRODUCTION

Natural hazards are phenomena that have essential consequences for society and its settlement areas. Increasingly, human settlement activity takes into account the space required by natural events. Therefore, it is especially important to answer society's need for protection with constructional measures. During the first phase, however, i.e., before constructional measures are taken, the possibilities regarding natural measures should be investigated. While the effects of natural events can be dramatic, as we all have experienced recently, it is hard to measure the elements involved in natural hazards and the structures withstanding them. This paper aims at showing how uncertainties with natural hazards can be included in assessments, and it critically discusses existing approaches to safety. The following discussion of acting and resistance is based on torrent barriers.

2 DETERMINING A LOAD INCREASE FACTOR FOR DEBRIS FLOW RESISTANCE BY RISK ANALYSIS

When assessing torrent barriers, various stress combinations are taken into consideration. The active soil pressure (G_a) for the state of backfill and the maximum water pressure for the unfilled are assessed. The highest load on this kind of construction, however, occurs in the unfilled state when hit by a debris flow. Currently, the load caused by debris flow impact is integrated into assessments by using the hydrostatic water pressure, increased by a load increase factor (k_{LI}) (equation (1)). The coefficient k_{LI} takes into account both the increased density in relation to pure water and the dynamic effects occurring when debris mixture hits the barrier. The pertinent literature has various specifications of the value of this load increase factor. Following the Swiss Guidelines for Assessing Torrent Barriers [1], a load increase factor between seven and ten can be assumed. WLS Report 50 [2] [3] [4] has a load increase factor between one and three.

$$p_{deb} = k_{LE} \cdot \gamma_W \cdot h_W \tag{1}$$

with: $k_{LE} = 7 \div 10$ according to [1], $k_{LE} = 3 \div 11$ according to [2][3][4] where γ_G is the specific gravity of the water, which is assumed to be 11 kN/m³ in order to take into account the presence of particular matter in the water. h_W is the storage level of water behind the barrier. In the following discussion, h_W is arranged to be equal to the barrier height (h). In the following, a load increase factor is calculated by risk assessment. The model structure is described in Figure 1. The model consists of three modules: the model definition, the probabilistic calculation module, and the safety analysis module. The model definition module is used to adapt the given location of a barrier to the general model. This involves the definition of an analytic model of acting and resistance. In order to achieve realistic predictions, a calibrated resistance model has to be used.

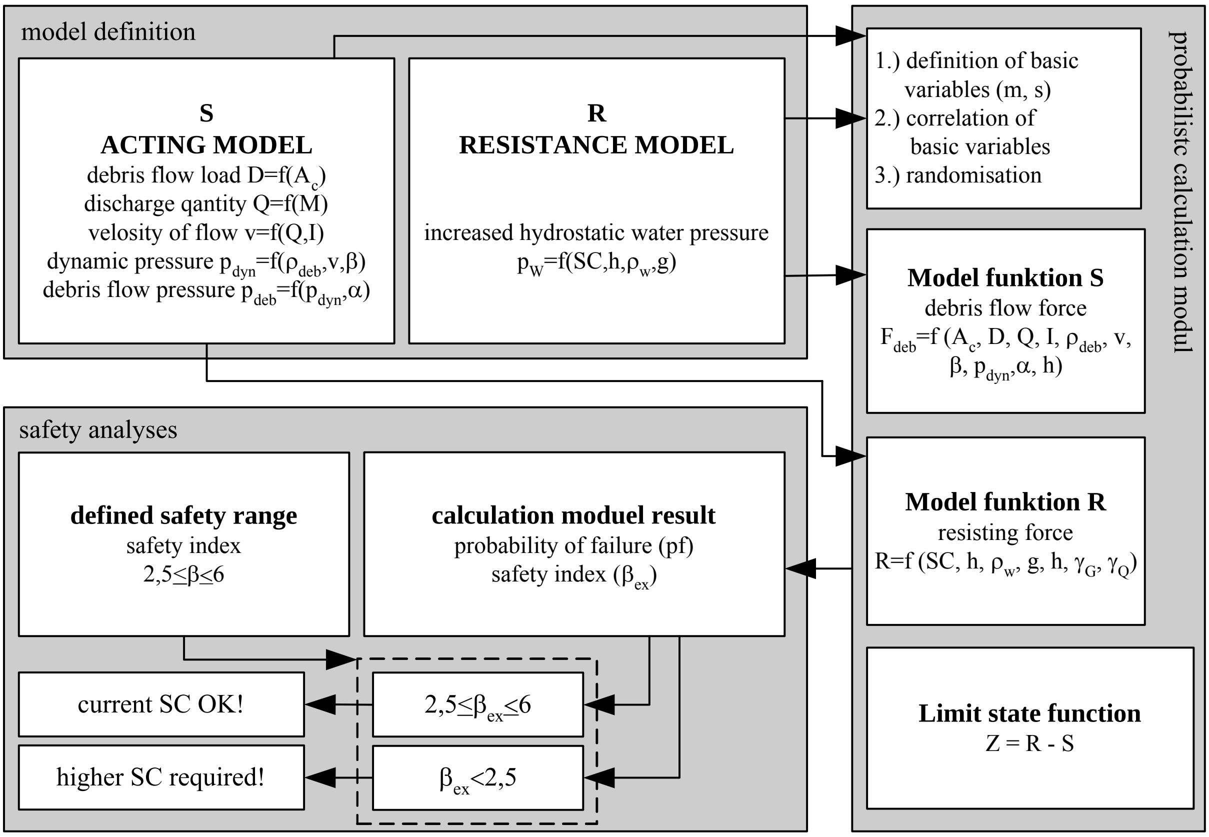


Figure 1: Structure of the model

2.1 ACTING MODEL

First aim is the analytically definition of the maximum possible debris flow pressure (p_{deb}), acting upon the back side of the barrier. The basis for determining a probable velocity of flow is the load of the debris flow (D) in [m³]. This value describes the maximum possible load of the debris flow as a result of a given catchment area variable, whose volume is the sum of particular matter and water. The catchment area, which significantly influences the above-mentioned parameters, in general has a value between 1.5 km² and 20 km² and a mean inclination between 5.0 and 45.0 degrees. With ZELLER and RICKENMAN [5], we can arrange the equation for determining the load of the debris flow of a given catchment area as shown in equation (2). The debris flow loads determined depending on the size of the catchment area in this way are illustrated in Figure 4.

$$D = 27 \cdot A_c^{0,78} \quad [\mathrm{m}^3] \tag{2}$$

The debris flow load (D) is used to determine the maximum discharge (Q) in [m³/sec]. MIZUYAMA 1992 has demonstrated with empirical formulae that event load and maximum discharge are correlated. He distinguishes between granular flows (Equation (3)) and muddy flows (Equation (4)). (cf Figure 2 and Figure 3) While the equations are based on Japanese research, they can be used for other regions of the world according to RICKENMANN [5].

Figure 2: Example for muddy flow

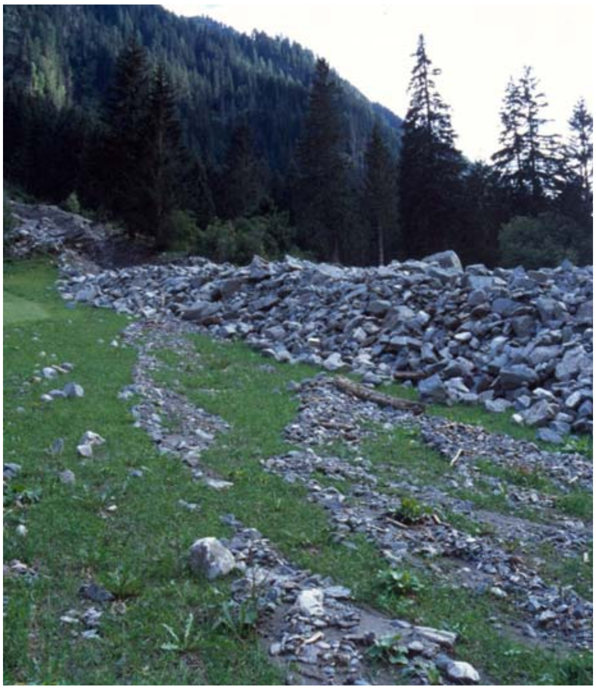


Figure 3: Example for ranular flow

$$Q = 0{,}135 \cdot D^{0{,}78} \qquad [\text{m}^3/\text{sec}] \qquad \text{granular flow} \tag{3}$$

$$Q = 0{,}0188 \cdot D^{0{,}79} \qquad [\text{m}^3/\text{sec}] \qquad \text{muddy flow} \tag{4}$$

This lets us construe the discharge trend lines as shown in Figure 2 and 3.

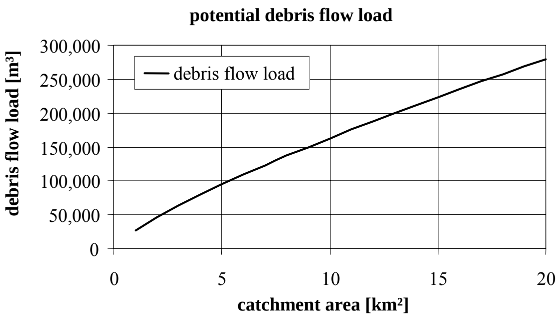


Figure 4: Debris flow loads (maximum event loads) for catchment areas of sizes 1.0 to 20.0 km², according to ZELLER and RICKENMANN

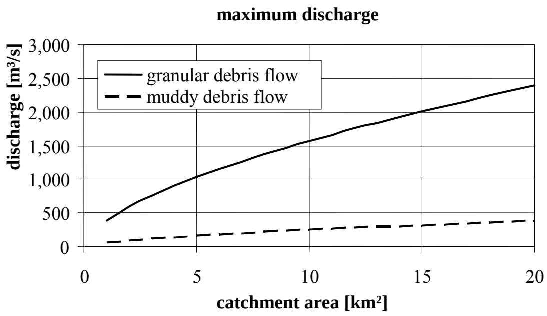


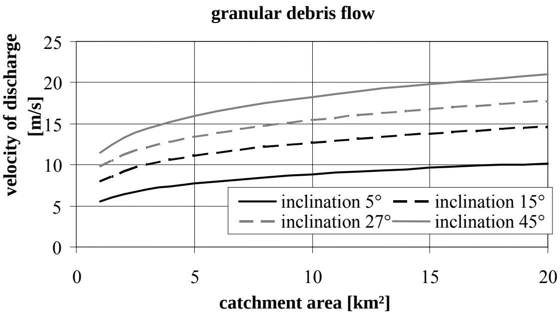
Figure 5: Maximum discharge for catchment areas between 1.0 and 20.0 km²

According to RICKENMANN [5], the velocity of flow (v), used as an input parameter for the dynamic pressure formula and depending on discharge quantity (Q) and inclination (I), can be assessed with Equation (5) for natural channels:

$$v = 2{,}1 \cdot Q^{0{,}33} \cdot I^{0{,}33} \qquad [\text{m/sec}] \tag{5}$$

This formula is valid for slope inclinations (J) between 5 and 45°. Figure 6 and Figure 7 illustrate the velocity distributions depending on catchment area and mean channel inclinations of 5, 15, 27 and 45°.

These figures show that granular debris flows display higher velocities than muddy debris flows.



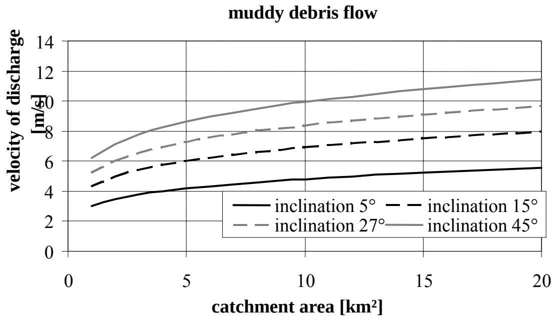


Figure 6: Velocity of granular debris flow

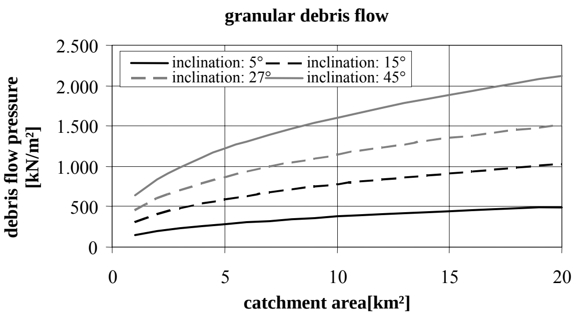
Figure 7: Velocity of muddy debris flow

Dynamic pressure (p_{dyn}), based on the parameters discussed above, can be determined with Equation (6). This calculated value, however, represents reality inadequately. Because of its particular dynamics, a debris flow's front area transports rocks up to several meters in diameter. If such a rock hits a torrent barrier, local peaks of dynamic pressure occur. Based on empirical studies of debris flow stress on permanent obstacles, flow pressure (p_{deb}) can be assumed to average between two and four times the dynamic pressure (p_{dyn}) [5]. This deviate is taken into account by a matching coefficient (α).

$$p_{dyn} = \rho_{deb} \cdot v^2 \cdot sin\beta \qquad \text{[N/m²]} \tag{6}$$

$$p_{deb} = \alpha \cdot p_{dyn} \qquad \text{[N/m²] mit } \alpha = 2 \div 4 \tag{7}$$

The specific gravity of the debris flow mixture (ρ_{deb}) in [kg/m³] can be assumed to range from 1600 to 2600 kg/m³ for Austrian debris flows. For simplicity's sake, the debris flow's impact angle (β) can be assumed to be 90°. Figure 8 and Figure 9 show debris flows' dynamic pressures depending on the size of the catchment area and the mean inclination. This analysis assumes a density of 1600 kg/m³ for granular debris flows and one of 1150 kg/m³ for muddy flows, as well as a matching coefficient α of 3.0.



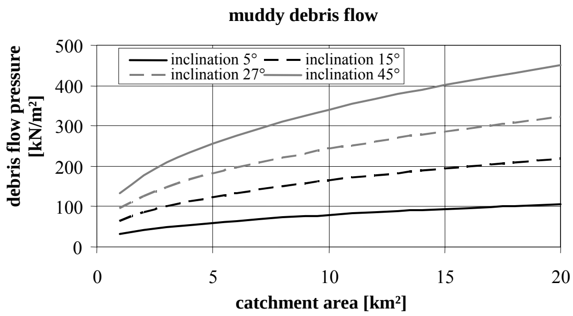


Figure 8: Resulting flow pressure for granular flow cases (ρ = 1600 kg/m³, α = 3)

Figure 9: Resulting flow pressure for muddy flow cases (ρ = 1150 kg/m³, α = 3)

As seen from the algorithms for determining the specific gravity of debris flows acting on barrier structures, there is a wide range of parameters that enter into the models. Classifications, according to catchment areas and inclinations yield, show only partly satisfactory results, since the subjective uncertainty involved in the classification process does not enter the models. These shortcomings gave reason to take a look at the problem from a probability theoretic perspective (cf. chapter 3).

2.2 RESISTANCE MODEL

In practice, barrier structures are designed for increased hydrostatic water pressure. In the following, the empirically assessed load increase factor (k_{LI}) will be designated as safety coefficient (*SC*). A major aim of the following analyses was a critically discussion of this safety coefficient. Therefore, SC has been analyzed for values ranging from 1 to 15. The linear increase of the safety coefficient on the impact side causes a linear increase of structure resistance (Equation (8)), which results from the assessment of a higher load level.

One of the most frequent constructions found in torrent barriers is the tall reinforced concrete barrier. When probabilistically analyzing the safety coefficient *SC* in detail, it is important to take into account that the assessment of reinforced concrete barriers is carried out according to a semi-probabilistic safety model. This means that the partial safety factors of reinforced concrete $\gamma_C = 1.5$ and the variable stress factor $\gamma_Q = 1.5$ enter into the calculated resistance, used in order to increase the characteristic resistance. Since the safety factors cannot be included in a probabilistic analysis, we can increase the hydrostatic load by these partial safety factors, interpreting them as an existing resistance (see the right side of Equation (8)).

$$R = SC \cdot F_{d,hydr} = SC \cdot \left(\gamma_C \cdot \gamma_Q \frac{\rho_W \cdot g \cdot h^2}{2} \right) \text{ in [N/m]} \tag{8}$$

ρ_W Density of water (assumed to be 1100 kg/m³, water + particular matter)
g Gravitational acceleration
SC Safety coefficient (SC = 1 to 11)

The following probabilistic analyses are carried out for three different barrier heights (h) of five, ten and 15 meters. Structural resistance (R) per one-meter segment is calculated according to Equation (8) for the defined wall heights as shown in Figure 11.

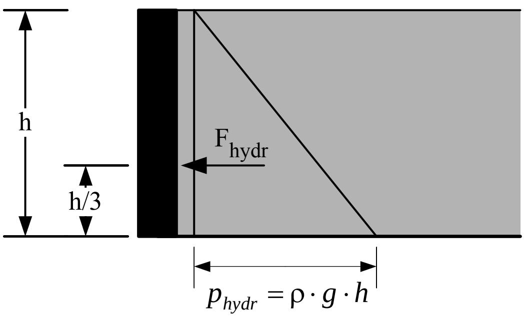


Figure 10: Definition of hydrostatic water pressure

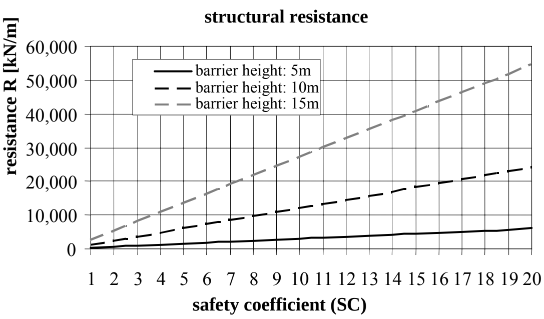


Figure 11: Increase of structural resistance depending on the safety coefficient for barriers 5, 10 and 15 meters high

2.3 LIMIT STATE FUNCTION

In order to analyze the effects of the safety coefficient *SC* on the probability of failure p_f, the calculated resulting forces of the flow pressures (F_{deb}), called stress forces (S), are compared to the resistance (R) of the barrier structure. The resistance results from the assessment of the hydrostatic water pressure, increased by *SC*. Thus, the limit state function can be arranged:

$$Z = R - S \text{ with } R = F_{d,hydr} \cdot SC \text{ and } S = F_{deb} = p_{deb} \cdot h \tag{9}$$

In this comparison, a value of Z that is less than zero indicates failure. The variables in Equation (9), in its simplest form, can be assumed to be deterministic quantities. Thus, the minimum value of *SC* is calculated from the constraint $Z = 0$. The disadvantage of with this approach is that it wrongly suggests clarity regarding stress and resistance. Chapter 3 will present an extended model that treats resistances and forces as uncertain quantities, thus coupling *SC* with probability of failure.

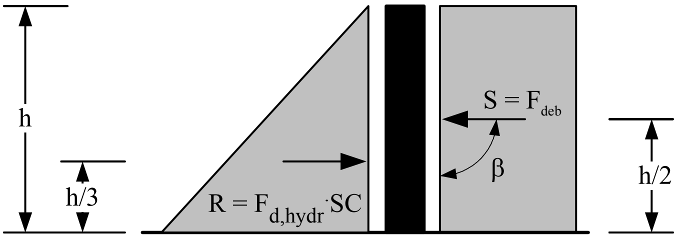


Figure 12: Comparison of resistance and stress models

For simplicity's sake, hydrostatic water pressure is conceived of as a triangular load in all models used; similarly, debris flow stress is modeled as a uniformly distributed load (Figure 12). For the moment, the offset of the result of resistance and stress will be ignored in the following analyses.

3 PROBABILISTIC ANALYSIS

In order to take into account the spread of the input variables, it is essential to base the assessment of the safety coefficient on a probabilistic analysis. These probabilistic analyses have been carried out using the FReEt software [6] [7] using the Latin hypercube sampling (LHS). The advantage of this method is that it focuses simulations on areas of failure and essentially reduces the number of simulations in comparison with other methods.

The probabilistic simulation runs as follows: definition of basic variables ("random variables") according to the models. Definition of the dependency of the basic variables by means of the correlation relationships. Randomization – design of samples for each basic variable according to the specifications of the first two steps (mean, standard deviation, correlation factor). Randomization is achieved by means of the LHS method, the correlation method is set up with the Simulated Annealing method. After that, the model function for the resistance part *R* and the stress part *S* is construed and the limit state function is arranged. This basis is used to calculate the probability of failure and the sensitivity factors.

3.1 RANDOMIZATION OF STRESS

The variables shown in Figure 13 have been introduced to the probabilistic analysis as stochastic models. The impact angle (β) is specified to be 90° degrees, as this is the most probable and, in addition to this, the most damaging case. The specific gravities for granular and muddy debris flows have been estimated according to experience. The discharge velocities (v) are a function of the flow load (D), the size of the catchment area (A_c), the discharge quantity (Q), the channel inclination (J) and the debris flow type (granular, muddy). The scattering variables in Figure 13 have been assessed for catchment areas of 5 km² and have been separately randomized based on the stress model (chapter 2.1).

The debris flow pressure (p_{deb}) was calculated in FReET according to equations (6) and (7), using the scattering values shown in Figure 13. The results based on the debris flow load are summed up in Figure 14.

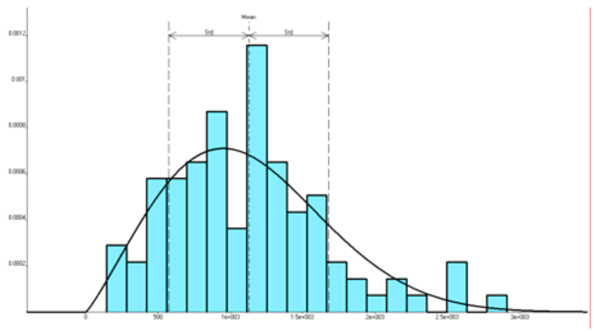
No.	basic variable (S)			unit	distribution type	m	s	cov
1	specific gravity (gran.)			kg/m³	lognormal	1362.5	335.1	-
2	α			degrees	normal	3.0	0.6	0.2
3	β			degrees	deterministic	90.0	-	-
4	velocity v	5	granular	m/s	negative half-normal	7.7	1.54	0.2
5			muddy	m/s	negative half-normal	4.2	0.84	0.2
6		15	granular	m/s	negative half-normal	11.1	2.22	0.2
7			muddy	m/s	negative half-normal	6.0	1.2	0.2
8		27	granular	m/s	negative half-normal	13.4	2.68	0.2
9			muddy	m/s	negative half-normal	7.3	1.46	0.2
10		45	granular	m/s	negative half-normal	15.9	3.18	0.2
11			muddy	m/s	negative half-normal	8.6	1.72	0.2
12	specific gravity (muddy)			kg/m³	logarithmic	1150.0	208.17	0.18
13	barrier height			M	deterministic	5.0	-	-

Figure 13: Basic variables of the stress model for the 5-meter barrier

Figure 15 and Figure 16 illustrate the distribution of the debris flow pressure for a granular and a muddy flow type. The smooth tests performed on the sampled histograms yield the Gumbel distribution for granular debris flows and the Gaussian distribution for muddy flows as best-fitting distribution types.

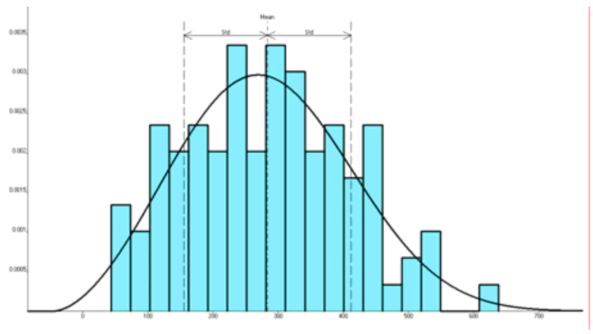
No.	mean inclination	debris flow type	m	s	cov
	[degrees]	[-]	[kN/m²]	[kN/m²]	[-]
1	5	granular	1126.00	551.34	0.49
2		muddy	283.02	127.80	0.45
3	15	granular	2339.20	1130.50	0.48
4		muddy	576.51	255.83	0.44
5	27	granular	3407.60	1653.60	0.49
6		muddy	854.87	387.47	0.45
7	45	granular	4799.80	2422.30	0.50
8		muddy	1186.10	525.96	0.44

Figure 14: Resulting forces (F_{deb}) based on debris flow pressures (p_{deb}) for varying inclinations and a barrier 5 meters high, simulated with FreET



m = 1126 kN/m, s = 551 kN/m

Figure 15: Distribution of forces resulting from a granular debris flow, for an inclination of 5°



m = 283 kN/m, s = 128 kN/m

Figure 16: Distribution of forces resulting from a muddy debris flow, for an inclination of 5°

3.2 RANDOMIZATION OF RESISTANCE

Figure 18 sums up the resistance values, depending on the hydrostatic water pressure increased by the safety coefficient (*SC*). The scattering of resistance could be approximated with a covariance of 0.05 (cov).

No.	basic variable (R)	unit	distribution type	m	s	cov
1	normal water pressure	kN/m	normal	303.19	15.16	0.05
2	double water pressure	kN/m	normal	606.38	30.32	0.05
3	triple water pressure	kN/m	normal	909.56	45.48	0.05
4	fourfold water pressure	kN/m	normal	1212.75	60.64	0.05
5	fivefold water pressure	kN/m	normal	1515.94	75.80	0.05
6	sixfold water pressure	kN/m	normal	1819.13	90.96	0.05
7	sevenfold water pressure	kN/m	normal	2122.31	106.12	0.05
8	eightfold water pressure	kN/m	normal	2425.50	121.28	0.05
9	ninefold water pressure	kN/m	normal	2728.69	136.43	0.05
10	tenfold water pressure	kN/m	normal	3031.88	151.59	0.05
11	elevenfold water pressure	kN/m	normal	3335.06	166.75	0.05
12	twelvefold water pressure	kN/m	normal	3638.25	181.91	0.05
13	thirteenfold water pressure	kN/m	normal	3941.44	197.07	0.05
14	fourteenfold water pressure	kN/m	normal	4244.63	212.23	0.05
15	fifteenfold water pressure	kN/m	normal	4547.81	227.39	0.05

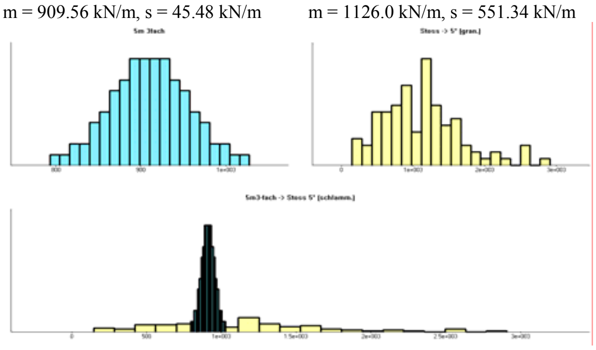
Figure 17: Basic variables of the resistance model for a barrier 5 meters high

3.3 LIMIT STATE FUNCTION

The Limit State Function for deterministic analysis has been presented in Equation (9) above. For the purpose of this analysis, the variables are treated as uncertain quantities by means of the stochastic models. Figure 18 and Figure 19 illustrate the results yielded by the limit state analysis assuming a barrier wall five meters high, designed to withstand three times the water pressure (SC = 3) under a debris flow impact at a slope inclination of 5°, both for granular and muddy debris flows. The resistance distribution (*R*) appears top left, stress (*S*) top right, and the safety range (*Z*) below. Comparing the muddy debris flow in Figure 19 with the granular one in Figure 18 shows that the probability of failure is higher for granular debris flows.

R
m = 909.56 kN/m, s = 45.48 kN/m

S
m = 1126.0 kN/m, s = 551.34 kN/m

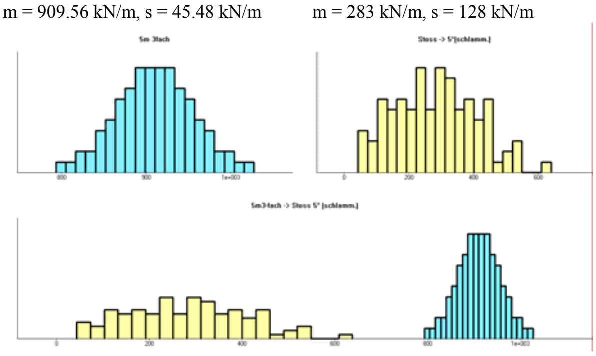


Z: m = -215.98 kN/m, s = 553.96 kN/m, Cornell-β = -3.9

Figure 18: Limit State Function for a granular debris flow: 5° inclination, 5°m wall height, triple water pressure

R
m = 909.56 kN/m, s = 45.48 kN/m

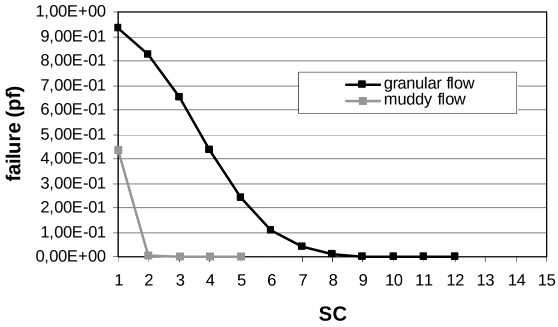
S
m = 283 kN/m, s = 128 kN/m



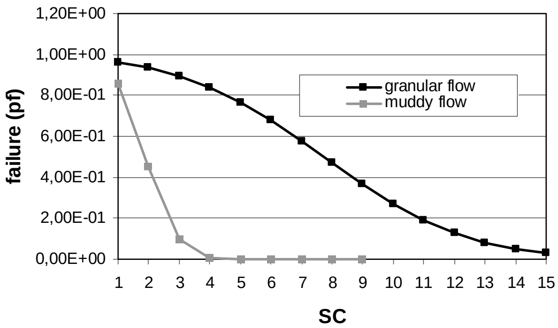
Z: m = 626.98 kN/m, s = 135.34 kN/m, Cornell-β = 4.6

Figure 19: Limit State Function for a granular debris flow: 5° inclination, 5°m wall height, triple water pressure

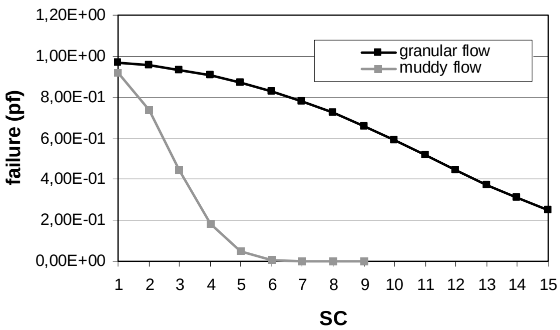
5°	granular flow		muddy flow	
SC	β	pf	β	pf
1	-1,49	9,322E-01	0,15	4,384E-01
2	-0,94	8,270E-01	2,45	7,190E-03
3	-0,39	6,517E-01	4,63	1,805E-06
4	0,16	4,379E-01	6,62	1,772E-11
5	0,70	2,421E-01	8,28	6,289E-17
6	1,24	1,069E-01	9,70	1,571E-22
7	1,77	3,856E-02	11,15	3,552E-29
8	2,29	1,103E-02	11,94	3,514E-33
9	2,81	2,490E-03	13,24	2,687E-40
10	3,33	4,341E-04	13,74	2,861E-43
11	3,87	5,536E-05	14,56	2,678E-48
12	4,29	8,890E-06	15,08	1,171E-51
13	3,36	3,955E-04	13,97	1,163E-44
14	5,26	7,229E-08	16,31	4,481E-60
15	5,79	3,490E-09	16,34	2,645E-60



15°	granular flow		muddy flow	
SC	β	pf	β	pf
1	-1,80	9,642E-01	-1,07	8,571E-01
2	-1,53	9,373E-01	0,11	4,544E-01
3	-1,26	8,967E-01	1,28	9,947E-02
4	-0,99	8,400E-01	2,43	7,522E-03
5	-0,73	7,668E-01	3,52	2,186E-04
6	-0,46	6,772E-01	4,54	2,783E-06
7	-0,19	5,759E-01	5,59	1,166E-08
8	0,08	4,694E-01	6,57	2,459E-11
9	0,34	3,659E-01	7,48	3,606E-14
10	0,61	2,712E-01	8,27	6,777E-17
11	0,88	1,908E-01	9,19	1,905E-20
12	1,14	1,280E-01	9,73	1,082E-22
13	1,40	8,125E-02	10,29	3,891E-25
14	1,65	4,938E-02	11,04	1,185E-28
15	1,92	2,767E-02	11,74	4,214E-32



27°	granular flow		muddy flow	
SC	β	pf	β	pf
1	-1,88	9,697E-01	-1,42	9,227E-01
2	-1,69	9,549E-01	-0,64	7,391E-01
3	-1,51	9,347E-01	0,14	4,439E-01
4	-1,33	9,076E-01	0,91	1,823E-01
5	-1,14	8,735E-01	1,67	4,713E-02
6	-0,96	8,313E-01	2,40	8,181E-03
7	-0,78	7,811E-01	3,15	8,261E-04
8	-0,59	7,230E-01	3,84	6,101E-05
9	-0,41	6,591E-01	4,55	2,679E-06
10	-0,23	5,892E-01	5,31	5,357E-08
11	-0,04	5,173E-01	5,85	2,486E-09
12	0,14	4,453E-01	6,47	4,836E-11
13	0,32	3,744E-01	6,93	2,134E-12
14	0,50	3,078E-01	7,57	1,879E-14
15	0,68	2,477E-01	8,23	9,571E-17



45°	granular flow		muddy flow	
SC	β	pf	β	pf
1	-1,86	9,683E-01	-1,68	9,533E-01
2	-1,73	9,583E-01	-1,10	8,647E-01
3	-1,61	9,458E-01	-0,52	6,990E-01
4	-1,48	9,303E-01	0,05	4,796E-01
5	-1,35	9,121E-01	0,62	2,678E-01
6	-1,23	8,909E-01	1,18	1,184E-01
7	-1,11	8,661E-01	1,74	4,137E-02
8	-0,98	8,364E-01	2,31	1,031E-02
9	-0,85	8,037E-01	2,83	2,352E-03
10	-0,73	7,674E-01	3,38	3,631E-04
11	-0,60	7,273E-01	3,90	4,786E-05
12	-0,48	6,840E-01	4,41	5,152E-06
13	-0,35	6,385E-01	4,97	3,434E-07
14	-0,23	5,904E-01	5,40	3,372E-08
15	-0,10	5,411E-01	5,89	1,902E-09

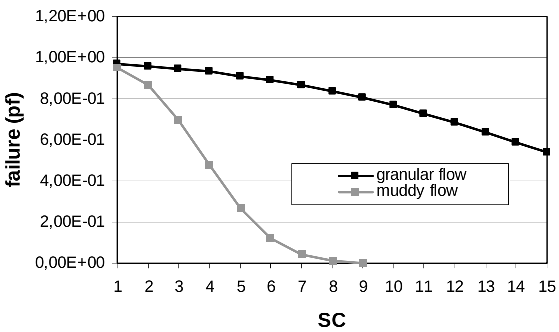


Figure 20: Simulation results for barrier five meters high and inclinations of 5, 15, 17 and 45°

The resulting reliability levels can be expressed using safety indices, "Cornell β values", a measure for probability of failure. The safety index (β) is calculated from the mean resistance (m_R), the mean stress (m_S) and the two pertinent standard deviations according to Equation (10). Figure 10 shows the results of the FReET simulations for barrier heights of 5.00 m and the inclinations discussed. It also includes a comprehensive progression of the safety index analysis and the probabilities of failure (*pf*).

$$\beta = \frac{m_Z}{s_Z} = \frac{m_R - m_S}{\sqrt{\left(s_R^2 + s_S^2\right)}} \tag{10}$$

β	5.2	4.7	4.2	3.7	3.0	2.5	2.0
pf	~ 10^{-7}	~ 10^{-6}	~ 10^{-5}	~ 10^{-4}	~ 10^{-3}	~ $5 \cdot 10^{-3}$	~ 10^{-2}

Figure 21: Correlation of safety index and probabilities of failure

3.3.1 Discussion

In structural engineering, a safety index in the range of 2.5 and 6.0 is commonly assumed, which we will also assume as the range of accepted risk in this paper. Figure 20 shows these values highlighted in gray. A β value of 2.5 corresponds to a probability of failure (*pf*) of 0.005% (pf = $5 \cdot 10^{-3}$), Figure 21. Assuming that the barriers analyzed are designed for triple water pressure, which is common practice, the model predicts that they will only withstand muddy debris flows in 5° inclined slopes. A granular debris flow leads to stress outside the safety range. A safety index of 2.5 is achieved only with designs for eight- to nine-fold water pressure. For more highly inclined slopes the β values are below the defined safety range. However, the ranges of accepted risk for torrent barriers have to be adapted to the local circumstances. The accepted risk depends on the importance of the barrier and of the object it is supposed to protect. For barriers protecting residential areas and important infrastructures, smaller probabilities of failure will be assumed than for those protecting, e.g., grass- or farmland.

3.4 VARIABLE RESISTANCE

The analyses presented so far have treated the uncertainties in the resistance model as stochastic variables constant in time. However, while stress and the uncertain variables it depends on play a dominant part in safety assessment, it is of essential importance to more closely investigate temporal change of resistance and the resulting changes in its mean and standard deviation. The stress side of traditional barriers is designed on the basis of deterministic calculation models with a pre-defined, constant safety level, as proposed in standard specifications for newly-built structures. However, the barriers' resistance changes in time. In the following, we will discuss how this temporal variability of resistance can be taken into account. This method consists of three levels, cf. Figure 22.

Levels	Factors assessed (influences)	Models applied	Statements
Level 1 – visual inspection + barrier does not display any changes (H0) → OK + barrier does display changes (H1) → measures → analysis according to level 2	**barrier surface** (cracks, spalling, corrosion, …) **bearing conditions** (integrity of foundations, **integrity of flanks**, tilted position, displacement) **rock behavior** (identification of movement) **backfill level**	visual methods (inspection, data sheets)	current state of the structure
Level 2 – Conceptual static assessment + barrier is safe (H0) → OK + barrier is not safe (H1) → measures → analyses according to level 3	Changes in the static system (changes of internal and external forces and bearing conditions) residual carrying capacity (concrete- and steel cross sections)	deterministic and semi-probabilistic models according to the present standard specifications safety level of new structure required	global statement on changes of the structure in time and on its integrity (global behavior of the structure)
Level 3 – probability-theoretic assessment	Taking into account uncertainties regarding models, model factors, subjects, and safety needs	Probabilistic models with higher accuracy than level-two models adapted safety level =f(age of structure, importance)	statement on changes of the structure in time and on probability of failure

Figure 22: Method for assessing temporal variability of resistance of torrent barriers

3.4.1 Level 1 – Visual Inspection

The assessment of the condition of barrier structures is based on visual inspections after natural events or according to a fixed schedule (e.g., every 2 years). These visual inspections are restricted to:

- an assessment of the barrier surface – cracks, spalling, corrosion, other phenomena (Figure 23 and Figure 24)
- an assessment of the bearing conditions – integrity of foundations, integrity of flanks, tilted position, displacement
- an assessment of the rock behavior – identification of tectonic movement through observing vegetation or earth movement
- an assessment of the backfill level

The assessment has to be based on the experience of experts. These inspections can yield the following results:

- The barrier does not display any damage – the null hypothesis *Ho* applies: "Barrier does not display any change in resistance"
- The barrier does display damage – the alternative hypothesis *H1* applies: "Barrier does display change in resistance"
    - shorter inspection intervals
    - immediate measures
        - damage of barrier surface – sampling, recalculation
        - bedding conditions – probing, recalculation
        - tectonic movement – …
        - backfill – …

Figure 23: Rettenbachsperre: concrete damage due to rock thrust

Figure 24: Rettenbachsperre: damage of steel reinforcement

3.4.2 Level 2 – Conceptual static assessment

The occurrence of an alternative hypothesis H1 on level 1 is the basis of conceptual static analyses. Depending on the given damage, these analyses consist of verifying the inner and outer stability. The basis for these verifications consist mostly in the principles of mechanics, pertinent standard specifications and state-of-the-art guidelines. The statements attained by these analyses are: "The property analyzed fulfills the requirements or it does not fulfill the requirements". Given temporal change and potential reserves, these statements are very unsatisfactory. Furthermore, these approaches hardly allow for taking into consideration the uncertainties involved in sampling and in assessing the process of modeling.

Permanent or discrete observation lend themselves to tracking the temporal change of resistance. Controlled monitoring of the surface processes and global structure processes would seem to be an adequate measure. The function of such a monitoring process is to register and assess chemical, physical and mechanical states of the structure's surface. In this area, the optimization of data collection- and sensor technologies for registering important physical and chemical processes and mechanical damage plays an essential role. A major concern for such a project will be the administration of data streams and collected data, the specific spatial localization of the data collected (e.g. by GPS), the assessment of the data collected and the definition of damage thresholds.

In a model it is difficult to relate the collected data of the surface processes, visible damage and mechanic with the model components. Neural network engineering has promising approaches for the description of these relations. Despite using neural network engineering, the question remains which results have to enter or should enter the modeling process for scientific applications. Aspects of damage such as change of the sectional area (decrease of static effective cross section, decrease of reinforcement section, etc.) and loss of bedding requirements enter in level 2 (cf. Figure 26).

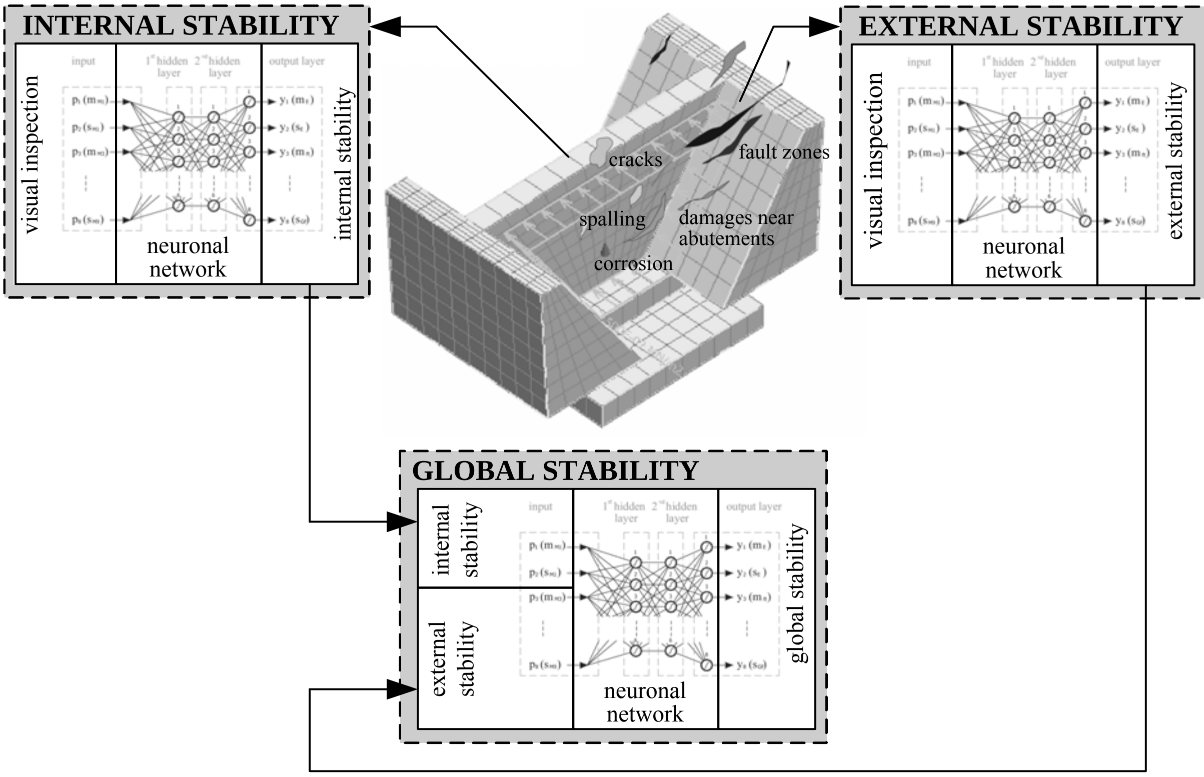


Figure 25: Correlation between visual inspected data from the surfaces of the barrier and the hill flanks with model parameters by dint of neuronal networks to get prediction to global stability of the barrier structure.

Using this method, it is possible to analyze the temporal changes of the structure in a first approximation, using the models of the standard specifications and their safety levels as a basis. As with level 1, the assessment can lead to the null hypothesis *H0* – "the structure is safe" – or to the alternative hypothesis *H1* – "the structure fails".

As mentioned above, this approach does not take into account uncertainties of sampling, of the assessment of specimens, etc. Neither does it incorporate the extended information gained from the surface inspections or the information contained in the structure itself. Finally, one might ask if the safety level of the standard specifications used is required.

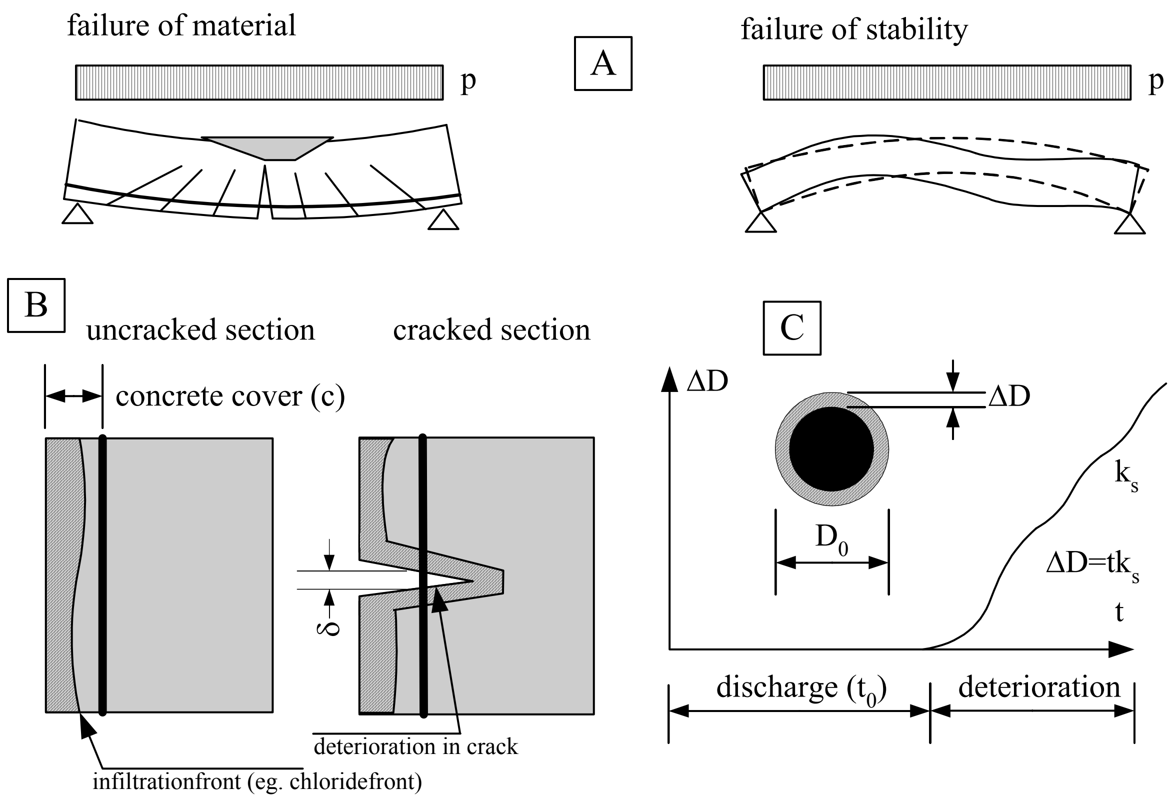


Figure 26: (A) Change in statical system due to type of failure (B) Surface processes: initiation process for corrosion, (C) deterioration model for reinforcement

The probabilistic treatment introduced in level 3 partly allows for taking into consideration the aforementioned aspects. It applies when level 2 leads to the alternative hypothesis *H1*.

3.4.3 Level 3 – Probabilistic Assessment

The probabilistic assessment makes it possible to include uncertainties regarding materials of the assessment methods and the models in the decision process. The principle is shown in Figure 27.

Another advantage of this method consists in the possibilities regarding the presentation of results. It is no longer necessary to distinguish between failure and safety as a 0–1 decision; the probability of failure is the main statement. This allows for assessments matching the local requirements, apart from the standard specifications that are designed for newly-built structures. In doing so, variable safety levels can be assumed, since the safety level for a residential area is higher than for forestry and agricultural areas.

Since the model's variables have to be treated as scattering quantities, the probabilistically based assay has higher computational costs. The probability of failure can be computed by means of Monte-Carlo simulations or more advanced methods (e.g. Chapter 2). The assessment and definition of the descriptive models and their basic variables require professional expertise. The model is a dynamic instrument taking into account:

- model uncertainty
- uncertainty of variables used
- assessor's uncertainty
- variability of stress
- need for protection by society.

The model has the following advantages:

- independence of standard specifications
- applicability to aging structures
- possibility to treat input variables (loads, resisting forces [material parameters, geometric

data], human behavior, production information and sampling data as uncertain, scattering variables

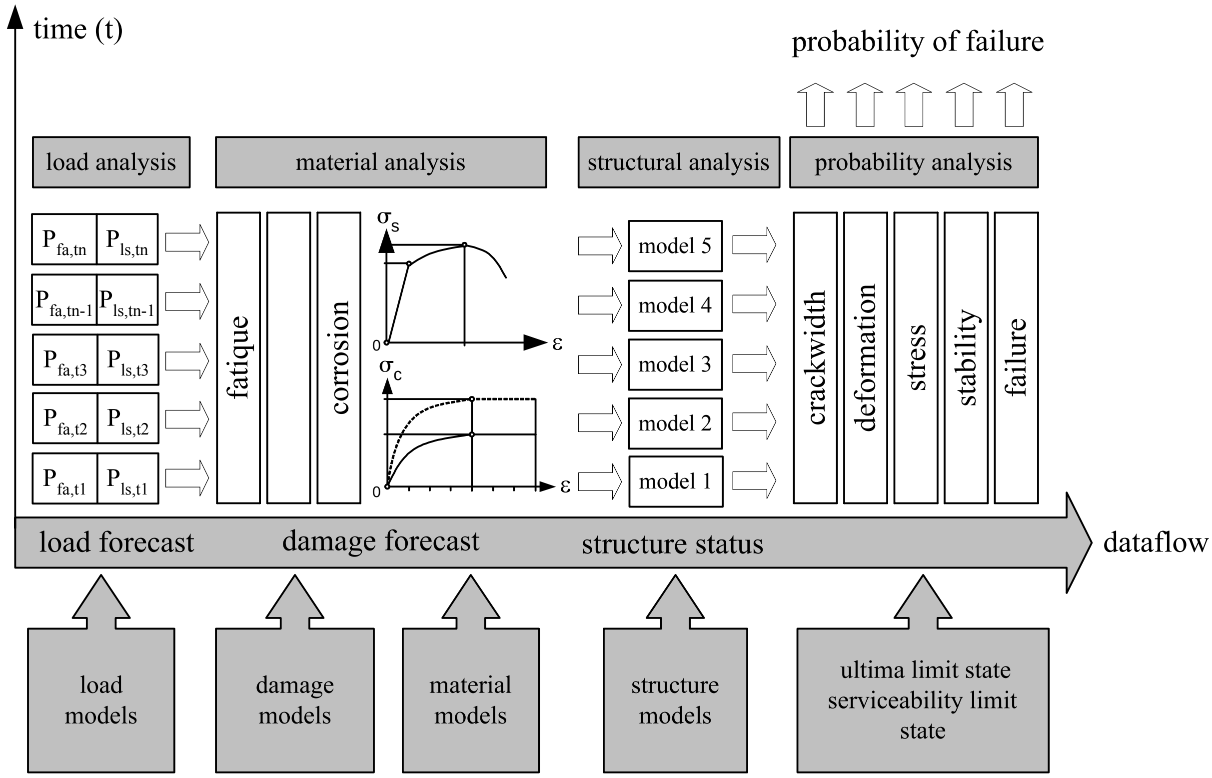


Figure 27: Probabilistic approach

3.4.4 Further advantages of the probabilistic approach

1. The results gained from controlled monitoring of surface processes mentioned in level 2 can be comprehensively included in the reliability analysis by using identification algorithms [8], making temporally variable reliability assessments possible.

2. Ongoing degradation influences the static and dynamic behavior of structures. A resonance-frequency vibration analysis makes it possible to detect structural changes. The changes in frequency characteristics can also be included in the modeling process for reliability assessments through neural network engineering methods. This allows for a temporally variable reliability assessment. This way, the inspection of surface processes can be avoided.

3. Controlled monitoring of surface processes and of the global behavior of structures for determining the probability of failure allow for a recursively built assessment catalogue of degradation processes based on stochastic models. This catalogue should serve inspectors in determining inspection intervals.

The temporally variable safety level that can be inferred from the elements described above makes it possible to work out detailed statements regarding

- life cycle planning
- planning of inspections / strengthening.

3.5 CONCLUSION

The analysis of stress models has shown that the commonly used approach of triple hydrostatic water pressure has to be critically questioned. In particular, the type of the debris flow to be expected has to be taken into consideration. The results in this paper are based on models and assumptions describimg the characteristics of the catchment areas. In order to achieve a general assessment, a larger number of real-life situations would have to be analyzed according to the method presented.

The major advantages of the probabilistic assessment concepts lie in the inclusion of sampling uncertainties, assessment uncertainties, model uncertainties and of temporally variable aspects. Therefore, the authors recommend this method for barriers in sensitive areas both for their construction from the start and for assessments according to fixed schedules.

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WIRKUNG UND LEBENSDAUER VON SCHNEENETZEN

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Abstract

This paper provides a first presentation of the “Snow net-Project in Hochfügen and Hafelekar in Tyrol”, performed by the Forest Technical Service for Avalanche and Torrent Control (WLV) in cooperation with the Federal Research and training Center for Forests, Natural Hazards and Landscape (BFW). For the snow pack stabilisation in the avalanche starting zones principally two systems are applied: snow bridges and snow nets. The construction of snow bridges is already well applied in Austria. In contrast the snow net systems are very rarely in use, because there are still many uncertainties in the construction and the foundation of nets. Therefore a test site in Hochfügen was installed to analyse the different systems of the companies Geobrugg, Trumer and EI Montage. The installation of another test site at the Hafelekar near Innsbruck is in progress. On this site are extensive measurements of the forces in the snow nets planned to obtain further information of the forces in the different systems. Based on the experiences in France and in Switzerland with the snow net measurements this project should provide additional and further data of the forces on the snow nets. Additionally tests with the anchors and the analysis of the materials in terms of corrosion protection should help to estimate the durability of the different systems, which are available on the market. In a long-term view the project is laid out to define an Austrian guideline for snow nets.

1. Einleitung

Der Forsttechnische Dienst für Wildbach und Lawinenverbauung (WLV) setzt seit gut 50 Jahren Schneestützkonstruktionen im Lawinenanbruchgebiet ein. Diese Maßnahmen haben die Aufgabe, das Anbrechen von Lawinen zu verhindern bzw. entstehende Schneebewegungen auf ein unschädliches Maß zu beschränken. Bisher wurden meist starre Konstruktionen, die so genannten Schneebrücken errichtet und wirksam eingesetzt. Dagegen die flexiblen Konstruktionen der Schneenetze wurden in Österreich bisher nur im geringen Ausmaß verwendet. In Italien oder Slowenien werden hingegen zum Großteil Schneenetze errichtet.
Die Ansichten, welche Konstruktion zu bevorzugen sei, sind sehr unterschiedlich. Derzeit spricht gegen den Einsatz von Schneenetzen die Unsicherheit im Bereich der Wirksamkeit und Lebensdauer der Netze und somit wird in Österreich verstärkt auf das bewährte System der Schneebrücken zurückgegriffen. Der Erfahrungsschatz mit Schneebrücken, die auch von der WLV zum Teil mitentwickelt wurden, kann mit mehr als 200km beziffert werden – dem gegenüber stehen einige hundert Laufmeter an Schneenetzen.

Die Beurteilung der Wirksamkeit der verschiedenen Schneestützkonstruktionen gestaltet sich jedoch schwierig. Bezüglich konstruktiver Gesichtspunkte und Materialeigenschaften kann je nach

Umständen der eine oder andere Konstruktionstyp bevorzugt werden. Die Wahl eines der beiden Konstruktionstypen Schneebrücken oder Schneenetze sollte letztendlich aufgrund objektiver Entscheidungskriterien hinsichtlich der Schutzwirkung, des technischen Standards, der Sicherheit des Systems, der Nachhaltigkeit, der Kosten und auch der Vereinbarkeit mit den anderen Zielen im Naturraum, z. B. Natur- und Landschaftsschutz, getroffen werden.

Es muss festgehalten werden, dass in der heutigen Zeit ein Paradigmenwechsel stattfindet: so verbreiteten bisher die weithin sichtbaren Lawinenverbauungen für die Bewohner ein Gefühl der Sicherheit vor Lawinen, nun aber werden auch die Aspekte des Landschaftsschutzes und der Landschaftsökologie immer wichtiger. Die Maßnahmen mit reinen Schutzfunktionen müssen mit den erweiterten Aspekten in Einklang gebracht werden. Von der Bevölkerung werden Schutzmaßnahmen gefordert, die sich bei gleicher Schutzwirksamkeit besser in die Landschaft integrieren können. Es stellt sich nun die Frage, wie kann diese geforderte Schutzwirksamkeit effizient erfüllt werden?

Großflächige Schneenetzverbauungen wie z.B. in Frankreich zeigen, dass diese aus größeren Entfernungen kaum sichtbar sind und somit die eigentliche Charakteristik der Landschaft nur wenig verändern. Die weiteren Vorteile der Konstruktion der Schneenetze sind, dass sie gegenüber Steinschlag unempfindlicher sind und ein kleineres Transportgewicht als Schneebrücken aufweisen. Nachteilig wirken sich die schwierigen Fundierungen aus, die einen Schwachpunkt darstellen können; sowie der Schneerückhalt bei Lockerschneeanrissen, der bei zu großen Maschenweiten mangelhaft ist. Während die Stahlschneebrücken nach klassischen baustatischen Methoden entworfen und berechnet werden können, liegt bei Schneenetzen eine diffizilere Situation vor: das flexible System der Schneenetze ist statisch schwieriger zu erfassen als das der starren Stahlschneebrücken. Denn infolge der Verformbarkeit der Stützfläche bilden sich im Schneenetz räumliche Kraftwirkungen verbunden mit großen Geometrieänderungen aus, die im Zuge der Dimensionierungen beachtet werden müssen.

Im Projekt „Lawinenschutzmaßnahmen Breitlehnerlawine“ bei Telfs (Inntal) in Tirol wurde die Errichtung von 8,5 Kilometern an Schneenetzen aus Gründen der guten Fundierbarkeit und des Landschaftsschutzes veranschlagt. Das Projekt, das seit Frühjahr 2005 umgesetzt wird, stellt die erste großflächige Verbauungsmaßnahme rein mit Schneenetzen in Österreich dar. Im Laufe des Genehmigungsverfahrens wurde daher auch eine wissenschaftliche Begleitung der Maßnahmen festgeschrieben, um die praktischen Erfahrungen mit den Netzen für den gesamten Dienstzweig der Wildbach- und Lawinenverbauung zur Verfügung zu stellen.

Die vielen offenen Fragen am Sektor Schneenetze in der Anbruchsverbauung haben im Fachschwerpunkt Lawinenschutz der WLV in Kooperation mit dem Bundesforschungs- und Ausbildungszentrum für Wald, Naturgefahren und Landschaft (BFW) zu einer Ausarbeitung eines umfangreichen Projektes zur Verwendung von Schneenetzen geführt. Ziel des Projektes ist es, ein Anforderungsprofil bzw. einen Standard für Schneenetze zu definieren. Die Erkenntnisse aus dem „Projekt Schneenetze“ sollen auch als Entscheidungshilfe bei der Umsetzung der Schutzmaßnahmen auf der Breitlehnerlawine und auch für weitere derartige Baufelder dienen. Es bedarf einer Abklärung, welches System verstärkt zum Einsatz kommen soll, da die verschiedenen Schneenetztypen sich sowohl hinsichtlich der verwendeten Netze (Dreiecksnetze, Omeganetze, Rechtecksnetze) und Stützenausformung, als auch in den Krafteinleitungen unterscheiden.

Zusammenfassend können vier generelle Ziele des Projektes genannt werden:

- Kraftnachweise in den verschiedenen Schneenetztypen
- Analyse der Kräfte und Spannungen anhand einfacher Methoden
- Erfassung des „State of the Art“ und einer Schadensanalyse
- Ausarbeitung einer Richtlinie

Bisher bilden die Richtlinien für den Lawinenverbau im Anbruchgebiet, welche vom Bundesamt für Umwelt, Wald und Landschaft (BUWAL) und von der Eidgenössischen Forschungsanstalt für Wald, Schnee und Landschaft (WSL) 1990 in der Schweiz herausgegeben wurden, die technische Grundlage für Schneenetze. In weiterer Folge soll eine Richtlinie für Österreich adaptiert und als Grundlage zur Typenwahl und der Dimensionierung des entsprechenden Netzsystems erarbeitet werden. Es soll damit auch eine Ausgangsbasis zur Prüfung von neu auf den Markt kommenden Produkten im Bereich der Schneenetze geschaffen werden, um den laufenden Stand der Technik gewährleisten zu können.

2. Beschreibung des Projektes – Methodik und Analyse der Grundlagen

2.1. Schneenetz-Testfelder

Die WLV hat ein erstes Testfeld von unterschiedlichen Schneenetztypen in Hochfügen im Zillertal eingerichtet, um im Bereich der Schneenetze konzentriert Informationen sammeln und die verschiedenen Systeme in situ testen zu können. Das Testfeld liegt auf 2200m Seehöhe, bei einer Neigung von 38-40° und mit Exposition nach Nord-Osten. Es können laufend Wetterdaten einer automatisierten Wetterstation in der Umgebung abgerufen werden. Im Winter erfolgt der Zugang für Aufnahmen vor Ort über die Skiliftanlage Hochfügen, im Sommer führt für Installationen und Reparaturen ein Forstweg direkt ins Testgelände. Aufgrund der Lage im Skigebiet besteht auch die Möglichkeit Schnee künstlich einzubringen, wie z.B. mit einer Pistenraupe.

Es wurden bereits vier verschiedene Typen an Schneenetzen aufgestellt:

1x Geobrugg Schneenetz Dk 3, N 1,8	Dreiecksnetz
1x Geobrugg Decco Netz (Versuchsanlage)	Rechtecksnetz
1x Trumer Dk3, N 1,8	Rechtecksnetz (Omeganetz)
1x EI Montagne Dk3, N 1,8	Dreiecksnetz

Um die Schneesicherheit (Einschneiung) zu erhöhen und eine bessere Vergleichbarkeit mit der Breitlehnerlawine zu erreichen, werden Schneenetze der Firmen Geobrugg, Trumer und EI Montagne auch am bereits seit 1957 bestehenden Testfeld Hafelekar bei Innsbruck errichtet. Dieser Bereich liegt auf 2200m Seehöhe, bei einer Neigung von 37° und mit Exposition nach Süden. Der Zugang wird im Sommer wie im Winter über die Nordkettenbahn erfolgen. In unmittelbarer Nähe der geplanten Netzreihe steht ein Schneepegel, der durchschnittliche Werte von 3-6,5m aufweist. Somit wird mit häufiger Überschneiung der Netzreihen gerechnet.

2.2. Arbeitsbereiche und -methoden

Das Projekt „Schneenetze“ kann gemäß den definierten Zielen in vier Arbeitsbereiche unterteilt werden:

I. Erfassung des „State of the Art“ und Datenerhebung

Als Grundlage für die weiteren Untersuchungen und Ausführungen soll zunächst der Stand der Technik des Stützverbaus im Anbruchgebiet (Schneebrücken und Schneenetze) erfasst werden.

Umfassende Messungen und Datenerhebungen an Schneenetzen im Gelände sollen einen Vergleich mit den Ergebnissen der theoretischen Untersuchungen – z. B. der statischen Analysen – ermöglichen. Somit können die Unterschiede der Netztypen, im Besonderen der Dreiecksnetze und der Rechtecksnetze aufgezeigt werden. Die Instrumentierung wird an den grundsätzlich verschiedenen Schneenetzen der Firma Geobrugg (Dk3, N 1,8) und am Rechtecksnetz der Firma Trumer (Dk 3, N 1,8 mit Omeganetz) umgesetzt, um die unterschiedlichen Kraftwirkungen in den Netzen messbar zu machen. Die beiden Systeme wurden bereits für den Einsatz im Gelände mit den für die Messanlagen notwendigen Aufbauten vorbereitet, Testmessungen werden noch im Winter 2005/ 2006 in Hochfügen durchgeführt. Im Sommer 2006 wird am Hafelekar das umfassende Testfeld errichtet und mit den Messeinrichtungen ausgestattet. Mit der Messtechnik bzw. Instrumentierung der Netze wurde das BFW beauftragt. Gemessen werden sämtliche Kräfte in den Seilen, aber auch die Neigung der Stützen bzw. der Netze, um auch die Kraftrichtungen besser abschätzen zu können. Die Messeinrichtungen an den Schneenetzen sollen im Herbst 2006 in Betrieb genommen werden und während der gesamten Projektdauer von 10 Jahren Daten über die Kräfte in den Schneenetzen liefern, um kontinuierliche Messungen und Zeitreihen bei verschiedensten Belastungssituationen zu erhalten.

Vorgesehene Instrumentierung der Schneenetze (Geobrugg/ Trumer):

- am Stützenfuß: über Lastmessbolzen in x-, und y- Richtung, zur Messung der Stützenkräfte
- am Stützenkopf: Messung am Tragseil: li. und re. der Klemmung
- Stützenneigungen
- Bergseitige Anker
- Tangentenneigung im Netz
- Zugkraft Topseil
- Zugmessungen in den talseitigen Abspannseilen
- Jeweils 1x Bodenabspannungen der Druckplatten
- Bergseitige Anker der seitlichen Abspannung

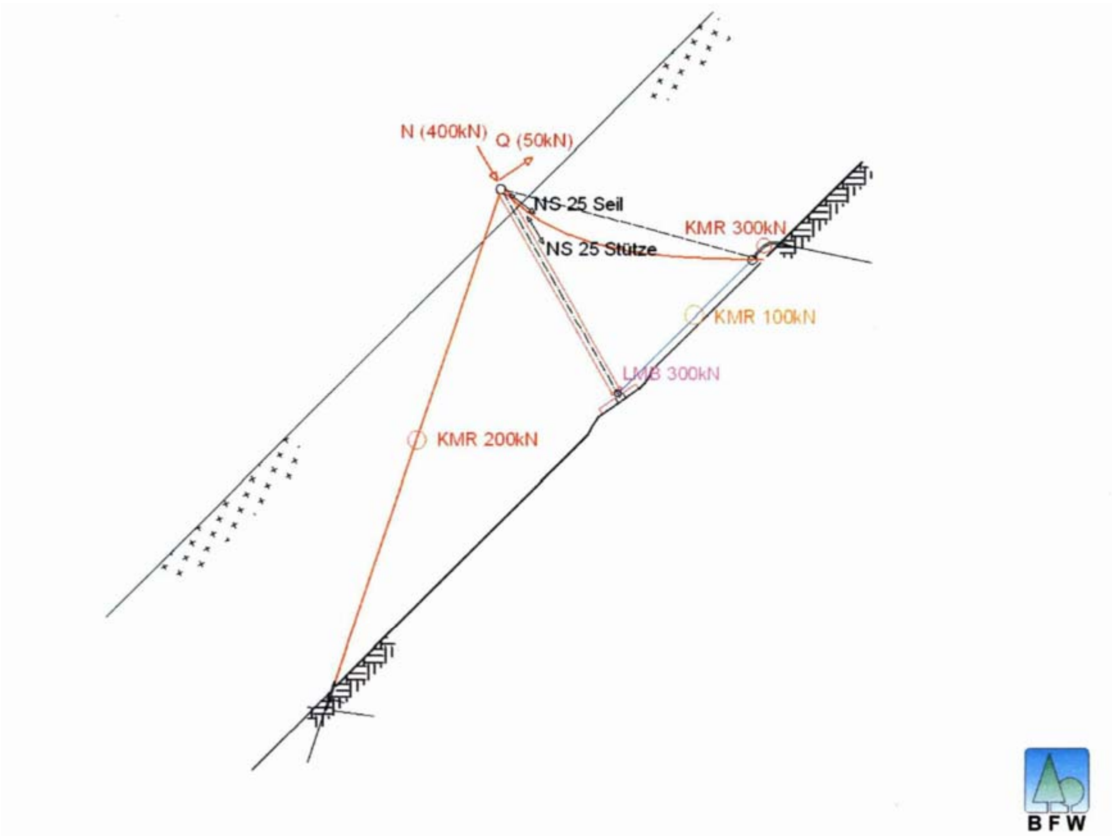

Abbildung1: Schematische Seitenansicht der Schneenetze mit Instrumentierung

Die Messdaten werden über GSM Verbindung übermittelt, um die Werte schneller einer Plausibilitätsüberprüfung unterziehen zu können. Aufgrund der leichten Zugänglichkeit über die Nordkettenbahn auf das Hafelekar können Messungen der Schneeverteilungen, der Schneedichte anhand von Profilen, etc. in regelmäßigen Abständen durchgeführt werden. In der Nähe befinden sich die automatisierten Wetterstationen der Zentralanstalt für Meteorologie, die laufend meteorologische Daten (Temperatur, Windstärke und Windrichtung, Schneehöhe, etc. liefern, welche auch über das Internet jederzeit abrufbar sind. Die Stromversorgung wird mit einem unter der Oberfläche verlegtem Kabel zur Bergstation der Nordkettenbahn sichergestellt.

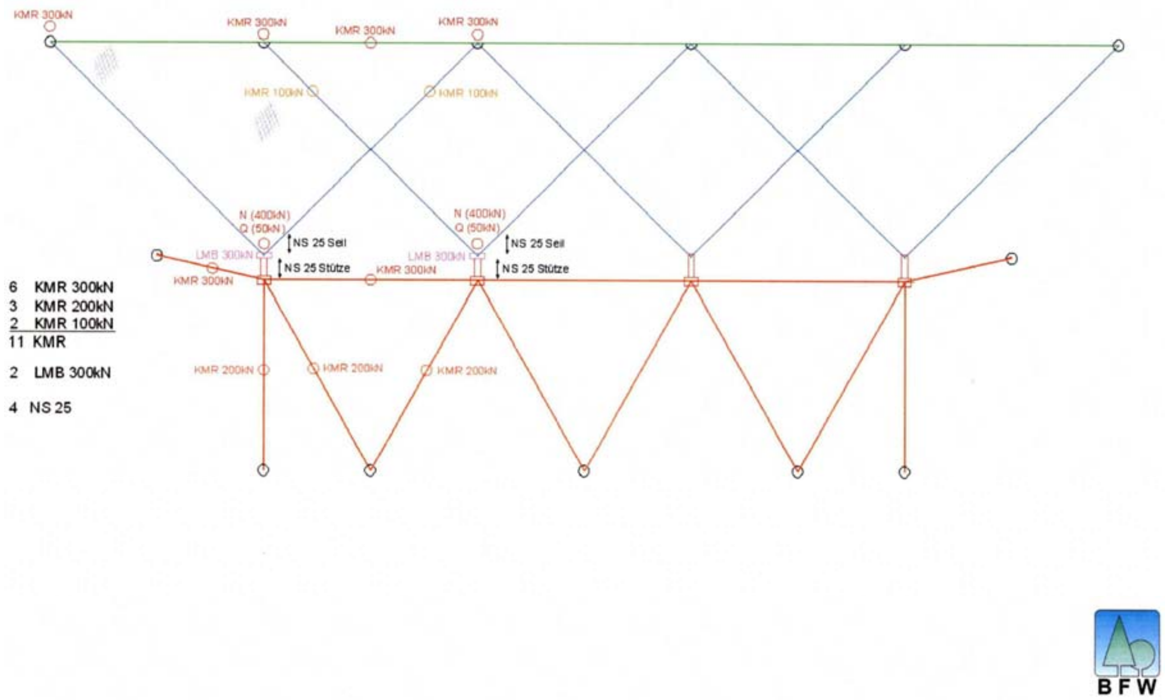

Abbildung2: Schematische Übersicht der Schneenetze mit Instrumentierung
(NS: Neigungsmessung; LMB: Lastmessbolzen; KMR: Zugmessungen (versch. Stärken))

Im Bereich der Verankerungen gibt es ebenfalls noch offene Fragen, daher wurde ein Teilprojekt „Ankerzugversuche“ am Erzberg in der Steiermark initiiert. Dabei wurden verschiedene Typen von Verankerungen von der WLV gebohrt und gesetzt und nach der Aushärtungsphase wieder gezogen, um die dafür notwendigen Kräfte messbar zu machen. Es wurden insgesamt 15 Verankerungen vorgesehen, mit 6 Zugankern und mit 9 aufgelösten Zug/Druckverankerungen, die hangparallel, bzw. 4 Versuche mit einem Ablenkwinkel von 15°, gezogen wurden. Im Detail wurde die Messreihe mit folgenden Ankern durchgeführt:

- 3x Seilanker zu je 4m Länge
- 3x Drahtbündelanker zu je 4m Länge
- 3x Gewi 25 aufgelöster Zug/Druck-Anker zu je +4m Länge bzw. -3m Länge
- 6x Gewi 32 aufgelöster Zug/Druck-Anker zu je +5m Länge bzw. -3m Länge

Das Ausgangsmaterial besteht aus Schüttgut des Erzabbaus, wobei es sich um ein extrem lockeres Material mit geringen Bindungen handelt und somit den Extremfall für Fundierungen im Lockermaterial darstellt. Der Erzberg bietet sehr gute Verhältnisse für die Versuchszwecke mit der notwendigen Infrastruktur und Zugänglichkeit, um relativ rasch eine Reihe von Messungen mit Zugversuchen durchführen zu können, die auch dass völlige Herausziehen der Anker zu Analysezwecken erlauben.

II. Statische Analysen:

Es werden statische Untersuchungen an drei verschiedenen Schneenetztypen durchgeführt:

- Dreiecksnetz der Fa. Geobrugg
- Rechtecksnetz der Fa. Trumer
- Dreiecksnetz der Fa. EI-Montagne

Im Zuge der ersten Projektphase werden einfache statische Voranalysen durchgeführt werden. Dabei soll die Seilkräfteverteilung realitätsnah berechnet werden. Untersucht wird auch die Gesamtstabilität von Schneenetzen, d. h. die Auswirkungen des Versagens einzelner Bauwerksteile auf die Gesamtstabilität. Diese statischen Voranalysen werden mittels des zweidimensionalen graphostatischen Lösungsansatzes nach Haefeli realisiert und anhand von R-Stab-Berechnungen mit dem Institut für Konstruktiven Ingenieurbau an der Universität für Bodenkultur, umgesetzt. In

Betracht gezogen werden auch FE Analysen, die jedoch erst nach vorliegen von ersten brauchbaren Messergebnissen aus den Testanlagen spezifisch definiert werden können.

III. Materialfestigkeiten:

Für die WLV und in weiterer Folge für die Bewohner der Lawinengebiete ist die Frage der Korrosions- und Langzeitbeständigkeit der verschiedenen Schneenetze von sehr hoher Bedeutung. Die ständige mechanische Beanspruchung der Netze unter Wind- und Schneebelastung besonders an den Kontaktstellen der Seile führt verstärkt zu einem Abtrag der Korrosionsschutzschichten. Ebenso führen die chemischen Belastungen in der Luft, in den Niederschlägen, etc. zu Verwitterungen des Materials. Eine Untersuchung der Materialfestigkeiten soll nun Rückschlüsse auf die langfristige Schutzwirkung, Lebensdauer und den Aufwand der Instandhaltung der unterschiedlichen Schneenetztypen ermöglichen. Für diesen Arbeitsbereich wurde das Institut für Betonbau, Baustoffe und Bauphysik der Uni in Innsbruck beauftragt. Im Labor des Institutes werden zunächst die Schneenetze auf ihre Verzinkungsstärken, auf die angewendeten Verzinkungsverfahren und auf die Art und Weise der verwendeten Verbindungsmittel, wie Klemmen, Kauschen, etc. untersucht.
Am Hafelekar bei Innsbruck stehen seit 1957 einige Prototypen von Schneenetzen der Firma Teufelberger, welche zu Testzwecken installiert wurden. Trotz der recht widrigen Verhältnisse am Hafelekar (2200m Seehöhe) erfüllen die meisten Werke erstaunlicher Weise noch immer ihre Aufgabe der Schnee-Stabilisierung. Im Zuge des Projektes werden diese Netze demontiert und ebenfalls im Labor überprüft, wobei besonders die Materialeigenschaften und die verbleibenden Zugfestigkeiten der Seile geprüft werden. Obwohl sich Materialtechnologie und Bauweise in den letzten knappen 50 Jahren wesentlich verändert und verbessert hat, erhofft man sich durch die Materialtests Rückschlüsse auf die Langlebigkeit von Schneenetzkonstruktionen.

IV. Synthese der Daten:

Aus den Ergebnissen der Arbeitsbereiche Erfassung des „State of the Art“ und Datenerhebung, Statische Analysen und Materialfestigkeiten ist es möglich Schlussfolgerungen auf die Lebenserwartung der verschiedenen Schneenetztypen, die Gesamtkosten der Errichtung und Instandhaltung, etc. zu ziehen. Aus der Gegenüberstellung und Zusammenfassung der Ergebnisse der ersten Projektphase sollen vorläufige Typenempfehlungen resultieren, um die gewonnenen Informationen bereits am Baufeld Breitlehnerlawine umzusetzen zu können und in der weiteren Projektphase als Richtlinie zu dienen.

2.3. Analyse der bestehenden Literaturgrundlage

Für das geplante „Projekt Schneenetze“ soll im Überblick (Kleemayr et al. 2005) dargestellt werden, welche wissenschaftlichen Arbeiten bereits durchgeführt worden sind und welche Messungen bzw. Analysen vorliegen. Überraschenderweise lassen sich zum Thema Schneenetze nur sehr wenige wissenschaftliche Arbeiten finden.
Dieses Kapitel gibt einen groben Überblick über die wissenschaftlichen Veröffentlichungen zu diesem Thema. Darauf aufbauend ist das „Projekt Schneenetze“ definiert, um zu erweiterten und verbesserten Erkenntnissen und in Folge zu einer klaren Richtlinie zu gelangen.

Für Schneenetze sind nur drei Arbeiten bzw. Arbeitsgruppen zu finden, die sich ursächlich mit dem Problem beschäftigt haben:

a) Haefeli (1954): stellt die erste und lange Zeit einzige Arbeit zu diesem Thema dar. Die Arbeit liefert keine konkreten Hinweise über Messungen, sondern eine stark vereinfachende aber praktikable Methode zur Berechnung der Kräfte im Bauwerk und folglich zur Dimensionierung, welche auch heute noch verwendet wird.
b) Margreth (1995): Margreth konnte als erster gemessene Kräfte vorweisen. Dabei wurde das statische Konzept von Haefeli nicht verändert. Die Interpretation der Messwerte erfolgt auf der Basis dieses statischen Konzeptes. Margreth 1995 hat an drei Systempunkten Messungen durchgeführt: bergseitiger Zuganker, Druckkräfte normal auf die Stütze und talseitiger Zuganker.

c) Nicot et al. (2000, 2004): Um von Francois Nicot und die Gruppe EI-Montagne hat sich in den letzten Jahren eine intensive Forschungstätigkeit entwickelt. Nicot et al. messen an den Dreiecksnetzen von EI-Montagne die bergseitigen Zugkräfte. Im Gegensatz zu den Schweizer Ansätzen verwendet die französische Gruppe aufwendigere und neuere mathematische Methoden und mechanische Konzepte, um die Statik der Bauwerke zu beschreiben. Die Analysen bleiben aber auf Dreiecksnetze beschränkt.

Allgemeine Bewertung der Arbeiten und Schlussfolgerungen aus österreichischer Sicht (Kleemayr et al. 2005): Vorweg muss betont werden, dass es sich hier nicht um eine detaillierte wissenschaftliche Analyse handelt, sondern um den Versuch, aus den bisherigen Erfahrungen wichtige Gesichtspunkte und Argumente für einen österreichischen Weg abzuleiten.

1) Messeinrichtung

Margreth 1995 hat bisher die umfangreichsten Messungen an drei Punkten durchgeführt. Allerdings lassen auch seine Messungen aus heutiger Sicht noch „Wünsche“ offen. Die Deformation des Bauwerks bei den unterschiedlichen Einschneiungszuständen könnte heute mit viel höherer Genauigkeit (ohne signifikante Kostenerhöhung) gemessen werden. Neigungsmessungen der Stütze und des Netzes könnten wesentlich dazu beitragen, die Veränderung der Geometrie bei den unterschiedlichen Belastungszuständen genauer und kontinuierlicher zu verfolgen. Die stetige Beobachtung der Geometrie würde die Genauigkeit der Dateninterpretation wesentlich erhöhen. Eine wesentliche Verbesserungsmöglichkeit im Vergleich zu den anderen Untersuchungen stellt auch die Messung der Scherspannung am Stützenboden dar. Theoretisch sollten bei der gegebenen Seilbelastung keine Scherkräfte auftreten, da das Seil die Stütze immer normal in Richtung Boden zieht. Sowohl Haefeli 1954, als auch Nicot 2000, 2004 gehen davon in idealistischer Weise aus. Die Realität zeigt jedoch sehr wohl Schäden im Bereich des Stützenfußes. Gründe dafür können entweder ein Versagen in der Bodenfuge, Anliegen des Netzes an der Stütze, oder einfach ein eingeschneiter Zustand sein. Die Messung der tatsächlichen Scherkräfte in diesem Bereich führen jedenfalls zu einer Erhöhung der Realitätsnähe bei Modellierung der Kräfte. Anzustreben wäre, im Gegensatz zur französischen Gruppe, ein gutes Mittelmaß aus messtechnischem Aufwand einerseits und mechanisch-theoretischem Modellierungsaufwand andererseits zu finden. Nicot 2000, 2004 misst nur die bergseitigen Zugspannungen, modelliert diese dann aber mit beträchtlichem mathematischem Aufwand.

2) Vergleich der bisherigen Messergebnisse

Der Vergleich der gemessenen bergseitigen Kräfte am Anker nach Margreth 1995: max. 72 KN und nach Nicot 2000, 2004 max.: 126 KN lässt noch einige Fragen offen. Die Interpretation, welche maximalen Kräfte auftreten können, ist nur mangelhaft mit Daten belegt und spiegelt eher die mechanischen Konzepte wieder.

3) Beschränkung ausschließlich auf Dreiecksnetze

Einer der größten Mängel der bisherigen Untersuchungen ist die ausschließliche Beschränkung auf Dreiecksnetze. Für neue Netztypen, wie dem Omeganetz der Firma Trumer, das eine bedeutend höhere Flexibilität aufweist, können die Ergebnisse nicht übertragen werden!

4) Randbedingungen – Schneemechanische Modellierung

Aus den bisherigen Messungen kann die Belastungssituation der Netze nicht allgemeingültig definiert werden (Kleemayr et al. 2005). Daher werden „Hochrechnungen“ auf mögliche maximale Belastungszustände unter Annahme eines mehr oder weniger vereinfachten statischen Konzeptes des Netzes und der schneemechanischen Modellannahmen (z.B. Schweizer Richtlinien für den Stützverbau) durchgeführt. So legt etwa Nicot 2000, 2004 der Berechnung des Schneedrucks ein Mohr-Coulomb’sches Reibungsmodell der Schneedecke zugrunde. Dies entspricht jedoch nicht dem heutigen Stand der Wissenschaft, da seit den Untersuchungen von Salm, Gubler aber auch Kleemayr, klar gezeigt werden konnte, dass für die typischen Deformationsraten der Schneedecke ein viskoses Schneedeckenmodell deutlich realitätsnähere Ergebnisse liefert. Für die Berechnung der maximalen Schneedrücke ist zu vermuten, dass das Schneegleiten eine nicht zu vernachlässigende Rolle spielt. Auch Nicot 2000, 2004 nimmt für die Spannung am Stützenfuß eine Belastung an, die überwiegend auf Schneegleiten zurückzuführen ist. Der Anteil des Schneegleitens am Schneedruck gemäß

Schweizer Formel stellt jedoch keine physikalische Größe dar, sondern hat sich in Verbindung mit den Stahlschneebrücken und deren Dimensionierung bewährt. Es scheint aber durchaus gefährlich zu sein, für die Erstellung von Dimensionierungsgrundlagen, die Methoden unkritisch zu übernehmen.

3. Ergebnisse und Diskussion

Die Installation der Messeinrichtungen ist in der Umsetzungsphase, daher können an dieser Stelle noch keine konkreten Messwerte zur Diskussion gestellt werden. Ziel dieser Arbeit ist es, das Projekt vorzustellen und die vorgesehene Methodik zu hinterfragen. Der Messzeitraum von vorerst 10 Jahren ist sehr lang - Änderungen in den Messanordnungen führen zu Verlusten an vergleichbaren Ergebnissen, daher ist es notwendig die Messanordnungen bereits im Vorfeld zu optimieren.

Die Materialtests der Uni Innsbruck zur Abschätzung der Lebensdauer sind ebenfalls noch in der Bearbeitungsphase. Hopf 2004 beziffert die Lebensdauer der Schneenetze in Österreich auf etwa 20 Jahre und verweist dabei auf die Erfahrungen mit Schneenetzen in Island. Doch die Verhältnisse sind aufgrund der Meeresnähe kaum auf Mitteleuropa übertragbar und gerade das von Hopf 1957 selbst installierte Testfeld mit verschiedenen Schneenetzen am Hafelekar lässt eine doch höhere Lebenserwartung der Netzsysteme vermuten. Die knapp 50 Jahre alten Werke erfüllen zum Großteil noch immer die vorgesehene Aufgabe des Schneerückhaltes am Hafelekar. Bedenkt man die stark fortgeschrittene Materialtechnologie und die verbesserten Fundierungsmöglichkeiten, so werden doch wesentlich längere Lebenszeiten auch für Schneenetze erwartet. Die Laboruntersuchungen der alten Netze werden besseren Aufschluss über die Dauerhaftigkeiten der Systeme geben. Versagenswahrscheinlichkeiten aufgrund des Materials können derzeit noch nicht angegeben werden.

Die Ankerzugversuche wurden am 7.November 2005 erfolgreich durchgeführt, die Ergebnisse müssen jedoch noch ausreichend analysiert werden und daher können hier nur erste Vorabinformationen wiedergegeben werden. Die Beurteilung der geologischen Verhältnisse wurde von der Geologischen Stelle in Innsbruck sichergestellt, um die Messergebnisse auch qualitativ bezogen auf das Ausgangsmaterial beurteilen zu können. Die wissenschaftliche Begleitung führte die Universität Leoben aus. Die Gutachten der beiden Stellen sind noch in der Ausarbeitungsphase, daher werden hier nur einige qualitative Beobachtungen beschrieben. Grundsätzlich kann angeführt werden, dass sich die notwendigen hangparallelen Zugkräfte im Wesentlichen zwischen 118kN und 353kN bewegt haben, um die oben angeführten Anker ziehen zu können. Die Werte sind überraschenderweise gering, jedoch handelt es sich bei dem Ausgangsmaterial um ein extrem lockeres Schuttmaterial mit unterschiedlicher Körnung und teilweise wurden auch Findlinge angebohrt, die sich auch entsprechend der Größe in den erhöhten Zugkräften auswirkten. Es konnte festgestellt werden, dass die Verwendung von Hutprofilen aus Stahl zur Verringerung des Einschnüreffekts der Anker nicht den gewünschten Effekt erzielt. Die Aufteilung der Anker in Zug und Druckglieder zeigt sich nicht widerstandsfähiger als reine Zuganker. Dabei wurde immer zuerst das Druckglied durch Ausknickung zerstört, worauf der Zuganker gezogen werden konnte. Diese bei Schneebrücken bewährte Methode bewirkt bei hangparallelen Kraftrichtungen der Schneenetze keine wesentliche Erhöhung der Widerstandsfähigkeit.
Einen großen Einfluss muss den verwendeten Mörtelstrümpfen zugeschrieben werden; hohe Zugkräfte wurden gemessen, wenn sich die entsprechenden Ausbauchungen und Vermengungen mit dem umgebenden Material einstellen konnten, sowie bei Durchbohrung von Findlingen. Dies ist weniger überraschend, jedoch muss erst die richtige Mischung aus Mörtelkonsistenz und Maschenweite der Strümpfe gefunden werden, die auch ein effizientes Einsetzen des Ankers ermöglicht. Diese ersten Zugversuche zeigen, dass die Fundierungen tatsächlich ein Problem darstellen können. Vorerst müssen die Ergebnisse noch statistisch ausgewertet werden, um auch quantitative Aussagen treffen zu können. Weitere Versuchsreihen sind notwendig, um die Frage der Fundierung systematisch abklären und um diese schlussendlich auch in eine Richtlinie betten zu können.

Die Literaturanalyse hat gezeigt, dass zwar die Kräfte im Berg- und im Talanker, sowie in den Stützen gemessen wurden, jedoch noch große Defizite in der Bestimmung der Kraftrichtungen und der umfassenden Analyse der möglichen Kräfte in Kombination mit den sich verändernden

Netzgeometrien bestehen. Rechtecksnetze wurden bisher noch nicht instrumentiert und gemessen, daher liegt auch hier großer Bedarf zur Abschätzung der Wirkung und der Kräfte in den einzelnen Seilen vor.

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Risk analysis of natural hazards

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ABSTRACT

Risk analysis is an important element of the integral risk management of natural hazards. The risk analysis is based on a hazard assessment and a comparison with elements at risk, typically evaluated in terms of human life and property values. For the hazard assessment of a given process such as a flood, debris flow or snow avalanche, the various aspects of the natural hazard process have to be defined, namely intensity (e.g. flow velocity, impact pressure), spatial extent and probability of occurrence. The potential damage of elements at risk largely depends on their vulnerability, i.e. the degree of destruction or harm resulting from a hazardous process of given intensity. Some major sources of uncertainty are pointed out, as related for example to the prediction of magnitude and frequency of natural hazard events and to the vulnerability which has to be assessed separately for each hazard type.

1. INTRODUCTION

In many countries there is an increasing interest to deal with the mountain natural hazards in a comprehensive way from a perspective of an integral risk management.

In the last decades, damages due to mountain natural disasters increased for example in Europe due to intense human activities in mountain areas (e.g. tourism, traffic, and vulnerable industries). Together with limited financial resources available for protection measures, this calls for an integral risk management of natural hazards. Both prediction and evaluation of natural hazard events and disaster management in a broad sense require knowledge from many different disciplines, and the decisions to be taken with regard to hazard mitigation measures imply discussion and communication between different stakeholders (technical experts, government representatives, politicians, the affected population).

Sustainable development of the environment with a parallel use of natural resources will be one of the major challenges in the 21st century. This points at the importance of improving the management of natural disasters on an international level, which is also reflected by several activities supported by United Nations Organisations: The International Decade of Natural Disaster Reduction (1990 – 2000) is now followed by the International Strategy for Disaster

Reduction. The expected increase of natural disasters as a result of the changing climate has contributed to a growing awareness about this topic.

The integral risk management of natural disasters typically involves a number of important tasks, as shown on Figure 1. After the occurrence of major disasters, a re-assessment may be useful or necessary, reflecting the cyclic nature of the integral risk management approach. In mountain environments of the European Alps, the natural hazard assessment mainly concerns the understanding, description and prediction of hydro-meteorological phenomena and gravitational mass movements including flooding processes. In the risk analysis the hazard assessment is combined with a vulnerability assessment of both objects and persons at risk and the environment. Risk evaluation is important with regard to the decision about mitigation concepts; risk evaluation is related to the definition of acceptable risk levels (e.g. Fell & Hartford, 1997; Jonkman et al., 2003).

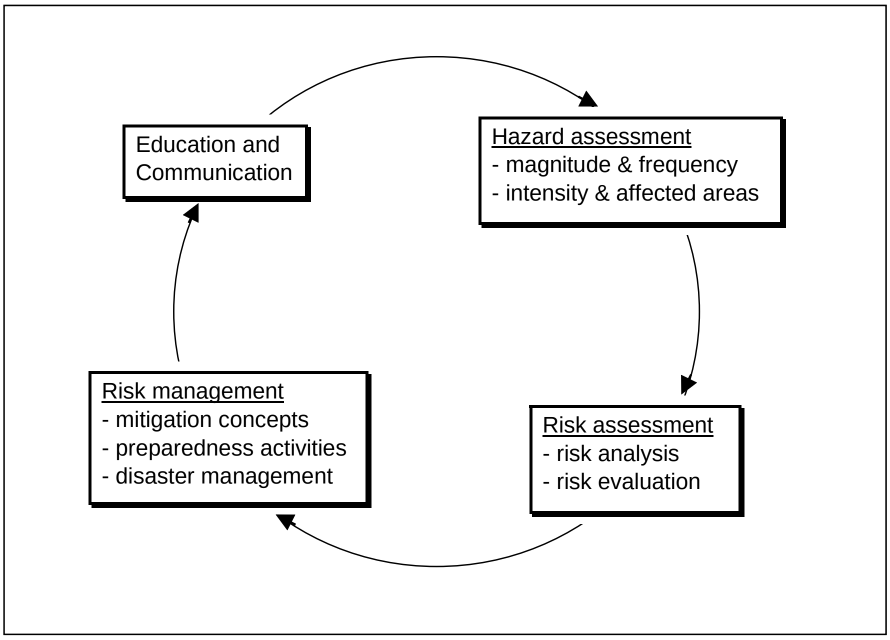

Fig. 1: Cycle of integral risk management of natural hazards.

Both hazard and risk assessment are essential elements to plan mitigation measures which aim at reducing the negative impacts of natural disasters. In the past, a number of methods have been developed to better assess hazards caused by avalanches, debris flows and flood events in torrential catchment areas (e.g. Fell & Hartford, 1997; BUWAL, 1999; Dai et al., 2002). The planning and development of mitigation concepts includes technical protection measures, warning and forecast methods, and integral watershed management (Hübl & Steinwendtner, 2000). The risk management is primarily concerned with the decision about the best mitigation strategy. This involves communication with the stakeholders and consideration of technical, ecological, political, socio-economic and cultural issues (Greminger, 2003 Ammann, 2003). Preparedness activities mainly help to decrease the vulnerability and to improve the disaster management. They are the more important the less resources are available for the implementation of technical protection measures.

The disaster management aims at reducing or limiting injuries, victims and damage during a hazardous event, and includes post-disaster recovery activities. It requires planning and training of adequate organizational structures, evacuation plans, and communication systems. Documentation of mountain disasters (Hübl et al., 2002) is another important element related to disaster management as it provides essential information and data to improve hazard and risk assessment. Education is a long term task with mainly two objectives: Formation of technical experts to deal with natural hazard management, and education of politicians, government agencies and the population to increase the awareness about natural hazards and potential consequences.

2. RISK ANALYSIS OF NATURAL HAZARDS

Risk is defined here in the sense of the IUGS Working Group on Landslides, Committee on Risk Assessment (IUGS, 1997) as "a measure of the probability and severity of an adverse effect to health, property or the environment".

Typically, the risk analysis of natural hazards includes the following steps: (i) hazard analysis, (ii) presence analysis, (iii) analysis of consequences, (iv) determination of risk (BUWAL, 1999). The hazard (i) is determined independently of human activity and then compared with the elements at risk (property, people) within the hazardous areas (ii, iii) to arrive at a risk estimation (iv). In the hazard analysis the areas potentially affected by dangerous processes have to delineated. The processes are described in terms of the probability of occurrence and the magnitude. The magnitude of an event largely determines the intensity of gravitational mass movements or flow processes. Intensity maps should be prepared for different locations in the potential impact zone; intensities of flow processes such as floods or debris flows may be described for example by the product of flow velocity and flow depth, whereas the intensity of snow avalanches may be characterized by impact pressure and in case of rock fall impact energy may be used (BWW, BRP, BUWAL 1997; BUWAL, BWW, BRP, 1997). Risk can be quantitatively expressed as:

$$R = C \; p = E \; v(I) \; p_H \; p_E \qquad (1)$$

where R = risk, C = consequences, i.e. potential loss as a result of hazard impact, p = probability of adverse event, E = element at risk (e.g. property value at risk), I = hazard intensity, v = vulnerability, expressed as relative degree of damage as function of I, p_H = recurrence probability of hazard, p_E = presence probability of element at risk. Often annual probabilities are used.

Similar definitions as given in equ. (1) have been proposed in the context of risk assessment of landslides (Fell & Hartford, 1997; Dai et al., 2002), of debris flows (Morgan et al, 1992; Romang et al., 2003), of floods (FLOODSITE, 2005) or of natural hazards in general (BUWAL, 1999; Heinimann, 2003; Bell & Glade, 2004).

The total risk for a given hazard type and area is the sum of all damaging events over a given time period. Infrequent events of large magnitude tend to produce high damage whereas frequent events of smaller magnitude are typically associated with smaller damage. It is possible that the total (collective) risk is dominated by smaller and more frequent landslide events (Fell & Hartford, 1997). A similar conclusion was made in some case studies analyzing the risk of flood and sediment transport events in small mountain catchments in Switzerland (Gächter & Bart, 2002; Bart, 2004). However, construction of protection measures such as flood dykes may eliminate or reduce damages due to small events and lead to increased human activities close to the stream channel, resulting in high damages for flood events exceeding the (elevated) threshold of the channel conveyance due to the protection dykes.

When determining the risk over an entire area the hazard probability p_H has to be further differentiated according to the spatial probability that a given site is affected by a hazard of a given magnitude. For example, flow processes in torrent channels can be strongly influenced by the presence of woody debris which may cause a blockage for example at limited flow cross-sections underneath bridges. It is often difficult to assign probabilities to such scenarios influencing the depositional behavior of a torrential flow process. Special considerations with regard to spatial occurrence probabilities of different mountain natural hazard processes are presented by Bart and Ackermann (2004).

In a comprehensive overview on landslide risk assessment, Dai et al. (2002) conclude that there are few reliable techniques available for assessing landslide hazard in terms of probability of occurrence for a given magnitude of failure. Therefore the probability and runout behavior of landslides are often assessed by historically based, largely stochastic analysis of earlier events. The accuracy of this assessment depends largely on the length, quality and nature of the historic data, and the uncertainty will increase if findings of such an empirical assessment are transferred to a site where less or no data is available. Another source of uncertainty is the assessment of vulnerability of constructions to natural hazard events for which only relatively few studies are available (see also section 3 below). This is also acknowledged by a summary report of IUGS (1997) where it is stated that for landslide risk analysis “the state of the art for assessment of vulnerability is in general relatively primitive”.

A simple method for a pragmatic risk management (BUWAL et al., 2001) is based on a common assessment of the risks with experts, local authorities and persons of the endangered population, and uses existing information on the hazard situation. In a participative dialogue hazard probabilities and expected damages are roughly quantified. The idea is to use local experience and come up with a first approximation of quantitative risk estimates also for situations where no detailed studies have yet been made. The method has been successfully tested in Switzerland, and recently a software tool has been developed which supports the application of the pragmatic risk management approach and which is available on the internet (BABS, 2005).

3. ASSESSMENT OF VULNERABILITY

An important element in the risk management of natural hazards is the assessment of the vulnerability (Glade, 2003; Alexander, 2005). In a general sense, vulnerability is related to the consequences or negative impacts of natural disasters. Vulnerability may be defined from a social-science perspective or from an engineering and natural science perspective. In the more general social-science perspective natural disasters are considered to be a result of a bad or false adaptation of human activities to nature, and vulnerability is also related the resilience of a system or the ability to respond to, cope with, recover from and adapt to natural events (Cutter et al., 2003; Alexander, 2005). Numerous definitions and examples of vulnerability are discussed for instance by Weichselgartner (2001), Glade (2003) and Alexander (2005).

From an engineering and natural science perspective, risk is typically defined as the product of hazard probability times consequences, as indicated by equ. (1) . Vulnerability then refers to the degree of damage caused by a hazardous event of given magnitude or intensity. It may be expressed as a dimensionless value between 0 and 1, where a vulnerability of 1 implies a complete destruction of a material value or the death of persons. Comparatively few studies are available which analyse types and extent of damage caused by natural hazard events such as debris flows, landslides and snow avalanches. Summaries of vulnerability values proposed for landslide risk analysis can be found in Fell & Hartford (1997) and in Glade (2003). It is generally recognized that the vulnerability depends on the process intensity, which typically varies as a function of event frequency, impact location and structural quality of the affected property. Only few attempts have been made to quantify the vulnerability of different object categories (buildings, infrastructure etc.) due to hazardous events in small mountain torrent catchments. For example, in the case of debris flow impact to buildings, proposed vulnerability values for low, medium and high intensities range from 0.005 – 0.70 (BUWAL, 1999), 0.1 – 1.0 (Michael-Leiba et al., 2003; Moon et al., 1992, in Fell & Hartford, 1997), and 0.1 – 0.5 (Bell & Glade, 2004). Apart from the semi-quantitative consideration of process intensity, the proposed vulnerability values are typically only crudely related to structural stability in the reviewed literature, at best considering different building types.

In a study by Kraus et al. (in press), an analysis is presented of damage caused by avalanches, floods and debris flows in 14 Austrian torrent catchments for rare events with estimated recurrence intervals between 100 and 150 years. As a result of the limited data availability, average damage values per natural hazard event were used. In general, the vulnerability values from the Austrian case study are lower than those proposed in Swiss studies (BUWAL, 1999; Romang et al., 2003).

For a more detailed evaluation, the damage and vulnerability values need to be more closely examined with regard to the intensity of the natural hazard event. In the Austrian study, only extreme events were considered and a simplified hazard intensity was taken into account according to the affected location (red or yellow zones). Including more detailed observations for a given event, Zanchetta et al. (2004) examined the relationship between the pressures associated with debris flows and its link to the damage caused to buildings; this study is based on disastrous debris flows in Sarno in the vicinity of Naples in Italy, where more than 100 people lost their lives.

To establish vulnerability relations for use in risk assessment, Keylock and Barbolini (2001) proposed a somewhat different approach for snow avalanches. They developed a method for deriving relative vulnerability values as a function of the avalanche position in the runout zone for a range of avalanche sizes. Information on size and frequency of avalanche events is used to determine a probability distribution of runout distances. By back-calculating the same runout distances with an avalanche dynamics model, impact pressures were determined, providing a link between avalanche size and vulnerability values.

There is a need to more accurately determine vulnerability values as a function of process type, intensity of impact and type of building structure, as is also recognized in other studies (IUGS, 1997; Glade, 2003).

4. UNCERTAINTY IN HAZARD AND RISK ANALYSIS

In the state-of-the-art report, the IUGS Working Group on Landslides, Committee on Risk Assessment (IUGS, 1997) concludes that "estimates of risk are inevitably approximate, and should not be considered as absolute values", recommends to emphasize the probabilistic characterization of hazard and quantification of risk, and welcomes discussion of limitations and error bounds of the quantitative risk analysis. The main uncertainties in risk quantification are related to the determination of:

- Event probabilities and magnitudes
- Prediction of runout behavior (or impact intensities at give locations)
- Assessing vulnerabilities (as a function of impact intensities)

In a review on risk assessment and uncertainties, Einstein and Karam (2001) point out that "describing uncertainties poses significant problems" which they illustrate with a series of examples primarily related to slope instabilities. In order to address this problem, they propose two possible approaches: a systematic structuring of the decision making process or to use stability charts and/or prioritizations. Nadim (2002) also recognizes the need to better account for uncertainties in landslide risk analysis, and he discusses a number of probabilistic tools and methods which may be used for this purpose.

A similar conclusion is drawn by Begum and van Gelder (2005) related to flood risk management: "Flood simulation is a major strategic planning tool for effective reduction of risk and damage due to flooding. However, there are uncertainties inherent in such prediction of flooding. Probabilistic analysis and uncertainty will play a major part in the decision making process of determining the flood risk." In this section reference is made to some studies which illustrate approaches to better account for uncertainties in the hazard assessment of a number of different process types.

Barbolini and Keylock (2002) present a new method for avalanche hazard mapping using a combination of statistical and deterministic modelling tools. The method may be used for avalanche sites with only few or lacking historic data on past avalanches, and can provide confidence limits on the proposed hazard zoning. A similar approach is proposed by Keylock and Barbolini (2001) to estimate vulnerabilities due to avalanches. In a further study, Barbolini et al. (2002) use a Monte Carlo procedure to consider uncertainties in avalanche release conditions. The statistical sampling-analysis method is used to evaluate the probability distributions of relevant variables for avalanche hazard assessment (e.g. runout distance and impact pressure). The release depth and release length are expressed in terms of probability distributions, accounting explicitly for inherent uncertainties in their definition.

Monte Carlo simulations are also used by Meunier and Ancey (2004) to account for uncertainties in avalanche dynamic modeling. They used a conceptual avalanche dynamics numerical models (a Coulomb-like and a Voellmy-fluid-like model), and fitted the model parameters (friction coefficients and avalanche volume) to the field data. The parameters are assumed to vary randomly and are adjusted to appropriate statistical distributions. In the case of the Voellmy fluid model, either the basal friction coefficient or the velocity-dependent friction coefficient is held constant. A large number of fictitious avalanches are simulated using the Monte Carlo approach, and the cumulative distribution function of the run-out distance are computed over a much wider range than represented by the historic data.

In a follow-up study, Ancey (2005) calibrated the friction coefficient μ of the Coulomb-like fluid flow model for snow avalanches. Since the bulk friction coefficient cannot be measured directly in the field, the friction coefficient was calibrated by adjusting the model outputs to closely match the runout data for 173 events taken from seven paths in the French Alps. A Bayesian inference techniques was used to specify the model uncertainty relative to data uncertainty and to robustly and efficiently solve the inverse problem. It was found that the overall empirical distribution function of the friction coefficient behaved as a random variable. The variations in the distribution function of μ remained larger than data uncertainty for the Coulomb model. This suggests that μ depends on other parameters. Finally, a probabilistic description of μ for a given avalanche volume was proposed.

Blazkova and Beven (2002) proposed a general methodology for flood frequency estimation based on continuous simulation. The methodology is illustrated with an application to a gauged site in the Czech Republic treated as if it was ungauged. Stochastic temperature and precipitation models are used to drive the rainfall-runoff TOPMODEL to simulate stream discharges. Using Monte Carlo simulations, the coupled model parameters are varied randomly across specified ranges. The simulations were run both for a 100 year and a 10,000 year period, and prediction limits for flood magnitudes and other response variables at different return periods were obtained. The results were then compared with a historical series of annual maximum discharges available from a gauging site for a period before it was destroyed. A concise summary on methods to consider uncertainty in rainfall-runoff modeling is presented by Uhl and Henrichs (2004).

5. CONCLUSIONS

The risk analysis of natural hazards requires a hazard assessment and a comparison with elements at risk. The potential damage is usually evaluated in terms of human life and property values. In practical hazard assessment, there are relatively large uncertainties regarding the quantification of the magnitude and frequency of natural hazard events such as floods, debris flows or snow avalanches. Another source of uncertainty is related to the estimation of intensity parameters and affected areas, which may be made with the help of simulations models; but even more sophisticated modeling approaches usually cannot account for special scenarios such as for example obstruction of a flood or debris flow by woody debris, causing overtopping at unexpected locations. Finally, only few studies have been made to assess the vulnerability of property or human life to hazard processes of different intensities. Some recent attempts to use probabilistic approaches in natural hazard assessment have been discussed. There is clearly a need to better integrate probabilistic methods into hazard and risk analysis.

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UNCERTAINTY IN FLOOD ESTIMATION

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ABSTRACT

The objective of this contribution is to make a clear picture of uncertainties we encounter in flood estimation, including both real-time flood forecasting and simulation for flood risk estimation. The reasons why, in simulation, we prefer the thesis of equifinality to looking for global optima are presented. The procedure of the GLUE methodology is illustrated on examples. The usability for practical problems is suggested and future development is outlined.

1. INTRODUCTION

The way of dealing with uncertainty in flood estimation depends largely on the purpose of the estimation, i.e. either real-time forecasting or simulation for flood risk estimation. In this contribution we will only briefly mention flood forecasting and concentrate mainly on uncertainty in simulation.

2. UNCERTAINTY IN FLOOD FORECASTING

In principle for real time forecasting any well-functioning precipitation-runoff model could be used – conceptual (e.g. Burnash, 1995) or even distributed „physically based“ (see e.g. Pappenberger et al., 2005). A variety of real-time forecasting approaches have been proposed but a Data-Based Mechanistic (DBM) model approach (Young, 2002a,b; Romanowicz et al., 2004) seems to have great advantages. It concentrates on the identification and estimation of the „dominant modes“ of dynamic behaviour represented as linear transfer functions. Nonlinearities are treated by filtering the inputs to the transfer functions and Young (2002a,b) has developed methods for non-parametric estimation of the nonlinearity using a „State Dependent Parameter“ (SDP) estimation methodology. From the point of view of dealing with uncertainty the aim is to minimise the variance of the forecast error, given real-time information about rainfalls and/or water levels during an event. An important part of the procedure consists of data assimilation using a Kalman filter-based approach (Kalman, 1960) to update the forecasts <u>and</u> the estimates of the forecast uncertainties.

DBM models can be used in real-time forecasting to predict both rainfall-flow and upstream to downstream propagation of the flood wave in the river. An interesting aspect of the DBM methodology is that it has been found that direct prediction of water levels is as successful for real-time forecasting as prediction of discharges. This has the advantage that the updating uses level directly, which is measured directly, rather than relying on a rating curve to convert level to discharge. By avoiding a source of nonlinearity in the rating curve, this tends to stabilize the heteroscedasticity that is expected in the forecast variance, particular as the flow starts to go overbank.

However it is still necessary to account for the nonlinearity in both rainfall-level and level to level predictions. Studies of the effective nonlinearity demonstrated by the SDP approach (e.g. Young and Beven, 1994; Young, 2002a,b) suggest that the nonlinearity in the rainfall-level (or rainfall-flow) case can be parameterized by a power-law relationship

$$u_k = c \, . \, y_k^{\gamma} \, . \, r_k$$

where u_k denotes the filtered "effective" rainfall, r_k denotes measured rainfall, y_k is the water level or flow as a soil moisture surrogate, c is a constant and γ is a power-law exponent.

The relationship between the flow or water level and effective rainfall can take the form of the linear transfer function

$$y_k = -a_1 y_{k-1} - a_2 y_{k-2} - \ldots - a_n y_{k-n} + b_0 u_{k-\delta} + b_1 u_{k-\delta-1} + \ldots + b_m u_{k-\delta-m} + \eta_k$$

where $a_1 \ldots a_n$, and $b_0 \ldots b_m$, are coefficients, δ is a time delay, k is a time step index and η_k is a noise input.

Experience suggests that it is possible to identify robustly a rainfall-level or flow model with more than 2 a coefficients. This can be decomposed into a fast and a slow component, representing the dominant modes of behaviour of the system. The simplest way of integrating this model with the Kalman filter data assimilation is to treat the updating in terms of a time variable gain. The aim is then to minimize the prediction variance for the required lead time.

A flood warning system using a DBM approach has been operative for Dumfries in UK since the early 1990s (Lees et al., 1994). The theory can be found e.g. in Young (2001, 2002a,b, 2003) and Romanowicz et al. (2004). For the identification of parameters the RIV (Refined Instrumental Variable) algorithm from the CAPTAIN toolbox for Matlab™ can be used.

3. UNCERTAINTY IN SIMULATION

Simulation in flood prediction is used for various design data where a design hydrograph with a certain return period is required, for the estimation of the flooded area during floods of certain return periods or for scientific purposes (together with some spatially distributed observations) aiming at finding out how the flood generation really works (e.g. Blazkova et al., 2002b).

In general, the simulation problem consists of two parts, both of which are associated with multiple sources of uncertainty. The first is to predict the discharge expected under given conditions, and particularly the extreme discharges to be expected for different return periods. The second is to predict the impact of those events, in terms of some variable of interest (such as inflow volumes to a reservoir, areas inundated on a flood plain, sediment transport, damage to property). This may require additional (uncertain) model components.

In both cases, predictions will be subject to error in the inputs to the models, the characterization of a particular system in terms of model parameters, errors in the data used to calibrate the model, and errors in the model structure itself as a representation of the dominant processes in the system (Beven, 2005a). A variety of models exist for the complex runoff generation processes on a catchment area, from the conceptual models, based mostly on the idea of a number of conceptual reservoirs to the so called „physically based" models which use differential equations in a distributed way (Beven, 2001). The effort to incorporate individual processes of runoff generation leads to the inclusion of more and more parameters which cannot be properly identified because there is not enough information in the measured data available for calibration (the models are overparameterized). The parameters of the models are found out by calibration on observed precipitation runoff data. The attempts to provide simulations without calibration with some reasonable degree of accuracy inevitably fail (see Parkin et al., 1996).

The calibration has originally been done either manually by an experienced hydrologist who used the physical interpretation of the parameters during the process or automatically by optimisation procedures. Because of the complexity of the modelled processes the response surface of the model is also complex with a number of local optima. Large efforts have been spent on the development of robust optimisation procedures able to find the global optimum (see e.g. Yapo et al., 1998, Gupta et al, 1998, Thiemann et al., 2001, Vrugt et al., 2003) but experience suggests that different parameter sets of the same model and different models can give equally adequate results. The model identification is non-unique. Precipitation runoff models and environmental models in general are mathematically ill-posed in this sense. It is impossible to deconstruct the error into its various sources.

An optimal parameter set is conditioned on a particular input series. Different errors compensate for each other, e.g. error in parameters can compensate for an input error. With another input series one would get another parameter set as the optimal one.

The way out of this problem is to give up the idea of the optimal parameter set or the optimal model and evaluate many parameter sets which produce acceptable results, the so called behavioural parameter sets. The result of the computations are then prediction bounds read from a cumulative distribution of behavioural predictions (Fig.1). For details see Blazkova and Beven (2002, 2004).

4. THE EQUIFINALITY THESIS AND EVALUATION OF MODELS

The equifinality thesis is intended to focus attention on the fact that there are many acceptable representations that cannot be easily rejected and that should be considered in assessing the uncertainty associated with predictions (Beven, 1993, 2005b). This is the basis of the Generalised Likelihood Uncertainty Estimation (GLUE) methodology of Beven and Binley (1992).

In GLUE the parameter sets are sampled randomly from physically reasonable ranges, often using uniform sampling where there is no strong information about prior expectations of parameter values. The parameter sets are then used to generate different realisations of the model outputs which are then evaluated using some criteria (measures of likelihood) to provide a weight associated with each parameter set. An important part of the process is the rejection of parameter sets that do not give acceptable results as non-behavioural. „Likelihood" here is used in a much broader sense than in statistical inference (see Beven, 2005b, for a discussion of the differences in approach). Measures of likelihood can be e.g. the determination coefficient between observed and modelled discharges, sum of absolute differences between a frequency curve based on observed data and on modelled flows, or various fuzzy measures to express similarity between observed and modelled variables. It is advantageous if some limits of effective observation error can be specified prior to running any simulations. Models predicting outside those limits can then be rejected as non-behavioural (Beven, 2005b).

An excellent illustration of the ill-posedness of environmental models and a help in modifying the parameter ranges are the dotty plots (projections of points on a likelihood surface onto a single parameter axis). In most applications only one or two parameters are sensitive (show an optimum). The insensitive parameters show behavioural and non-behavioural simulations over the whole range (Fig.2).

Sometimes more variables are evaluated than just the flow at the outlet. In evaluating hydraulic models, for example, both the outflow hydrograph and the extent of flooding can be used as measures of likelihood. For evaluating the flooded area simulations Aronica et al. (2002) used a difference between an interpolated inundation level and the observed one in the form

F=(No. of correct predict. cells – No. of incorrect predict. cells)/ Tot. no of observed flooded cells

Romanowicz and Beven (2003) and Pappenberger et al. (2005) applied fuzzy based measures in evaluating a flood inundation model to allow for measurements with high uncertainties. The result is a map which shows the distributed probabilities of inundation for a given flow event and provides an easy way to evaluate the risk of inundation in future events.

In hydrological models which provide spatial distribution of saturation, the likelihood measure is a combination of two criteria: goodness of fit of flow at the outlet and the comparison of modelled and observed saturated areas (e.g. Blazkova et al., 2002a).

5. A CASE STUDY

The application of the GLUE methodology to estimate prediction uncertainties will be briefly described using a study of the flood frequency on the Zelivka catchment (Blazkova and Beven, 2004).

This catchment (1186 km^2) has been subdivided into 7 subcatchments. In four of them the flow information was available. The computation procedure is described in Figures 3 and 4. Simulations have been carried out with the frequency version of TOPMODEL for a large catchment (the simulated storms move over the catchment) and with accumulation and melt of snow.

For each of the subcatchments with observed data a fuzzy model has been set up combining 3 criteria: goodness of fit of the flood frequency curve, of the flow duration curve and of the maximum annual snow water equivalent (schematically shown in Fig.3). The result is the likelihood of the simulation on the subcatchment in question. The likelihoods from 3 subcatchments plus likelihoods of flow frequency from a station Dolni Kralovice with a long observed series, likelihood of duration at the dam site and likelihood of snow water equivalent in interbasins are inputs into a second fuzzy system. On this basis the short (100-years simulations in hourly timestep) have been evaluated. With the behavioural parameter sets 10-thousand years series have been modeled to get better estimates of the longer term frequency statistics. A final evaluation, again in a fuzzy form, has been used – the relation of the simulated rainfall to the Probable Maximum Precipitation (PMP has been derived by the Institute of Atmospheric Physics, Rezacova et al., 2000). Prediction quantiles are then obtained by weighting the predictions from all the behavioural models with their associated final fuzzy likelihood measure. On Fig. 4 the prediction bounds of flood frequency at Dolni Kralovice together with the observed data are shown.

6. CONCLUSIONS AND FUTURE WORK

Clearly, subjective choices must be made through the GLUE methodology such as deciding which parameters of the model will be fixed and which will be varied, the parameter ranges, the likelihood criteria and their combinations, the treatment of the fuzzy area between behavioural and non-behavioural simulations, etc. All these choices, however, are made explicit so that it is possible to discuss them and to change them. For the problem of flood frequency estimation, for example, it would be beneficial to discuss with end-users the prediction quantiles of the design frequency curve: do they consider the 95% prediction bound too high?

In recent years GLUE methodology has been extended to rely more on prior evaluations of model acceptability relative to observations and less on likelihood measures based on model residuals after a model has been run (see Beven, 2005b). This focuses attention, for example, on incommensurability errors (differences between the nature of observed and predicted variables due to scale and heterogeneity effects) and the effects of input errors, both of which are often ignored. Simulations are only then considered behavioural if predictions lie in the range of the effective observational error. This development has already been used e.g. in the study of Page et al. (2003).

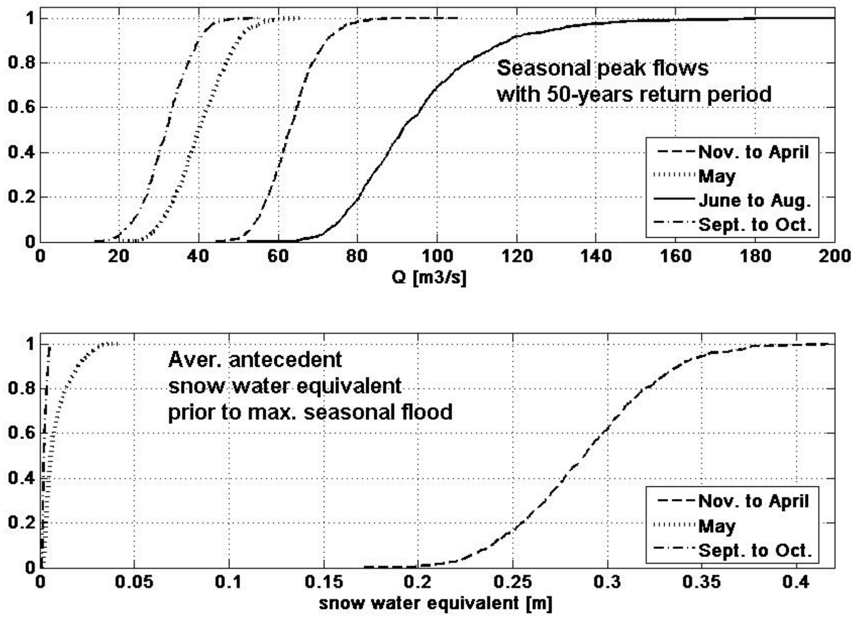

Fig 1. Predicted cumulative distributions of seasonal floods

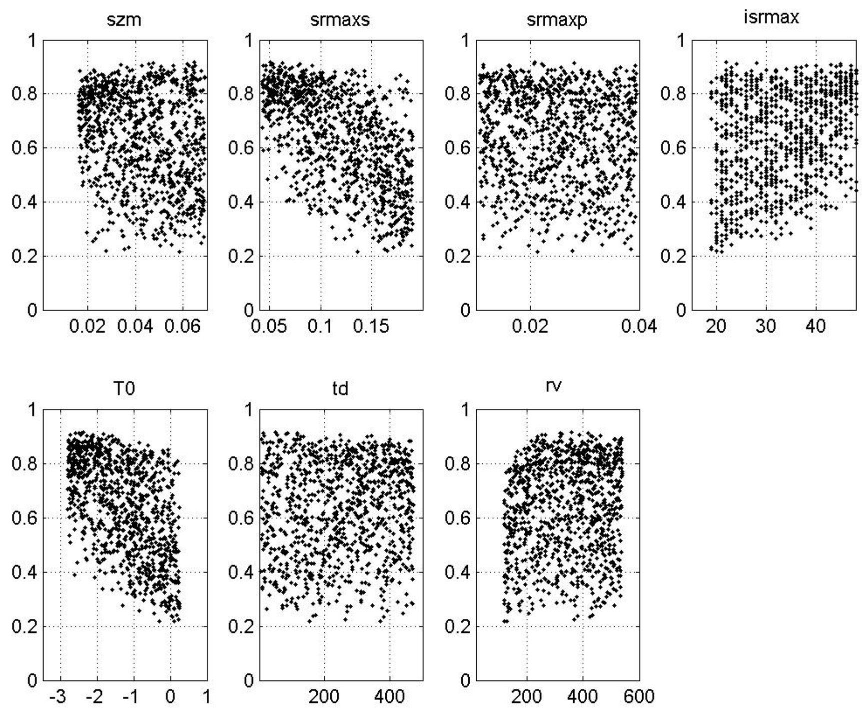

Fig. 2 Dotty plots. Likelihood measure: coefficient of determination. Horizontal axis – parameter range, vertical axis – likelihood measure. Each dot is one simulation.

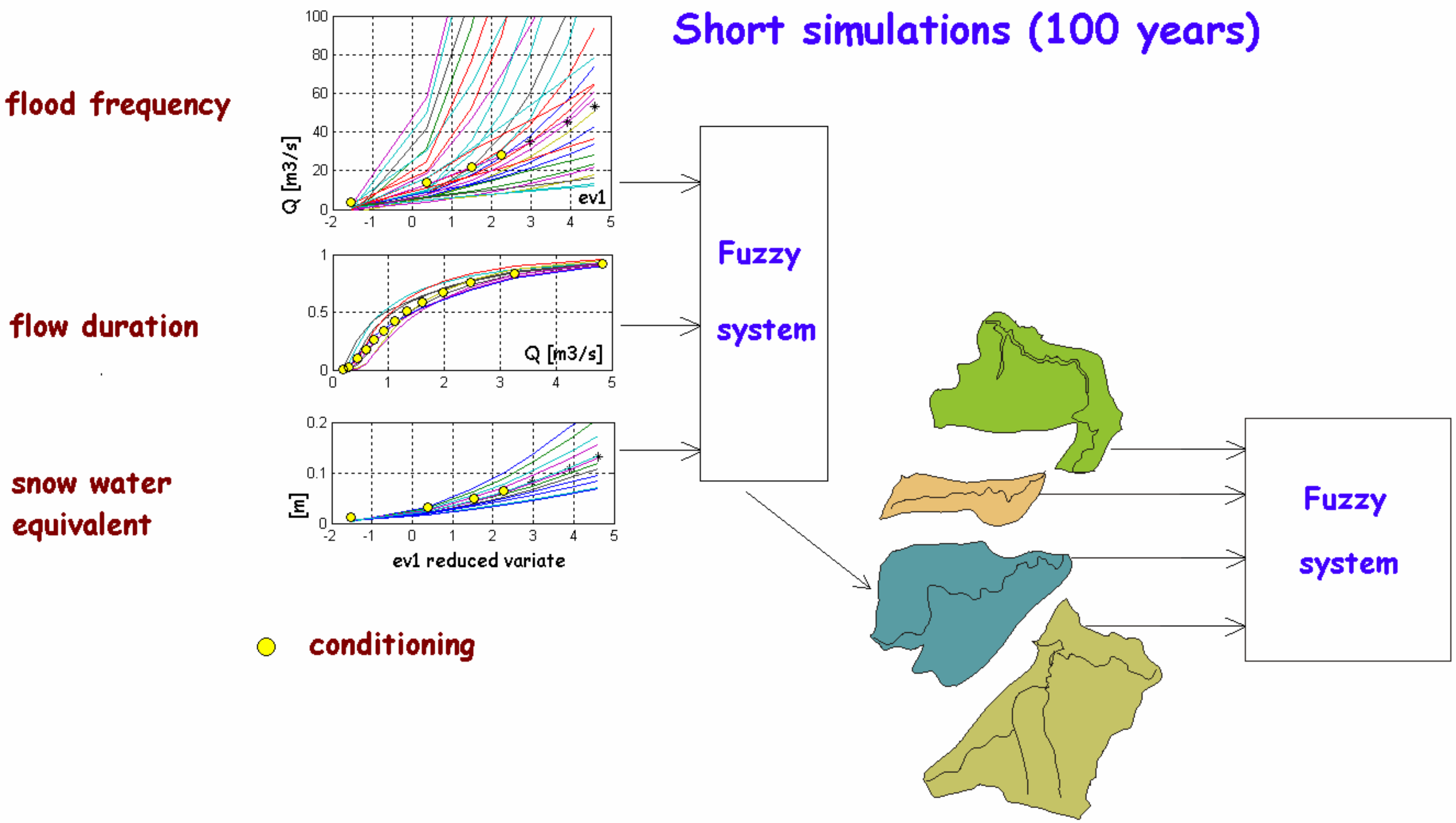

Fig. 3 Computation of likelihoods from short (100 years) simulations. Flood frequency - the exceedence curves of annual flow maxima, ev1 is Gumbel distribution, ev1=4.6 is for 100years flood, snow water equivalent – exceedence curve of annual maxima of snow water equivalent, conditioning – points on which the likelihood has been computed

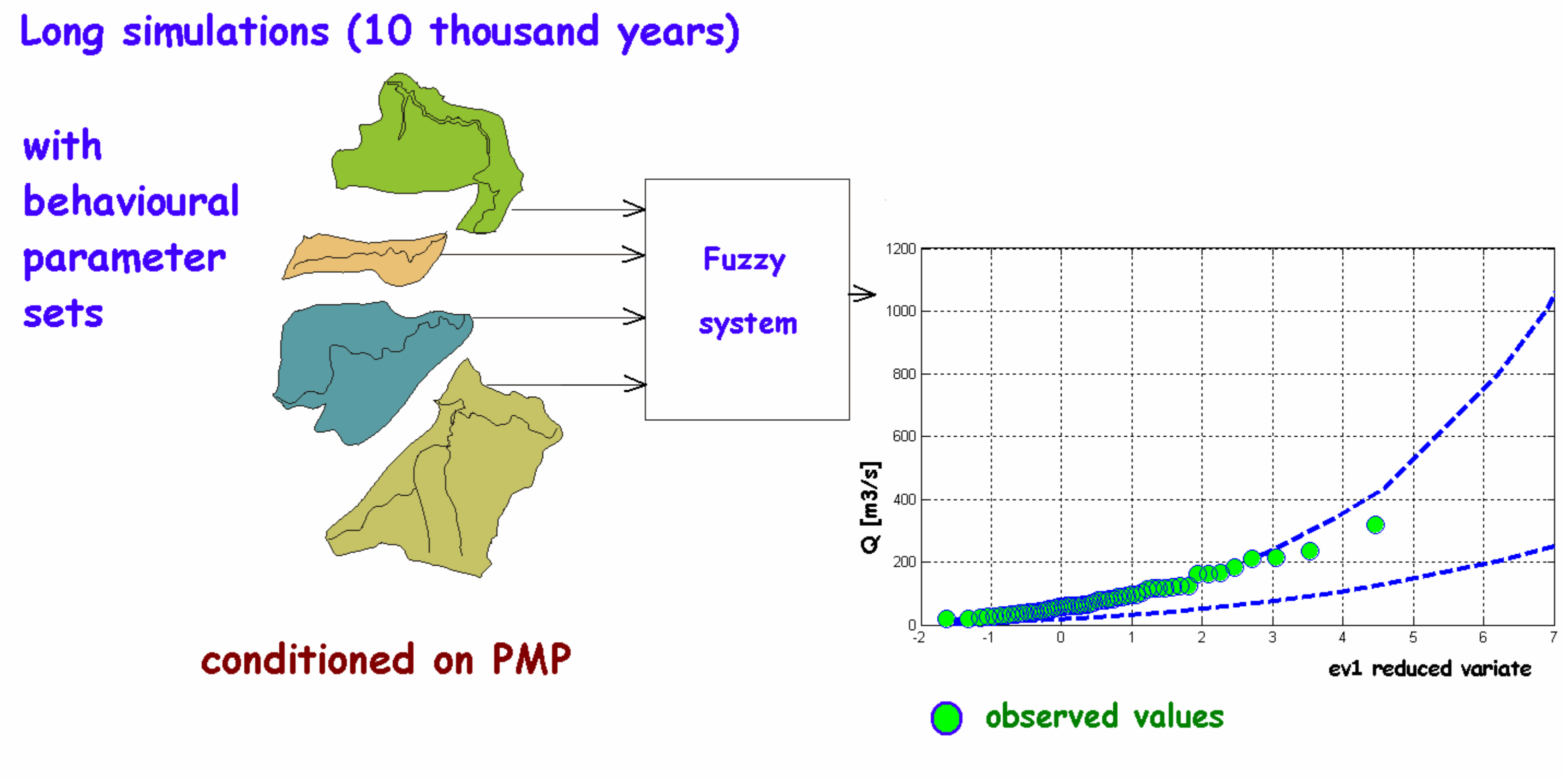

Fig. 4 Computation of likelihoods from long (10 thousand years) simulations, behavioural parameter sets – parameter sets found acceptable in 100 years simulations, PMP – probable maximum precipitation

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PROBABILISTIC LANDSLIDE HAZARD ASSESSMENT: AN EXAMPLE IN THE COLLAZZONE AREA, CENTRAL ITALY

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ABSTRACT

We present a probabilistic model to determine landslide hazard. The model predicts where landslides will occur, how frequently they will occur, and how large they will be in a given area. We apply the model in the Collazzone area in central Umbria. For this area, we prepared a multi-temporal inventory map through the interpretation of multiple sets of aerial photographs and field surveys. We partitioned the study area into 894 slope units, and obtained the probability of spatial occurrence of landslides by discriminant analysis of thematic variables. For each slope unit, adopting a Poisson probability model for the temporal occurrence of landslides, we determined the probability of having one or more landslides in different periods. We obtained the probability of landslide size by analyzing the frequency-area statistics of landslides. Assuming independence, we determined landslide hazard as the joint probability of landslide size, of landslide temporal occurrence, and of landslide spatial occurrence.

1. INTRODUCTION

Assessment of landslide hazard involves determining "where" landslides are expected, "when" or how frequently they will occur, and how large or destructive the slope failures will be, i.e. the "magnitude" of the expected landslides. Different methods have been proposed to evaluate where landslides are expected (e.g., Carrara et al. 1995, Soeters and van Westen 1996, Chung and Fabbri 1999, Guzzetti et al. 1999). To predict the location of landslides, these methods use statistical classification techniques and exploit the known relationship between past landslides in an area and a set of geo-environmental thematic variables in the same area. Attempts have been made to predict "when" landslides will occur by establishing the probability of landslide occurrence in a given period (e.g., Keaton et al. 1988, Lips and Wieczorek 1990, Crovelli 2000, Guzzetti et al. 2002b, 2005). Most commonly, the temporal probability of landslide occurrence is obtained from catalogues of historical landslide events or multi-temporal landslide inventory maps. No single measure of landslide "magnitude" exists. For some landslide types, landslide size (i.e., area or volume) is a reasonable proxy for landslide magnitude. The frequency-area statistics of landslides can be obtained from landslide inventory maps (Stark and Hovius 2001, Guzzetti et al. 2002a, Malamud et al. 2004), and this information can be used as a proxy for the distribution of landslide magnitude in an area.

2. PROBABILISTIC MODEL OF LANDSLIDE HAZARD

Guzzetti et al. (1999) defined landslide hazard H_L as "the probability of occurrence within a specified period and within a given area of a potentially damaging landslide of a given magnitude". This definition can be written as:

$$H_L = P[A_L \geq a_L \textit{ in a time interval } t\textit{, given \{ morphology, lithology, structure, land use, ...\}}] \quad (1)$$

where, A_L is the area of a landslide greater or equal than a minimum size, a_L. For any given area, proposition (1) is equivalent to:

$$H_L = P(A_L) \cdot P(N_L) \cdot P(S) \tag{2}$$

that expresses landslide hazard as the conditional probability of landslide size $P(A_L)$, of landslide occurrence in an established period $P(N_L)$, and of landslide spatial occurrence $P(S)$, given the local environmental setting. Equation (2) assumes independence of the three individual probabilities. From a geomorphological point of view, this assumption is severe and may not hold, always and everywhere (Guzzetti et al. 2005). However, given the lack of understanding of the landslide phenomena, independence is an acceptable approximation that makes the problem mathematically tractable and easier to work with.

2.1 Probability of landslide size

The probability that a landslide will have an area greater or equal than a_L is:

$$P(A_L) = P\left[A_L \geq a_L\right] \tag{3}$$

and can be estimated from the analysis of the frequency-area distribution of known landslides, obtained from landslide inventory maps. Malamud et al. (2004) proposed a truncated inverse Gamma probability distribution to approximate the probability density of landslide area. Using this distribution, the probability of landslide area $P(A_L)$ is given by:

$$P(A_L) = \int_{a_L}^{\infty} p(A_L;\rho,a,s)dA_L = \int_{a_L}^{\infty} \frac{1}{a\Gamma(\rho)}\left[\frac{a}{A_L - s}\right]^{\rho+1} exp\left[-\frac{a}{A_L - s}\right]dA_L \tag{4}$$

where: $\Gamma(\rho)$ is the gamma function of ρ, and $\rho > 0$, $a > 0$, and $s \leq A_L < \infty$ are parameters of the distribution. In equation (4), ρ controls the power-law decay for medium and large landslide areas, a primarily controls the location of the maximum of the probability distribution, and s primarily controls the exponential decay for small landslide areas.

In another study of frequency-area statistics of landslides, Stark and Hovius (2001) proposed the probability density function of landslide area to be in good agreement with a double Pareto probability distribution. Using this distribution, $P(A_L)$ is given by:

$$P(A_L)=\int_{a_L}^{\infty} p(A_L;\alpha,\beta,l,m,c)dA_L=\int_{a_L}^{\infty}\frac{\beta}{l(1-\delta)}\left[\frac{\left[1+(m/l)^{-\alpha}\right]^{\beta/\alpha}}{\left[1+(A_L/l)^{-\alpha}\right]^{1+(\beta/\alpha)}}\right](A_L/l)^{-(\alpha+1)}dA_L \qquad (5)$$

where: $\alpha>0$, $\beta>0$, $0\le c\le l\le m\le\infty$, and with $\delta=y(c)=\left[\frac{1+(m/l)^{-\alpha}}{1+(A_L/l)^{-\alpha}}\right]^{\beta/\alpha}$. Note that α in equation (5) is the same as ρ in equations (4) and controls the power-law decay of landslide probability for large landslide areas. Also, β in equation (5) controls the power-law decays for small landslide areas.

2.2 Temporal probability of landslides

Landslides can be considered as independent random point-events in time (Crovelli 2000). In this framework, the exceedance probability of occurrence of landslide events during time t is:

$$P(N_L)=P\left[N_L(t)\ge 1\right] \qquad (6)$$

where $N_L(t)$ is the number of landslides that occur during time t in a given area. Adopting a Poisson model for the temporal occurrence of landslides, the probability of experiencing one or more landslides during time t is:

$$P[N(t)\ge 1]=1-P[N(t)=0]=1-\exp(-\lambda t)=1-\exp(-t/\mu) \qquad (7)$$

where λ is the estimated average rate of occurrence of landslides which corresponds to $1/\mu$, with μ the estimated mean recurrence interval between successive failure events. The variables λ and μ can be obtained from a historical catalogue of landslide events, or from a multi-temporal landslide inventory map. The Poisson model holds under the following assumptions (Crovelli 2000): (i) the number of landslide events that occur in disjoint time intervals are independent; (ii) the probability of an event occurring in a very short time is proportional to the length of the time interval; (iii) the probability of more than one event in a short time interval is negligible; (iv) the probability distribution of the number of events is the same for all time intervals of fixed length; and (v) the mean recurrence of events will remain the same in the future as it was observed in the past. The consequences of these assumptions, which may not always hold for landslide events, should be considered when interpreting the results of the probability model.

2.3 Spatial probability of landslide occurrence

The spatial probability of landslide occurrence, also known as landslide susceptibility, is the probability that any given region will be affected by landslides, given a set of environmental conditions. Defining L: "a given region will be affected by landslides", susceptibility, S, becomes:

$$S = P\,[L \text{ is true, given } \{ \text{ morphology, lithology, structure, land use, etc. } \}] \quad (8)$$

or,

$$S = P\,[L \mid v_1(r), v_2(r), \ldots, v_m(r)] \quad (9)$$

which is the joint conditional probability that a region *r* will be affected by future landslides given the *m* environmental variables $v_1, v_2, \ldots, v_m$ in the same region.

Susceptibility can be estimated using a variety of statistical techniques, which include discriminant analysis, logistic regression analysis, and conditional analysis based on a variety of favourability functions. Depending on the type of statistical technique, the meaning of the probability changes slightly. When using discriminant analysis or logistic regression analysis, the probability assigned to any given area (i.e., to each terrain or mapping unit) is the probability that the area pertains to one of two groups, namely: (i) the group of mapping units having landslides, G_1, or (ii) the group of mapping units free of landslides, G_0, given the set of environmental conditions used in the analysis. At the beginning of a study only past landslides in a region are known. Hence, classification of mapping units free or having landslides is made based on the known distribution of past slope failures. A straightforward deduction is to assume $S= P[r \in G_1] = 1-P[r \in G_0]$. In other words, if a region *r* pertains to the group of mapping units having known landslides because of the local environmental conditions, it is likely that the same region will experience slope failures again in the future. Equally, if a region pertains to the group of mapping units free of known landslides it is unlikely that the same region will experience mass movements. Chung and Fabbri (1999) proposed to estimate the probability of future landslides in any given region, *S*, from the probability of past landslides in the same region, given a set of environmental variables. Letting *F*: "a given region has been affected by landslides", the joint conditional probability of past landslides in a region *r*, given the *m* environmental variables $v_1, v_2, \ldots, v_m$ in the same region is:

$$D = P\,[F \mid v_1(r), v_2(r), \ldots, v_m(r)] \quad (10)$$

From equations (9) and (10) it follows that:

$$P\,[L \mid v_1(r), v_2(r), \ldots, v_m(r)] = P\,[F \mid v_1(r), v_2(r), \ldots, v_m(r)], \quad (11)$$

or $S = D$.

Quantitative susceptibility models can predict the spatial occurrence of future landslides under the general assumption that in any given area slope failures will occur in the future under the same circumstances and because of the same conditions that caused them in the past. This is a geomorphological rephrase of "the past is the key to the future", a direct consequence of the principle of uniformitarianism largely accepted in the Earth Sciences. However, the principle may not hold for landslides. New, first-time failures occur under conditions of peak resistance (friction and cohesion), whereas reactivations occur under intermediate or residual conditions. It is well know that terrain gradient is an important factor for the occurrence of landslides. An obvious effect of a slope failure is to change the morphology of the terrain where the failure occurs. In addition, when a landslide moves it may change the hydrological conditions of the slope. It is also well known that landslides can change their type of movement and velocity with time. Lastly, landslide occurrence and abundance are a function of environmental conditions that vary with time at different rates. Some of the environmental variables are affected by human actions (e.g., land use, deforestation, irrigation, etc.), which are also highly changeable. Because of these complications, each landslide occurs in a distinct environmental context, which may have been different in the past and that might be different in the future. Despite these limitations, we assume that the principle of uniformity hold "statistically", i.e., that in the investigated region future landslides will occur on average under the same circumstances and because of the same conditions that triggered them in the past. We further assume that our knowledge of the distribution of past failures is reasonably accurate and complete. We accept these simplifications to make the problem tractable.

3. HAZARD ASSESSMENT FOR THE COLLAZZONE AREA

The Collazzone area extends for 78.89 km^2 in Umbria, central Italy, with elevations ranging between 145 m and 634 m (Figure 1). Landscape is hilly, and lithology and structure control the morphology of the slopes. In the area crop out: (1) recent fluvial deposits along the valley bottoms, (2) continental gravel, sand and clay, Plio-Pleistocene in age, (3) travertine deposits, Pleistocene in age, (4) layered sandstone and marl in various percentages, Miocene in age, and (5) thinly layered limestone, Lias to Oligocene in age. Soils range in thickness from a few decimetres to more than one meter, and exhibit a xenic moisture regime, typical of the Mediterranean climate. Annual precipitation averages 885 mm, and rainfall is most abundant in the period from September to December. Landslides are abundant in the area, and range in type and volume from very old and partly eroded large deep-seated slides to shallow slides and flows.

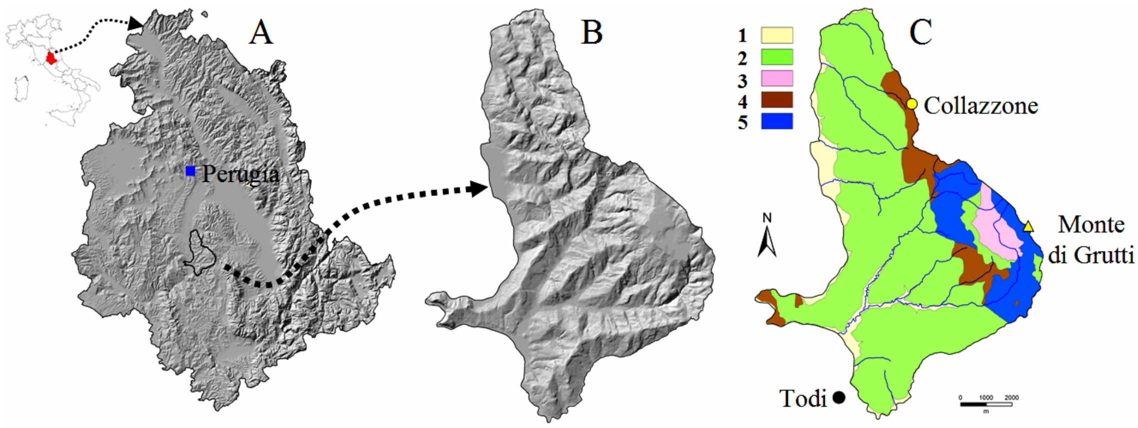

Figure 1. (A) Location of the study area in Umbria. (B) Shaded relief image showing morphology in the Collazzone area. (C) Lithological map for the Collazzone area: (1) Alluvial deposits, (2) Continental deposits, (3) Travertine, (4) Layered sandstone and marl, (5) Thinly layered limestone.

For the Collazzone area, a detailed multi-temporal landslide inventory map was obtained at 1:10,000 scale through the interpretation of multiple sets of aerial photographs and detailed geological and geomorphological field mapping (Figure 2). To prepare the landslide inventory, we used five sets of aerial photographs ranging in scale from 1:13,000 to 1:33,000, and covering unsystematically the period from 1941 to 1997. The inventory map obtained from the analysis of the aerial photographs was updated to cover the period from 1998 to 2004 through field surveys conducted following periods of prolonged rainfall. In the multi-temporal inventory, landslides are classified according to the type of movement, and the estimated age, activity and depth. Landslide type was defined according to Cruden and Varnes (1996) and the WP/WLI (1990).

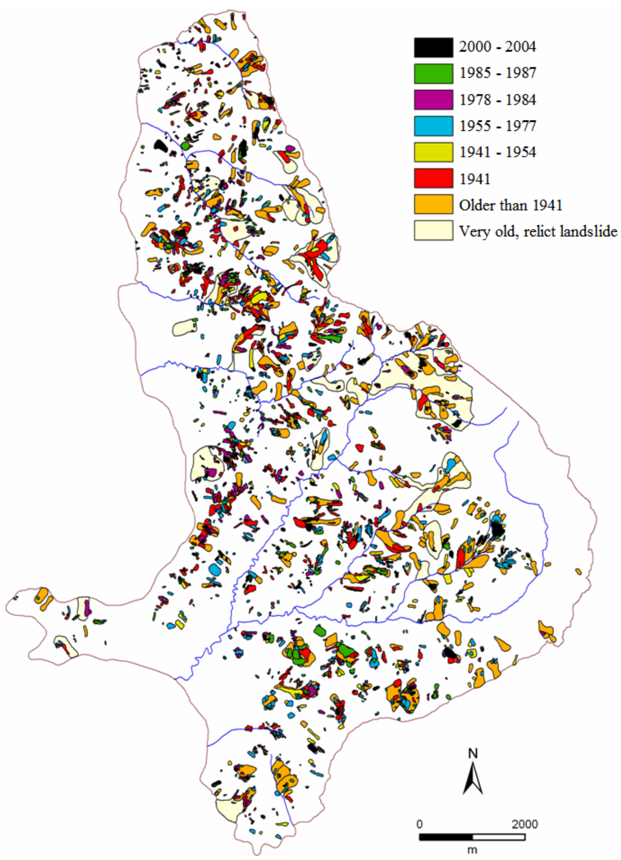

Figure 2. Multi-temporal landslide inventory map for the Collazzone area. Colors show landslides of different dates or periods, determined from the dates of the aerial photographs and the morphological appearance of the landslides.

Figure 3 summarises the data, models and products used to ascertain landslide hazard in the Collazzone area. The proposed probabilistic model requires an estimate of the probability of landslide area, a proxy for landslide magnitude. We obtained this estimate using the truncated inverse Gamma probability distribution of Malamud et al. (2004), and selecting the 2490 landslides shown in the multi-temporal inventory covering the 64-year period from 1941 to 2004. The hazard model requires an estimate of the temporal probability of slope failures. To obtain this estimate, we counted the number of landslides shown in the multi-temporal inventory in each slope unit. Considering only the recent landslides, we prepared a map of the total number of landslide events in the 64-year period between 1941 and 2004. For each slope unit, based on the past rate of landslide occurrence, we obtained the landslide recurrence, i.e., the expected time between successive failures. Knowing the mean recurrence interval of landslides in each mapping unit (from 1941 to 2004), assuming the rate of slope failures will remain the same for the future, and adopting a Poisson probability model, we computed the probability of having one or more landslides in each slope unit. The adopted hazard model requires a probabilistic estimate of the spatial occurrence of landslides. We obtained landslide susceptibility through discriminant analysis of 46 thematic variables, including morphology, lithology, structure, land use, and the presence of large relict landslide. As the dependent variable for the multivariate analysis, we selected the presence or absence of landslides in each slope unit, as shown by the multi-temporal inventory map (Figure 2).

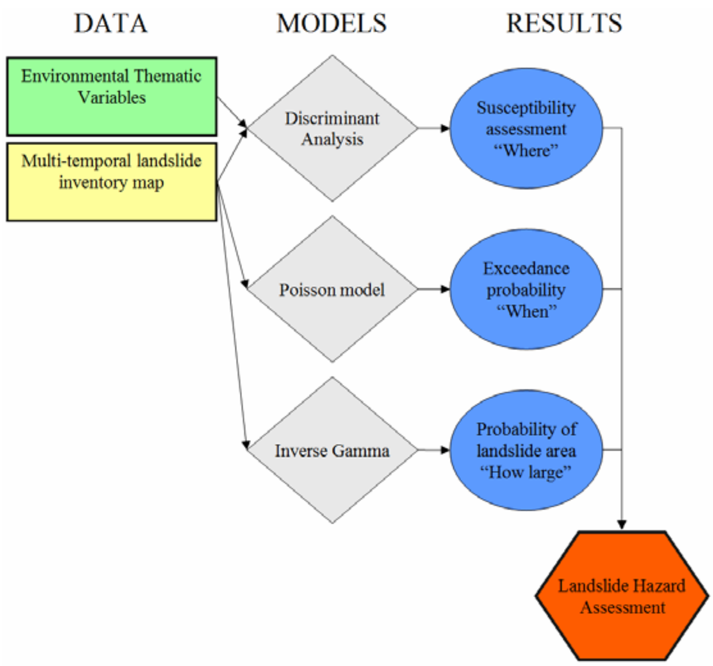

Figure 3. Block diagram exemplifying the work flow adopted to determine landslide hazard. Rectangles indicate input data. Diamonds indicate individual models, for landslide susceptibility, for the temporal probability of landslides, and for landslide size. Ellipses indicate intermediate results. Hexagon indicates the final result.

Assuming independence, we multiplied the probability of landslide size (eq. 4), the probability of landslide temporal occurrence (eq. 7), and the probability of spatial occurrence (eq. 11), and we obtained landslide hazard (eq. 2) i.e., the joint probability that a slope unit will be affected by future landslides that exceed a given size, in a given time, and because of the local environmental setting. Figure 4 shows examples of the landslide hazard assessment prepared for the Collazzone area. The Figure portrays landslide hazard for four periods (i.e., 5, 10, 25 and 50 years), and for two different landslide sizes, greater or equal than 1000 m^2, and greater or equal than 10,000 m^2.

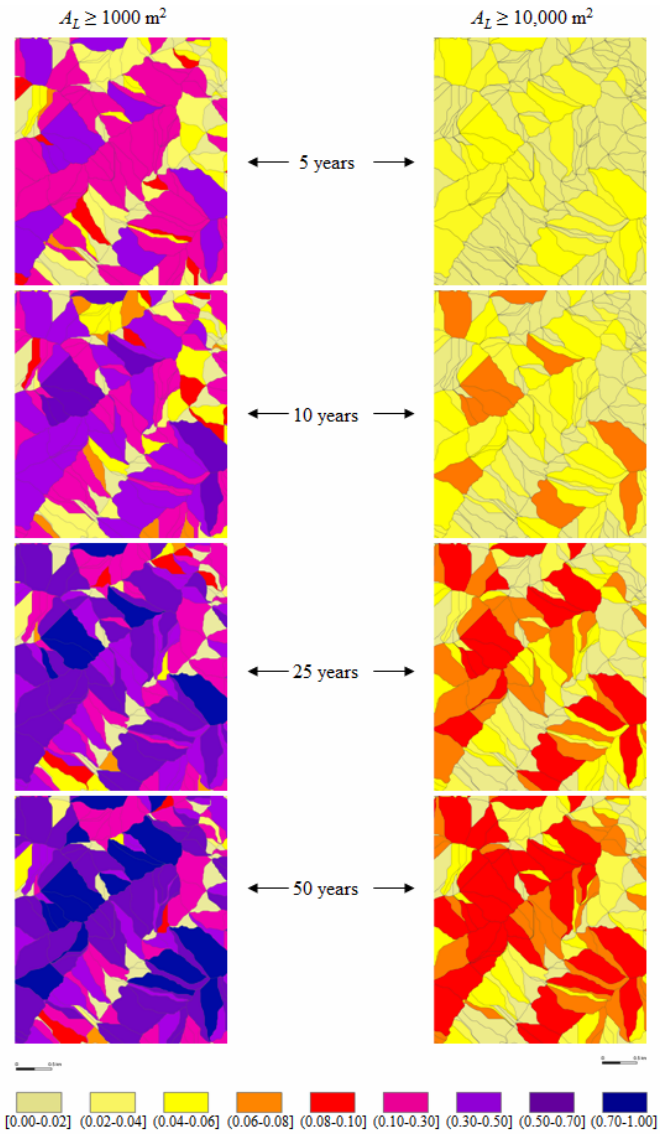

Figure 4. Examples of landslide hazard maps for four periods, from 5 to 50 years, and for two landslide sizes, $A_L \geq 1000$ m^2 and $A_L \geq 10,000$ m^2. Colors show different joint probabilities of landslide size, of landslide temporal occurrence, and of landslide spatial occurrence.

CONCLUSIONS

To ascertain landslide hazard in the Collazzone area we have adopted the probabilistic model proposed by Guzzetti et al. (2005). The adopted model expresses landslide hazard as the joint probability of landslide size, considered a proxy for landslide magnitude, of landslide occurrence in an established period, and of landslide spatial occurrence, given the local environmental setting. For the study area we have obtained most of the information used to determine landslide hazard from a detailed multi-temporal inventory map, prepared through the interpretation of five sets of aerial photographs and field surveys. The adopted model proved applicable in the test area. We judge the model appropriate in similar areas, and chiefly where a multi-temporal landslide inventory captures the types, sizes, and expected recurrence of slope failures. We conclude by pointing out that the main scope of a landslide hazard assessment is to provide probabilistic expertise on future slope failures to planners, decision makers, civil defence authorities, insurance companies, land developers, and individual landowners. The adopted method allowed us to prepare a large number of different hazard maps, depending on the adopted susceptibility model, the established period, and the minimum size of the expected landslide. How to combine such a large number of hazard scenarios efficiently, producing cartographic, digital, or thematic products useful for the large range of interested users, remains an open problem that needs further investigation.

ACKNOWLEDGMENTS

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BEURTEILUNG UND MANAGEMENT VON HOCHWASSERRISKEN

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ABSTRACT

In diesem Beitrag wird zuerst eine Darstellung der Quantifizierung von Hochwasserrisken gegeben. Dabei wird auf die Modellunsicherheit, Datenfehler und die zeitliche Variabilität der zu schätzenden Größen eingegangen. Im zweiten Teil wird das Risikomanagement behandelt, wobei der Rahmen für ein „Integriertes Management von Hochwasser Risken“ erläutert wird. Dieses beinhaltet als wesentliche Elemente die Risikoanalyse, Möglichkeiten zur Gefahrenreduktion, Möglichkeiten zur Schadensreduktion, Notfallmaßnahmen und die Beteiligung der Betroffenen. Durch das Zusammenwirken aller sektoralen Alternativen sollten die steigenden Hochwasserschäden reduziert werden können.

Berücksichtigt man, dass jedwede Schutzmaßnahme nur maximal bis zu einem Ausbauwert Schutz bietet, so verbleibt weiterhin ein Gefährdungs- und Schadenspotential, das durch Nutzungsänderungen sogar deutlich erhöht werden kann. Der Umgang mit dem ursprünglichen Risiko, dem verbleibenden Risiko, sowie den nicht monetär quantifizierten Folgewirkungen wird ebenfalls im Rahmen des Risikomanagements behandelt.

1. EINLEITUNG

Zwischen 1998 und 2005 traten in Europa mehr als 100 größere Hochwasserereignisse auf, wobei insbesondere das Hochwasser 2002 großflächig auch Österreich betraf und zum Verlust an Menschenleben und großen Sachschäden führte. Die Gesamtkosten liegen für dieses Ereignis allein in Österreich bei 3,2 Milliarden Euro.

Hydrologische Analysen zeigen bisher keine großflächige Zunahme extremer Hochwässer, wenngleich eine Zunahme infolge von bereits beobachtbaren Klimaänderungen zukünftig zu erwarten ist. Es ist jedenfalls eine deutliche Zunahme der Schadwirkungen von Hochwässern zu beobachten, obwohl jährliche Investitionen von etwa 250 Mio. Euro im Bereich des vorbeugenden Schutzes vor wasserbezogenen Naturgefahren in Österreich getätigt werden (Zenar, 2003).

Aus diesem Grunde ist ein umfassendes Konzept zu entwickeln, das aufbauend auf den bisherigen Erfahrungen und Strategien die Konsequenzen von Naturereignissen wie Hochwässern reduziert. Diese Konzept wird unter dem Begriff „Integriertes Risikomanagement“ zusammen gefasst. Dies beinhaltet alle Maßnahmen zur Entwicklung eines wirtschaftlich gerechtfertigten Hochwasserschutzes, der von einer flussgebietsbezogenen Gesamtschau ausgeht. Es sind dabei sowohl die ökonomischen und sozialen Bedürfnissen der Bevölkerung als auch der Erhalt und die Verbesserung der ökologischen Funktionen des Gewässerraumes zu berücksichtigen. Eine erfolgreiche Umsetzung des Konzeptes setzt eine

- enge Kooperation zwischen Planern, Verwaltung und der Bevölkerung,
- eine Abstimmung unterschiedlicher Planungsbereiche wie Infrastrukturentwicklung, Raumordnung und Flächenwidmung,
- eine Abstimmung von Lenkungsmaßnahmen, die sich auf die Schadenskompensation bei Naturereignissen, Förderungsmaßnahmen in der Schutzwasserwirtschaft, und auf die private Vorsorge beziehen,

voraus.

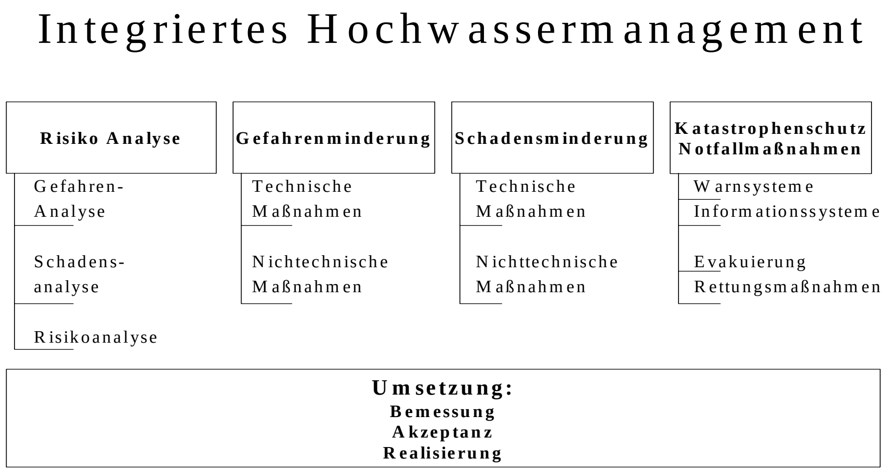

Abb. 1 Elemente eines „Integrierten Hochwassermanagements“

Nachfolgend wird auf den ersten Teil, die Risiko-Analyse, eingegangen.

2. RISIKO ANALYSE

Hochwasserschutzbauwerke können nach verschiedenen Kriterien bemessen werden. In einigen Ländern, auch in Österreich, werden normative Zielsetzungen verfolgt und Gebiete ihrer Nutzung entsprechend geschützt. Gemäß den derzeit gültigen RIWA-T Richtlinien (BMfLFUW, 1994; DVWK, 1987)

- sind Hohe Lebens-, Kultur- und Wirtschaftswerte nach Möglichkeit vor jedem Hochwasserereignis zu schützen (RHHQ);
- Für Siedlungen und bedeutende Wirtschafts- und Verkehrsanlagen ist im allgemeinen die Gewährleistung eines Schutzes bis zu Hochwasserereignissen mit 100-jährlicher Häufigkeit anzustreben (HQ_{100}).

- In begründeten Fällen (Einzelanwesen, einzelne Wirtschaftsanlagen) ist eine Abminderung auf Ereignisse 30-jährlicher Häufigkeit (HQ_{30}) zulässig.
- Eine Unterschreitung des HQ_{30} ist nur dann vertretbar, wenn an das Gewässer anschließend keine roten Gefahrenzonen verbleiben.
- Sonstige örtliche Anlagen von geringerer Bedeutung sind im allgemeinen vor Ereignissen bis zu 30-jährlicher Häufigkeit (HQ_{30}) zu schützen.
- Land- und forstwirtschaftlich genutzte Flächen sind nicht gesondert zu schützen.

Der Index gibt die Jährlichkeit des Bemessungsereignisses, sein Reziprokwert die jährliche Auftretenswahrscheinlichkeit an. In der praktischen Anwendung wird zum Bemessungsereignis noch ein Sicherheitszuschlag hinzugezählt, der die hydraulischen Unsicherheiten berücksichtigt (Klenkhart und Nachtnebel, 2005).

Das Risiko eines Hochwasserereignisses wird in obigen Überlegungen nur indirekt angesprochen, als der Nachweis der Wirtschaftlichkeit von Schutzmaßnahmen zu erbringen ist. Eine häufig verwendete Definition des Risikos ist durch Gl. (1) ausgedrückt.

$$R(X^*) = \int_{X^*}^{\infty} f(Q) \cdot D(Q) \cdot dQ \qquad \text{Gl (1)}$$

Dabei stellt Q die Größe eines Hochwasserereignisses, f(Q) die Dichtefunktion in einem Gewässerbereich dar, D(Q) definiert die Konsequenzen, die mit dem Auftritt eines Ereignisses der Größe Q verbunden sind, und X* gibt die Obergrenze für die Belastbarkeit eines Schutzbauwerkes an. Die Konsequenzen werden meist in monetär quantifizierten Schäden bzw. in Gefahren für den Verlust an Menschenleben angegeben.

Dem Ausdruck in Gl. (1) liegen nun einige Annahmen zu Grunde, die in der Praxis nicht erfüllt werden können. Es wird angenommen, dass die Hochwasserwahrscheinlichkeit f(Q) und die Schadensfunktion D(Q) zeitlich invariant sind. Hochwässer weisen offensichtlich, sowohl nach Auftrittszeit als auch nach Auftrittsgröße, einen Jahresgang auf. Weiters veränderten anthropogen bedingte Eingriffe im Einzugsgebiet den Abflussverlauf von Hochwässern nachhaltig. Die Hochwässer werden beschleunigt und ihre Spitzen erhöht, wobei bei extremen Ereignissen dieser Effekt wieder abklingt, wenn der gesamte Talbereich überflutet ist. Durch die intensivierte Nutzung der Talbereiche wird laufend das mögliche Schadenspotential und in der Folge der tatsächlich auftretende Schaden erhöht. Die in Abb. 2 dargestellten Schäden zeigen diesen Trend deutlich, während sich die Auftrittshäufigkeiten von Hochwässern kaum verändert haben.

Berücksichtigt man die Instationäritäten in den beiden Funktionen, so wird Gl. (1) etwas komplexer, wie in Gl. (2) dargestellt ist, wobei DF den Diskontierungsfaktor symbolisiert.

$$R(X^*) = \int_{T_{start}}^{T_{End}} DF(t) \cdot \int_{X^*}^{\infty} f(Q|t) \cdot D(Q|t) \cdot dQ \cdot dt \qquad \text{Gl. (2)}$$

Eine weitere Annahme liegt in der Beurteilung der Zuverlässigkeit eines Schutzbauwerkes. Ein Bauwerk kann aus einer Reihe von Gründen, die in mangelhafter Konstruktion und Wartung begründet sein können, bereits bei niedrigeren Lastfällen versagen. Ebenso kann es auf Grund eines Sicherheitszuschlages und infolge von Notmaßnahmen einem höheren Lastfall widerstehen. Die Größe X ist somit eine Zufallsvariable, die durch eine Verteilung g(x) zu beschreiben ist (Faber et al., 2003). Außerdem ist zu berücksichtigen, dass bei Überbelastung und Bruch eines Schutzbauwerkes die Schadwirkungen sogar verstärkt werden können.

$$R(X) = \int_{X_{Min}}^{X_{Max}} g(X) \cdot \int_{X}^{\infty} f(Q|X) \cdot D(Q) \cdot dQ \cdot dX \qquad \text{Gl. (3)}$$

Schließlich ist noch zu berücksichtigen, dass auf Grund von zeitlich begrenzten Beobachtungen, des Messfehlers und der Willkürlichkeit in der Wahl einer Verteilungsfunktion f(Q) noch zusätzliche Unsicherheiten in die Risikoerfassung eingehen.

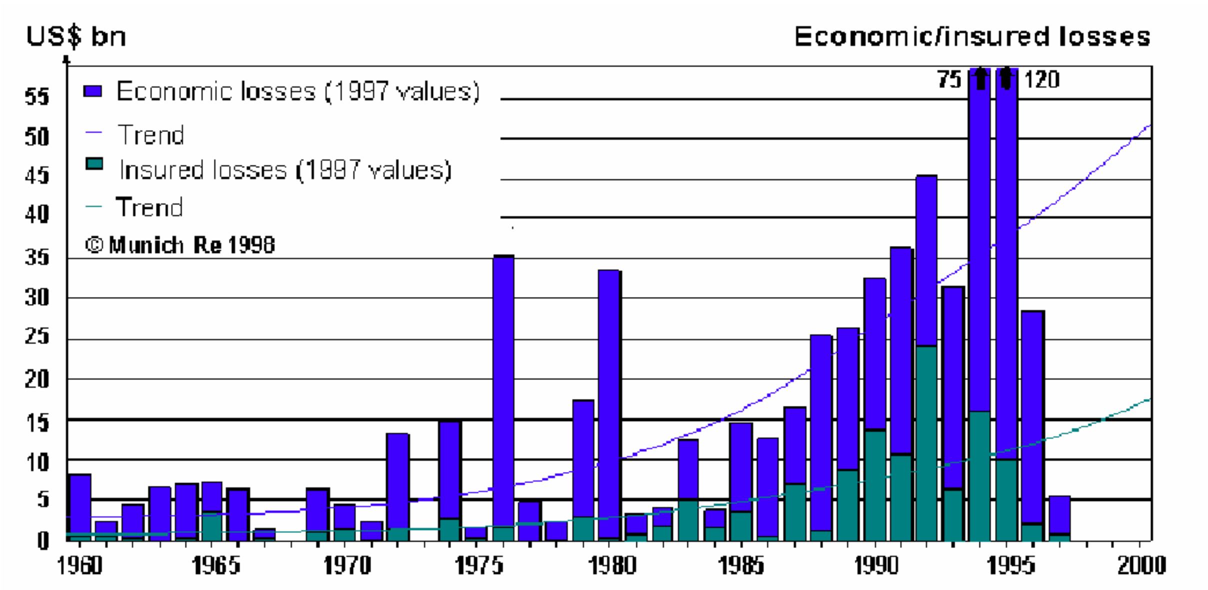

Abb.2 Tatsächliche und versicherte Hochwasserschäden (Munich Re, 1998)

Auch bei der Erfassung des Schadens D(Q), selbst bei einer ex-post Analyse, treten zusätzliche Unsicherheiten hervor, die durch unterschiedliche Erhebungsmethoden verursacht sind. Versucht man all diesen praktischen Überlegungen Rechnung zu tragen, so wäre Gl. (2) mit Gl. (3) zu kombinieren und zusätzlich noch zu parametrisieren, um die Modellungenauigkeiten (Verteilungstyp) und Datenunsicherheiten zu berücksichtigen.

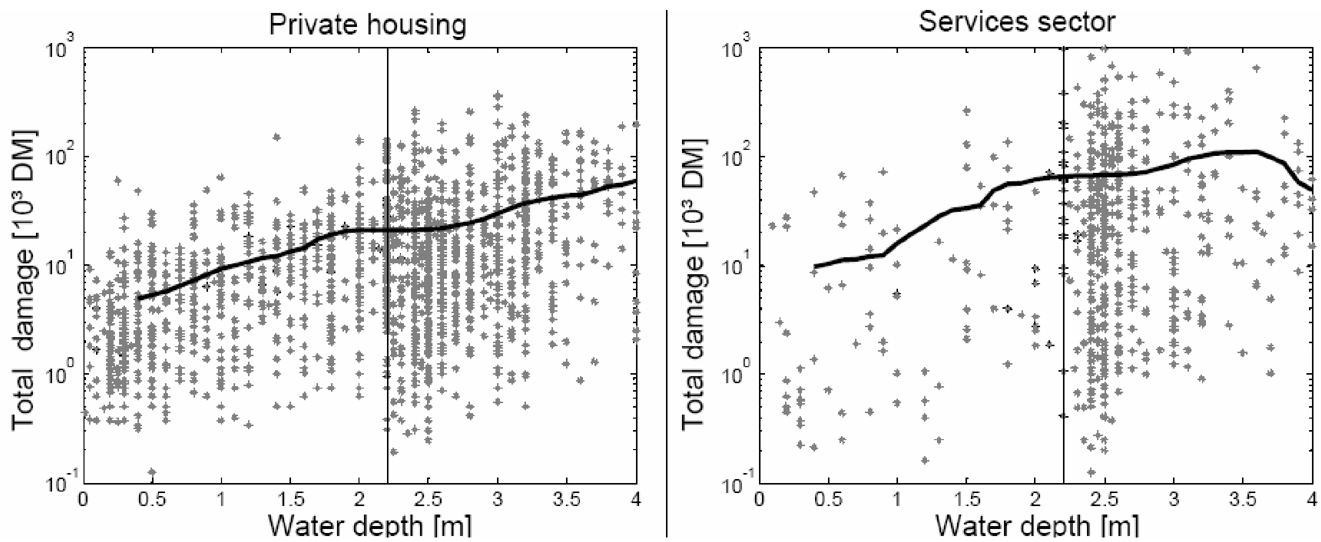

Abb. 3 Variabilität in den Schadensangaben (aus HOWAS, Merz et al., 2004)

Teilweise wird der Unsicherheit in der Erfassung des Risikos dadurch begegnet, dass bei der technischen Umsetzung von Schutzmaßnahmen Sicherheitszuschläge zur Bemessungsgröße zugerechnet werden. Dieser Freibord genannte Zuschlag hat nicht die hydrologischen Unsicherheiten, sondern hydraulisch verursachte Einflüsse wie turbulente Abflussvorgänge, starke Querströmung oder variable Sohllage, etc. zu berücksichtigen. Dementsprechend sind hydraulische Überlegungen bei der Wahl des Freibordes bestimmend, wobei von der Lage der Energielinie auszugehen ist.

Die Wahl des Freibordes soll nicht zu einer versteckten Erhöhung der Ausbauwassermenge dienen und soll ebenfalls nicht zu einer Verschärfung der Hochwassersituation in Unterliegerbereichen führen.

3. RISIKO MANAGEMENT

In diesem Abschnitt werden aus dem Themenfeld des Integrierten Hochwasserschutzes einige Punkte heraus gegriffen, die entweder bisher zu wenig Beachtung fanden, oder die derzeit einer intensiven Diskussion unterliegen (Nachtnebel, 2003). Gleichzeitig bieten die angeführten Beispiele Ansatzpunkte für den Umgang mit den angeführten Unsicherheiten bei der Quantifizierung des Risikos.

Im Unterschied zu anderen Naturrisken kann beim Hochwassermanagement sowohl die Größe des Ereignisses reduziert werden, als auch das Schadenspotential verändert werden. Ebenso bestehen noch Entscheidungsmöglichkeiten im Hinblick auf die Wahl des Schutzgrades und für die Streuung des Risikos. Dies sei kurz an Hand einiger Beispiele erläutert.

Die Größe eines Hochwasserereignisses lässt sich durch den Erhalt und die Schaffung von Retentionsraum für die flußab gelegenen Gewässerabschnitte dämpfen. Dies ist insoferne wichtig, als lineare Maßnahmen, wie Deichbauten, Retentionsraum laufend eliminieren und damit die Hochwassergefährdung für die Unterlieger erhöhen. Zur Reduktion des Schadenspotentials bestehen mehrere Möglichkeiten, die von der Absiedlung, der rechtzeitigen Hochwasserwarnung, bis zu Objekt bezogenen Maßnahmen reichen. Die Wahl des Schutzgrades wurde im voran gegangenen Kapitel kurz diskutiert. Das verbleibende Risiko, also jenes bei Überschreiten des Schutzgrades, kann durch Katastrophenfonds, durch Versicherungsstrategien oder individuell getragen werden. Die in Mitteleuropa vorherrschende Strategie ist die Teilabgeltung des Schadens aus Mitteln eines Katastrophenfonds.

3.1 AUSWEISUNG VON GEFÄHRDETEN BEREICHEN

Für dessen Quantifizierung ist die Angabe oder Modellierung des Lastfalles nötig, wozu in Abb. 4 die räumliche Verteilung der Überflutungshöhen bei einem außergewöhnlichen Ereignis dargestellt ist. Das gewählte Ereignis hat im Beispiel eine jährliche Auftrittswahrscheinlichkeit von etwa 0.001 und überschreitet somit den bestehenden Hochwasserschutz, der auf ein HQ100 ausgelegt ist, also eine Auftrittswahrscheinlichkeit von 0.01 besitzt.

Für die Charakterisierung des Lastfalles sind neben der Überflutungshöhe noch die Überflutungsdauer, die Dynamik der Einströmung, die Sedimentkonzentration und etwaige begleitend auftretende Schadstoffkonzentrationen wesentlich. Etliche dieser Eigenschaften treten besonders nachteilig bei Deichbrüchen hervor, die durch Überströmen bewirkt werden. Beim Hochwasser 2002 wurden insgesamt 45,2 Mio € an schutzwasserbaulichen Einrichtungen verursacht (Zenar, 2003), wobei auch viele Deiche und kleinere Dämme zerstört wurden. Durch die Deichbauten war der Abstrom der ins Hinterland eingeströmten Wassermassen erschwert, wodurch zusätzliche Schäden verursacht wurden.

Es sind für alle größeren Gewässerläufe Überflutungs- bzw. Überflutungs- und Gefahrenzonenpläne nach einem vereinheitlichten Konzept zu erstellen, die flächenhaft die Hochwassergefährdung für HQ30, HQ100 und eventuell HQ_{300} ausweisen.

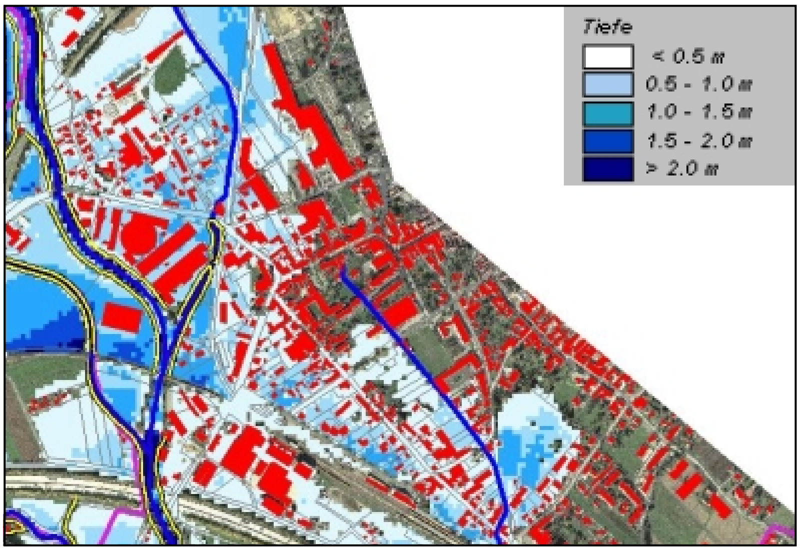

Abb. 4 Darstellung der Überflutungshöhen bei einem Extremereignis

Damit wird die Basis für eine verbesserte ex-ante Schadensermittlung geschaffen. Bei der derzeitigen Erfassung von Hochwasserschäden (bzw. des Schadenspotentials) bestehen trotz vorhandener Richtlinien und umfangreicher Dokumentationen methodische Probleme, die auch eine ex-post Analyse von Hochwasserschäden erschweren. Es ist ein Instrumentarium mit aktualisierten Richtlinien zu entwickeln, das zur Beurteilung des Schadenspotentials, der Wirtschaftlichkeit von Schutzmaßnahmen sowie des Restrisikos heranzuziehen ist. Weiters kann durch Abstimmung mit der Raumplanung das Schadenspotential deutlich reduziert werden.

3.2 SCHUTZBAUTEN MIT ENTLASTUNGSEINRICHTUNGEN

Für den Fall des Auftretens eines Ereignisses, das größer als das Bemessungsereignis ist, sind Maßnahmen zum Schutz des Bauwerkes vorzusehen (Klenkhart und Nachtnebel, 2005). Erst durch den Erhalt des Bauwerkes trotz extremer Belastungen kann der Schaden im Hinterland durch eine kontrollierte Überflutung reduziert und der verbleibende Zeitraum für Evakuierungsmaßnahmen verlängert werden. Durch die kontrollierte Wasserableitung bei einer Überströmstrecke sinkt der Wasserstand stromabwärts bis etwa zum Bemessungswasserstand.

Es kommen verschiedene Entlastungseinrichtungen in Betracht. Insbesondere sind regelbare Durchlässe, Wehranlagen und Überströmstrecken möglich. Die des öfteren angeführte mechanische Öffnung des Deiches durch Sprengung oder durch vorgesehene Bruchstellen ist als problematisch zu beurteilen. Die Gründe dafür liegen in der geringen sozialen Akzeptanz im Ernstfall und im unkontrollierten Abflussgeschehen im Hinterland. Überströmstrecken werden an einigen Stauhaltungen an der Donau und an einigen Hochwasserdeichen, z.B. an der Gail, seit Jahren ausgeführt.

Die Situierung von Überströmstrecken hängt von hydrologischen, topografischen, nutzungsbezogenen Eigenschaften und von ökologischen Gesichtspunkten ab. Aus hydrologischer Sicht sind im

Längsverlauf des Fließgewässers Überströmstrecken jeweils dort zu situieren, wo es in Folge größerer Zubringer zu einer nennenswerten Erhöhung des Hochwasserabflusses kommt (Abb. 5).

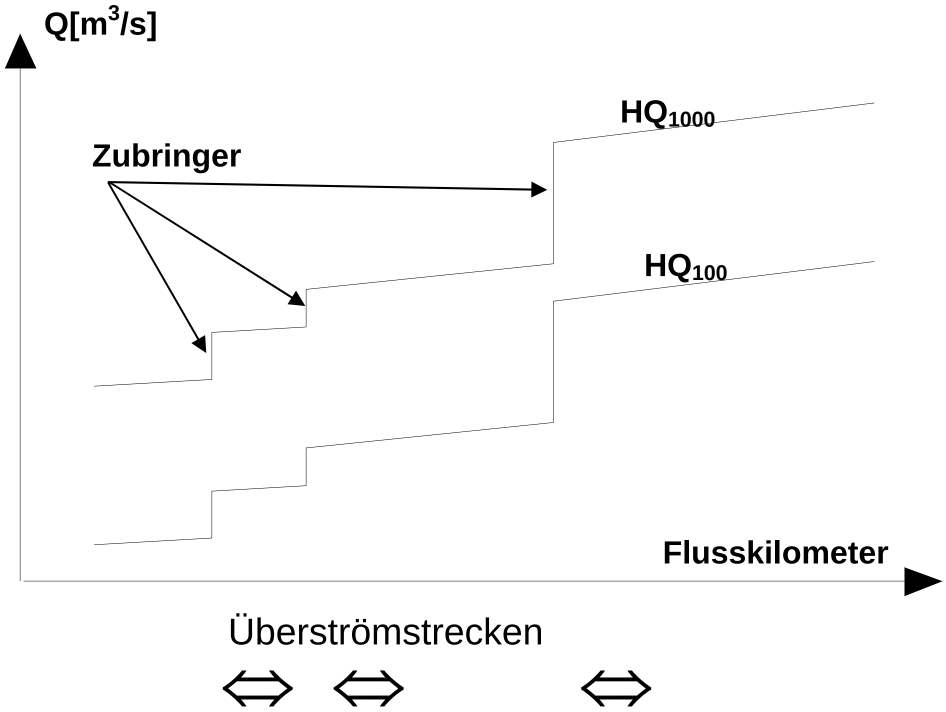

Abb.5 : Darstellung der Bemessungsdurchflüsse im Längenschnitt.

Ein weiterer Aspekt für die Situierung von Überströmstrecken ist der im Hinterland vorhandene Retentionsraum. Überall wo größere Retentionsräume vorhanden sind, ist eine Entlastung vorzusehen, und wenn nicht anders möglich, auch im Bereich von Siedlungsgebieten. Weiters ist bei der Situierung von Überströmstrecken auf die Abflusskapazität im Hinterland und auf die Bebauung Bedacht zu nehmen. Es ist darauf zu achten, dass Altarme, Mulden im Vorland den Abfluss und die Rückgabe ermöglichen. Werden Überströmstrecken gezielt nach ökologischen Gesichtspunkten angeordnet, ist zumeist ein relativ häufiges Ausufern gewünscht. Auch in einem solchen Fall sind die für ein kontrolliertes Ausufern nötigen Überströmstrecken sorgfältig zu bemessen und konstruktiv auszuführen.

Dimensionierungsgrößen (Bieberstein et al., 2002) sind die Länge, die Profilform (Neigung) und bei der technischen Ausführung ist noch der Erosionsschutz des Deiches und des Böschungsfußes wichtig. Alle diese Größen hängen von der Abflusscharakteristik, der Häufigkeit und Dauer der Beanspruchung und der Höhe der Spiegellage über dem Gelände ab. Generell ist festzustellen, dass für die Überströmstrecken selbst kein Freibord vorzusehen ist. Bei der Abflusscharakteristik sind die Länge der Überflutungsstrecke, die Rauhigkeit der Dammoberfläche, die luftseitigen Böschungsneigungen (Neigung der Energielinie) sowie die maximal mögliche spezifische Beaufschlagung maßgebend.

Der maximal mögliche Abfluss, für den die Überströmstrecke zu dimensionieren ist, ist abhängig von der Gerinnecharakteristik (vor allem dem Verhältnis zwischen Gerinnebreite und Abflusstiefe). Aus der von bautechnischen Gesichtspunkten abhängigen zulässigen spezifischen Beaufschlagung ergibt sich die Länge der jeweiligen Überströmstrecke. Die Wasseranschlagslinie flussabwärts des Steichwehrs entspricht dem Bemessungswasserstand.

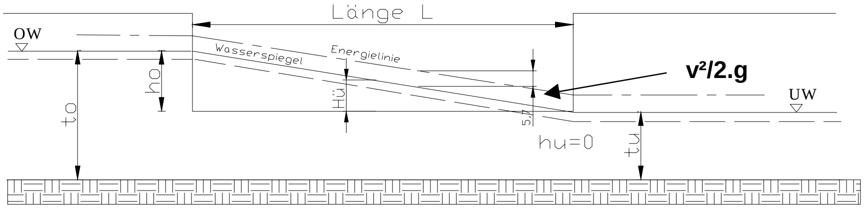

Abb. 6 Schematischer Längenschnitt einer Überströmstrecke.

3.3 REDUKTION DES LASTFALLES

Bei den Maßnahmen zur Reduktion der Größe und Häufigkeit von Ereignissen bieten sich nichttechnische und technische Maßnahmen an.

Die nichttechnischen Maßnahmen sind auf die Fläche bezogen und haben die Aufgabe, die natürliche Retentionswirkung eines Einzugsgebietes zu vergrößern, den Abflussvorgang zu verzögern, die Versickerung zu vergrößern und einen mittelfristigen Ausgleich von Perioden mit hohen und niedrigen Abflüssen zu erzielen. Hier besteht ein enger Konnex zu ökologischen Zielsetzungen, die derartige Maßnahmen voll unterstützen. Zur Umsetzung sind Flussgebietspläne zu erstellen und raumordnerische Maßnahmen zu formulieren. Ein Inventar und ein Konzept zur Erhaltung, Erweiterung und Bewirtschaftung des natürlichen Retentionsraumes ist anzustreben.

Technische Maßnahmen beinhalten die Errichtung von Hochwasserrückhaltebecken an den Oberläufen und den Bau von Hochwasserschutzdeichen in den Talniederungen. Die Trassierung der Deiche ist in größtmöglicher Entfernung vom Gewässer zu wählen, so dass sich eine standortsgerechte Vegetation ausbilden kann ohne dass die Abflussleistung für das Bemessungsereignis beeinträchtigt wird. Zu berücksichtigen ist hier der Feststoffhaushalt, da es bei Überbreiten zu Ablagerungen kommen und dadurch der Bemessungswasserspiegel verändert werden kann.

3.4 OBJEKTSCHUTZ

Die durch Hochwässer verursachten Folgeschäden sind beträchtlich. Dazu zählen Kontaminationen durch austretende Schadstoffe, insbesondere in Wohnbauten, wo die Heizungsanlagen nicht entsprechend den technischen Standards errichtet wurden. Durch aufschwimmende Tanks, durch unsachgemäße Lagerung von Brennstoffen werden diese Kontaminationen verursacht. Ein wesentliches Ziel muss daher die Reduktion der Folgeschäden sein. Dazu zählt die entsprechende bauliche Ausführung und Nutzung von Kellerräumen und Erdgeschossen. Die Problematik wird sowohl im Defizit von Auflagen (zum Teil werden im Rahmen von wasserrechtlichen Bewilligungen von Bauwerken im Hochwasserabflussgebiet keine Vorschreibungen bezüglich der Auftriebssicherheit und Undichtheit von Öltanks vorgesehen) als auch in der mangelhaften Durchsetzung bestehender Auflagen und Nutzungseinschränkungen, gesehen.

Für den Objektschutz bestehen etliche technische Möglichkeiten (Proverbs, D.G. & R. Soetanto; 2004), die durch Förderungsmaßnahmen, Kontrollen oder Sanktionen verstärkt umzusetzen sind.

3.5 RISIKO KOMMUNIKATION

Die Schäden eines katastrophalen Ereignisses können durch entsprechende Vorbereitung reduziert werden. Dazu zählt die frühzeitige Kenntnis über eine bevorstehende Katastrophe, die Information der Bevölkerung und das Anlaufen von Rettungs- und Evakuierungsmaßnahmen. Die beiden etwa gleich großen Rheinhochwässer in den Neunziger Jahren verursachten deutlich unterschiedliche Schäden, da beim zweiten Ereignis die Bevölkerung entsprechend vorbereitet war.

Die Gefahrensituation (Restrisiko, siehe Abb. 4), sowie das Verhalten im Katastrophenfall ist den potentiell Betroffenen nachdrücklich zu vermitteln. Neben diesen ortsbezogenen Informationen sind auch zeitbezogene Informationen für die Schadensreduktion wichtig. Die durch verbesserte Prognosen mögliche Vorbereitung auf ein Ereignis kann monetäre Schäden, sowie die Gefährdung von Menschenleben deutlich reduzieren.

3.6 EINBEZIEHUNG DER BETROFFENEN

Jede Planung ist nur soweit effizient, als sie umgesetzt und von der Bevölkerung akzeptiert wird. Gleiches gilt für technische Auflagen für Gebäude. Es sind daher die potentiell Betroffenen in die Planung, die Umsetzung und die Finanzierung mit einzubeziehen. Dies impliziert eine ausreichende Information und eine Beteiligung der Bevölkerung an der Entscheidungsfindung selbst. Gleichzeitig heißt dies aber auch, dass anteilig Kosten für Bau und Instandhaltung von der örtlichen Bevölkerung zu tragen sind.

Dementsprechend hat die Planung großräumig, nach Möglichkeit flussgebietsbezogen, zu erfolgen. Planungsvarianten sind frühzeitig, solange noch generelle Planungen durchgeführt werden, zu diskutieren, die Kostenbeiträge sind in ihrer Größenordnung offen zu legen, und das verbleibende Risiko ist den Betroffenen zu vermitteln.

4. ZUSAMMENFASSUNG

Obwohl beträchtliche Summen in den Schutz vor Naturgefahren, insbesondere in den Hochwasserschutz, laufend investiert werden, steigt das Schadenspotential deutlich an. In den letzten Jahren wurden nun Ansätze zu einem Integrierten Risikomanagement entwickelt. In diesem Beitrag wurde zuerst der Rahmen für ein derartiges Management dargestellt und dann einige Punkte heraus gegriffen, die derzeit einer Diskussion unterworfen sind. Die wesentlichen Punkte sind dabei der Umgang mit Unsicherheiten in der Quantifizierung des Risikos selbst und die Maßnahmen zur Reduktion des Risikos. Erst durch das Zusammenwirken von unterschiedlichen Maßnahmen, die technische, nicht technische, administrative, planerische, legistische und kommunikative Elemente beinhalten, wird es möglich, das Hochwasserrisiko zumindest teilweise zu steuern.

Ein absoluter Hochwasserschutz ist unmöglich, und die Sicherheitserwartungen der Betroffenen können daher nur zum Teil erfüllt werden. Es bleibt daher die Frage, wie das verbleibende Risiko erfasst und kommuniziert werden kann und wie im Schadensfall eine möglichst breite Streuung des Risikos erfolgen kann.

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GLETSCHERGEFAHREN IM HIMALAYA – GEFAHRENANALYSE UND SCHUTZMASSNAHMEN

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ABSTRACT
Die außerordentlichen klimatischen Bedingungen und die hohe Reliefenergie auf der Südabdachung der Himalayakette führen, neben anderen Faktoren, zu einer erhöhten Geo-Hazard-Aktivität und damit verbundenen Risiken für Bevölkerung und Infrastruktur. In Nordindien, Nepal, Tibet und Bhutan schmelzen die Gletscher bedingt durch die regionale Erwärmung der letzten Jahrzehnte sehr schnell ab, wobei jährliche Rückzugsraten von über 25 Metern erreicht werden. Dadurch entstehen unzählige supraglaziale und proglaziale Gletscherseen. Die Analyse von Satellitenbildern hat alleine in Nepal über 2300 Gletscherseen ergeben, wobei von 20 eine akute Gefährdung ausgeht. In Bhutan sind von über 2600 Gletscherseen 24 als gefährlich einzustufen.

Diese Gletscherseen sind meist durch instabile Moränendämme begrenzt, deren Versagen immer wieder zu Gletscherseeausbrüchen, mit teilweise katastrophalen Auswirkungen in den flussabwärts gelegenen Talbereichen geführt hat. Als Auslösemechanismen für derartige Flutwellen können abschmelzendes Toteis in den Moränen, hydrostatischer Druck und Piping-Effekte in Moränenquellen, sowie Massenbewegungen in die Seen vermutet werden.

Im Rahmen eines von der Österreichischen Entwicklungszusammenarbeit geförderten Österreichisch-Bhutanischen Kooperationsprojektes welches am Institut für Geologische Wissenschaften der Universität Wien durchgeführt wurde (unter der Leitung von Univ.-Prof. Dr. Hermann Häusler und Dr. Mag. Diethard Leber), wurde im Nordwesten Bhutans dieses geogene Gefährdungspotential näher untersucht.

Durch die Integration von Daten aus der geomorphologisch/geotechnischen und hydrogeologischen Geländeaufnahme, dem Einsatz von ingenieurgeophysikalischen Methoden (Reflexions- und Refraktionsseismik, Geoelektrik und Bodenradar) und der Integration von meteorologischen und hydrologischen Parametern war eine fundierte Beurteilung des Geohazard-Potentials möglich. Die Beurteilung des geogenen Prozessgefüges bildete so die Grundlage für die Planung geeigneter Schutz- und Sanierungsmaßnahmen.

Bedingt durch die Unzugänglichkeit der hochalpinen Gebiete von Lunana und Tarina im Nordwesten Bhutans kommt der Auswertung von hochauflösenden panchromatischen und multispektralen Satellitenbilddaten und von satellitengestützen Radardaten eine große Bedeutung für eine erste Abschätzung des Gefahrenpotentiales von Gletschersseen zu.

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