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Ufficio federale delle strade

Level of safety required for the assessment of existing highway bridges

Erforderliches Sicherheitsniveau für die Überprüfung bestehender Brücken

Swiss Federal Institute of Technology, Lausanne

MCS - Maintenance, Construction and Safety of Structures
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Research mandate 84/99 commissioned by the Bridge Research Working G r o u p

April 2002

FEDERAL ROADS OFFICE
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i FOREWORD PROPOS

When designing a *new bridge,* the level of safety is not explicitly considered. Experience shows that the level of safety recommended by design standards is probably more than sufficient. Since optimizing the quantity of materials in relation to the safety margin is not economically justifiable, finding the required level of safety has never been of prime importance.

On the other hand, when assessing an *existing bridge,* the decision to intervene (restoring structural safety, increasing the strength of structural elements) is motivated by an unsatisfied verification of the structural safety of one or more load-bearing bridge elements. In addition, there is often a fine line between heavy and light intervention; for example, the additional weight of a deck upgrade may also require reinforcement of the primary load-bearing structure. A more in-depth study of structural safety could therefore make it possible to limit, or even avoid, heavy construction interventions.

As a result, there is a need to better understand the minimum acceptable level of safety. In a more comprehensive framework, adequate reliability is strongly linked to optimization in terms of the cost-benefit ratio of (non-)intervention. In addition, structural safety aspects need to be complemented by considering the performance (in terms of serviceability) and economic value of a bridge. A comprehensive approach is therefore required to determine the optimum intervention for an existing bridge, while respecting the required level of safety.

As part of the research mandate 84/99 awarded by the Swiss Federal Roads Authority (FEDRO), a methodology is being developed to define the required level of safety as a function of the risk associated with bridge failures. The present research is also a contribution to a *risk-based* approach to safety which considers the probability of failure <u>and</u> the extent of damage following failure.

The authors would like to thank the Swiss Federal Roads Office (FEDRO) and the members of the research commission, namely P. Matt (chairman), M. Donzel, Prof. R. Favre, Prof. A. Muttoni, H. Fleischer, P. Wüst and H. Figi.

Lausanne, April 2002Prof . E. Brühwiler

ii SUMMARY

This report presents the results of a study into the level of safety required for the assessment of existing road bridges. The philosophy of the study is to define target reliability as a function of the risk associated with a failure, rather than considering the level of safety imposed by construction standards. The study therefore focused on an analysis of the risk associated with road-bridge failures and the risks accepted by the public in everyday activities. This risk comparison is then used to define an acceptable level of risk for the evaluation of existing highway bridges.

The motivation behind the study is to improve the assessment of existing bridges by means of a more detailed and accurate evaluation, with the aim of avoiding interventions on structures that are already sufficiently safe. This is the danger when construction standards or safety levels for new buildings are applied as-is to the assessment of existing bridges.

Compared with sizing new structures, there are many reasons to treat existing structures differently. Risks and uncertainties are reduced once the structure has been successfully put into service and is operating satisfactorily. Around 40% of bridge accidents occur during construction, and are mainly due to human error. There is therefore no reason to cover these risks when assessing an existing bridge. Interventions to increase the load-bearing capacity of existing bridges are relatively costly, which justifies a more detailed assessment.

It is important to note that the aim of the study is not to reduce the overall safety level of the bridge fleet, but rather to target a uniform level of acceptable risk. The approach proposed in this report is to define a required safety level as a function of risk situations, rather than applying the same required safety level to all bridges and risk scenarios. This approach requires the following steps:

- identification of predominant risk situations.
- definition of the consequences of a risk situation in terms of damage and the economic importance of the bridge.
- selection of a required safety level as a function of the magnitude of these consequences.

The required level of safety is thus defined as a function of "external" parameters representing the value and importance of a structure. This required safety level is then compared with the estimated safety, calculated from the "internal" parameters describing the bridge's condition. Methods for calculating bridge safety are also briefly presented in the report, with references to other sources of guidance on the subject.

The report concludes with a concise, practical guide to selecting a required safety level, and application examples are given for highway bridges.

iii SUMMARY

This report presents the results of a study of the target safety level required for the evaluation of existing highway bridges. The philosophy of the study is to define target safety levels as a function of the risk associated with bridge failures, rather than considering the target safety level implied by design codes. The study has therefore focused on surveys of the risk associated with bridge failures and the risk accepted by the public in daily activities. These risks are then used to define an acceptable level of risk to be used for evaluating existing road bridges.

The motivation for the study is to improve the evaluation of existing bridges with a view to avoiding interventions on structures that are already adequately safe. This is the danger when design codes, or design levels of target safety, are applied to the evaluation of existing bridges.

Compared to the design of new structures, there are the following reasons for treating existing structures differently. There are fewer hazards and less uncertainty once a structure has successfully entered service and performed satisfactorily. For example, 40% of bridge accidents occur during construction, mainly due to human error, and there is no need to cover this hazard when evaluating an existing bridge. Also, measures to increase the safety of an existing bridge are relatively costly.

It is important to note that the aim of the study is not to reduce safety levels globally throughout the bridge stock, but rather to target a uniform level of acceptable risk. The approach proposed in this report is to define target safety levels as a function of the hazard scenario under consideration, rather than applying a uniform target safety level to all scenarios and bridges. This approach involves the following steps :

- Identification of hazard scenarios.
- Definition of the consequences of a given hazard scenario with respect to damage and the economic importance of the bridge.
- Selection of the target safety level as a function of the magnitude of these consequences.

The target safety level is thus derived as a function of "external" parameters representing the value and importance of a structure. This target safety level is then compared to the estimated safety, which is calculated using "internal" parameters describing the state of the bridge. Methods for the calculation of bridge safety are also presented briefly in the report, making reference to other sources of guidance on the subject.

The report concludes with a concise practical guide to the selection of target safety level and a number of examples for road bridges.

iv ZUSAMMENFASSUNG

Der vorliegende Bericht enthält die Ergebnisse einer Studie über das erforderliche Sicherheitsniveau für die Überprüfung bestehender Strassenbrücken. Ziel dieser Forschung war es, die Zuverlässigkeit einer Brücke in Abhängigkeit des Versagensrisikos zu bestimmen und nicht, wie beim üblichen Vorgehen, das aus der Anwendung der Konstruktionsnormen resultierende Sicherheitsniveau zu übernehmen. Um dieses Ziel zu erreichen, wurden Brückenunfälle analysiert und das von der Gesellschaft akzeptierte Risiko für diverse Aktivitäten des täglichen Lebens ermittelt. Diese Risiken wurden mit dem Risiko eines Brückenunfalls verglichen, um daraus das akzeptierte Risiko zur Überprüfung bestehender Strassenbrücken abzuleiten.

Mit den gewonnenen Erkenntnissen soll die Überprüfung bestehender Strassenbrücken verbessert werden, indem die Tragsicherheit einer Brücke mit einem detaillierteren Nachweis eher nachgewiesen werden kann als wenn einzig basierend auf den Konstruktionsnormen der Tragsicherheitsnachweis geführt wird. Damit sollen bauliche Eingriffe (Instandsetzungen, Verstärkungen) möglichst vermieden werden.

Im Vergleich zur Bemessung neuer Tragwerke gibt es mehrere Gründe, die bestehenden Bauwerke anders zu behandeln. Bei einer bestehenden Brücke gibt es weniger Unsicherheiten, da sie ja ihre Gebrauchstauglichkeit bereits bewiesen hat. Zudem ereignen sich 40 % aller Brückenunfälle bereits während dem Bau. Zur Beurteilung bestehender Brücken sind diese meistens auf menschliches Versagen zurückzuführende Unfälle nicht zu berücksichen. Schliesslich sind bauliche Massnahmen zur Erhöhung der Tragfähigkeit bestehender Brücken vergleichsweise kostspielig, was einen weitergehenden Nachweis gerechtfertigt.

Es ist wichtig zu präzisieren, dass das Ziel dieser Studie nicht darin besteht, das globale Sicherheitsniveau von Brücken zu vermindern, sondern ein gleichmässiges akzeptiertes Risiko für Versagensszenarien anzustreben. In diesem Bericht werden deshalb akzeptierte Sicherheitsniveaus in Abhängigkeit des Gefährdungsbilds bestimmt, indem wie folgt vorgegangen wurde :

- Ermittlung der massgebenden Gefährdungsbilder
- Beurteilung eines gegebenen Gefährdungsbilds bezüglich mögliche Schadensgrösse und wirtschaftliche Bedeutung der Brücke
- Ermittlung des akzeptierten Sicherheitsniveaus in Abhängigkeit des Schadensausmasses

Das akzeptierte Sicherheitsniveau wird aufgrund " äusserer " Parameter definiert, die den Wert und die wirtschaftliche Bedeutung der Brücke beschreiben. Dieses Ziel-Sicherheitsniveau wird im Sicherheitsnachweis mit der rechnerisch ermittelten, effektiven Tragsicherheit verglichen. Diese wird aufgrund " innerer " Parameter ermittelt, die den Zustand der untersuchten Brücke beschreiben. Entsprechende Methoden werden kurz dargestellt, und es wird auf entsprechende Literatur hingewiesen.

Der Bericht schliesst mit einem Leitfaden zur Ermittlung des akzeptierten Sicherheitsniveaus. Einige Beispiele veranschaulichen die Anwendung.

v DEFINITIONS

Failure state	Failure	Versagen	Inadequate performance with respect to a limit, such as structural safety or serviceability.
Performance	Performance	Leistungsfähig-keit	The ability of a structure to meet requirements.
Reliability	Reliability	Zuverlässigkeit	The probability that the performance of an structure meets requirements over a given period and with a defined probability.
Reliability required	Target reliability	Erforderliche Zuverlässigkeit	The level of reliability to aim for, based on society's expectations and requirements in terms of public safety.
Optimum reliability	Optimum reliability	Optimale Zuverlässigkeit	The level of reliability achieved by optimizing costs and benefits during construction or intervention.
Risk	Risk	Risiko	The expected consequences of failure, being the failure damage multiplied by the failure probability.
Ruin	Structural failure	Tragwerksver-sagen	Structural failure, e.g. failure of a component or collapse of a structure.
Element breakage	Element failure	Bauteilversa-gen	Ruin of an element, limiting the performance of a structure.
Collapse	Collapse	Einsturz	Total collapse , rendering a structure unusable.
Risk situation	Hazard scenario	Gefährdungsbild	A situation (combination of actions) which could cause a failure resulting in a certain consequence (damage).
Risk category	Risk category	Risiko-Kategorie	The classification of a structure according to the level of risk it presents.
Damage	Damage	Schaden	The consequence of a failure, expressed as for example, in terms of the number of deaths or the cost of inadequate performance.

1 INTRODUCTION

1.1 MOTIVATION

When designing and building a *new bridge,* the level of safety is not explicitly considered, e.g. by applying the design rules of the standards. Experience shows that the level of safety recommended by design standards is probably more than sufficient. Optimizing the quantity of material in relation to the safety margin is not economically justifiable, and therefore the search for the required level of safety has never been of prime importance.

On the other hand, when assessing an *existing bridge,* the decision to intervene (restore structural safety, increase load-bearing capacity of structural elements) is motivated by an unsatisfied verification of the structural safety of one or more load-bearing bridge elements. What's more, there seems to be little understanding of the boundary between heavy and light intervention. For example, a light intervention (deck thickening) could lead to heavy reinforcement of the primary load-bearing structure. In this case, a more in-depth study of structural safety could demonstrate that it is possible to limit, or even avoid, heavy interventions.

As a result, there is a need for a better understanding of the level of safety required for bridges. Furthermore, as the load-bearing elements of a structure are checked independently, it is advantageous to analyze the overall reliability of a structure using a "system" approach. The notion of time must also be integrated to take account of the reduction in section strength due to effects such as corrosion and fatigue.

In a more comprehensive framework, adequate reliability is strongly linked to optimization in terms of the cost-benefit ratio of (non-)intervention. In addition, structural safety aspects need to be complemented by consideration of a bridge's performance in terms of serviceability and durability. Consequently, a comprehensive approach is required to determine the optimum intervention for an existing bridge.

1.2 AIMS AND LIMITS OF RESEARCH

The basic idea of the study is to define target reliability as a function of the risk associated with bridge failures, rather than considering the level of safety imposed by construction standards. The study focuses on an analysis of the risk associated with bridge failures and the risks accepted by the public during everyday activities. These risks are then used to define an acceptable level of risk for the assessment of existing highway bridges.

The motivation behind the study is to rationalize the assessment of existing bridges, with the aim of avoiding interventions on structures that are already sufficiently safe. This is the danger when construction standards or safety levels for new buildings are applied as-is to the assessment of existing bridges.

1.3 APPROACH

The search encompasses the following three stages:

- a study of bridge failures
- a study of the level of safety associated with other activities
- development of a methodology for defining the level of safety required for a given risk situation

The minimum safety required by society in relation to public safety must be identified. This is an important criterion for determining minimum intervention or justifying non-intervention. The study includes a comparison of the reliability of highway bridges worldwide with that of various means of transport and other areas of activity. The aim of the study is to justify a minimum risk.

Rather than applying a uniform level of safety required for all bridges and risk scenarios in the final phase, a methodology has been developed for defining the level of safety required for a given risk situation. This approach involves the following steps:

- identification of predominant risk situations.
- definition of the consequences of a given risk situation in terms of damage and the economic importance of the bridge.
- selection of a required safety level as a function of the magnitude of these consequences.

The required level of safety is thus defined as a function of "external" parameters representing the value and importance of a structure. This required level of safety is then compared with the estimated safety, calculated using "internal" parameters describing the state of the bridge. Methods for calculating bridge safety are also briefly presented in the report, with references to other sources and guidance on the subject.

2 STATE OF KNOWLEDGE

2.1 INTRODUCTION

The aim of this part of the research is to draw up an inventory of current knowledge in the field of target reliability when assessing existing highway bridges. The conclusions to be drawn from this bibliographical study are presented as follows:

- **Section 2.2** ***Bibliographical research***

  Commentary on the most interesting publications on the subject of bridge assessment, risk, bridge systems and structural deterioration.

- **Section 2.3** ***Foreign directives***

  Summary of guidelines used for bridge design and evaluation.

- **Section 2.4** ***Conclusions***

  Synthesis of the main conclusions of this chapter, focusing on the applicability of the documents examined to Switzerland.

2.2 RESEARCH BIBLIOGRAPHY

We have reviewed over 40 publications dealing with bridge safety, management, evaluation and probabilistic analysis. A review of the most interesting articles from Europe, North America and Australia is presented below. In this review, we deal with the articles in the chronological order of our research (table of contents of the report). At present, the need for research lies mainly in the interaction between the fields of safety analysis, deterioration and management of existing structures. The common goal is to develop an integrated approach that can be used for planning maintenance interventions on existing road bridges.

2.2.1 Study of accidents and risks

The collapse of a bridge is a rare event. One might conclude that bridges have an acceptable level of safety. A recent study carried out in England [Menzies, 1996].

[Schneider, 1994] analyzed 800 civil engineering damages. He classified them according to cause and possible measures to be taken. The results are very telling, and can be used to identify effective measures to guarantee the safety of structures. According to Schneider, 75% of accidents are due to human error. He also proposes a detailed risk classification scheme.

An overview of accidents in construction is given in [Carper, 1997]. It shows that the main risks affecting the safety of structures are as follows: Inadequate dimensioning or insufficient knowledge, inappropriate choice of site, errors during construction, collapse during construction, extreme actions (earthquake, wind, snow, fire, etc.) and unexpected combinations of actions, unexpected deterioration or deterioration faster than expected.

Several authors compare the different risks of death [Allen, 1972] [Thoft-Christensen, 1982] [Melchers, 1999] [Menzies, 1996] [Schneider, 1994]. Most of these comparisons are made at the level of deaths from a certain activity relative to a population, taking into account exposure time. These studies show that the risk of death due to structural failure is negligible compared with other hazards. Several authors set risk limits. [Schuler, 1999] proposes as an upper limit the risk of death in general with a probability of 10^{-5} .

2.2.2 Parameters influencing reliability

The assessment process is of great importance for bridge maintenance. Most researchers in this field agree that when faced with uncertainties, decision-making can be facilitated by risk-based verification of structures. The difficulties lie in modelling, human error and engineering office habits [Menzies, 1999].

The big difference between the evaluation of an existing bridge and the design of a new one is the amount of data/information on the bridge. [Faber, 2000] gives an overview of reliability-based methods for evaluating existing structures. His summary also includes applications to real structures.

If we obtain additional (measured) data from an existing structure or its components, we can improve the a priori estimate of the structure's reliability. This is the domain of Bayesian statistics, using Bayes' theorem [Scheiwiller, 1998][Melchers, 1999][Faber, 2000].

As actions and resistances are random variables, deterministic approaches do not take the safety reserve into account. Methods for assessing reliability can be found in various publications [Stewart, 1997], [Thoft-Christensen, 1982], [Schneider, 1994], [Melchers, 1999]. Analysis can be performed by numerical integration, Monte Carlo simulation or approximation methods such as First Order and Second Order Reliability Methods (FORM/SORM) [Ditlevsen, 1996]. [Haldi, 1998] and [Stewart, 1997] review the main methods used in the field of industrial system dependability (cause tree, consequence tree, etc.).

There are few applications of probabilistic analysis of structural failures, because of the great sensitivity to accepted distributions, the difficulty of taking into account human behavior and other factors that have a major influence on current risk. In addition, there is still the problem of recognizing failure risks. Researchers have tried to circumvent these problems through use:

- a reliability index to overcome the sensitivity of risk calculations to accepted distribution functions,
- of Bayesian variables whose means and standard deviations can be estimated by judgment (thus taking into account human behavior and simplifications in structural analysis),
- calibration procedures that adapt the safety level of existing dimensioning procedures [Bassetti, 1998]. This makes it possible to establish more uniform safety levels. [Nowak, 1995] established the load and resistance factors of the new American standards so as to have a predefined level of safety. The target reliability index was set on the basis of reliability indices obtained on bridges designed to the old standards.

[Tabsh, 1991] proposes a method for calculating the reliability of multi-girder highway bridges. This bridge system is composed of elements in series and in parallel. The difference between the ultimate load that can be applied to an element (element reliability) and the ultimate load of the system (system reliability) is called **redundancy**. The reliability of bridges designed to American standards varies with the span and materials used. For steel bridges, [Tabsh, 1991] found reliability indices of the order of 3 to 3.5, for composite bridges from 2.5 to 3.5 and for reinforced and prestressed concrete bridges from 3.5 to 4.

[Ghosn, 1996] demonstrates the difference between the reliability of an element and the reliability of the typical bridge system. Current design procedures assume that the bridge system is always in an elastic state, whereas the strength of a component is determined on the basis of limit state considerations. This assumption underestimates the true capacities of a bridge system, and therefore gives lower limits for reliability. If, for example, the bearing moment of a two-span bridge reaches the plastic moment, the section will undergo inelastic deformations and a redistribution of forces to the others. Ghosn's approach assumes that an explicit relationship can be found between all possible failure mechanisms. Bridges are often composed of a large number of structural elements, and it is often extremely difficult to find expressions for their predominant failure mechanisms. This problem can be overcome by using an efficient numerical simulation technique (e.g. the response surface method [Johannis, 1999]).

[Ghosn, 1998] has also developed a method for taking redundancy into account when sizing and evaluating existing highway bridges. The elements of a bridge are not independent, but act together to form a system. The method penalizes bridges with insufficient redundancy by applying larger system factors during traditional dimensioning. The limit states analyzed for adequate bridge system safety are: element failure, ultimate limit state, service limit state, damaged limit state.

[Schneider, 1994] suggests subdividing a system into series and parallel elements.

The reserve due to redundancy is very high for multi-girder bridges. After the failure of one of the girders, the loads are taken up by the others. However, this type of bridge is not very common in Switzerland, and redundancy in the longitudinal direction is not very high.

Reliability-based techniques are excellent tools for assessing deteriorated structures. In particular, they can be used to determine the right time to intervene, thereby minimizing maintenance and repair costs. [Sarveswaran, 1999] uses an empirical deterioration model based on values measured on site to predict the evolution of deterioration in reinforced concrete beams (loss of reinforcement cross-section and detachment of concrete cover).

[Ciampoli, 1998] has formulated a probabilistic method for assessing the reliability of elements of a structure subject to deterioration. This is **time-dependent** and can be updated in the event of maintenance or repair. In his approach, he distinguishes between deterioration due to ageing (continuous) and deterioration due to impact (punctual). Once the reliability of each component has been defined, the reliability of the system as a whole can be assessed, taking into account its functional logic and structural behavior.

[Enright, 1998] combines values measured in situ with numerical integration. His method can be used to predict the reliability of reinforced concrete bridges under environmental actions such as alkali-silicate reactions, corrosion or frost. It is an approach in which loads and resistance are time-dependent.

[Kunz, 1992] has established a method for assessing the fatigue safety of existing steel bridges. For this purpose, the probability of failure is determined as a function of the number of trains expected in the future. The probability of fatigue failure of a construction detail can thus be calculated by taking into account the probability of crack detection. This can then be compared with a desired value.

2.2.3 Safety level required

A maintenance strategy is based on considerations of minimum acceptable safety. If this is too conservative, the structures will be reinforced or the working load limits lowered. On the other hand, if it is too optimistic, there is a risk that the bridge will fail in service [Shetty, 1999]. In the field of bridges, there are few studies on the level of safety required.

The acceptability of the risk of a bridge collapse is highly dependent on the importance of its intangible value, the amount of traffic and the cause of failure. For a loss of life linked to a bridge collapse, [Menzies, 1996] proposes a maximum annual probability accepted by society of 10^{-6} (a single death) or 10^{-7} (several deaths). The acceptability of a risk is linked to whether it is voluntary (the individual freely decides to engage in a potentially dangerous activity) or involuntary (control or mastery of risk exposure is beyond the individual's control) [Haldi, 1998] [Schneider, 2000].

In order to use a reliability index in the evaluation of a bridge, it is necessary to specify a **target reliability** index above which an acceptable level of safety is achieved. Three approaches have been pursued to determine this [Shetty, 1999]:

- risk levels accepted by the company based on historical data
- calibration with existing standards
- economic optimization [Nowak, 1996]

The target reliability index must also take into account the type of failure and its consequences. The same probabilistic models used to determine the target reliability index should be used to compare a bridge reliability index with the target index [Stewart, 1999].

The Committee Draft of the future Standard [ISO/CD 13822] lists the fundamental differences between the design of new structures and the assessment of existing structures. It also gives examples of target reliability indices. It distinguishes between serviceability, fatigue and ultimate limit state. Target reliability indices are given as a function of failure consequences.

[Sertler, 1999] recommends target reliability values of between 2.8 and 3.5 for the assessment of existing railway bridges. Values are chosen according to the type of failure and the importance of a bridge element in terms of the consequences of failure.

[Kunz, 1992] takes into account the redistribution of forces and gives target values $\beta_{t,I}$ for the probabilistic assessment of the fatigue strength of a load-bearing element as a function of the number of elements and the target value of the system $\beta_{t,s}$.

[Allen, 1991] suggests using the same semi-probabilistic concept used for the design and evaluation of existing structures. It should take into account the quality and quantity of inspections, potential failure modes and possible consequences. The target reliability index β_t is then adjusted by values $\Delta\iota$ that take these parameters into account.

A more rational approach is to use socio-economic arguments to find target reliability values [Melchers, 1999]. The costs of various possible intervention options are compared: no intervention, reinforcement of the structure or change of use, demolition of the structure and replacement by a new one. Obviously, the results of this approach must be compared with the values accepted by society.

In his book, [Melchers, 1999] also cites the approach of CIRIA (Construction Industry Research and Information Association, London). It proposes calculating the target value for the probability of failure using a formula dependent on the remaining life of the bridge, the average number of people on or around the bridge during this period, and a social factor.

According to [Nowak, 1996], the optimum level of safety depends on the consequences of failure and the costs to safety. It corresponds to the minimum expected cost. The serviceability limit state has a lower level of failure consequences. For this reason, lower values of the target reliability index are chosen for the service state (target value = 1.0) than for the ultimate limit state (target value = 3.5 for an element, target value = 5.5 for the system).

The major problem, however, remains the impossibility of knowing the quality of the target reliability index. Only when a failure occurs is it known that the reliability index was at too low a level. The only solution would therefore be to progressively lower the reliability of a bridge until it fails [Flaig, 1999].

2.3 GUIDELINES AVAILABLE AT ABROAD

The basic idea behind design standards is that the structure should be able to withstand the actions applied to it. The risk comes from the variability of these actions and resistances, which cannot be accurately described. To reduce the risk of failure to an acceptable level, partial factors are applied to the actions and resistances. The values of these factors are given in the standards. Initially, they were based on experience. Later, they were progressively lowered in line with new knowledge [Allen, 1972], in particular to guarantee a certain level of safety (target reliability). Today, there are standards that specify the level of safety required for different types of structure.

The Czech standard for steel construction specifies target reliability index values according to the importance of a building [CSN 73 140].

The Nordic Committee on Building Regulations gives recommendations based on economic optimization [NKB 36]. The target reliability is determined by the consequences of failure and the **nature of the fracture** (brittle or ductile fracture). Lower safety levels are required for ductile fractures because such a mode of failure is accompanied by warning signs.

The Canadian standard CAN/CSA-S6-88: Design of Highway Bridges, in its chapter on the evaluation of existing bridges, provides a procedure for determining load and resistance factors which differs from the rest of the standard [Buckland, 1990]. First, the target reliability index is determined as a function of:

- inspection level (non-inspectable, routine, critical),
- system behavior (influence of failure of one element on other elements),
- element behavior (brittle fracture with no warning signs, ductile fracture with probable warning).

Load factors and resistance coefficients are given according to the target reliability index and bridge traffic.

[Stewart, 1997] gives examples of target values for the safety of civil engineering structures (nuclear power plants in the USA and Great Britain, potentially hazardous industries in Australia and the Netherlands, etc.).

2.4 CONCLUSIONS

As we have seen from the review of the main bibliographical references cited in sections 2.2 and 2.3, there is as yet no definitive answer to the question at hand. Namely, what level of safety should be guaranteed when evaluating existing highway bridges? The following trends can be discerned:

- We want to carry out risk-based structural verification.
- We want to be able to define the target reliability as a function of several parameters (bridge size, time, consequences of failure, etc.).
- Approximation methods such as FORM or SORM are efficient and sufficiently accurate to determine the reliability index β.
- The economic aspect plays a non-negligible role in determining the required level of safety, represented by the target reliability index β_t .

3 BRIDGE FAILURE CASE STUDY

3.1 INTRODUCTION

Civil engineers are called upon to ensure the *safety* and *reliability of* structures. In order to fulfill this task, *hazard identification* is of prime importance. The engineer must then analyze and evaluate the hazards, and decide on the *measures* to be taken to guarantee the required safety and reliability.

In this chapter, 138 bridge failure cases (see appendices A1 and A2: bridge failure cases) have been studied with the aim of :

- learn from real cases
- identify the causes of accidents

This list of 138 failure cases is by no means exhaustive, but we consider the cases analyzed to be fully representative for our study. Among the cases studied, some failures occurred on bridges under construction, others on bridges in service. Our interest in this study lies in the analysis of failures on bridges in service. We have also studied failure cases (or "near misses"), i.e. cases where warning signs of danger have been spotted during inspections and appropriate measures taken to prevent failure.

The failure cases are then listed according to the hazard classification of [Schneider, 1994] (Figure 3.1), which can be described as follows:

- *Accepted hazards* are represented by risks of which the engineer was aware (earthquake, train derailment, etc.), but which were considered acceptable on the basis of a risk assessment.
- *Residual hazards* are due to unknown or undetected hazards such as fatigue damage, dynamic or resonance effects that were unknown at the time the structure was built. These residual hazards may also be due to neglected hazards, such as poor design, lack of monitoring during use, neglect in the face of a significant increase in traffic loads, or in the face of a zone at risk from natural events (scouring, earthquake, wind, etc.).

There may also be hazards which have been identified and considered, but for which the measures taken are inadequate or defective. This is the case for dimensioning errors, design faults or underestimation of certain risks (buckling, warping, delicate construction phases, etc.).

[Schneider, 1994] gives the following classification of hazards (figure 3.1): Hazards can be *accepted*, avoided or reduced by safety *measures.* Non-adapted measures or incorrect application of measures result in *residual hazards.* How this figure is applied to accident analysis is detailed in chapter 3.2.

These various cases of bridge failure are then listed according to their technical causes. These causes, which are of various kinds (scour, earthquake, impact, excessive load, corrosion, fatigue, instability, dynamic effects, dimensioning or design error) can be summed up in two main actions: firstly, the actions of *the natural environment* acting on bridges, and secondly, the *erroneous human handling* to which bridges are exposed in all phases of use. These human errors are due on the one hand to the engineer who has acted negligently or inefficiently, and on the other to the user whose actions have not respected the bridge's intended use.

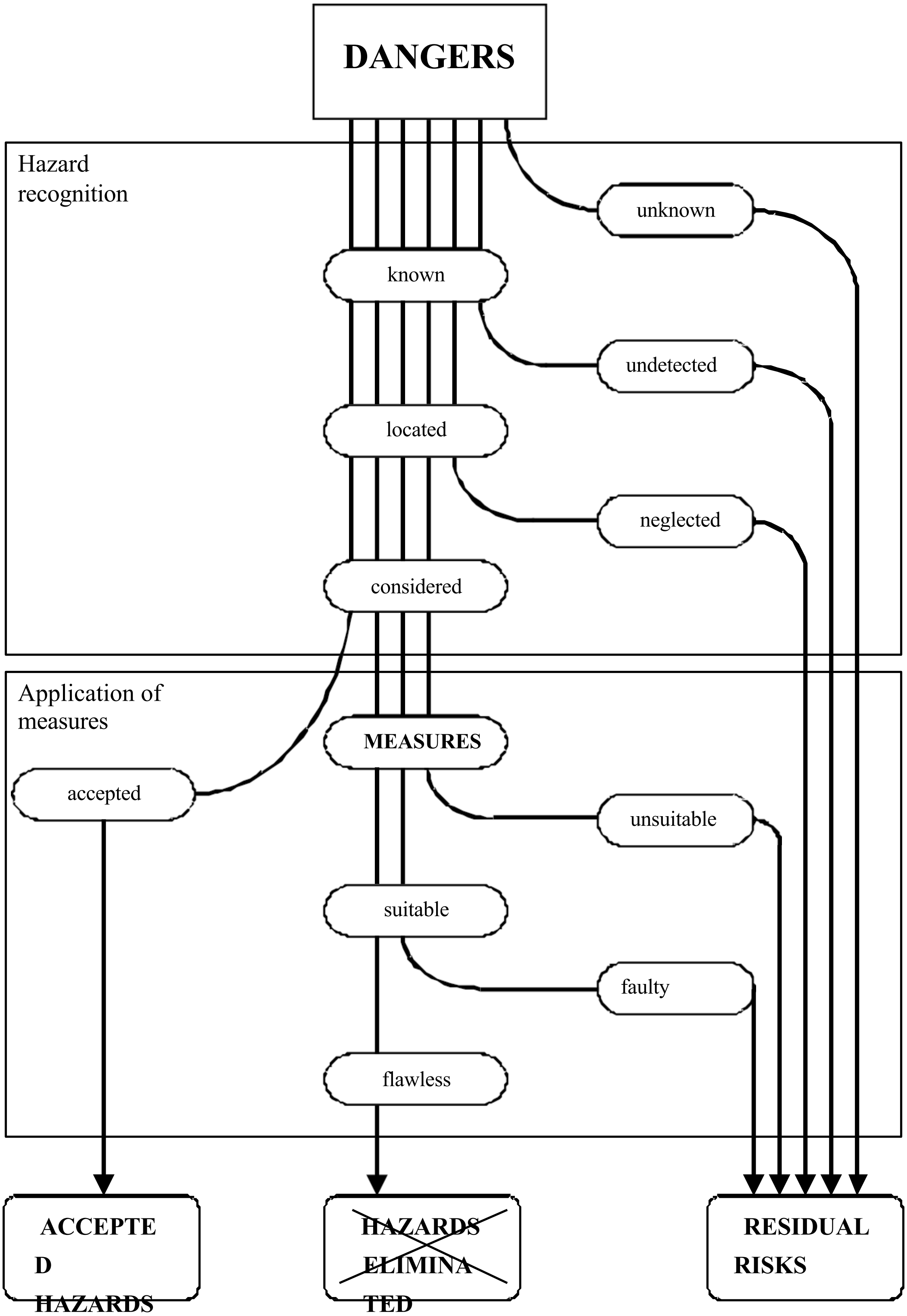


***Figure 3.1** - Hazard classification [Schneider, 1994].*

3.2 CASES OF BRIDGE COLLAPSE

3.2.1 Bridge collapses at construction

Collapses on bridges under construction alone account for 40% of all bridge collapses. In fact, instability and failure are most likely to occur when bridges are pushed or underpinned. In recent years, two bridges under construction in Switzerland have collapsed: The Illarsaz bridge over the Rhône in Valais in 1973, on which steel main girders failed when the concrete deck was being pushed, and the Valangin bridge over the Sorge in the canton of Neuchâtel, also in 1973, where the bridge was being pushed up a slope of over 6%. Fortunately, neither of these collapses caused any loss of life, but they did cause considerable damage.

3.2.2 Damage

Damages are cases where hazards are identified by warning signs during inspections, after which appropriate measures are taken to avoid ruin. However, as in the case of in-service bridge collapses, negligence and errors were made during the design and/or construction of these bridges. A number of such cases of damage have been documented [PIARC, C11 - Committee on Road Bridges, 1999], particularly in Switzerland, where corrosion of reinforcement and prestressing tendons has been identified as the main problem. In most cases, this corrosion is caused by the presence of water contaminated by de-icing salts, which finds its way inside bridges.

As the ideal protection against corrosion has yet to be discovered, this underlines the importance of a *regular monitoring* and *maintenance* of existing bridges.

3.2.3 Bridge collapses in service

In the case of collapses occurring on bridges in service, we can see that a large proportion of these collapses are due to inadequate measures. This points to the involvement of the engineer who, in over 95% of cases, bears a heavy responsibility for bridge collapse (figure 3.2).

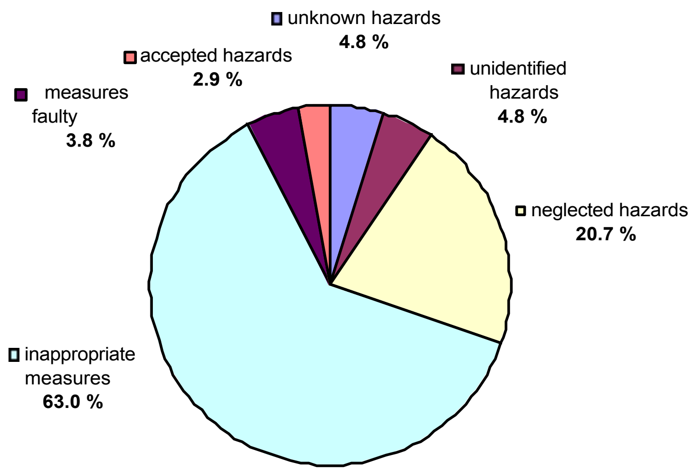


Figure 3.2 *- Hazards and measurements on bridges in service*

3.2.4 Technical causes of bridge collapses at service

Among the bridge collapses in service, it is interesting to make a breakdown according to the technical causes of the collapses, which can be listed as shown in figure 3.3. These different causes are also divided according to the breakdown in [Schneider, 1994], where it can be seen that, in virtually a l l cases, inappropriate measures had been taken. In this figure, we can see that the engineer is largely responsible for the collapse of bridges in service.

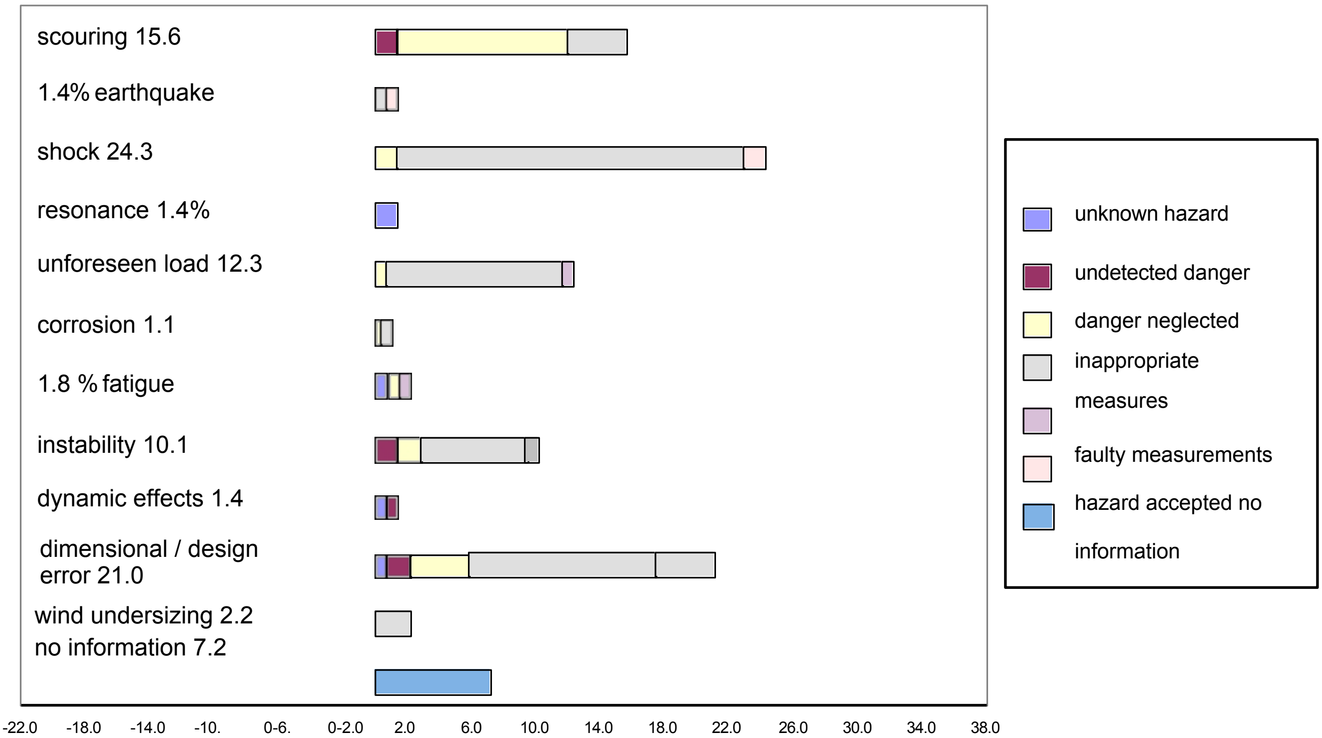


Figure 3.3 *- Technical causes of bridge collapse in service*

It's important to point out that the low proportion of bridge collapses attributed to earthquakes (1.4%) is due to the fact that, in the case of natural disasters such as earthquakes, the total material damage to a town or region is usually described, without any specific description of the damage or collapses attributed to bridges alone. However, it's clear that earthquake-related collapses account for a much larger proportion of the total than shown above.

Collapses attributed to corrosion problems are usually limited to a failure that is detected in time, and for which remedial measures or replacement of reinforcement and prestressing tendons are carried out early enough to prevent collapse. Even if the collapse is actually caused by corrosion, this is often not recognized, as it becomes the triggering element, but is often undetectable at the time of collapse.

Of the technical causes described above, these can be grouped into three categories (figure 3.4), encompassing, on the one hand, exposure of bridges to *erroneous human handling* in all phases of bridge use. These manipulations include, for example, uncontrolled use in relation to forecasts, incorrect operation or misuse, explosions, uncontrolled execution in relation to the planned construction process, inadequate or faulty measures on the part of the engineer during design, dimensioning or construction. This category represents the vast majority of causes of bridge collapse in service, accounting for 72% of cases.

A second category encompasses the actions of the *natural environment* acting on bridges, such as water, snow, ice, wind, earthquakes,.... It should be emphasized that these actions, although natural, does not absolve the engineer of any responsibility to identify these hazards and take appropriate measures to prevent any risk of collapse. These natural actions account for 19.4% of bridge collapses in service.

In addition to these categories, there were of course ten or so percentages of cases where insufficient information was available to define the cause of the collapse.

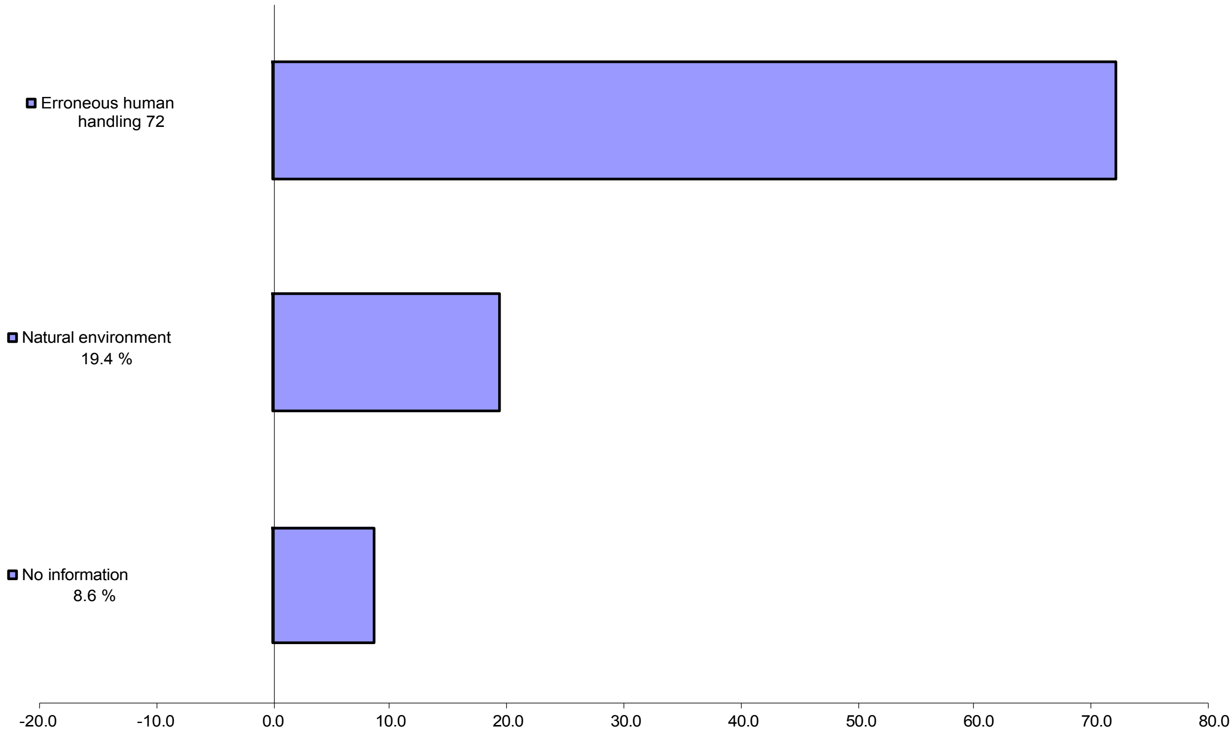


Figure 3.4 - *Causes of bridge collapses in service*

3.3 SYNTHESIS

The bridge collapse case study identifies characteristics and information to provide insight into the level of safety required and the likelihood of bridge collapses:

- Half of all bridge collapses occurred during the construction phase (40%) or during the first two years of service. This means that most defects should already be detected during the project phase, during execution or directly after commissioning. This demonstrates the importance of checks to avoid defects, both during the project phase and during construction. Careful acceptance of the bridge before it is put into service, combined with intensified monitoring during the bridge's initial service phase, is a sensible way of reducing the likelihood of collapse.
- Collapses caused by corrosion and fatigue logically occur at an already advanced age of bridges. This underlines the importance of proper monitoring and maintenance.
- Accidents due to natural causes such as earthquakes, wind or scour generally affect older bridges, for which the hazards had been accepted, not identified or considered by taking inappropriate measures due to lack of knowledge, taken at the time or later. Consequently, to improve the safety of the bridge stock, we need to systematically identify and check bridges with design and dimensioning faults, due to insufficient knowledge at the time of their construction.

- Most bridge collapses (75%) are due to human error on the part of the engineer, or to inappropriate or unforeseen use on the part of the user. Human errors include: ignorance, carelessness, negligence, imperfect knowledge, underestimation of effects, forgetfulness or problems of information flow [Schneider 1994]. This leads us to conclude that, quite clearly, accident reduction is first and foremost a human factor challenge!
- Accepted and objectively unknown hazards account for only 8% of collapse cases.
- No collapse was the result of an inappropriate measure not covered by the standard. This leads us to conclude that the level of safety recommended by the standards is sufficiently high.

These bridge failure characteristics lead to the conclusion that a higher level of inherent safety exists for bridges that meet the following conditions:

- The bridge is designed, dimensioned and built according to current knowledge, and measures have been taken to challenge the human factor.
- The bridge behaved normally during the first years of service.
- The bridge is monitored and maintained accordingly.

In this case, it is justified to adapt the safety level required for a given risk situation for an existing bridge. This specific safety level may be quantitatively lower than that implicitly given in the standards for the construction of new bridges. A methodology for determining the required specific safety level is proposed in the following chapters.

4 RISKS AT COMPANY

4.1 INTRODUCTION

In this chapter, we look at the various risks faced by mankind, and how they are perceived and accepted. We begin with a general study, then attempt to derive values applicable to civil engineering and road bridges in particular.

4.2 STUDIES STATISTICS

Risk is inseparably linked to life. It can be due to natural causes or to the consequences of human activities. The latter can also expose third parties to dangers beyond their control. Generally speaking, it can be said that **there is no such thing as zero risk**.

It's not easy to compare risks. First, we need to agree on how to calculate the probability of occurrence and the consequences of a risky event, as well as on how to view exposure to risk. As far as consequences are concerned, we'll be looking at **causes of death in what** follows.

Numerous statistics are available in the literature. For example, Table 4.1, based on a 1978 study by the American Nuclear Society, gives the estimated probabilities and consequences of the most deadly disasters that actually occurred.

Nature of the disaster	**Estimated probability [per year]**	**Estimated number of deaths (upper bound)**
Earthquake	10^{-3} to 10^{-4}	100'000 à 1'000'000
Flooding	10^{-2}	200'000 à 1'000'000
Raz de marée, hurricane	10^{-2} to 10^{-3}	50'000 à 500
Tornadoes	10^{-2} to 10^{-3}	1'000 à 10'000

Table 4.1 *- Possible disasters according to a study by the Amercian Nucl. Soc. 1978 [Haldi, 1998]*

If we now turn our attention to individual risks, we can compare different causes of death in Switzerland, using the Swiss Federal Statistical Office's (SFSO) Statistical Memento of Switzerland 1999.

Population of Switzerland (1997): 7,096,500

Deaths in Switzerland (1997): 62,839 (0.89% of the population)

Nature of death	Number of deaths	Individual probability [deaths/head/year]
Infectious diseases	905	$1.3 \cdot 10^{-4}$
Tumors	15'047	$2.1 \cdot 10^{-3}$
Diseases of the circulatory system	25'755	$3.6 \cdot 10^{-3}$
Accidents	2'064	$2.9 \cdot 10^{-4}$
Suicides	1'431	$2.0 \cdot 10^{-4}$
Other	17'637	$2.5 \cdot 10^{-3}$
Total	62'839	$8.9 \cdot 10^{-3}$

Table 4.2 - *Causes of death in Switzerland in 1996*

For information, in relation to these values, a probability of 10^{-6} would correspond to 0.011% of deaths, or 7 people dying each year.

More detailed and based on a larger sample (approx. 200 million people), albeit a little old, Table 4.3 gives details of the causes of death due to accidents in the US population in 1969.

Nature of accident	Number of deaths	Individual probability [deaths/head/year]
Vehicles	55'791	$3 \cdot 10^{-4}$
Fires	7'451	$4 \cdot 10^{-5}$
Drownings	6'181	$3 \cdot 10^{-5}$
Poisoning	4'516	$2 \cdot 10^{-5}$
Firearms	2'309	$1 \cdot 10^{-5}$
Machines (1968)	2'054	$1 \cdot 10^{-5}$
Transport on water	1'743	$9 \cdot 10^{-6}$
Air travel	1'778	$9 \cdot 10^{-6}$
Falling objects	1'271	$6 \cdot 10^{-6}$
Electrocutions	1'148	$6 \cdot 10^{-6}$
Railways	884	$4 \cdot 10^{-6}$
Lightning	160	$5 \cdot 10^{-7}$
Tornadoes (average 1953-1971)	118	$4 \cdot 10^{-7}$
Hurricanes (average 1901-1972)	90	$4 \cdot 10^{-7}$
Miscellaneous	8'695	$4 \cdot 10^{-5}$
Total	Approx. 115,000	$6 \cdot 10^{-4}$

Table 4.3 - *Causes of accidental death in the USA in 1969 [Haldi, 1998].*

It is often more meaningful to calculate risks per hour of exposure and per person exposed. This is the case when the risks are linked to a particular activity and only a specific group of the population is concerned. In this case, we can speak of **fatality rates**.

Table 4.4, taken from [Melchers, 1999], shows the approximation of such risks for specific activities. There is a difference of about a factor of 10 between "voluntary" and "involuntary" risks. Risk also depends on the degree of exposure.

Activity	Death rate [death/h. of exposure]	Estimated typical exposure [h/year]	Individual probability for an exposed person [deaths/year]
Mountaineering	3-4-10^{-5}	50	1.5-2-10^{-3}
Boating	1.5-10^{-6}	80	1.2-10^{-4}
Swimming	3.5-10^{-6}	50	1.7-10^{-4}
Cigarette	2.5-10^{-6}	400	1-10^{-3}
Air transport	1.2-10^{-6}	20	2.4-10^{-5}
Car transport	7-10^{-7}	300	2-10^{-4}
Rail transport	8-10^{-8}	200	1.5-10^{-5}
Building work	7-20-10^{-8}	2200	1.5-4.4-10^{-4}
Factory work	2-10^{-8}	2000	4-10^{-5}
Fires	1-3-10^{-9}	8000	8-24-10^{-6}
Collapses of structures*	2-10^{-11}	6000	1-10^{-7}

*Estimated exposure for an average person

Table 4.4 - Risks associated with specific activities [Melchers, 1999].

This table shows just how important the choice of reference unit is. For example, airplanes are generally considered safer than cars. This is true if we compare the annual risks of an average person, but false if we consider the death rate per hour of exposure. It is therefore important to choose the right reference magnitude, depending on the context.

There are also other approaches to risk assessment, such as the FAR (Fatal Accident Rate) developed in Great Britain for occupational activities. This is defined as the average number of accidental deaths recorded in 10^8 hours of exposure to a particular activity (i.e. 1,000 workers for 2,500 hours a year over 40 years).

4.3 RISK PERCEPTION

The analysis of people's perception of risk is an area that has been little studied. We know, for example, that human beings are more impressed by major disasters than by less spectacular but more frequent accidents, even when the risk (probability - damage) is equal.

Factors influencing risk perception include :

- the control that can be exercised over the course of the accident in question
- the extent of the accident (which is given more importance than frequency)
- the seriousness of personal injury
- the spectacular consequences of the accident
- the publicity surrounding the risk in question

- the novelty of the risk (unfamiliarity)
- minimizing future risks that are far in the future
- the difficulty of revising judgments to incorporate new data
- the often erroneous nature of intuitive assessments (tendency to overestimate the reliability resulting from a small number of observations)
- systematically critical or hostile attitudes of certain groups of people towards certain organizations or institutions

Risk perception can also vary according to occupation, level of education, social status, gender and cultural background.

4.4 ACCEPT RISK

The acceptance of risk by individuals and society is influenced by many factors (see table 4.5), the most important being the voluntary or involuntary nature of the risk incurred.

POSITIVE	**NEGATIVE**
Voluntarily assumed	Unintentionally suffered
Immediate effects	Deferred effects
No alternatives	Existence of alternatives
Known danger	Unknown danger
Linked to an essential activity	Linked to an ancillary activity
For specific groups	For everyone
Good use	Misuse
Reversible consequences	Irreversible consequences

Table 4.5 *- Factors likely to affect risk acceptance [Starr, 1976].*

According to a study by [Otway 1970], the population's tolerance of individual annual risks can be quantified schematically as follows:

Individual probability [deaths/head/year]	Characteristic opinion
10-3	This level of risk is unacceptable; as soon as a risk approaches this level, immediate action is taken to reduce it, or the activity in question is discontinued.
10-4	The company commits resources (often public) to p u t in place measures to reduce this risk (e.g. laws).
10-5	Risks of this kind (e.g. fire, drowning, poisoning) only lead to warnings (authorities to citizens, parents to children).
10-6	In principle, risks of this level do not worry the average person, who is aware of their existence but doesn't really feel concerned. They are resigned to such risks, which are similar to those associated with natural elements (e.g. lightning, floods, earthquakes).

Table 4.6 - *Indication of risk tolerance [Otway et al., 1970].*

4.5 RISKS FOR BRIDGES- ROADS

From the previous chapter, we'll try to assign individual probability values to death on a highway bridge, by comparison with other areas. We will establish two bounds: lower and upper. The target reliability value for the evaluation of an existing road bridge will be based on an individual probability of death between these two bounds, according to the approach described in detail in Chapter 5.

4.5.1 Individual probability of dying on a highway bridge: Lower limit

The risk of an individual dying on a road bridge with a probability of **10^{-6} [deaths/inhabitant-year]** can be considered a ***lower limit***. In principle, risks of this level do not worry the average person, who is aware of their existence but does not really feel concerned. They are resigned to such risks, which are similar to those associated with natural hazards. By way of comparison, the annual probability of a person being killed by lightning is $5 \cdot 10^{-6}$. This value of 10^{-6} has already been proposed by other authors, such as [Menzies, 1996].

4.5.2 Individual probability of dying on a highway bridge: Upper limit

We consider that the risk of an individual dying on a road bridge should not exceed that of dying in a car accident. We will therefore take as our ***upper limit*** the corresponding individual probability of **$3\ 10^{-4}$ [deaths/inhabitant-year]**. Note that for values of this order, society generally takes steps (such as legislation) to contain or reduce the risks.

5 PARAMETERS INFLUENCING RELIABILITY

5.1 INTRODUCTION

The aim of the study is not to reduce the overall safety level of the bridge fleet, but rather to ***target a uniform level of acceptable risk****.*

The approach proposed in this report is to thoughtfully define a required safety level as a function of risk situations. The required safety level is thus defined as a function of "external" parameters representing the value and importance of a structure. This required safety level is then compared with the estimated safety, which is calculated using "internal" parameters describing the bridge's condition.

5.2 FUNCTION-RELATED PARAMETERS ("EXTERNAL")

5.2.1 Description of parameters

Civil engineering structures, and bridges in particular, are unique objects. This is why, when assessing an existing bridge, the level of safety required must be adapted to the particular conditions of the bridge in question.

Target reliability depends essentially on three major criteria:

1. the extent of the damage caused by ruin
2. value in use
3. intangible values

5.2.2 Extent of damage ruin

As a bridge is always part of a traffic system, its usage characteristics must be taken into account when assessing the damage caused by collapse. The consequences of a bridge's collapse can most easily be quantified by the number of fatalities it causes. This depends on the intrinsic values of the bridge, such as traffic, geometry, location and mode of failure. As far as traffic is concerned, we need to consider the case of peak traffic and the case of a traffic jam, which give the maximum number of fatalities. The number of people likely to die in the 'peak traffic' risk situation is a linear function of average daily traffic (ADT) and total bridge length. For the 'traffic jam' risk situation, this number depends linearly on the number of lanes and the span.

If the bridge is located in an area where large numbers of people congregate (squares, residential areas, proximity to another road), not only the people on the bridge, but also those underneath it and in its vicinity can be killed.

Figure 5.1 shows the parameters influencing the number of fatalities in a bridge collapse.

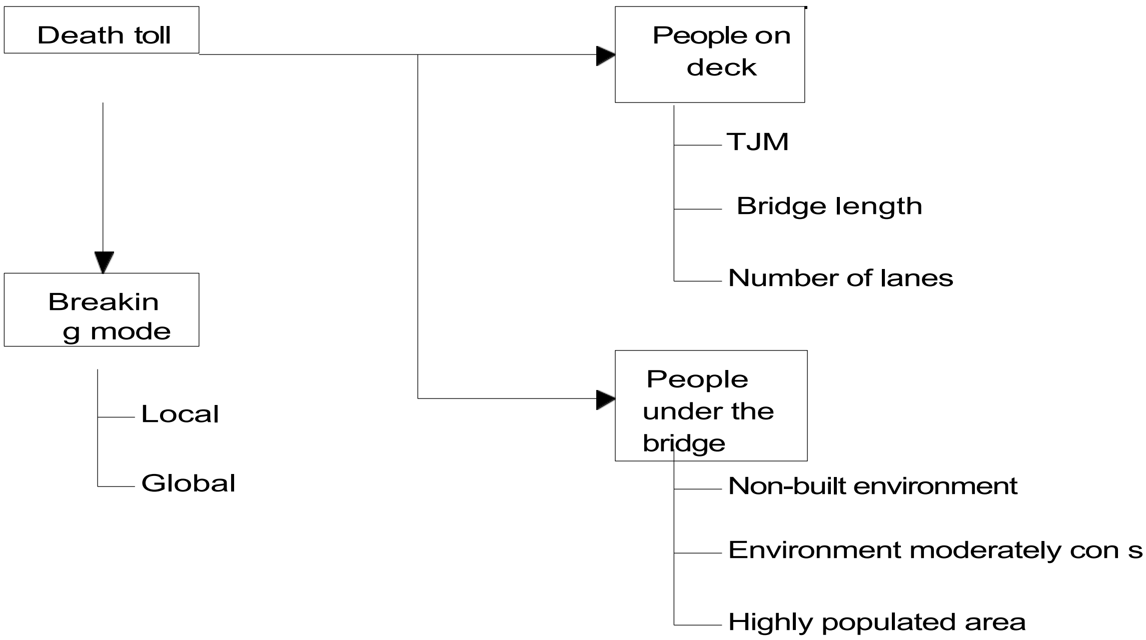


Figure 5.1 *- Parameters influencing the number of fatalities (ADT: Average Daily Traffic)*

The number of fatalities depends directly on the risk situation, so it includes not only the people who were on the bridge when it collapsed, but also all those who died as a result of the ruin (e.g. cars driving at night who didn't see that the bridge had collapsed).

To determine the probability of failure corresponding to the criterion 'damage following ruin', a certain number of acceptable deaths is chosen, and depending on the parameters influencing it, we find the probability of failure. We take the most severe case (with a failure probability of 10^{-6}) and adapt it to our risk situation. All the parameters listed in figure 5.1 are cumulative. The sum of the fatalities for each parameter gives the total number of fatalities.

Target failure probability limits

Let's recall the limits we accepted in Chapter 4 for the individual probability of dying on a highway bridge, by comparison with other fields:

- lower limit: 10^{-6} [deaths/capita-year].
- upper limit: $3 \cdot 10^{-4}$ [deaths/capita-year]

Definition of the most serious case

We propose the following specifications to describe the most severe case. The ruin of the entire structure is accepted, as it results in the maximum number of fatalities.

Probability of failure p_f	:10^{-6}
deaths/year TJM	90,000 veh/d
Bridge length	:1000 m
Number of lanes	: 4
Breaking mode	: Collapse
Bridge location	:Moderately built-up environment

When assessing a specific bridge, we adapt the target probability of the worst case to take into account the parameters specific to the bridge in question.

Bridge location

The location of the bridge influences the number of fatalities due to bridge collapse. A medium-sized bridge may cause the same number of deaths as a large bridge if it is located in a densely populated area (crossing a heavily trafficked road, crossing places where large numbers of people gather [public squares, hospitals, schools, shopping centers, etc.]). A moderately built-up area could be that of a bridge crossing a low-traffic road or a residential area. This is taken into account in the classification criteria.

Breaking mode

The failure of a bridge element (punching of a slab, failure of a girder in a multi-girder bridge, failure of an overhang) has far less serious consequences than the ruin of the bridge (the entire bridge collapses following the complete failure of a section).

Target failure probability

Given the values of the parameters corresponding to the risk situation to be assessed and the acceptable number of fatalities, it is possible to determine the probability of failure.

With the lower limit of target probabilities and a population of $7 \cdot 10^6$ in Switzerland, there are 7 deaths per year due to bridge failure. If we assume 10 hours of annual exposure to the risk of bridge failure (on average), we find a FAR (Fatal Accident Rate) of 10.

Taking into account the number of bridges, the annual exposure to the risk of bridge failure and the assumption that the failure of the bridge considered as the most serious case causes 600 deaths, we find the relationship between the target probabilities and the number of deaths illustrated in figure 5.2.

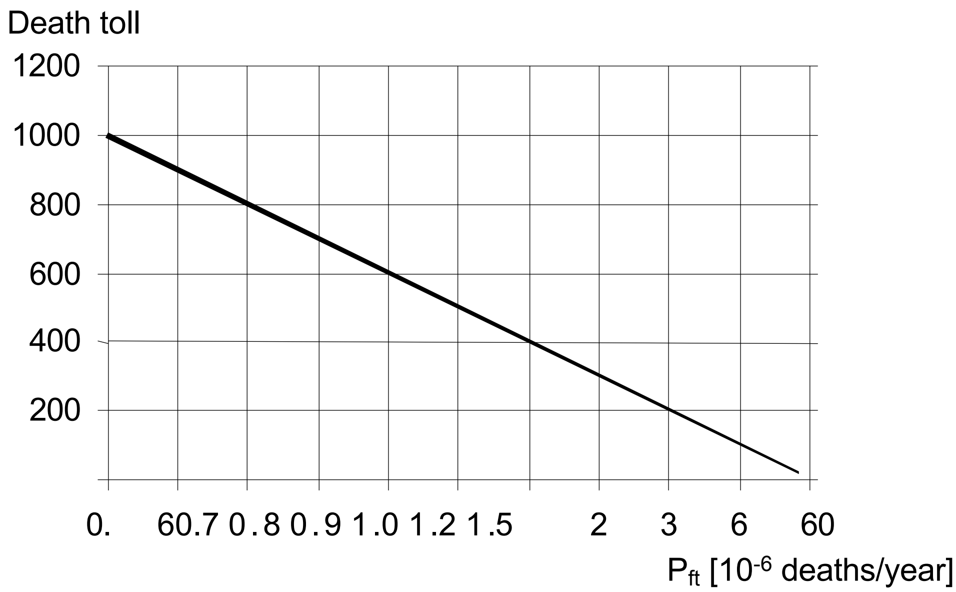


Figure 5.2 *- Relationship between target probability and number of fatalities (rounded values)*

The target probabilities given in table 5.1 are defined on the basis of the relationship illustrated in figure 5.2. These are used to define the risk categories presented in chapter 6.

Death toll probable	**Target probability**
<1	10^{-3}
1	$5 \cdot 10^{-4}$
5	10^{-4}
10	$5 \cdot 10^{-5}$
50	10^{-5}
100	$5 \cdot 10^{-6}$
500	10^{-6}

Table 5.1 *- Target probabilities as a function of the number of fatalities*

The number of traffic lanes has little influence on the number of fatalities. Indeed, going from the most serious case (e.g. 6 lanes) to 2 lanes only reduces the number of fatalities by a factor of three.

Similarly, the mode of failure has little influence on the risk category. For example, if one of the 6 girders of a multi-girder bridge fails (fracture), there are only 6 times fewer fatalities than in the most serious case (caisson failure). To this end, we distinguish only between rupture (one element) and ruin (structural failure of the whole), with no intermediate state.

5.2.3 Value of use

The use value is the value of a bridge in the context of the road network and the importance of the section. It can be determined by judging feasibility, the costs of construction or operating measures, and the user costs incurred by bridge failure. For this purpose, cost-benefit analyses (economic optimization) are carried out.

The cost-benefit analysis in [Diamantidis, 2001]) optimizes the Z(p) function:

$$Z(p) = B(p) - C(p) - D(p) \tag{5.1}$$

With

B(p) profit due to the existence of the

structure C(p) construction cost

D(p) expected cost of ruin

p vector including all parameters controlling costs and reliability

The theory of decision statistics dictates that the averages of B(p), C(p) and D(p) should be taken for the calculations. For all parties involved (engineer, client and user), Z(p) should be greater than 0. Benefits and costs are not necessarily the same for all parties.

Several cost-benefit analyses have been carried out [Diamantidis, 2001]. The conclusions can be summarized in a table containing optimal ruin probabilities. Table 6.2 (adapted from [Diamantidis, 2001]) shows the CR risk category$_U$ as a function of the consequences of ruin and the relative costs of safety measures. The rasterized category should be considered as the most usual (category V corresponds to a probability of ruin of 10^{-5}). This result is less conservative than the usual target reliability values, but the difference with the latter is not too great. In the Eurocode, for example, we find a probability of failure of $0.7 \cdot 10^{-5}$ for a reference period of 50 years, which corresponds to an annual probability of $7 \cdot 10^{-5}$ (total dependence) to $1.2 \cdot 10^{-6}$ (total independence).

The target probabilities in Table 5.2 depend on the parameter ρ, which is defined as the ratio between the costs of ruin and the costs of construction: $\rho = C_{ruine} / C_{const}$. Ruin costs include the cost of rebuilding the bridge or element and the cost of loss of life. Typical examples for the different classes are mountain bridges, agricultural structures or masts for the minor consequences class; cantonal road bridges outside towns, offices, industrial buildings and apartments for moderate consequences and large freeway bridges, theaters, hospitals and large buildings for major consequences. For ρ values greater than 10, and especially if the absolute value of C_{rupt} is also large, the consequences must be considered extreme, and a full cost-benefit analysis is recommended. Intervention costs include everything necessary to avoid bridge failure. For low intervention costs and high consequence costs, a higher reliability of the bridge may be required, and thus a higher risk category.

	Consequences		
Relative costs of safety measures	Minors $\rho < 2$	Moderate $2 < \rho < 5$	Major $5 < \rho < 10$
Large	10^{-3}	$5 \cdot 10^{-4}$	10^{-4}
Normal	10^{-4}	10^{-5}	$5 \cdot 10^{-6}$
Small	10^{-5}	$5 \cdot 10^{-6}$	10^{-6}

Table 5.2 *- Target probabilities corresponding to the 'value in use' criterion (adapted from [Diamantidis, 2001])*

The values given in Table 5.2 apply to a system. If the analysis is carried out at element level, the same values can be used, provided that system failure is dominated by element failure. In general, in such cases, the target probabilities will decrease, as the relative costs of failure for an element are greater than for system failure. The costs of failure of an element can be low only for structures with high redundancy. The categories in Table 5.1 are given for structures or elements at the design stage (not at the construction stage). Ruin due to human error or ignorance and ruin due to causes unrelated to the structure are not covered by this table.

The relative costs of safety measures depend above all on the variability of loads and resistances. The 'normal cost' class is associated with medium variability ($0.1 < V < 0.3$). It is interesting to note that the greater this variability (and therefore the relative costs of measurement), the greater the target probability. The Committee Draft of standard [ISO/CD 13822] also specifies p values_t for serviceability and fatigue.

5.2.4 Values intangible

In addition to the criteria of economic value, each work has certain intangible values. These consist of various aspects, which we will examine in this chapter. They must be considered from the point of view of both current condition and future potential. The Swiss Federal Roads Office has published a guideline for assessing the conservation value of engineering structures [FEDRO, 1998].

The value of target reliability is a consequence of *safety requirements* alone, whereas intangible values are assessed by *society*. Intangible values therefore have no influence on target reliability. Rather, they play a role in the (re-)definition of the utilization plan, or in the choice of the type of intervention, if any.

This can be illustrated by a fictitious example: suppose we have a historic bridge with significant intangible value. If this high intangible value were to lead to an increase in target reliability, we might then be forced to carry out structural interventions on the structure to meet reliability requirements. Such interventions would then have a negative impact on the preservation of this historic bridge in its original state, which would ultimately defeat the original purpose. What's more, the non-modification of the target reliability by virtue of the intangible criteria might not have led to any intervention at all!

It is the engineer's responsibility to reflect on the intangible values of the structure he is studying, in order to apply the most appropriate solutions when choosing an intervention. The following intangible values should be considered:

5.2.4.1 Historical-cultural value

The cultural-historical value of a structure derives from its position within the economic, political or social development of an era. As a representative of a certain way of building and a witness to a technical development, a structure refers to a certain cultural era: the era of reinforced and prestressed concrete in the construction of the National Road network in the 2ème thirds of the XX^{ème} century for most of the bridges considered here.

We can also mention the work's relationship to a famous builder.

Cultural-historical value therefore goes beyond purely stylistic value.

5.2.4.2 Aesthetic value

The aesthetic value of a work is the result of its architectural and artistic qualities, the composition and form of its structure, the particularities of its style and the aesthetic application of its materials.

Public opinion on aesthetic value sometimes varies from one generation to the next.

The aesthetic quality of construction details can have a major influence on the overall impression.

5.2.4.3 Technical value

The technical value of a structure lies in the materials used in its construction and in its design features. In the case of bridges on national highways, the main features are :

- unique, daring, innovative or pioneering buildings and structures
- quality and uniqueness of materials and techniques used
- unmistakable character

5.2.4.4 Socio-cultural value

The socio-cultural value of a structure results from its readiness to be used by groups of people linked by their profession, society, age, origin or for specific public purposes. In the case of national highways, for example, we think of the socio-cultural value for the regions served by the network.

5.2.4.5 Emotional value

Emotional values encompass aspects such as affective value, prestige, agreement with the personal principles of the builder, users or local residents.

Emotional values can be decisive when making decisions. Everyone involved has specific preferences and prejudices for or against the conservation of structures.

5.2.4.6 Situation value

The situational value of a structure reflects its spatial interaction with its environment (delimitation of space, separation of territory, striking appearance). Aesthetic landmarks play a secondary role here.

Structures are landmarks. They mark the environment, facilitate orientation and help to identify the place.

5.2.4.7 Image value

This value is somewhat different from other intangible values, in that it has more to do with the owner than with the structure itself. By image, we mean the opinion that the general public, and in particular bridge users, may form of the administration in charge of bridges, and of the bridge itself.

the entire civil engineering profession. Typically, a ruined bridge or, to a lesser extent, maintenance or repair work that causes a nuisance will be detrimental, as will a structure that does not inspire confidence.

5.3 STRUCTURE-RELATED PARAMETERS ("INTERNAL")

5.3.1 Introduction

The aim of this section is to give an introduction to the aspects to be considered in finding the actual reliability with respect to a given risk situation. The actual reliability is compared with the target reliability in order to decide whether the level of safety is adequate when probabilistically assessing an existing highway bridge. This section presents an overview of probabilistic concepts and approaches, with reference to other sources of information. Aspects to be considered when assessing the actual reliability are as follows:

- uncertainties related to base variables
- reliability of structural systems
- inspection and monitoring of structures

5.3.2 Uncertainties related to base variables

5.3.2.1 Introduction

A structure is evaluated taking into account the uncertainty associated with its condition, use and exposure. This uncertainty is described by basic variables such as the dimensions of a structure, the properties of materials and the magnitude of actions.

For a deterministic assessment, the basic variables are described by representative values and partial factors. The representative values, partial factors and models applied provide conservative efforts to take account of the high degree of uncertainty when dimensioning a structure.

For probabilistic evaluation, we consider the probability density of a base variable, represented, for example, by the mean and standard deviation for a given probability distribution. Two values are useful for representing a basic variable:

- the bias, represented by the ratio between the mean and the representative value.
- the coefficient of variation, represented by the ratio between the standard deviation and the

mean. The uncertainty associated with a base variable is represented by its coefficient of variation.

5.3.2.2 Sources of uncertainty

Sources of uncertainty are due to various causes:

- intrinsic variability, such as concrete density, which is difficult to reduce and varies over time and/or space.
- estimation errors when data are incomplete, invalid or too general (e.g. wind action on a bridge). It also happens that the source of the data does not correspond to the case in question. In all these cases, the error, and therefore the uncertainty, is reduced by increasing the data and/or taking measurements on site.
- an imperfection in the mathematical models used to represent reality, such as a poor distribution of load effects in a bridge due to a lack of knowledge, or

by using a simplified model. Here again, on-site measurements reduce error and hence uncertainty.

- human error during design, construction or operation. In such cases, the error, and therefore the uncertainty, is reduced by quality assurance, on-site measurements or protective devices.

It is therefore important to recognize the sources of uncertainty in order to identify ways of reducing them. The importance of the different sources of uncertainty varies according to the type of load.

The uncertainty associated with permanent loads is a function of the dimensions and density of the elements in a structure. For reliability analysis, the permanent actions can be represented by a normal law. The uncertainty associated with permanent loads, at the time of design, is given for each source in [Bailey 1996 and Diamantidis 2001].

For a deterministic analysis, we consider a representative value (mean/nominal) and a partial factor. For target reliability, the partial factors vary according to the importance of the permanent load in a limit function, as does the coefficient of variation.

5.3.3 Reliability of structural systems

5.3.3.1 Introduction

Bridges are made up of several elements, and therefore represent a 'structural system'. The reliability of a structural system is a function of the reliability of its elements, for the following reasons:

- Loads and resistances can be dependent (e.g. loads can be section-dependent and resistance can be a function of previously applied loads).
- There may be a correlation between the properties of elements (e.g. ultimate strength and stiffness) located in different parts of the bridge.
- If one element has reached its ultimate strength, it doesn't mean that the whole bridge has. There will be a redistribution of forces, and another element will come to the aid of the first to take on part of its load (=> redundancy).
- There are limit states that apply to the whole system, rather than to individual elements (e.g. foundation settlements, total deflection).

Even in conventional deterministic analysis, the structural system is simplified. For example, in a lattice structure, elements are idealized by their center of gravity, connections are points and critical sections for stress control are predefined locations of a limited number. Ruin of a structural system can be defined in a number of ways:

- maximum stress reached everywhere
- (plastic) failure mechanism formed (therefore rigidity = 0)
- limit rigidity reached
- permissible deflection achieved
- cumulative damage limit value reached (e.g. in fatigue)

5.3.3.2 Structural analysis methods

Analysis of structural systems is facilitated by simplified modeling of loads, loading sequence, static system and material characteristics. When dimensioning many structures, the extreme values (envelopes) have been obtained by *elastic calculation*. General methods for finding the *ultimate load are* based on the two fundamental theorems of limit analysis, which can be used to find approximate values of the solution when not all the following conditions are met [Frey, 1994]:

- statics (balance)
- kinematics (compatibility) and
- constitutive laws (elasto-plastic)

The static method provides a lower value for the ultimate load. This method is based on the static theorem of plasticity theory, which states that any load to which a statically permissible distribution of internal forces corresponds is less than or equal to the actual limit load.

The kinematic theorem provides an upper value for the ultimate load, found for a structure that transforms into a mechanism made up of rigid parts and plastic hinges. The latter may form where the stress/strength ratios are greatest. The second theorem of plasticity theory states that any load at which a kinematically permissible failure mechanism is present is greater than the ultimate load. The application of this principle is generally very simple and elegant, but it overestimates the ultimate load. This is why, for design purposes, the combination of decisive loads (risk situation) is generally determined using the static ultimate load calculation method.

When *dimensioning* structures according to the theory of plasticity, it is best to use the static method, which provides a lower bound for the ultimate load, i.e. a result on the side of safety. When *assessing* existing structures, the kinematic theorem is often used, as it allows the load-bearing capacity of a structure to be exploited to the full. However, the ultimate load may be overestimated.

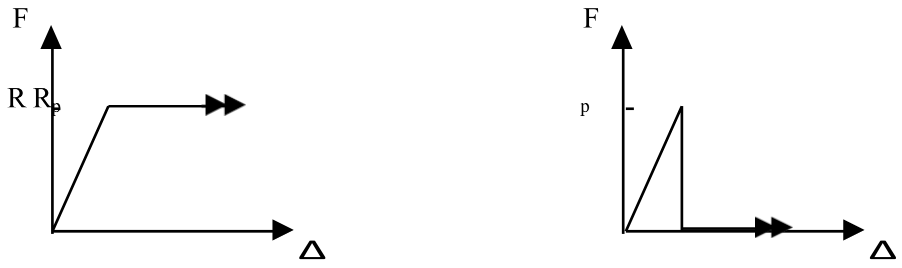
Table 5.3 summarizes the different methods used to verify a structure. For the service condition, actions and resistances are calculated according to the theory of elasticity. For the ultimate state, resistance is established using plasticity theory, and forces can be determined either elastically or plastically.

	Theory of elasticity		**Plasticity theory**	
Service status	S	R		
Ultimate state		S	R	
			S	R

Table 5.3 *- Analysis methods (S: stress, R: section strength)*

5.3.3.3 Redundant structural systems

Because of its complexity, the behavior of materials in structures is usually simplified. *Redundant* structures (with redistribution of forces), such as a beam on three supports, can show two types of behavior depending on the type of failure (fig. 5.2). Hyperstatic structures are typically redundant systems.



(a) ductile fracture *(b) brittle fracture*

Figure 5.2 - *Behaviors of a redundant system element*

For ductile fracture, the behavior is elasto-plastic. Once plastic strength has been reached at one point, the load no longer increases. Displacements continue to increase under constant load. This behavior allows the elements of the system to remain at maximum stress while deforming (fig. 5.2(a)). Because of redundancy, brittle failure of an element does not necessarily lead to the ruin of the system. The behavior of an element can be modeled by elastic-brittle behavior. For this type of behavior, deformation can be found with zero load, even after the maximum load has been reached (fig. 5.2(b)).

Non-redundant structures (without redistribution of forces), such as a simple beam, behave differently:

(a) ductile fracture *(b) brittle fracture*

Figure 5.3 - *Behaviors of an element in a non-redundant system*

In the case of ductile failure, the behavior is elasto-plastic, but with a much shorter rigid-plastic part. Without redundancy, forces cannot be taken up elsewhere, and the structure collapses soon after the strength of one element has been reached. Brittle failure is rather similar: the structure collapses immediately when the maximum load is reached. This is modeled by purely elastic behavior.

Table 5.4 summarizes the collapse of different systems for brittle or ductile failure of an element. It shows that failure of a non-redundant system does not announce itself. A lower probability of collapse must therefore be imposed, as measures cannot be taken to prevent the death of people.

	Redundant system	**Non-redundant system**
Ductile fracture	Gradual collapse	Almost instantaneous collapse
A fragile break	Gradual collapse	Instant collapse

Table 5.4 - *Type of structural collapse*

The above considerations of brittle or ductile failure should be used with caution. In fact, the behavior of accepted elements assumes controlled displacement loading. If force control is imposed, even an element with elasto-plastic behavior may fail suddenly (and therefore brittle!). What's more, in the bridge field, the predominant action is the payload (traffic). In the ultimate state, behavior is therefore controlled by force, and consequently brittle failure is always observed when the system reaches its limit load. On the other hand, in order to reach its limit load, a certain ductility is required to form plastic hinges.

Bridges should be designed with a minimum level of **redundancy, so** that the failure of one element does not necessarily lead to the ruin of the entire system. Redundancy is defined as the bridge's ability to resist loads after the failure (or damage) of a bridge element. Fragile or ductile failure can occur. The reasons for such failure may be the application of high live loads, sudden loss of an element after brittle failure, or an accident.

Redundancy is particularly important for bridges with several parallel girders. If one of these girders fails, the loads will be transferred to the other girders, insofar as the construction permits; there will thus be a redistribution of forces. In Switzerland, systems with a high degree of redundancy are commonplace, such as multi-girder bridges. This redistribution also occurs in the longitudinal direction: when the ultimate span resistance is reached, there is redistribution to the supports and vice versa. But this longitudinal redundancy is less important.

5.3.3.4 Analysis of redundant structural systems

Once all the different possible modes of failure have been identified, the events (failure of elements or in a section) contributing to this failure can be listed systematically with a *cause tree* or a *consequence tree* [Haldi, 1998][Melchers, 1999].

For the cause tree, the procedure is to take each failure event and break it down into sub-events, which are also broken down. The lowest sub-events in the tree correspond to element or section failures.

The operation of a system can also be modeled using a *success diagram* [Haldi, 1998] [Schneider, 1994]. This analysis consists of modeling the system by breaking it down into blocks, representing elements, subsystems or functions, and specifying the links between these blocks. Blocks representing components whose failure alone is sufficient to cause system failure are connected in series. Blocks representing components whose failure alone is sufficient to cause system failure are connected in series. Blocks representing components whose simultaneous failure is sufficient to cause system failure are connected in parallel.

Figure 5.4 shows an example of the success diagram for an embedded-supported beam.

(a)

(b)

Figure 5.4 - *Success diagram of an embedded-supported beam [Schneider, 1994].*

In this example, the plastic moment at A is strongly correlated with that at B. For total system failure, we need to reach the plastic moment at A AND the plastic moment at B (after reaching it at A) OR vice versa.

The embedding moment at A is M_A = -3/16FL according to an elastic calculation. So, for a span L of 10 m: M_A = -1.875F. The condition for achieving the plastic moment at A is as follows: G_A = R - 1.875F < 0. Using software such as VaP, and knowing the statistical parameters of resistance and load, we can calculate the probability of failure p_{fA} of element A: p_{fA} = P (G_A < 0). As for the system, it fails only when element B also reaches its plastic strength (after A has already reached it). Elements A and B are therefore "connected" in parallel (see right-hand side of Fig. 5.4b). We need to determine the conditional probability p_{fBA} . With M_A = -R, the bending moment at B is M_{BA} = FL/4-R/2. The corresponding failure condition is G_{BA} = R- FL/4 + R/2 < 0. From this we derive the probability of failure of element B, given that the resistance at A has already been reached, p_{fBA} . The probability of failure of a system composed of elements in parallel is no greater than the probability of failure of the most reliable element. So if p_{fA} is greater than p_{fBA} , then the probability of failure of path A (left-hand side of Fig. 5.4b, ruin beginning with element A) is p_f (A) [p_{fBA} . Path A is one of two possible paths. For path B (right-hand side of Fig. 5.7b), the plastic moment is reached first for element B and then for element A. The probability of failure of the entire system can be deduced from the two paths A and B in series, and is therefore the sum of p_f (A) and p_f (B).

In the previous example, we assumed that the element that failed remains active, i.e. continues to support the plastic moment. For brittle (instantaneous) failure, this is no longer possible. In this case, the resistance of the element in question is reduced to zero (see fig. 5.2b)). The static system is considerably altered, with M_{BA} = FL/4. *The probability of failure is therefore much higher than in the case of ductile behavior (by deformation).*

5.3.3.5 Probabilistic analysis methods

When we calculate reliability, we take into account the variability of actions and resistances. The software used (e.g. VaP [VaP, 1996]) allows parameters (loads, elastic limits, geometries, etc.) to be introduced in the form of a statistical distribution (bias, C.O.V.). It is also possible to introduce the effects of time (corrosion, fatigue) by decreasing resistance values over time. Several deterioration models are given in the literature ([Ciampoli, 1998] [Roelfstra, 1999] [Kunz, 1992],...).

Unfortunately, it is not always possible to describe the limit function G(x) by one or more explicit limit equilibrium equations. This means that it can only be defined by trial and error, for example by repeated numerical analysis with different starting values. These values can be random, as in a Monte Carlo analysis, or in a specific order. In any case, it is clear that methods like FOSM cannot be applied directly, as they require an explicit, preferably derivable, form for the limit function. Such a form can be artificially created using a polynomial or other function tailored to the results obtained from a limited number of discrete numerical analyses. These 'response surfaces' approximate the responses of the structure in the vicinity of the design point, with a poorer match elsewhere. If this response surface approximates the system response well, a good estimate of reliability can be expected.

The failure modes of a structure are not always known. They can be established by methods such as Monte Carlo simulations. Of particular interest are the failure modes that have the greatest influence on the probability of system failure. The decisive load cases must be selected. For complex systems with multiple loads, the critical limit states may differ according to the loading sequence. To date, there is no known method for solving these systems.

5.3.4 Monitoring

It is important to note that the probability of failure of a structural element p_f can be expressed as follows:

$$p_f = p_{ruine}\,(1 - p_{dét}) < p_t \qquad (5.2)$$

with :

p_{ruine} :calculated probability of ruin

$p_{dét}$:probability of detection of unexpected action or damage reducing the strength of the structural element

p_t :(acceptable) limit value for the probability of ruin

Equation (5.2) shows that the probability of failure can be reduced by increasing monitoring and thus the probability of detecting an unexpected action or damage reducing the resistance of a structural element.

6 SAFETY LEVELS REQUIRED

In this chapter, the target reliability index β_t is used to determine the level of safety required for a given risk situation. Chapter 5.2 (External parameters) gives more details on the parameters influencing target reliability.

The approach proposed by the present research to establish the required level of safety is as follows: Starting from a *risk situation*, we determine the *risk category* related to the magnitude of the damage following ruin, as well as that related to the value in use. The highest risk category is decisive for the risk situation in question.

Knowledge of the risk category is used to determine the required safety level. This level is characterized by the target failure probability p_t or the *target reliability* index β_t. A different failure probability will therefore be established for each risk situation analyzed.

As the *'intangible values'* criterion is not directly quantifiable, the engineer will make a qualitative judgment of its importance in the assessment or intervention frameworks for establishing sufficient safety.

Figure 6.1 shows how to find the target failure probability p_t corresponding to a risk situation.

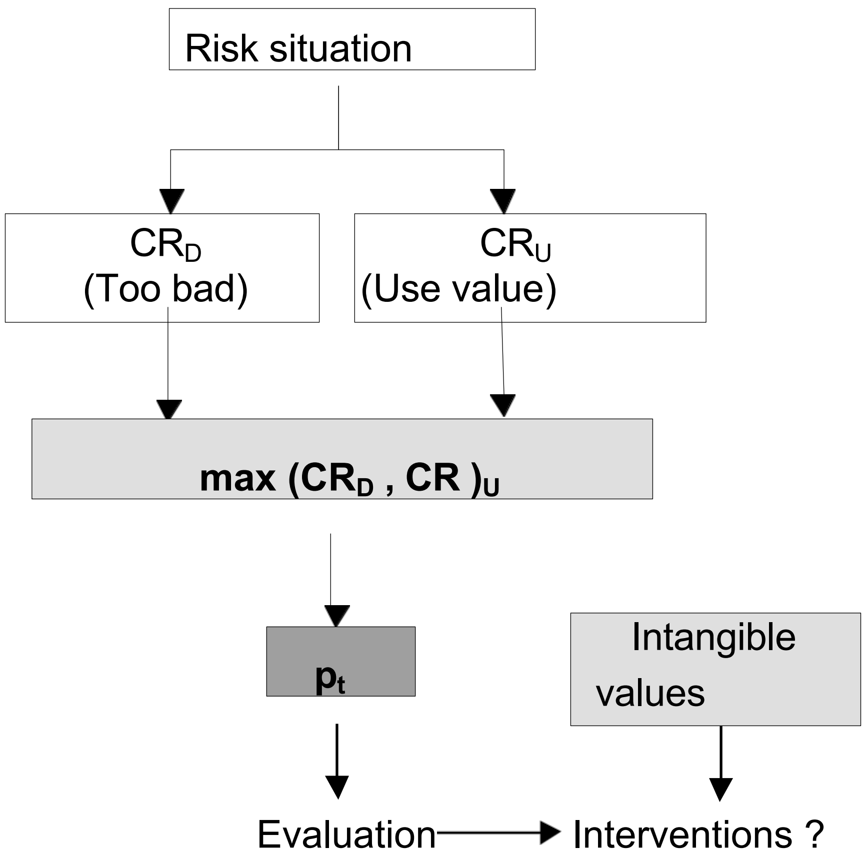


Figure 6.1 *- Procedure for determining the target failure probability p_t corresponding to a risk situation (CR: risk category)*

The following paragraphs are presented in "how to" format.

6.1 SITUATION RISK

The safety level is determined for a given risk situation. The term ' risk situation' (e.g. collapse) corresponds to the definition given in SIA 160 (§2.22) and should not be confused with the term 'cause of accidents' (e.g. scouring).

6.2 RISK CATEGORY

The CR risk category corresponding to the risk situation under consideration is the **maximum value of** the risk categories related to damage CR_D , respectively use CR_U .

$$CR = max\ [CR_D\ ,\ CR\ ]_U \qquad (6.1)$$

with :

CR risk category

CR_D :risk category related to the extent of damage following ruin

CR_U :risk category related to value in use

Figure 6.2 shows how the CR risk category is established to determine the required safety level.

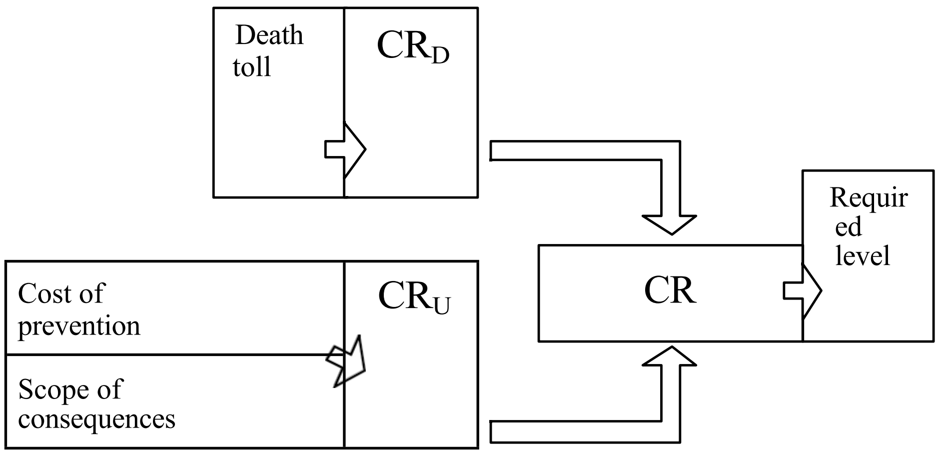


Figure 6.2 - *Determining the risk category*

Tables 6.1 and 6.2 show the risk categories for fatalities (CR_D) and consequences (CR_U) respectively. The tables are based on tables 5.1 and 5.2 respectively.

Death toll probable	**Risk category CR_D**
<1	I
1	II
5	III
10	IV
50	V
100	VI
500	VII

Table 6.1 - *CR damage risk category$_D$ by number of fatalities*

The consequence classes in Table 6.2 depend on the parameter ρ, which is defined as the ratio between the cost of ruin and the cost of construction: ρ = C /C$_{\text{ruineconst}}$ (see section 5.2.3).

	Consequences		
Relative costs of safety measures	Minors $\rho < 2$	Moderate $2 < \rho < 5$	Major $5 < \rho < 10$
large	I	II	III
means	III	V	VI
small	V	VI	VII

Table 6.2 - *CR risk category$_U$ corresponding to the 'value in use' criterion (adapted from table 5.1)*

The values given in Table 6.2 apply to a system. If the analysis is carried out at element level, the same values can be used, provided that system failure is dominated by element failure. In general, in such cases, the target probabilities will decrease, as the relative costs of failure for an element are greater than for system failure. The costs of failure of an element can be low only for structures with high redundancy. The categories in table 6.2 are given for structures or elements at the design stage (not at the construction stage). Ruin due to human error or ignorance and ruin due to causes unrelated to the structure are not covered by this table.

6.3 RELIABILITY INDEX TARGET

Table 6.3 shows the correspondence between the risk category and the target annual failure probability p_t, respectively the target annual reliability index β_t . The risk category CR is the largest value of the risk categories related to the damage CR_D and the use value CR_U .

Risk category CR	Target probability p_t	Target reliability β_t
I	10^{-3}	3.1
II	$5 \cdot 10^{-4}$	3.4
III	10^{-4}	3.7
IV	$5 \cdot 10^{-5}$	4.0
V	10^{-5}	4.2
VI	$5 \cdot 10^{-6}$	4.4
VII	10^{-6}	4.7

Table 6.3 *- Annual target probabilities and reliabilities by risk category*

6.4 NOTE FINAL

The proposed approach makes it possible to define the required level of safety, represented by a target reliability. This reliability must be compared with the actual reliability in the face of the risk situation to be verified. The target reliability cannot be used directly in a semi-probabilistic verification, which would require updating the partial factors (according to the target reliability) for the risk situation to be verified. The definition of the updated partial factors according to the target reliability should be done in an additional study.

7 EXAMPLES OF APPLICATIONS

7.1 INTRODUCTION

The aim of this chapter is to give several examples of the application of the method for selecting the required safety level described in chapter 6. This safety level is to be compared with the actual reliability for a given risk situation. The actual reliability of the structures is not calculated. The examples are based on risk situations for three bridges on the A1 freeway between Geneva and Lausanne (Figure 7.1):

- Perroy underpass
- Coude overpass
- Bridge over the Aubonne

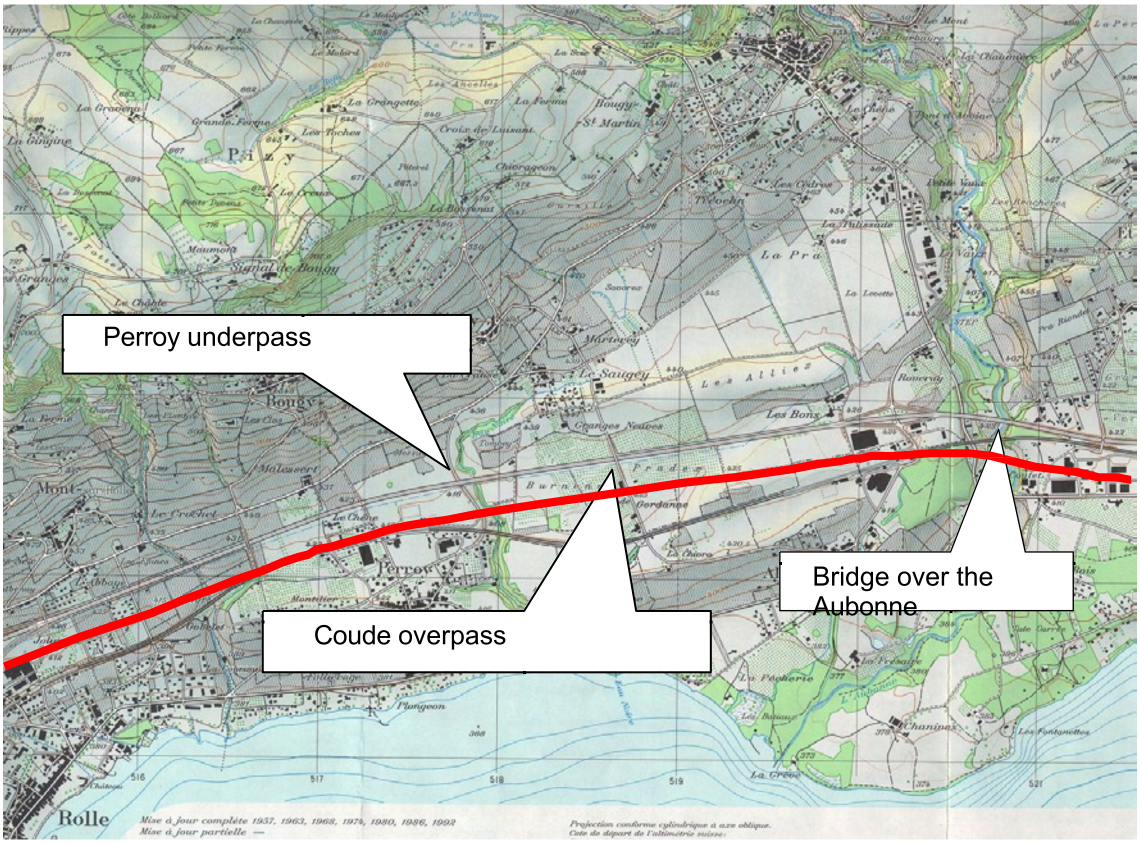


Figure 7.1 *Situations of the three bridges on the A1 freeway between Geneva and Lausanne.*

7.2 PERROY UNDERPASS

The Perroy underpass is located at km 47.914 of the A1 trunk road between Lausanne and Geneva, and crosses the 52nd cantonal road between Perroy and Féchy. The static system of the structure is a reinforced concrete frame with a span of 10 m and a width of 29.16 m. The bridge deck is a slab with a minimum thickness of 450 mm. The traffic gauge height is 4.20 m downstream and 5 m upstream on the Féchy side. A section of the bridge is shown in figure 7.2.

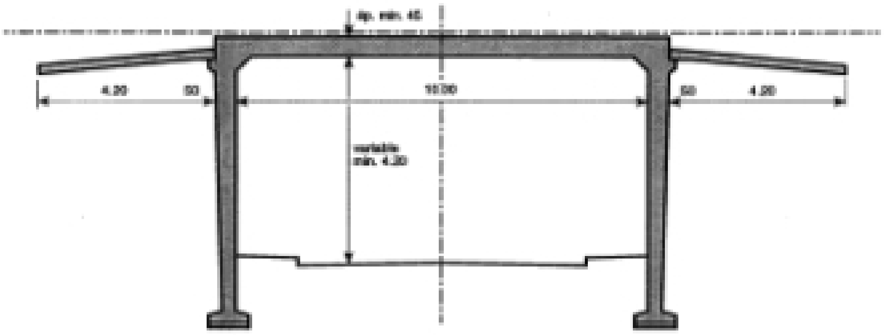


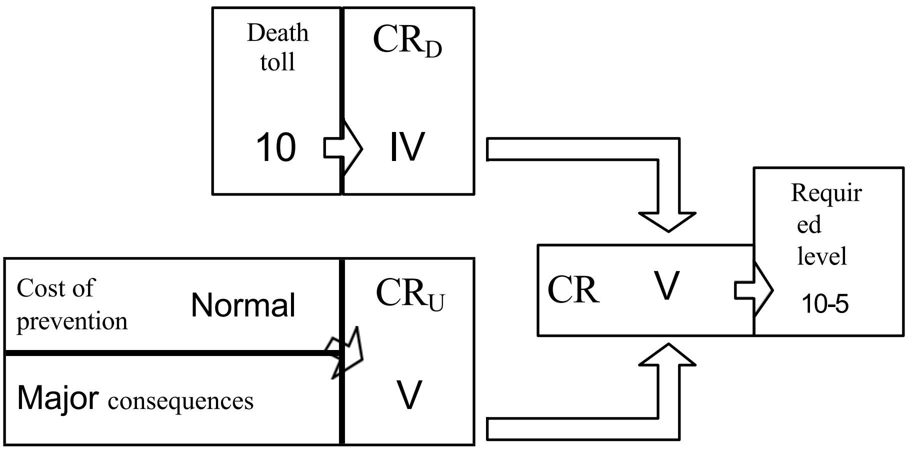
Figure 7.2 *Longitudinal section of the Perroy underpass.*

As a risk situation, we consider the ruin of one half of the slab by collapse, i.e. the formation of a mechanism, due to the presence of two extreme heavyweights.

It is assumed that the collapse of the slab would have the following consequences:

- Two trucks fall onto the lower road and there's a pile-up on the freeway, resulting in a dozen deaths.
- The freeway is cut off in one direction while half the slab is being rebuilt.

The level of safety required is determined on the basis of damage to a dozen or so people, and assuming that the cost of preventing collapse is average and the consequences from the user's point of view are major.



7.3 UPPER PASSAGE OF ELBOW

The Coude overpass is located at km 48.800 of the A1 trunk road between Geneva and Lausanne, and provides access to the AF 922 road between Féchy and Allaman. It is a typical crutch overpass, consisting of 3 pre-stressed prefabricated I-beams. During construction, the beams were placed on scaffolding, the deck slab was cast in place on prefabricated slabs (lost formwork) and the parabolic continuity prestressing cables were tensioned to create the structure's uniformity. This static system makes it possible to cross the main road with a slender structure, with 10.95 m edge spans and a 27.30 m central span. The elevation of the structure is shown in figure 7.3.

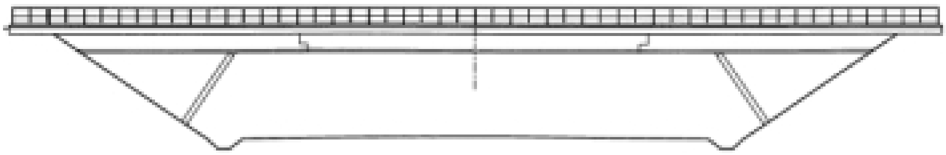


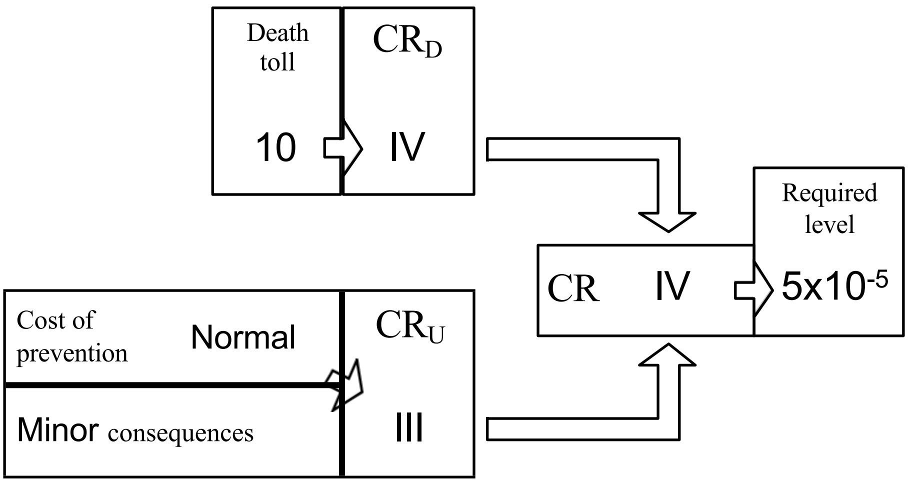
Figure 7.3 *Elevation of the upper elbow passage.*

As a risk situation, we consider the collapse of the cantilever beam due to the presence of two extreme heavy goods vehicles.

It is assumed that collapse would have the following consequences:

- A truck and the beam fall onto the four lanes of the freeway and there's a pile-up causing a dozen deaths.
- The freeway is closed in both directions for a day for cleaning.

The required level of safety is set for damage of around ten deaths, assuming that the cost of preventing collapse is normal and the consequences from the user's point of view are minor.



7.4 PONT SUR L'AUBONNE

The Aubonne bridge consists of two twin bridges, with only the abutments and foundation footings in common. The superstructure is made of prestressed concrete, and the piers of reinforced concrete. The structure forms part of a plan curve that includes a clothoid and an arc of a circle with a radius of 2,000 m. The total length of the bridge is 277 m over seven spans. The maximum height of the piers is 25 m, the spans of the river banks are 34 m, and the central spans 37 m. The longitudinal section of the bridge is shown i n figure 7.4.

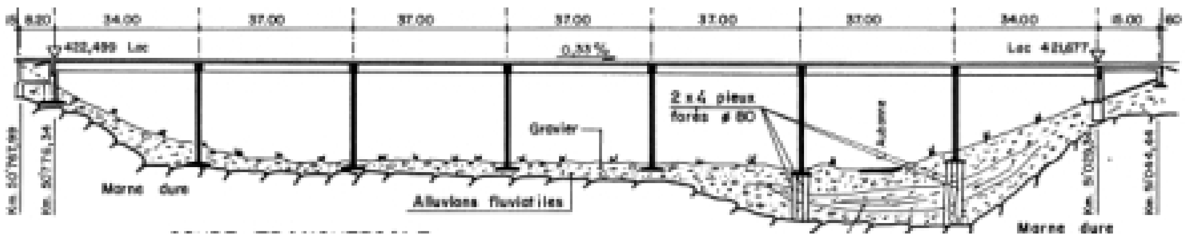


Figure 7.4 *Longitudinal section of the Aubonne bridge.*

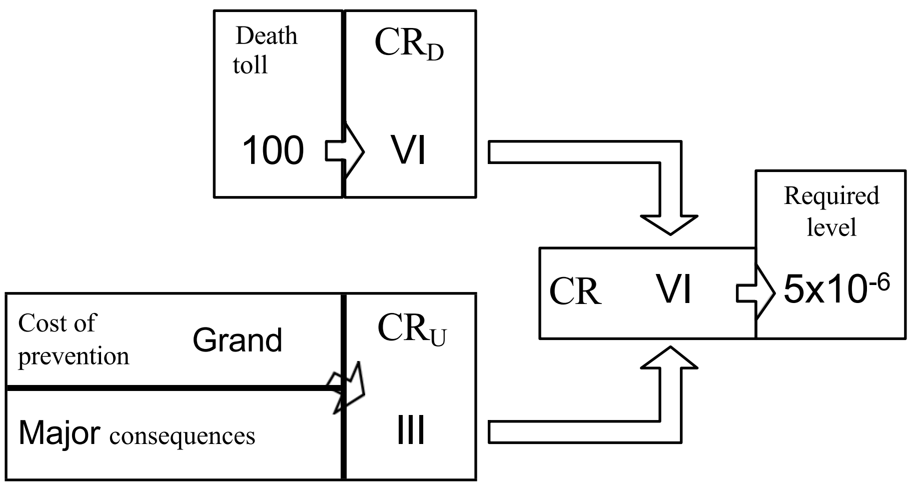
Two risk situations are presented:

1) The collapse of a central span, due to the formation of a mechanism caused by the presence of extreme heavy goods vehicles.
2) Plasticization of a beam due to the passage of an extreme heavyweight.

1) It is assumed that the collapse of the central span would have the following consequences:

- Two trucks and a coach fall twenty meters to the valley, and there's a pile-up on the freeway, resulting in around a hundred deaths.
- The highway is cut off in one direction while the bridge is being rebuilt.

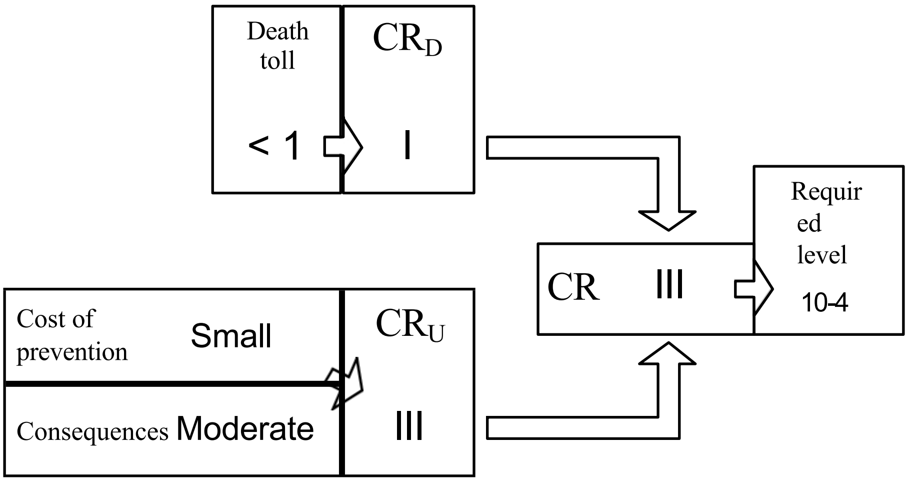
For this risk situation, we set the required safety level by assuming a hundred deaths, a high cost of collapse prevention and major consequences from the user's point of view.



2) For beam plasticization, the following consequences apply:

- The risk of a road accident is low.
- One lane of the freeway is closed in one direction while the beam is repaired.

The level of safety required with regard to beam plasticization is set on the assumption that there will be no fatalities, that the cost of preventing plasticization is small and that the consequences from the user's point of view are moderate.



7.5 COMPARISON OF RESULTS

The results of the analysis of the four risk situations are summarized in Table 7.1. The analyses show that, for the same section of freeway, the level of safety required can vary considerably depending on the risk situation.

Risk situation	**Dead**	**CRD**	**Costs & Consequences**	**CRU**	**Max**	β	P_f
Perroy underpass Slab ruin	10	IV	normal major	V	V	4.2	10-
Elbow overpass Ruined gerber joint	10	IV	normal minor	III	IV	4.0	5x
Bridge over Aubonne River Ruin due to collapse	100	VI	major	III	VI	4.4	5x
Bridge over the Aubonne River Plastification of a beam	< 1	I	small moderate s	III	III	3.7	10-

***Table 7.1** Required safety level for four risk situations*

8 CONCLUSIONS

This report presents the results of a study into the level of safety required for the assessment of existing road bridges. The basic idea behind the study was to define target reliability as a function of the risk associated with bridge ruins, rather than considering the level of safety imposed by construction standards. The study therefore focused on an analysis of the risk associated with bridge ruins and the risks accepted by the public during everyday activities. These risks are then used to define an acceptable level of risk for the assessment of existing highway bridges.

It is important to note that the aim of the study is not to reduce the overall safety level of the bridge fleet, but rather to target a uniform level of acceptable risk. The approach proposed in this report thoughtfully defines a required safety level as a function of risk situations.

The required safety level is thus defined as a function of "external" parameters representing the value and importance of a structure. This required safety level is then compared with the estimated safety, calculated from the "internal" parameters considering the bridge's condition.

The proposed approach makes it possible to define the required level of safety, represented by a target reliability. This reliability must be compared with the actual reliability in the face of the risk situation to be verified. The target reliability cannot therefore be used directly in a semi-probabilistic verification, which would require updating the partial factors (based on the target reliability) for the risk situation to be verified.

Examples of application of the proposed approach to highway bridges show that, for the same stretch of freeway, the level of safety required can vary considerably depending on the risk situation.

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APPENDIX A1: Bridge collapses in service

Name + Description + Location IN SERVICE	Year of breakup	Age + Monitoring	Causes	Liability	Consequences	Teaching	Hazards + Info quality [0-3]
Angers suspension bridge (France)	1850	12 years Step inspection	Cables break as 500 soldiers pass over them		220 dead	Questioning the principle of suspended bridges	Unknown danger (resonance) [3]
Railroad bridge in Ashtabula, OH (USA)	1876	11 years Pas inspection	Fatigue failure of a defective assembly propagated by cyclic train movements	Fatigue	80 dead	Detailed inspections would have prevented the ruin	Unknown danger (fatigue) [3]
Tay" railway bridge Dundee (Great Britain)	1876	1 year Step inspection	Gust of wind : Undersizing wind loads	Engineer and nature (wind)	75 dead	Detailed inspections would have prevented the ruin	Unsuitable measures (undersized in wind) [3]
Railway bridge Münchenstein (Switzerland)	1891	16 years old	Buckling instability steel uprights	Engineer	73 dead 170 injured	Watch out for design	Unsuitable measures (no bracing, instability) [2]
Railway bridge "Horseshoe (Tasmania)	1893		(No information)				[0]
Railway bridge between Angers and Poitier (France)	1907		A train derailed as it crossed the bridge	Shock	28 dead		Danger accepted [2]
Munich Bridge (Germany)	1910		High water	Scouring			Neglected danger [1]
Glen Loch Bridge Pennsylvania (USA)	1912		Fatigue failure of a vertical cable and train derailment	Fatigue	4 deaths		Unknown danger [2]
Railway bridge over the Yun River (China)	1923	22 years old	Resonance during the passage of 2 locomotives		No fatalities		Unknown danger (resonance) [3]
Webster Street weighbridge in California (USA)	1926		Collision with a boat	Shock	No fatalities		Unsuitable measures [2]
Bridge over the Rhine at Tavanasa, Graubünden (Switzerland)	1927	22 years old	Falling rocks	Nature	No fatalities		Hazard not identified [2]
Suspension bridge Whitesville, Virginia (USA)	1926		Cable fastener failure due to soldering defect	Engineer	6 deaths 24 injured		Faulty measurements [2]
Weighbridge on the Hackensack River New Jersey (USA)	1928	2 years Step inspection	Dynamic effects not taken into account structure in motion	Engineer	No fatalities		Undetected hazard (dynamic effects) [2]
Gateway near Koblenz (Germany)	1930		Party overload	Engineer	1 death		Unsuitable measures [2]
Bridge in New Mexico (USA)	1933		Erosion ofa submerged pile due to high water	Nature (scouring)			Unsuitable measures [2]
Bridge in Oregon (USA)	1937		A truck a little too high hit the bridge, causing it to collapse.	Shock			Unsuitable measures [2]
Bridge in Virginia (USA)	1937		A truck hit the bridge and caused its collapse	Shock			Unsuitable measures [2]
Bridge in Colorado (USA)	1937		Vehicle overload	Engineer			Unsuitable measures [2]
Bridge Rüdersdorf motorway (Germany)	1938	1 year	Choosing the wrong steel: St 52 Low-temperature fracture	Engineer	No fatalities	No more steel construction St 52	Unknown hazard (steel type) [3]
Bridge tramway in Hasselt (Belgium)	1938	1 year	Rupture after a passing tramway Poor materials	Engineer	No fatalities	Choice of steel type	Hazard not identified (embrittlement of welded steel) [3]
Steel arch bridge "the view from the falls, NY (USA)	1938	40 years Pas inspection	Foundation instability and ice block impacts	Engineer and nature (scouring)	No fatalities, Bridge closed	Better protection for foundations	Unsuitable measures (foundation protection) [2]
Suspension bridge Tacoma, Washington (USA)	1940	4 months Step inspection	Dynamic excitation of the deck due to wind frequency	Engineer and nature (wind)	No fatalities, Bridge closed	Greater rigidity o f suspension bridge decks	Unknown hazard (dynamic effects) [3]
Bridge over the Mississippi near Chester, Illinois (USA)	1944	2 1/2 years	Poor wind dimensioning	Nature (Wind) and Engineer	No fatalities		Unsuitable measurements (wind gusts) [2]
Lift bridge in New Jersey (USA)	1945		A train continued past the stop signal	Shock	1 death 68 injured		Unsuitable measures [2]
Road bridge near Fresno, California (USA)	1947		Overloading farm tractors	Engineer			Unsuitable measures [2]
Bridge near Koblenz (Germany)	1947		Ice blocks	Shock			Neglected danger [1]
Name + Description + Location IN SERVICE	Year of breakup	Age + Monitoring	Causes	Liability	Consequences	Teaching	Hazards + Info quality [0-3]
Bridge in Maine (USA)	1947		Truck collision	Shock			Unsuitable measures [2]
Bridge in Düsseldorf (Germany)	1947		Collision with a boat	Shock			Unsuitable measures [1]

Footbridge to Stresa (Italy)	1948		People overload	Engineer	12 dead		Unsuitable measures [2]
Bridge Elbow grade (USA)	1950		Bridge collapses shortly after erection	Engineer	no deaths		Unsuitable measures [2]

Bridge Dupplessis in the St-Maurice River, Quebec (Canada)	1951		(No information)				[0]
Bridge of Brooklyn to Harrodsburg (USA)	1953	80 years old	Overload	Engineer	1 injured		Unsuitable measures [2]
Bridge on the Peace River (Canada)	1957	No monitoring	Movement of anchors on foundations that were not properly secured	Engineer	No fatalities, Bridge closed		Unsuitable measures (unstable anchors) [2]
Arch bridge in Topeka, Kansas (USA)	1958	During demolition	Demolition equipment deadweight surcharge	Engineer	No fatalities,	Just as dangerous during demolition as during construction	Neglected danger (during demolition) [2]
Bridge near Bristol (England)	1960		Two boats collide in a bridge pier on a foggy day	Shock	5 deaths		Unsuitable measures [2]
Bridge near Kloster Moraca (Yugoslavia)	1962		No information		21 dead 17 injured		[0]
Bridge King Street on the Yarra River (USA)	1962	1 year Step inspection	3 factors : low-strength steel faulty design low ambient temperature	Engineer	No fatalities		Unsuitable measures or neglected hazard (faulty design) [2]
Bridge from Maracaibo (Venezuela)	1964		Boat collides with several bridge piers	Shock	6 deaths		Unsuitable measures [2]
Bridge near New Orleans (USA)	1964		Boat collision	Shock	6 deaths		Unsuitable measures [2]
Bridge between Antwerp and Aachen (Belgium)	1966	8 years old	Landslide	Nature	2 deaths 16 injured		Hazard not identified [2]
Bridge between Antwerp and Lüttich (Belgium)	1966	8 years	High water	Scouring	2 deaths 13 injured		Neglected danger [2]
Bridge to Punta Piedras (Venezuela)	1966		Overload	Engineer	20 dead		Unsuitable measures [2]
Ariccia Bridge (Italy)	1967	114 years old	Cause unknown		2 deaths		[0]
Suspension bridge on the Ohio the "silver bridge West virginia (USA)	1967	40 years Pas inspection	fatigue fracture + corrosion	Engineer: fatigue and corrosion	44 dead 2 missing 9 injured	Detailed inspections would have prevented the worst	Neglected hazard (lack of inspection) [2]
Bridge between Pisa and Florence (Italy)	1968		High water during repair work	Scouring	No fatalities		Neglected danger [2]
Bridge in the province of Udine (Italy)	1968		High water	Scouring			Neglected danger [2]
Bridge (Montenegro)	1968		Overload	Engineer	6 deaths 21 injured		Unsuitable measures [1]
Bridge in Illinois (USA)	1970		Bridge broken by 1st train pass (undersizing)	Engineer			Unsuitable measures [2]
Bridge from the A1 freeway at Hamborg (Germany)	1970		Successive failure of a pylon and then of the bridge after severe wind-induced oscillations	Engineer (wind)			Unsuitable measures [2]
Antelope Valley highway interchange (USA)	1971		Earthquake	Nature (earthquake)	Minor damage		Accepted hazard (earthquake)
Bridge in Georgia (USA)	1972		Boat collision	Shock	10 deaths		Unsuitable measures [2]
Bridge near Katerini (Greece)	1972		High water	Scouring	1 death		Neglected danger [2]
Bridge mixed wood-steel (Philippines)	1972		Rupture due to an overload o f people during a procession	Engineer	145 dead 200 injured		Unsuitable measures [2]
Gateway in Pinzgau (Austria)	1974		Collapse caused by schoolchildren crossing together	Engineer	8 deaths 16 injured		Unsuitable measures [2]
Name + Description + Location IN SERVICE	**Year of breakup**	**Age + Monitoring**	**Causes**	**Liability**	**Consequences**	**Teaching**	**Hazards + Info quality [0-3]**
Lake Pontchartrain bridge (USA)	1974		Collision with a boat	Shock	3 deaths		Unsuitable measures [2]
Bridge near Charleroi (Belgium)	1974		Train derailment and collision with bridge	Shock	17 deaths 80 injured		Unsuitable measures [2]
Weighbridge in Ontario (Canada)	1974		Collision with a boat	Shock	2 injured		Unsuitable measures [2]
Bamboo bridge over the Ganges (India)	1974		No information		40 dead		[0]
Bridge in Hobart, Tasmania (Australia)	1975		Boat collides with 2 bridge piers	Shock	15 dead		Unsuitable measures [2]
Bridge near Vranje (Yugoslavia)	1975		High water	Scouring	13 dead		Neglected danger [2]
Bridge on the M62 (England)	1975		A crane under the bridge toppled over onto it	Shock	2 deaths		Unsuitable measures [2]
Bridge on rue Lafayette in St-Paul, Minnesota (USA)	1975	7 years	Mid-span through crack (poor weld detail and low temperature)	Engineer	No fatalities		Neglected hazard (faulty design: cross-welds) [2]

Bridge over the Danube at Vienna (Austria)	1976	40 years old	Faulty design : Missing reinforcement and poor concreting	Site engineer	No fatalities		Defective measures (on site) [2]

Manchac Bridge in Louisiana (USA)	1976		Collision with a boat	Shock	2 deaths 2 injured		Unsuitable measures	[2]
Wooden footbridge in Vorarlberg (Austria)	1976		Collapse caused by schoolchildren crossing together	Engineer	8 wounded		Unsuitable measures	[2]
Bridge between Turin and Mailand (Italy)	1977		High water	Scouring			Neglected danger	[1]
Bridge north of Genoa (Italy)	1977		High water	Scouring			Neglected danger	[1]
Bridge near Sydney (Australia)	1977		Collision with a train	Shock	89 dead		Unsuitable measures	[2]
Bridge near Moscow (Russia)	1977		Insufficient restoration after first break in 1940	Engineer	20 dead 100 casualties		Unsuitable measures	[2]
Bridge in Punjab province (India)	1977		Breakage on passing omnibus	Engineer	22 dead		Unsuitable measures	[2]
Bridge in north-east India	1977		Train derailment and collision with bridge	Shock	50 dead		Unsuitable measures	[2]
Bridge in Assam (India)	1977		Collapse due to passing train	Engineer	45 dead 100 casualties		Unsuitable measures	[1]
Pont (Bangladesh)	1978		No embedding of lower deck reinforcement	Engineer			Unsuitable measures	
Bridge in San Sebastian (Spain)	1978		Rupture during a gathering of people	Engineer	7 deaths		Unsuitable measures	[1]
Bridge near Dortmund (Germany)	1979		A 39t truck collides with the bridge	Shock	1 death 6 injured		Unsuitable measures	[2]
Pont-mixte near Duisburg (Germany)	1979		A buldozer's mechanical shovel tore off and toppled the bridge.	Shock	8 deaths		Unsuitable measures	[2]
Bridge near Salvatierra (Mexico)	1979		No information		7 deaths			[0]
Bridge on Hood Canal in Washington (USA)	1979	21 years old	Wind + storm	Scouring Wind	No fatalities			[0]
Bridge in Göteborg (Sweden)	1980		Collision with a boat	Shock	8 deaths		Unsuitable measures	[2]
Bridge in Wiscontin (USA)	1980		Collision with a truck	Shock	1 injured		Unsuitable measures	[2]
Bridge in Florida (USA)	1980		Collision with a boat	Shock	35 dead		Unsuitable measures	[2]
Suspension bridge to Munster (Germany)	1980		Collision with truck in icy conditions	Shock	1 death		Unsuitable measures	[2]
10Decks Central China (China)	1981		High water	Scouring			Neglected danger	[1]

Name + Description + Location IN SERVICE	Year of breakup	Age + Monitoring	Causes	Liability	Consequences	Teaching	Hazards + Info quality [0-3]
Bridge in British Columbia (Canada)	1981		High water and tree trunk impacts	Shock	6 deaths		Neglected danger [2]
Bridge in Munich (Germany)	1981		Collision with a dump truck	Shock	4 injured		Unsuitable measures [2]
Bridge on the Brajmanbari (Bangladesh)	1982		The bridge broke when a full bus drove over it	Engineer	45 dead		Unsuitable measures [1]
Tubular bridge in Lorraine (France)	1982		Collision with a boat	Shock	7 deaths		Unsuitable measures [2]
Bridge in Ohio (USA)	1982		Inadequate and low-quality building materials	Engineer	5 deaths 4 injured		Unsuitable measures [2]
Wooden bridge on Cebu Island (Philippines)	1983		Overload	Engineer	20 dead		Unsuitable measures [1]
30 m span of the bias bridge over the Mianus river (USA)	1983	35 years Pas inspection	Faulty design: Pause lining 10 years before, clogging drainage systems	Engineer: corrosion	3 deaths 3 injured	Detailed inspections would have prevented the worst	Neglected hazard (negligent design) [2]
Aerial tramway bridge (China)	1983		Collision with a boat	Shock	7 deaths		Unsuitable measures [1]
Suspension bridge over the Iapo River (Brazil)	1984		No information		8 deaths		[0]
Bridge in central India (India)	1984		High water as a train passed over it	Shock Scouring	102 dead 100 casualties		Neglected hazards and inappropriate measures [2]
Suspension bridge near Munnar (India)	1984		No information		14 dead 11 injured		[0]
Suspension bridge at Sully-sur-Loire (France)	1985		Poor quality of cable steels, brittle at low temperatures	Engineer	No fatalities		Unsuitable measures [2]
Bridge over the Schoharie River in NY (USA)	1987	31 years Rehabilitation in 1981	Erosion at the base of a pile and poor static system (domino effect)	Nature (scouring) and Engineer	10 deaths	Adequate protection for immersed batteries + sys.st.	Undetected hazards and inappropriate measures [3]
Stone bridge on the Gotthard route (Switzerland)	1987	18 years old	The bridge was washed away the high waters of the Reuss	Scouring	No fatalities		Neglected danger [2]
Freeway overpass (Germany)	1989		Truck collides with pile	Shock	1 injured		Unsuitable measures [2]
Bridge south of Los Mochis (Mexico)	1989		The bridge was washed away as a train passed over it	Scouring	103 dead 200 injured		Neglected danger [2]
Upper deck between San Francisco and Oakland Bay, California (USA)	1989	53 years old	Earthquake	nature (earthquake)	1 death	Need to update earthquake sizing	Neglected hazard (earthquake) [2]
Cypress double highway viaduct, California (USA)	1989	32 years old	Earthquake + Inadequate design of reinforcement detail between column and the upper deck	Engineer and nature (earthquake)		Need to update earthquake sizing	Unsuitable measures (tremor of earth) [2]
Floating bridge Murrow, Washington (USA)	1990	50 years Maintenance work	Span immersion	Scouring	No fatalities		Neglected hazard (wave bursts)
Antelope Valley highway interchange (USA)	1992	21 years after 1er, No action taken	Earthquake	Engineer and nature (earthquake)	Bridge span failure	Update on structural safety verification	Neglected danger (earthquake 21 years after the first) [1]
Bridge to Kilosa (Tanzania)	1992		The bridge was washed away as a train passed over it	Scouring	100 dead		Neglected danger [2]
Bridge between Nairobi and Mombassa (Kenia)	1993	95 years old	The bridge was washed away as a train passed over it	Scouring	144 dead		Neglected danger [2]
Cicero Bridge in Sicily (Italy)	1993	< 100 years	High water	Scouring	4 deaths 1 injured		Neglected danger [2]
Bridge in trellis in Alabama (USA)	1993		Collision with a boat	Shock	47 dead		Unsuitable measures [2]
Span of the Songsu Bridge, Seoul (South Korea)	1994	15 years	Increased traffic load without checks prerequisites + construction details	Engineer	32 dead		Neglected hazard and def. measures (s-dim.) [2]
Bridge "Twin in California (USA)	1995		Scouring around its foundations on a high-water day	Nature (scouring)	7 deaths		[0]
Concrete bridge (Palau)	1996		Poor concrete quality and corrosion	Corrosion engineer	2 deaths 4 injured		Unsuitable measures
Bridge of "Walnut street" (USA)	1996	~ 90 years	Scouring of foundations due to high river water Susquehanna and the presence of lots of ice	Nature (scouring)			[0]

Name + Description + Location IN SERVICE	Year of breakup	Age + Monitoring	Causes	Liability	Consequences	Teaching	Hazards + Info quality [0-3]
Terrace Bridge (Canada)	1997		During maintenance work, the frame collapsed into the stream		1 death 1 disappeared 4 injured		[0]
Bridge over the Jarkon River (Israel)	1997		Poorly constructed and overloaded	Engineer	2 deaths 64 injured		Unsuitable measures [1]
Road bridge (Peru)	1998		No information		30 dead		[0]
Eschede overpass (Germany)	1998		Train derails, ripping out bridge pier	User (shock)	100 dead 88 casualties		Danger accepted (shock) [2]

APPENDIX A2: Bridge collapses during construction

Name + Description + Location UNDER CONSTRUCTION	Year of breakup	Age + Monitoring	Causes	Liability	Consequences	Teaching	Hazards + Info quality [0-3]
Bridge Morawa in Ljubitschewo (Serbia)	1893	End of construction	Breakage during load test (sub-samples)	Engineer	No fatalities		Unsuitable measures or neglected hazards [2]
Quebec City railway bridge (Canada)	1907	Under construction	Faulty design: underestimation of pp structure (sub-dimens.)	Engineer	76 dead		Defective measurements (defective design and calculation error) [3]
Quebec City railway bridge (Canada)	1916	Under construction	Faulty design	Engineer	13 dead		Defective measurements. (a support part has broken off) [3]
Arch from Sando (Sweden)	1939	Under construction	Poor design of wooden structures	Engineer	18 dead		Neglected hazard (poor concept.) [2]
Sullivan Square highway bridge in Boston (USA)	1952	Under construction	Instability during assembly (design and/or assembly error)	Engineer	No fatalities		Faulty measurements (faulty design, instab.) [1]
Narrows Bridge Vancouver (Canada)	1958	Under construction	Design fault	Engineer	15 dead 20 injured		Unsuitable measurements (design error) [2]
Floating bridge on Hood Canal in Washington (USA)	1958	Under construction	Holes in the formwork allowed water to seep in	Scouring	No fatalities		Neglected hazard (water ingress) [1]
Bridge over the Danube in Vienna (Austria)	1969	Under construction	Faulty design without taking temperature effects into account	Engineer	No fatalities		Unsuitable measures (failure to take account of the effects of a downturn) temperature) [2]
Bridge of Cleddau in Milford Haven (Wales)	1970	Under construction	Collapse during pushing	Engineer	4 deaths		Unsuitable measures (pushing instab.) [1]
Bridge of West Gate in Melbourne (Australia)	1970	Under construction	Collapse during assembly	Engineer	34 dead	Same office as for the bridge by Cleddau (1970)	Unsuitable measures (instab. during assembly) [1]
Bridge over the Rhine at Koblenz (Germany)	1971	Under construction	Excessive deformation of steel casing (instability: buckling)	Engineer	13 dead		Unsuitable measures (construction defects) [2]
Bridge over the Rhône at Illarsaz in Valais (Switzerland)	1973	Under construction	Fracture of steel girders during pushing concrete deck (instability)	Engineer	No fatalities		Unsuitable measures (slope and hillside) [2]
Viaduct over the Sorge at Valengin, Neuchâtel (Switzerland)	1973	Under construction	Pushing in the direction of too steep a gradient (6.5%) (instability: slippage)	Engineer	No fatalities		Neglected hazard (slippage) [1]
Exchanger Riley, east of Chicago (USA)	1982	Under construction	Faulty design: Overload on inadequate shoring system	Engineer	13 dead 18 injured		Unsuitable measures (poor concept.) [2]
Bridge in Elwood, Kansas (USA)	1982	Under construction	Faulty design	Engineer	1 death 8 wounded		Unsuitable measures [1]
Caisson bridge (Germany)	1985	Under construction	Undersizing: temporary abutment	Engineer	No fatalities		Unsuitable measures (sub-dim prov. batteries) [2]
Bridge in El Paso,Texas (USA)	1987	Under construction	Inadequate scaffolding	Engineer	1 death 7 injured		Unsuitable measures (faulty design) [2]
Highway bridge near Seattle (USA)	1988	Under construction	Beams not yet held together by spacers, domino effect	Engineer	No fatalities		Unsuitable measures (instability) [2]
Caisson bridge in Los Angeles (USA)	1989	Under construction	Collapse while dismantling scaffolding to lower a prefabricated voussoir	Engineer	5 injured		Unsuitable measures (instab. during assembly) [2]
Baltimore overpass (USA)	1989	Under construction	Faulty design: No prestressing yet and asymmetrical loads	Engineer	14 injured		Unsuitable measures (scaffolding + props) [2]
Bridge on the Mississippi (USA)	1990	Under construction	(No information)	Engineer	1 dead Several injured		[0]
Bridge in Hiroshima (Japan)	1991	Under construction	Stability problem (slippage)	Engineer	14 dead		Undetected danger (instability) [0]
Freeway interchange scaffolding in Los Angeles (USA)	1991	Under construction	Undersizing: Unexpected asymmetrical overloading	Engineer	No fatalities		Unidentified hazard (subimens.) [0]