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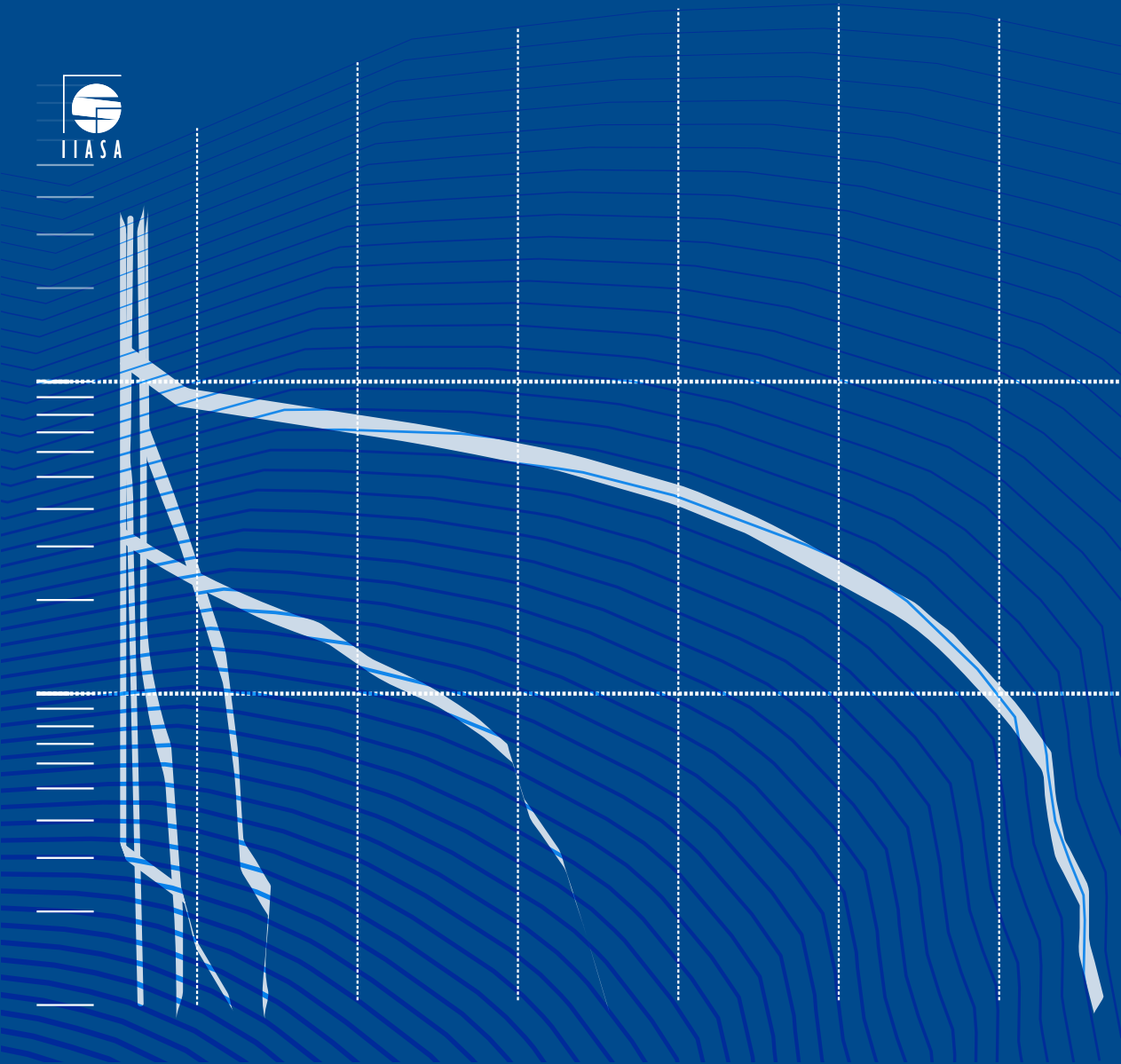

Uncertainty and Disaster Risk Management

Modeling the Flash Flood Risk to Vienna and Its Subway System

Keith L. Compton, Rudolf Faber, Tania Y. Ermolieva, Joanne Linnerooth-Bayer, and Hans-Peter Nachtnebel

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Joanne Linnerooth-Bayer

Abstract

This report describes an interdisciplinary approach to flood risk analysis and management that was developed by investigating flood risks in the city of Vienna, Austria. The purpose of the research was to analyze different policy paths (including both flood-prevention measures and risk-sharing financial provisions) in the presence of major uncertainties. A preliminary analysis resulted in the identification of two major methodological issues that needed to be resolved, namely:

- The concept of risk used in flood management varied subtly but significantly across the disciplines contributing to the assessment.
- Current assessment procedures did not give a full account of uncertainties and their different types.

For those reasons an approach was developed that allows the analyst: 1) to integrate the different disciplinary concepts of risk within a single interdisciplinary analysis; and 2) to take into account uncertainties in a way that not only allows their many characteristics to be distinguished but is also consistent across the component disciplines. The focus of this report is the phenomenon of flash flooding of the Vienna River. Our analysis demonstrated that, in this case, the greatest damage from flash flooding was to be expected in the Vienna city subway system. The report thus describes a detailed assessment of the flood risk to the subway and of related management measures, on which research to date has been scarce.

The results show that an approach based on catastrophe modeling and Monte Carlo simulation can not only integrate the risk perspectives of the different technical disciplines contributing to this study but also provide a useful framework for comparing the characteristics of different mitigation strategies. The results of the simulations suggest alternatives for combining different mitigation measures to ensure complementarity among the characteristics of different components of an overall strategy, and thereby decrease total costs and reduce the likelihood and the uncertainties of catastrophic financial losses.

Key words: Risk Assessment, Uncertainty Analysis, Risk Management, Catastrophe Models, Flash Flood, Subway Flood Risk, Catastrophe Insurance

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Introduction and Theoretical Background

The purpose of this report is to illustrate an interdisciplinary approach to flood risk analysis that combines hydrological flood risk assessment and simulation modeling with the finances of flood risk management. Using this interdisciplinary approach, we examined flood risks in the city of Vienna, Austria, together with some alternatives for mitigation of the damage caused by flooding.

While developing an interdisciplinary approach for examining catastrophic flood risks, we found that the concept of risk used in flood management varied subtly but significantly across the disciplines contributing to the study. Although such variations appear subtle, the way in which the term "risk" is conceptualized (e.g., as probability, as consequence, as expected value, etc.) can significantly affect the way in which an analysis produced within a particular discipline is structured. More importantly, it can significantly affect the conclusions reached about the courses of action recommended, particularly when a decision maker has to choose from among very different options developed on the basis of analyses prepared within different disciplinary frameworks. This can happen, for example, when a decision maker is trying to decide whether to implement a structural approach (e.g., raise the height of river levees) or a financial approach (e.g., transfer the risks through insurance). An important result of this study is the integration of these different disciplinary concepts of risk within a single interdisciplinary analysis. We also show that the way in which uncertainty is defined and represented is not consistent across different disciplines.

This project was carried out within the framework of catastrophe model development. In this section we will introduce the reader to the different concepts of risk that arise within catastrophe modeling. We will first discuss taxonomy of perspectives on risk, show how our approach fits into a larger taxonomy, and then discuss the way risk is conceptualized in the technical disciplines contributing to this study. Finally, we discuss the impact of uncertainty in catastrophe modeling and introduce an approach for integrating multiple concepts of uncertainty into catastrophe modeling. The rest—and majority—of the report (Chapters 2 to 5) presents a concrete implementation of these ideas in a case study which examines urban flooding in Vienna. A brief set of general observations and conclusions is presented in Chapter 6.

The approach illustrated in this study will be useful for examining policy paths, including flood risk mitigation and insurance, that are used to manage the risks of flooding in Vienna and elsewhere. Our results build on ongoing work at the *Universität für Bodenkultur* (BOKU) in Vienna and IIASA on the development and use of models for the management of catastrophic risks (Amendola *et al.*, 2000a, 2000b; Brouwers 2003; Ekenberg *et al.* 2003; Ermoliev *et al.*, 2000; Faber and Nachtnebel, 2002, 2003; Freeman *et al.*, 2002; Konecny and Nachtnebel, 1985; Nachtnebel and Faber, 2002; Nachtnebel, 2000; Mechler, 2003). These studies encompass a wide variety of disciplines, catastrophe types, and spatial and temporal scales.

As in any analysis, we have operated under significant constraints, some external and some self-imposed. One self-imposed constraint is that it is not our goal in this analysis to provide and implement a "true" definition of the term "risk" or "uncertainty." It is not even clear if such a task is possible. Nor do we include all possible concepts of risk within our larger analysis, although we do try to provide some glimpses of how this analysis might fit into a broader decision-making framework. As will become apparent, this report remains very firmly within a technical perspective and does not deal with non-technical (for example, psychological or sociological) perspectives on risk. Our intention is not to propose a canonical definition that fits any situation. Rather, we seek simply to clarify how we have used these terms and to show how a slightly broader conception allows integration across different technical (hydraulic and financial) disciplines. This type of integration, in turn, allows meaningful comparisons of very different flood mitigation alternatives to be produced. External constraints on the availability of resources and data during the study also restrict the usefulness of this analysis as a direct input into policy decisions on flooding for the city of Vienna. The study was not commissioned to provide input of this kind. This report is a case study that illustrates an approach to catastrophe modeling that relies on real data and addresses a real problem. Although every effort was made to use high-quality data, to produce accurate models, and to deal with issues of relevance to policymakers, the study lacks several of the elements critical to a decision support study. Quality assurance and quality control (QA/QC) reviews of data and codes were not undertaken; nor was there a review of the legal and regulatory requirements for a decision. These aspects often impose significant legal and scheduling constraints on the analyst and, together with the budgetary and time constraints typical of applied analyses, impede exploration of alternative approaches to the structuring and evaluation of problems. It is our hope, however, that the study raises some interesting questions and suggests possible courses of action in similar situations occurring elsewhere. We are grateful for the opportunity to explore an applied problem in the way that seemed most appropriate from the intellectual perspective and to have the freedom

to address issues and make decisions rather than be forced by external constraints to follow predefined approaches.

1.1 Concepts of Risk

In his risk taxonomy, Ortwin Renn (1992) distinguishes four perspectives: technical, economic, psychological, and sociological. As previously mentioned, the scope of this study is largely within the technical perspective. Renn subdivides the technical perspective into a statistical or actuarial approach (typically used in the insurance community), a modeling approach (typically used in the health and environmental protection community), and probabilistic models (typically used in safety engineering). One goal of this study is to integrate these distinct approaches within the technical perspective.

According to Covello and Merkhofer (1994, p. 20), "risk is, at a minimum, a two-dimensional concept involving: 1) the possibility of an adverse outcome; and 2) uncertainty over the occurrence, timing, or magnitude of that adverse outcome." This definition is appropriate for our purposes, as it offers fruitful opportunities for integrating the differing technical perspectives. Although it is largely consistent with the concept of risk used in the financial community, there are differences. Financial experts, going back to the definition provided by Frank Knight (1921), use the term "risk" to refer to a measurable (typically statistical) volatility and speak of "upside" and "downside" risks to refer to the possibility that an outcome may be either better or worse than the expected outcome. The differences are subtle but significant. The financial definition is narrower in that Knight's concept of risk explicitly excludes epistemic uncertainty and includes only variability (often called aleatory uncertainty). However, this concept is also broader in the sense that the possibility of unexpected positive outcomes is also included. The distinction is relevant to the extent that a policy oriented toward "loss prevention" or "loss reduction" can sometimes blind one to the possibilities that may exist for maximizing welfare.[1] The common theme is that both concepts of risk arising within the technical perspective include, either implicitly or explicitly, probability and consequences of occurrence as the two major risk components. Our goal is to implement a concept of risk that not only includes the probability/consequence distinction and the (implicit) full conception of uncertainty advocated by Covello and Merkhofer, but also broadens consequences to include both upside and downside risks. We emphasize that the psychological dimensions, such as the aversion that individuals might

[1] According to White *et al.* (2001), "there are very few efforts to estimate the net benefits of location of land use in hazard areas of the actual benefits of extreme events.... Land and locations in areas subject to hazard have market value, often high market value... some effort to calculate net gains and losses should be undertaken in the literature and its continuing absence in these texts reveals a prevailing state of ignorance that the research efforts have scarcely addressed."

have for certain types of risk, or the sociological aspects, such as the equitable distribution of risks, are not typically considered in technical risk analyses. For this reason, technical analyses are only one input into larger policy processes. Experience has also demonstrated the many dimensions to risks that are not included in estimates of probability and consequence, such as whether the risk is voluntary or controllable.

Technical disciplines concerned with standard setting have often emphasized one of the two component concepts of risk at the expense of the other. Some disciplines have focused most of their attention on probability of occurrence as a measure of risk. A scenario to be avoided is identified (e.g., destructive flooding, release of radioactivity from a nuclear reactor, etc.) and the "risk" is the probability of occurrence of the adverse event. Typical examples of this paradigm include traditional approaches to flood and earthquake protection. In traditional flood protection, for example, a typical goal is to reduce the probability of flooding to below a certain design value, such as a 100-year flood (i.e., the probability of flooding in any year should be less than 1 percent). Other disciplines have focused on the magnitude of the adverse consequences as a measure of risk, most frequently by attempting to keep consequences below a certain level determined to be "acceptable" or "safe," regardless of the likelihood of the effect. This approach is embodied, for example, in regulations banning substances found to be carcinogenic. Setting exposure levels to hazardous chemicals in the workplace or environment such that no adverse effects are expected, but without explicit regard as to the likelihood of that exposure, is an example of this paradigm. This reasoning, especially when the consequences may be very serious or catastrophic and the probabilities are difficult to assess, is the logic underlying the European Union's precautionary principle. Within the actuarial community, on the other hand, both probabilities and consequences are considered explicitly. However, they are typically telescoped together by the use of "expected value" as a measure of risk.

1.2 Aleatory Uncertainty, Epistemic Uncertainty, and Risk Curves

Uncertainty in the likelihood of floods arises from a number of sources. These uncertainties can be grouped into two fundamental types: aleatory and epistemic. Aleatory uncertainty, sometimes called irreducible uncertainty, arises from the natural variability of the system under study. Some systems are fundamentally stochastic in nature and their future state cannot be predicted deterministically. There are many examples of this in nature, such as the number of radioactive decay events observed within a specific time frame from a specific quantity of material or the time between earthquakes of a given magnitude on a particular fault. For our

study, the natural variability is the time expected until a storm of a certain magnitude occurs.[2] Rainfall patterns are not identical from year to year. This type of uncertainty is termed "irreducible" uncertainty because it is a property of the phenomenon itself. However, although the maximum rainfall cannot be predicted with precision, it has been found that these values follow regular statistical distributions. The likelihood that the worst storm in a year will exceed a certain level may, to a first approximation, be estimated simply by collecting information every year on the worst storm (e.g., the amount of rain falling within a 6-hour period) and developing an empirical distribution. The functional form of the distribution can be determined based on statistical principles or assigned based upon engineering judgment. The statistical problem is then to use the historical data to find the parameters of the distribution.

This example also illustrates the second source of uncertainty, namely, epistemic uncertainty. Epistemic uncertainty refers to a lack of knowledge about the system and can be introduced by errors in, or limitations to, the ability to collect samples. In many locations, reliable historical records may only cover a period of several decades. Even if it were reliable, measuring peak rainfall or river flow during a storm is subject to error. There is also no guarantee that the climatic conditions that generate the rainfall or land use patterns affecting the rate at which water drains into the river have not changed over the period of measurement; in fact, it is quite likely that such conditions have changed. Finally, the choice of a model to describe the variability distribution is not a clear-cut process. Fitting observed data to an incorrect model can lead to errors in prediction. These and other sources of error lead to epistemic uncertainty. Such uncertainty may not be severe when one is trying to estimate the expected annual maximum or the maximum to be expected once every 5–10 years. However, the uncertainty involved in estimating the magnitude of storms that recur over a period of centuries or of millennia is dramatically greater than estimating the magnitude of storms that recur over a period of years or decades. Although such uncertainties are also present in evaluating the magnitude of storms that recur over shorter periods, the range of possible values may not be terribly large. Extrapolation from short observation periods to very long observation periods amplifies the sources of uncertainties and progressively violates the assumptions of an underlying steady state made in developing the forecasts. The range of possible values of peak rainfall during a decadal storm (a storm that is expected to occur once every decade) may vary only over a few tens of millimeters

[2]The magnitude or severity of a rainstorm is often defined as the amount of rainfall averaged over a specific period of time. As rainfall is a stochastic process, the averaging time affects the peak rainfall. For example, a storm may produce bursts of rain at 100 mm/hr for periods of a few minutes, but will produce only 50 mm/hr when averaged over a period of three hours. In this study, we will use the 6-hour average rainfall as the indicator of the magnitude of a storm, as it is this period that corresponds to the response time of the watershed under study.

and may be managed by simply adding an appropriate design margin on to an engineered design. In the United States, the use of a safety margin on levee heights of three feet (approximately 1 m) was just such a consideration (National Research Council, 2000). However, when attempting to protect against storms that recur over periods of millennia, the peak rainfalls that might be reasonably expected can range over tens to hundreds of millimeters. The worst flood in a millennium may be only slightly more severe than the worst flood in a century, or it could be dramatically worse. If one applies the typical design margin or safety factor approach, one could end up installing a system in which most of the costs are directed at ensuring that the design margin was sufficiently large. On the other hand, if one simply uses a "best" estimate (such as an expected value or a most likely value), one might find that there is a significant probability that the protection system would not function if the storm were much larger than the best estimate.

However, once effective measures are taken to protect against the more frequent floods, it is precisely the rare and uncertain floods that may now pose most of the risk to the affected populations. The decision maker is therefore in a quandary, with pitfalls on all sides. If the true likelihood of a particularly severe flood is quite high and no mitigation efforts are undertaken, massive damage might result. On the other hand, if the true likelihood is low and expensive mitigation measure are undertaken, then the resources used to implement the mitigation may have been lost if the event fails to occur. In the worst of all possible worlds, expensive mitigation measures could be implemented but still fail when called upon to withstand the flood. In this case, losses are incurred both before the disaster (mitigation costs) and as a result of the disaster (in terms of damage to assets). Thus, in addition to the costs and benefits of different mitigation measures, the reliability of the mitigation measures is also a critical input to decision making. Determining the best course of action in such a case is problematic and depends sensitively on the preferences and values of the decision maker. When significant uncertainties are present about the timing or magnitude of the potential loss, it is not possible to simply compare the costs and benefits of different options. It is the specific goal of this chapter (and, more generally, of the whole report) to illustrate a way of structuring these uncertainties so that the decision maker can see the results of a decision and the extent to which the losses and attendant uncertainties change under different decisions.

The approach we have chosen uses a "risk curve" or complementary cumulative distribution function (CCDF) to characterize the risk. A single CCDF plots the magnitude of an event on the horizontal axis versus the probability of exceeding that magnitude on the vertical axis. This technique is widely used in other risk-analytic activities, most notably in reactor safety studies. This method was used in the 1975 Reactor Safety Study to illustrate the number of potential deaths from an accident at a nuclear reactor as a function of the likelihood of their occurrence. Typically, the plot is log-linear, with the exceedance probability as the ordinate

(vertical axis) on a logarithmic scale and the consequence plotted as the abscissa (horizontal axis). The use of a log-linear scale allows a much finer resolution of the characteristics of low probability events.[3] The risk curve is useful in this regard, as it explicitly represents both the probability and the consequence. For example, whereas a standard "safety margin" approach cannot distinguish between a system failure resulting in low damage and one resulting in high damage, a risk curve can. In contrast to an expected value approach, a risk curve can distinguish between an event with a low probability of occurrence and severe consequence versus a more frequent but less severe consequence. In our curves, we will represent the natural variability or irreducible uncertainty on the ordinate. The epistemic uncertainty is represented by error bands of any desired confidence level that surround that curve.

1.3 Catastrophe Models as Integrated Assessment Models

The catastrophe models examined and developed within IIASA's Risk, Modeling and Society (RMS) project offer a natural setting for applying this expanded conception of risk. Examination of the use of the term "catastrophe model" reveals that such models have evolved from the broadening of actuarial approaches for estimating risk to incorporating the modeling and probabilistic approaches of the other technical risk perspectives. The distinction between catastrophe models and earlier, public-policy-oriented simulation models is that (as pointed out by Renn, 1992) modeling and probabilistic safety assessment (PSA) approaches have historically been used for the purposes of standard setting or for improving technological systems. Catastrophe models differ in that the results are typically used within a risk-sharing framework such as insurance.

A common element in most catastrophe models is the use of decomposition,[4] a staple element in systems-analytic thinking (Raiffa, 1968; Bier *et al.*, 1999). In catastrophe modeling, decomposition is implemented by the creation of modules or submodels. Many authors (Walker, 1997; Kozlowski and Mathewson, 1997; Clark, 2002; Boyle, 2002) define three modules: a scientific or hazard module comprising an event generator and a local intensity calculation, an engineering module for damage estimation, and an insurance coverage module for insured loss calculation. Finally, most catastrophe models produce outputs that are distributional. That is, the results are typically not simply an expected loss but rather a full loss distribution curve that may or may not follow a particular statistical distribution. Based upon

[3] The user must simply keep in mind, when comparing two curves on such a plot, that the use of a logarithmic scale means that equal divisions on the ordinate represent order of magnitude changes. The intuitive understanding of the relative likelihood for a user accustomed to linear plots may be biased to exaggerate the likelihood of low-probability events if this is not consciously acknowledged.

[4] For a thought-provoking discussion of decomposition, see section 6.4 in Bier *et al.* (1999).

these observations, we define catastrophe modeling as a risk-analytic technique that has the four following characteristics:

1) The technique: catastrophe modeling makes use of simulations rather than purely historical actuarial data for the purposes of estimating probabilities and outcomes. One of the main reasons for developing a catastrophe model is that there are not enough historical data for actuarial estimates. Data must therefore be generated by simulating the physical events. This does not preclude the inclusion of actuarial data: it is enough that simulations based on theoretical models rather than statistical analysis of historical data be included as a primary element of the analysis.

2) The structure: catastrophe models are typically modular, that is, comprised of relatively independent submodels. For example, a "hazard" submodel drives the risk, a "loss" submodel estimates some type of loss dependent upon the hazard, and a "management" submodel examines the impact of different decisions. The modular nature of most catastrophe models is important in that it allows (a) the development of a model by interdisciplinary teams and (b) where appropriate, the substitution of a simple and computationally inexpensive reduced-form model for a more complex and computationally time-consuming mechanistic simulation model. The ability of the model to be developed by interdisciplinary teams allows the inclusion of the relevant expertise without requiring all members of the team to be experts in all the disciplines represented in the model. The important element is that all members of the team should have an understanding not only of how to properly interpret the output of the submodels but also of the ultimate use of the model. The ability to implement computationally inexpensive reduced-form models—referred to as "catastrophe generators" by Ermoliev *et al.* (2000)—allows for the use of numerical optimization models that would be analytically intractable and otherwise prohibitively expensive in computational resources.

3) The output: catastrophe models explicitly include both probabilities and consequences (typically, purely financial consequences rather than health and safety or broader economic consequences). In contrast to many deterministic models or probabilistic safety assessments, the catastrophe model does not focus solely on the probability of failure (e.g., the reliability of a system). In contrast to many actuarial methods, it does not collapse the probability and consequence into a single expected value but focuses attention on the entire combination of probabilities and consequences, namely, the probability distribution of consequences.

4) The use: the main difference between a catastrophe model and a more traditional natural hazard risk assessment as applied in public policy analysis is the application. Catastrophe (cat) models have thus mainly been developed for insurance or risk sharing settings. This contrasts with flood damage reduction analyses, which are often focused on loss prevention or loss reduction. Like the public policy models for natural hazard risk assessment described by Petak and Atkisson

(1982), cat models are typically modular simulation models producing a probability distribution of potential losses. The first two elements (a scientific or hazard module comprising an event generator and a local intensity calculation, as well as an engineering module for damage estimation) are essentially the same as the first two modules of the public-policy-oriented models discussed previously. However, a catastrophe model typically extends the public-policy-model approach by overlaying the exposure of the insurer over the distribution of damages to compute potential claims. In a rather novel application, a catastrophe model developed by IIASA for flooding on the Upper Tisza River in Hungary was used to illustrate the policy impacts of options for a nationwide insurance program. This proved useful at a stakeholder workshop, where local residents, insurance companies, and the central government reached a consensus on a policy direction (see Vari *et al.*, 2003; Linnerooth-Bayer and Vari, 2004; Ekenberg *et al.*, 2003).

1.4 Catastrophe Modeling and Uncertainty

Catastrophic risks are low-probability, high-consequences events. Often stemming from low probability, they are plagued by major uncertainties. One less-developed aspect of catastrophe modeling is accounting for epistemic uncertainty. Although many catastrophe models are probabilistic, they often include only aleatory uncertainty, perhaps reflecting the origin of these approaches within the insurance community. However, an explicit consideration of epistemic uncertainty is critically important. Physically based simulation of climate-driven catastrophes is challenging (Petak and Atkisson, 1982; Minnery and Smith, 1996), as no models are yet available that can synthesize accurate predictions of rainfalls, wind speeds, or other climatic phenomena with detailed resolution across the full range of spatial–temporal scales necessary for accurate risk analyses (e.g., from global scale to scales of the order of square kilometers and from annual to hourly scales). When the possibility of climate change is taken into account, the epistemic uncertainties increase dramatically. Petak and Atkisson (1982, p. 186) emphasize that "the results derived from the risk analysis models are not to be considered 'fact.' Much uncertainty is associated with the findings generated by the models." This statement remains as true today as when it was written 20 years ago. Pervasive uncertainties in the underlying science remain. In financial circles, this uncertainty is termed "ambiguity," and a high level of ambiguity is a stumbling block to the success of insurance programs because of the effect it has on insurability (Kunreuther and Roth, 1998, p. 33). One sometimes hears that uncertainty can be reduced by modeling. It is important to recognize that this is not always the case. There is a significant difference between using a model for prediction and using a model for information structuring. Using a model for pricing insurance can be difficult because it may force the

model to be used in predictive mode, where the model may be weak. Models do not necessarily reveal anything new about the world. What they are good at doing is structuring the information that is already available, allowing additional relevant information to be brought to bear on a problem. They may not be able to reduce uncertainty, though, and in fact they may reveal just how uncertain a situation is.

The good news is that there is long experience in risk analysis techniques for dealing with uncertainty and that this experience is being brought in to the field of catastrophe modeling. Considerable progress has been made in methods for the explicit analysis of uncertainty (cf. Morgan and Henrion, 1990; National Research Council, 2000; Bier *et al.*, 1999; and others). Model verification and validation exercises can be conducted to assist in the quantification of uncertainties in catastrophe models. Furthermore, multiple assessments can be carried out. According to Gary Venter (2003) Guy Carpenter stated that a "key to effective catastrophe modeling is understanding the uncertainties involved... it is critical to look at the results from a number of catastrophe models so that we can see what the range of results would be and how different approaches to a problem could lead to different outcomes." The integrated approach presented in this report draws heavily upon one of the authors' experience with the treatment of uncertainty in the field of human health risks from pollutants introduced into the environment as well as from approaches developed for characterizing uncertainty in nuclear power plant risk assessments (cf. Morgan and Henrion, 1990; Covello and Merkhofer, 1993). We are heartened to see that others are beginning to explore this topic as well; for an example of an approach similar to ours that examined the uncertainty in flood risks along the River Rhine, see Merz *et al.* (2002).

1.5 Motivation for Catastrophe Modeling

Given the potential costs and uncertainties associated with catastrophe modeling, what are the advantages? They are considerable. At a minimum, the use of a distributional technique allows a much better characterization of loss possibilities than that embodied in the annual expected loss or the probable maximum loss concept. However, Walker (1997) suggests that the true advantage of catastrophe modeling "lies in the step change described above in the information it provides, not the marginal improvement in a single point calculation... the benefits lie in the overall savings arising from an integrated approach to risk management." A major advantage of these types of integrated models (whether cat models for insurance purposes or public policy models commissioned by national or regional governments) is that they can produce outputs tailored to different stakeholders and multiple hazards simultaneously. "The primary output... may be the loss experienced by a single property or facility (single-site analysis), the aggregate portfolio loss in a particular

catastrophe zone (zone analysis), or the aggregate portfolio loss for a whole state or country, or worldwide, from a particular hazard (specific hazard analysis) or all hazards (multi-hazard analysis)" (Walker, 1997). The outputs from an integrated model of climate risk and seismic risk, for example, could show the distribution of impacts to farmers (both the distribution and across the whole sector), to urban dwellers, to insurers, and to the government treasuries. These distributions of impacts might be the basis for either negotiation or optimization, or both.

To realize these advantages, it is necessary to provide guidance, tools, and practical examples for the effective use of the new information within a risk-sharing context. This has been explored by Ermoliev *et al.* (2000) for the case of insurers, illustrating how catastrophe modeling can lead to improved policies on the part of insurers of their coverage of losses and premiums in an environment of spatial and temporal dependencies. By improved policies, the authors suggest some reasonable objectives on the part of insurers (profits, stability) and premium holders. Furthermore, in contrast to models that are focused on loss prevention or loss reduction, the risk-sharing orientation of catastrophe models leads naturally to their applicability to negotiation processes. The ability of a model to clarify the results of a particular decision on the distribution of risks and benefits or to reveal potential unintended consequences allows parties to a negotiation to examine how different policies and decisions might affect their own interests. The IIASA River Tisza study (see Vari *et al.*, 2003; Ekenberg *et al.*, 2003) examined the use of a catastrophe model in the negotiations between stakeholders (including citizens, local and national government officials, engineers, and insurers) dealing with flood risks on the Tisza River. The use of catastrophe models to examine the concrete impacts of different concepts of fairness as a tool in negotiations on risk may prove to be one of the more novel applications of the technique.

1.6 Objectives and Structure of the Report

This report applies these concepts of risk and uncertainty to a concrete case, namely, the risk of flooding along the Vienna River in Vienna, Austria. Our goal is to illustrate how the techniques discussed above can be applied to the problems of urban flooding. It thereby extends traditional engineering-based approaches to flood risk management, effectively integrating loss-spreading techniques, such as the purchase of flood insurance or the maintenance of a catastrophe fund, with traditional loss-reduction techniques, such as the construction of levees, floodwalls, or detention basins. Furthermore, by representing risk using a CCDF (Complementary Cumulative Distribution Function or "risk curve"), we illustrate: 1) an information-rich approach to dealing simultaneously with probabilities and consequences; and 2) the significant differences between policy alternatives. Finally,

we illustrate how Monte Carlo simulation techniques can be used to address both epistemic and aleatory uncertainty.

The remainder of this report therefore focuses on the elaboration of a catastrophe model for management of flood risks on the Vienna River that fully addresses the range of uncertainties in possible financial losses. We begin with a discussion of the potential problems associated with flooding along the Vienna River and identify flooding of a subway line as the major area at risk. We then briefly examine case studies of previous catastrophic subway floods and use these to develop an empirical model for the estimation of damages from flooding. This model is then integrated with the hydraulic analyses prepared by BOKU/IWHW (Institute of Water Management, Hydrology, and Hydraulic Engineering) to provide an integrated catastrophe model. Following this, the model is used to evaluate a number of different hypothetical mitigation options, both structural and financial, for managing flood risks. Emphasis is placed on the ability to quantitatively compare the results of different options with the results of options integrating both structural and non-structural measures. Both epistemic and aleatory uncertainties are handled explicitly throughout. The report concludes with a discussion of the insights provided by this exercise.

2
Background

The following discussion is summarized from Faber and Nachtnebel (2003), where technical details of the data and models can be found.

2.1 General Description

The Vienna River is one of the largest rivers in the city of Vienna with a catchment area of 230 km^2. As shown in *Figure 2.1*, the river flows through some of the most densely populated districts of the city. The most exposed infrastructure is located along a reach of over 8 km, namely, the subway line, which is constructed in an open section on the right river bank, and the main roads on both sides. From a hydrological viewpoint, flood hazards from the Vienna River are critical because of the large number of impervious surfaces covering wide parts of the catchment, low geological infiltration capacity, and little natural retention. These lead to rapid rises in water level, resulting in flash flooding.

The 12 km urban reach is currently a stone-work and concrete bed with tunneled river reaches. This system was constructed between 1895 and 1915 in parallel with the construction of the city railway. Two sections of 0.375 and 2.156 km were tunneled. The flood-related threat in the city is due to many factors, including large channel slopes and flow velocities, rapid increase of discharge, and the absence of natural retention areas. According to hydraulics estimates and laboratory tests, velocities up to 7–8 m/s and supercritical flow conditions in several sections are expected during extreme floods. Significant backwaters from arch bridges and tunneled sections, lateral waves of +/–0.75 m at 5.5–6.5 m/s mean velocity, as well as transverse water surface inclination in bends are expected to occur during large floods.

2.2 Rainfall Characteristics

As in many small mid-latitude catchments, flooding on the Vienna River is typically flash flooding due to small and meso-scale convective storms embedded in large-scale systems. The duration of these storms is typically from several hours to one day, and flooding is generated because of the fast watershed responses. Even low hills and mountains can intensify storm events in comparison with plain areas

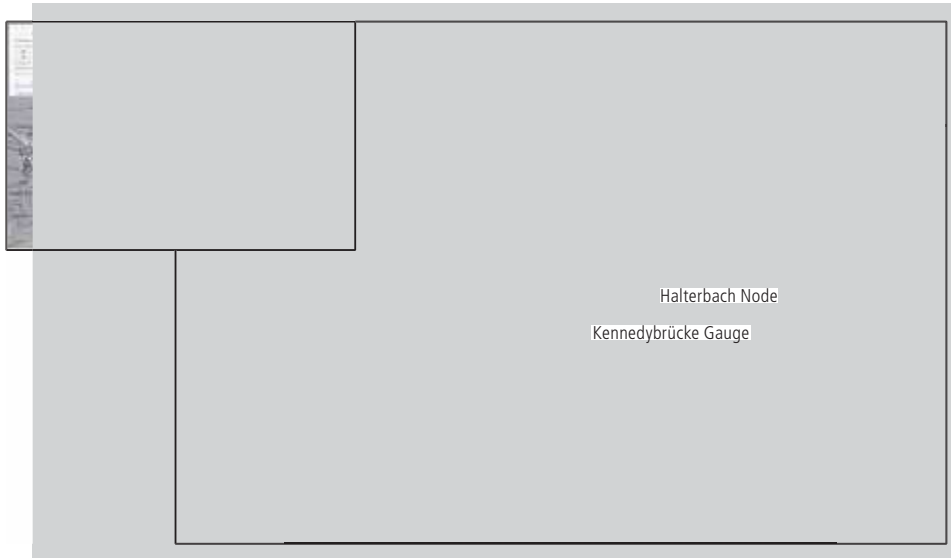

Figure 2.1. Vienna River watershed map.
Source: ÖK 200, BEV (1999).
The Vienna River watershed map shows the watershed with its rural (173 km^2) and urban character (57 km^2); the Halterbach Node which is the outlet of the rural catchment for rainfall runoff modeling the flood retention reservoirs: Auhof, Mauerbach, and Wienerwaldsee; and the Kennedybrücke (Kennedy Bridge) gauge in the urban river reach.

through the regeneration of convective cells (Kelsch, 2001). The orographically intensified convective movement of air masses in the western hills of the Vienna River basin is also documented in the Austrian Hydrographical Atlas (HAÖ, 2003).

As discussed in the Introduction, flood protection tends to rely on the identification of a design flood or design rainfall with a specified annual exceedance probability.[1] Applications of design rainfall data in flood protection and urban hydrology often use rain yield or rain depth relations. Intensity-depth-frequency (IDF) curves are developed for specified regions through fitting mostly exponential functions to recorded rainfall aggregates of partial series. Modeling of very rare storms uses design values developed from local records or regionalized data. These numbers represent conservative estimates of expected values; the parametrical uncertainty is

[1] A simple way to determine the annual exceedance probability is to count the number of years in which the flood exceeded a certain level and divide that by the total number of observations. In other words, a flood with an annual exceedance probability of 10 percent is a flood magnitude that is equaled or exceeded in one out of every 10 years of observation. It may then be referred to as the "10-year" flood.

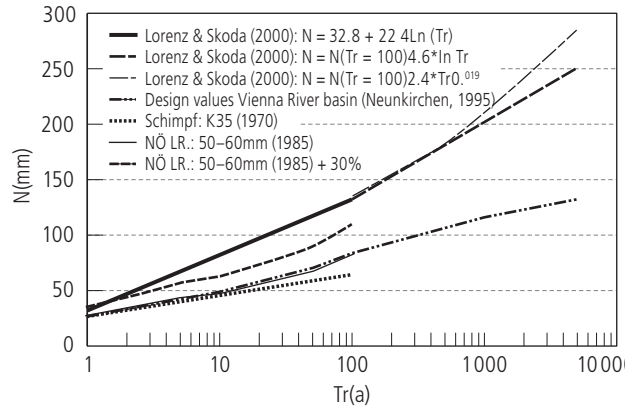

Figure 2.2. Comparison of 6-hour point design rainfall in the rural Vienna River catchment.
Note: Wien River = Vienna River

currently ignored in design and analysis of rainfall-runoff processes. A temporal change of design values can be seen from the one-hour rainfall at Vienna's oldest meteorological station Hohe Warte, which increased steadily from 1957 to 2000 (*Figure 2.2*). The relative extent to which climate change, measurement errors, data processing, and extrapolation uncertainties have contributed to this increase is unclear. According to the Vienna hydrographical service (Pekarek, 1998), the precipitation characteristics and recording and analyzing methods have changed in recent years so that return periods cannot currently be assigned to recently monitored extreme storms. A re-evaluation of the Schimpf criteria and design data, which have been widely used in Austria since the early 1970s, is recommended by that author. These criteria would imply that the 48-hour rain depth of 240 mm measured in the hills west of the city in July 1997 exceeded a 1,000-year event. There are also concerns about the accuracy of the extrapolation of the Lower Austrian 1901–1980 series (Lower Austria, 1985). This concern has led to efforts to establish new design rainfall data for Lower Austria by combining atmospheric models and measurements (Salzer, 2002). In the discussion of design values, attention should be paid to the length of the underlying series, the date of establishment (state-of-the-art methodology), and if measurement errors were corrected, for example, by increasing the raw data by a certain amount. Design values for the greater region around the Vienna River basin have been published by a number of authors, mainly for and from Hohe Warte data. They are now given. However, for reasons of completeness, publications which are not directly relevant to this investigation are also listed.

Steinhauser (1957): Data from the 1901–1955 series were obtained by the Hellmann recorder, selected according to thresholds of half of the Wussow criterion and processed with the Reinhold guidelines (Wussow, 1922; Reinhold, 1935). Amounts for rainfall durations from 5 minutes to 48 hours are given with a maximum return period of 50 years for Hohe Warte.

Schimpf (1970): Values are published for rainfall durations from 30 minutes to 72 hours. For shorter intervals, the Wussow formula is recommended. The regional classification of Kreps and Schimpf (1965) assigns the K35 criterion to the western Vienna area and the Vienna River catchment and the K25 criterion to urban plains and the region with moderate hills. The accuracy of these design values is questionable.

Lower Austrian Government (Lower Austria, 1985): This publication uses the 1901–1980 series and recommends design values up to 48 hours and a exceedance probability of 0.01 for different zones. The western Vienna hills and the Vienna River catchment are located in the region of 50–60 mm mean extreme daily precipitation, whereas the urban areas are in the 40–50 mm zone. This database is no longer recommended, as the values seem too small (Salzer, 2002). It is assumed by experts that an increase of 20–40 percent leads to more accurate values.

Auer *et al.* (1989): Intensity-duration-frequency (IDF) relations are developed for Hohe Warte from 5-minute ombrograph aggregates of the partial series spanning 1973–1982, according to DVWK-ATV (1983). From the 10-year series up to 50-year values were extrapolated for rain durations from 5 minutes to 30 days.

Kadrnoska and Adam (1992): Design recommendations for conduits in Vienna are based on a maximum annual 15-minute rainfall intensity with 105 l/s/ha southwest of the River Danube and 90 l/s/ha northeast. These values are developed from the 1901–1955 series (Steinhauser, 1957). Other rain durations and return periods are usually obtained by using the Reinhold (1935 and 1940) coefficients. Reinhold's time coefficients are applicable for return periods up to 20 years. They are normally used as simplified pipe design tools.

Lorenz and Skoda (2000): Design rainfall is calculated by the OKM (Orographic Convective Model); Lorenz and Skoda, 2000; HAÖ, 2003) using partial series of the ÖKOSTRA project (for the city of Vienna, only the Hohe Warte series is long enough) and a meteorological prediction model for convective storms with orographic influence. Lorenz and Skoda corrected the measurement error by a 5 percent increase in raw data. The orographic influence is accounted for by incorporating a 1.5 km raster elevation model. Durations range from 5 minutes to 12 hours and return periods from 0.5 to 100 years. The authors recommend two formulas for return periods longer than 100 years and a re-evaluation of their results when improved convective models and a larger rainfall database are available. Electronic data were obtained from HZB via MA 45. These model data are available for the whole of Austria and are presently recommended in Lower Austria for durations

of up to 3 hours and return periods of up to 100 years. Values for other durations and return periods have been re-evaluated (Salzer, 2002). These numbers are also published in the digital Austrian Hydrologic Atlas (HAÖ, 2003). Data represent the lower limits of maximum convective precipitation inside a 6 x 6 km area.

Lower Austrian Government (Lower Austria, 2001): A review of the Lower Austria rainfall intensities for the one-year 15-minute storm was published in 2000. It shows values from 110 to 120 l/s/ha around the city and up to 130 l/s/ha in the Vienna River basin (Lower Austria, 2000).

ÖKLIM (2001): This database comprises extrapolated rain data of several durations of the 1991–1999 series of Hohe Warte.

The increase in the design values over time based on observations is evident when one compares Steinhauser (1957), Auer (1989), and ÖKLIM (2001). Higher values due to a different model approach are obtained by Lorenz and Skoda (2002). High values of the Lower Austrian series (1980) and Schimpf's data (1970) are explained by the geographical location of Hohe Warte on the boundary of two regions. The curves represent the higher precipitation class. This underlines the importance of spatial variability.

To establish the design rainfall amounts for flood investigations in the Vienna River basin and protection reservoir adaptation, an extrapolation from the Lower Austria series (1901–1980) and Schimpf's data was performed by Neukirchen (1995), as indicated in *Figure 2.2*. Both these analyses were reassessed, and it was concluded that the storm depths had been underestimated. *Figure 2.2* comprises the 30 percent increased values from the 1901–1980 Lower Austria series. It also shows the values proposed by Lorenz and Skoda (2000) for the urban Vienna River catchment consisting of a curve for return periods up to 100 years and two equations for larger values. Because of the orographic influence, the numbers for the rural Vienna River basin (which are not available) could be even larger, but they are currently re-evaluated for annual probabilities smaller than 0.01 and durations of more than 3 hours.

For this study, it is assumed that reliable values fall between the design values and the Lorenz and Skoda figures; however, there remains a considerable uncertainty concerning the design rainfall depth. This uncertainty is expressed by defining the design storm depth as a random variable following an extreme value distribution and by explicitly considering a normal distributed standard error about the parameters of that distribution.

As rainfall of a larger areal extension has a smaller intensity than a point rainfall of a given frequency, the design rainfall data have to be reduced to obtain estimates for the basin precipitation. For the rural (173 km^2) and the entire Vienna River catchment (230 km^2), areal reduction factors of 95–80 percent are found in Maniak (1988), Gutknecht (1982), and Lorenz and Skoda (2000). As this reduction applies to all point rainfall design values in the same way, it is not used in the project.

Table 2.1. Estimates of peak discharges during significant floods at the Kennedybrücke gauge, Km 7.65.

Peak discharge (m^3/s)	Return period (a)	Date	Reference
472	70	April 1951	Bauer (1993)
374	30–35	July 1975	Bauer (1993)
138	20–25	May 1991	Bauer (1993)
317		7 July 1997	Neukirchen (1997), according to rating curve
285	< 50	7 July 1997	Neukirchen (1997), adjusted
193		7 July 1997	HZB (1999)
125		21 May 1999	HZB (1999)

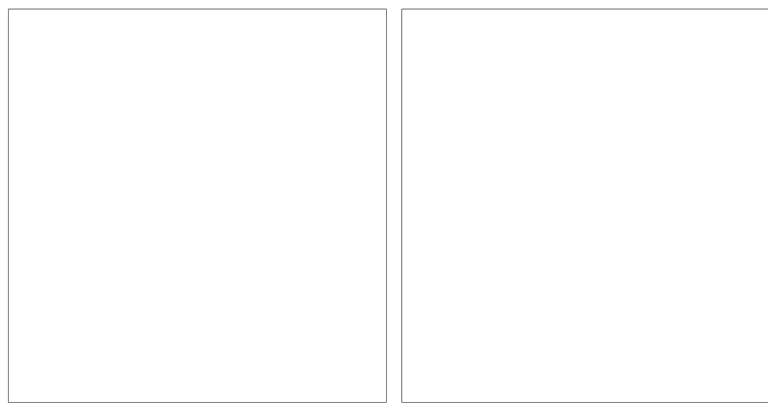

Figure 2.3. Vienna River at Km 8 during normal flow conditions (left) and during the 1975 flood (right).
Source: BMLFUW (2002).

The Vienna River has a mean annual flow, based on data from 1981 to 1999, of 1.16 m^3/s (HZB, 1999). The maximum discharge was estimated for the 18 May 1851 event as 600 m^3/s at the outlet of the Vienna River into the Danube (Bauer, 1993). Some of the larger events in the 20th century were estimated at the Kennedybrücke gauge at Km 7.65. Water surfaces have been recorded since 1904 and discharges since 1981. The Vienna River has experienced extremely large flows in the past, as illustrated in *Table 2.1* and *Figure 2.3*.

However, problems related to the estimation of the probability of larger discharges include undocumented changes in gauge zero before 1958, gradually varying flow conditions, and hydraulic jumps (MA 45, 2001a). Data from 1962 to 1971 are missing. As the available gauge series are not very long or reliable, rainfall-runoff models are used for design and analysis purposes. For the recent upgrades of the Vienna River flood protection system, which started in 1997, catchment

models were developed that account for rainfall-runoff, routing, and storage processes. These models provide flood hydrographs entering the urban river reach. The urban stormwater runoff is estimated and added along the river. It is assumed that the recurrence periods of rainfall and discharge are equal. Catchment models were established by Neukirchen (1985) with a simplified estimation of flood control basin performance, IWHW (1988) included a hydrologic retention basin model and Neukirchen (1995) established a rainfall-runoff model as a basis for the projected real-time control system. This model was calibrated by two flood events in 1991. The largest peak discharge and volume at the city's entrance were calculated for the 6-hour storm. The urban runoff contribution is calculated with a rainfall-runoff and hydrodynamic transport model (data, for example, in Neukirchen, 2000).

2.3 Elements at Risk

Several elements at risk (EAR) are located in the urban river vicinity. The most endangered is the U4 subway line on the right embankment. For 7.5 km it is situated mostly in open sections beside the river before it enters the underground track (*Figure 2.4*). A partition wall protects the subway line from floods. Portable flood barriers can be installed in two locations to prevent the overflowing water from being conveyed to underground sections of the line, which include major subway junctions. These emergency measures are now available; they require a 6-hour lead time for installation. There are main roads on the left embankment, together with densely populated areas. Various service pipes are located under the road embankments.

The construction of the first city railway along the Vienna River was started in 1894 and opened to the public in 1898. It was closed down in 1918 and reopened as an electric line in 1926. The gradual reconstruction to become the transport system of today was begun in 1976 and completed in 1981 (Prillinger, 2000). There are a variety of failure mechanisms that could lead to severe damage to the subway. The term "overflowing" is used for a situation where the mean water level is higher than the wall crest. This contrasts with "wave overtopping," which refers to the temporal and spatial oscillations of the water surface over the floodwall. Although no past inundation or other flood damage to the subway or the embankment has been reported, it is generally agreed that wave overtopping and overflowing of the subway wall may occur at floods slightly larger than a 100-year event. In the event of intensive overflowing and the absence or malfunction of the transverse portable flood barriers located at the track at Längenfeldgasse (upgraded 2001) and Naschmarkt (since 1999), the U4 subway line acts as a flood bypass conveying water downstream to the junctions at Längenfeldgasse, Karlsplatz, and Landstrasse where the tunnels of nearly all connected lines are inundated (see the three crossed circles on

Figure 2.4a. Vienna River.

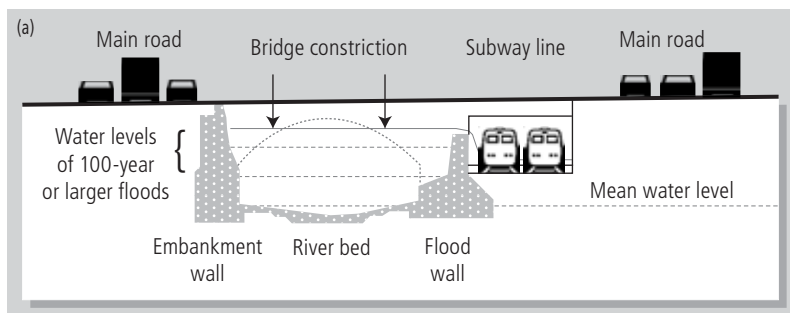

Figure 2.4b. Elements at risk.

Figure 2.4. General situation of the urban Vienna River with the main elements at risk. Map: BEV, ÖK50.

the map in *Figure 2.4a*). In addition, about 1 km downstream of the Auhof basins, local inundation of both embankment roads may occur.

Another failure mechanism is wall collapse. The subway's masonry partition wall was constructed about 100 years ago and subsequently restored. During floods, it is subjected to hydrostatic and dynamic horizontal water forces and also, in particularly adverse conditions, to pore water pressure acting in the wall joints and fissures. Considering the wall geometry of the bends in plan view, the strength also depends on the arch action: concave bends have a slightly higher resistance. Large horizontal forces appear only with extreme water levels, and the loss of equilibrium may cause rapid overflowing. A final failure mechanism is the collapse of the embankment wall on either the left or right bank. The stability of the embankment wall depends on intact subsoil supporting the concrete foundation, which may be affected by the development of large scours close to the foundations. This can happen after the invert material is destroyed by the stream's shear force. It is assumed that intensive foundation scouring results in wall failure, leading to severe damage on the left embankment, in consequence of which the conveying capacity will be reduced by wall and backfill material. The backwater effects will increase the probability of the above-mentioned failure modes. If they occur on the right bank, rapid overflowing into the subway line and stations could occur.

2.4 Flood Protection

Because of the problems discussed above, and because of the desire for the river to have an improved ecological and recreational character, a suite of flood protection activities has been identified. An interdisciplinary study (Bauer, 1993) combined ecological and technical issues to produce a solution that focuses on reconstructing, extending, and adaptively controlling the flood protection works. To improve the flood-carrying capacity of the channelized river and to improve water quality, the study further proposes a large urban stormwater bypass channel below the current river bed. Urban stormwater discharges can reach up to 200 m^3/s at the mouth of the Vienna River in extreme cases (Bauer, 1993; MA 45, 1996). The goal of this project is to reduce the 1,000-year design flood of the rural river basin from its original (pre-1990) value of 475 m^3/s to 380 m^3/s. All the urban stormwater will be conveyed in a bypass channel located in the current river bed. In addition, a forecast-based runoff model for reservoir control will be installed and the retention schemes (*Figure 2.5*) will be adapted. The Mauerbach and Auhof schemes have been rehabilitated to serve ecological and recreational purposes in addition to their flood-protection role. The redesign of the reservoirs was based on hydrologic simulations with a rainfall-runoff model that was calibrated by the May and August 1991 storms. Future work

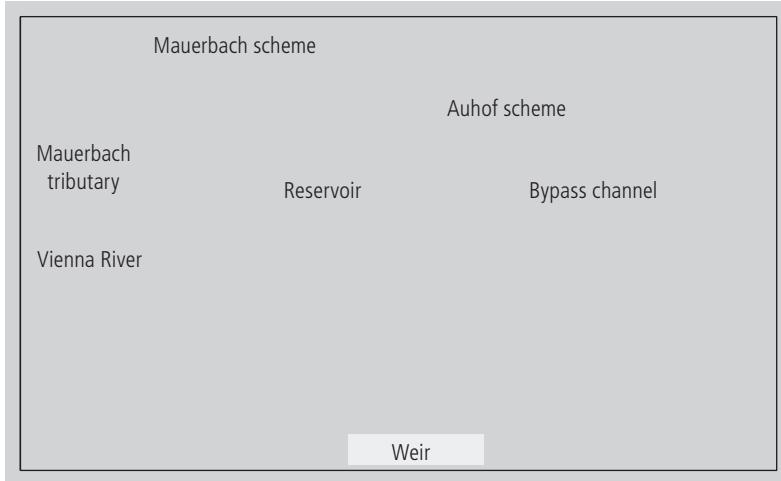

Figure 2.5. Auhof and Mauerbach retention schemes (MA 45, 1996).

will focus on rainfall forecasting for the real-time-controlled basin operation and the implementation of a warning and basin operation system.

The flood protection system in the Vienna River basin is characterized by a sequence of partly upgraded detention reservoirs and a 12 km channelized urban reach. Both the flood control basins and the urban river reaches were engineered from 1895 to 1902. Apart from repairs undertaken over the last century, the urban river is mainly in the form in which it was constructed in 1900. According to a critical analysis in the 1980s, the retention basin provided insufficient protection, as very large hydrograph peaks such as the 100- and 1,000-year events pass through the flood control basins without significant reduction of the flood peak (IWHW, 1988). This is because of insufficient storage volume and control capacity, which causes premature basin filling of the Auhof reservoirs by tributaries of the adjacent hills and by the increasing branch of the Vienna River hydrograph.

The Auhof flood storage system consists of an upstream basin distributing the discharge into the bypass channel or the five-basin storage cascade. During upgrading works completed in 2001, some of the weir crests were increased in height, and hydraulic steel structures were upgraded for adaptive control purposes. The landscape of the basins was redesigned from an ecological viewpoint. The Mauerbach basins consist of a distribution basin and one storage basin. Changes similar to those at Auhof were also carried out at Mauerbach reservoir and completed in 2001.

The Wienerwaldsee is an artificial reservoir with a 13.5 m high barrage constructed in 1894 to provide drinking water of up to 24,000 m^3 per day at times of peak demand and in emergencies (Bauer, 1993). Plans have been drawn up to

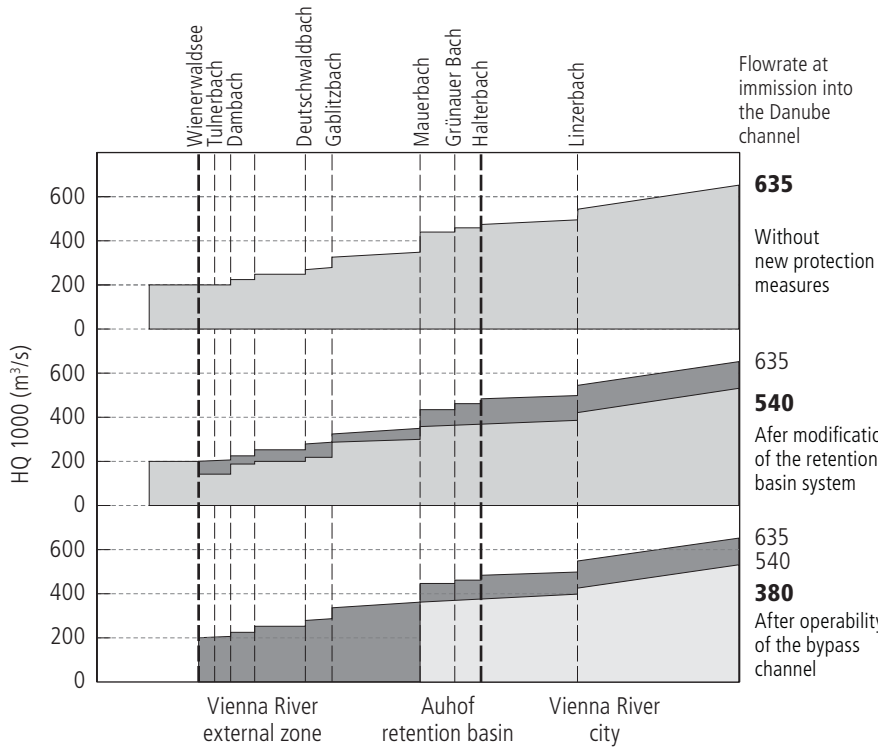

Figure 2.6. Hydrological profile of the 1,000-year design peak discharge (MA 45, 1996).

Table 2.2. Projected retention basin storage capacity along the Vienna River.

Retention Basin	Flood storage volume (m^3)	
	Neukirchen (1997)	Neukirchen (2001)
Auhof	1,160,000	720,000
Mauerbach	160,000	160,000
Wienerwaldsee	520,000	630,000
Total	1,840,000	1,510,000

adapt this basin to serve flood control purposes. These include an extension of the barrage and an expansion of control capacity. However, as of March 2003 these works had not been started. One reason is that other drinking water sources were due to take over its capacity in 2005, and the further utilization of Wienerwaldsee was thus in question. The options of selling the basin to the adjacent Lower Austria communities or using the basin purely for flood protection purposes have been widely discussed (*Kurier*, 2002).

The hydrologic investigations in this study distinguish among several construction and operational states of the retention basin system:

1. Hypothetical natural state without any artificial retention capacity;
2. Reservoir state before beginning of the upgrading works in 1997;
3. Recent (2002) state; and
4. Reservoir state after completed upgrading of Auhof, Mauerbach, and Wienerwaldsee

The effect of upgrading the protection system from no retention effect to full operation of all three reservoirs on the 1,000-year design flood peak is demonstrated in the upper and middle hydrologic profile from Wienerwaldsee to the mouth in *Figure 2.6*. The remaining discharge into the Vienna River in m^3/s can be decreased down to 380 m^3/s starting from 635 m^3/s. The lower profile exhibits the influence of the urban stormwater bypass channel. A detailed description of structural and operational basin conditions can be found in Bauer (1993), MA 45 (1996), and Neukirchen (1995; 1996; 1997). For the projected retention basins see *Table 2.2*.

3
Hydraulic Assessment Model Development

The following discussion has been adapted from the discussion in Faber *et al.* (2003) and summarizes the work described in much greater detail in Faber and Nachtnebel (2003), where the technical details of the data and models used can be found. The objective of the hydrologic and hydraulic analysis was to come up with an estimate of the frequency of failure of the protection system investigated and to give an approximation of the severity of a failure event. These are intended as inputs to the IIASA catastrophe model. Uncertainties in the input data are processed by Monte Carlo methods.

3.1 Stochastic Hydraulic Model: Summary Description

To model the watershed hydrology, the rainfall depth sampled for a defined return period is transferred into a peak discharge by stepwise deterministic relations for different constructional and operational detention reservoir states. These transfer functions were derived from rainfall-runoff models for the rural and the urban river reach (Neukirchen, 1995; 2000). The uncertainty in these models is not included in the Monte Carlo approach. The estimation of relevant flow parameters was carried out with a modified version of the hydraulic 1-D steady-flow model HEC-RAS (HEC, 2001). The HEC-RAS code computes water levels by accounting for sub- and supercritical flow conditions, backwater effects from channel constrictions, and transverse water surface inclination in bends. The computational kernel of HEC-RAS was used within a Monte Carlo simulation framework to assess the probability of failure conditional on user-defined return periods. In addition to the peak discharge described above, several basic random variables were introduced in the Monte Carlo simulations to incorporate uncertainties in the channel roughness, the river cross-section station and elevation, and the energy loss due to bridge constrictions. From the output of each hydraulic model run, the occurrence of several possible flood-induced failure modes was evaluated. These failure types comprise overflowing, structural damages like tipping of a floodwall, scouring of the river bed, and collapse of river bank structures. If a deterministic analysis is performed with expected values, the structural failure modes will not occur. However, the goal

of this section is to introduce parametric uncertainties that may result in system failure at river flow rates close to, but not exceeding, the design flow.

The system of equations describing water level as a function of channel parameters and river flow rate is too complex to propagate uncertainties analytically. Therefore, Monte Carlo simulations are performed by sampling inputs to the hydraulic model to provide empirical conditional probabilities of failure P(F|Tr), given specified return periods Tr = t (e.g., t = 50 or 100 years). A failure curve is fitted to the data points to obtain a continuous function. The simulated events have different but well defined return periods which represent the basic inequality for developing the weighting function of each simulated scenario: the probability of a variable X being larger than or equal to a defined value x indicated by the return period Tr = t:

$$P(X \geq x_t) = \frac{1}{\tau(x)} \tag{3.1}$$

As the equality P(X = xt) is usually described by a probability density function, which is not known, a numerical solution with Dt = 1 year is performed. Equation 3.2 is used as the weighting function assigned to the conditional probability described by the failure curve.

$$P(X = x_\tau) = P(T_r = \tau) \cong \frac{1}{\tau} - \frac{1}{\tau + 1} = \frac{1}{\tau(\tau + 1)} \tag{3.2}$$

a) Conditional probability of failure P(F|Tr)
b) Weighting function for conditional probabilities of failure P(Tr = t)

The total probability concept (e.g., in Ang and Tang, 1975; Plate, 1993) is used for the integration of all conditional probabilities weighted by their occurrence probability and gives an estimate of the probability that the system fails in one year:

$$P(F) = \sum_{\tau=1}^{\infty} P(F|T_r = \tau) \frac{1}{\tau(\tau + 1)} \tag{3.3}$$

For the failure assessment, basic random variables are introduced describing the water pressure in the floodwall, the critical river bed material's shear stress, and the scour depth and center for the failure mechanism. Further, the partly blocked flow profile due to collapsed bank structures and backfill material is explicitly modeled by a randomly changed cross-section geometry.

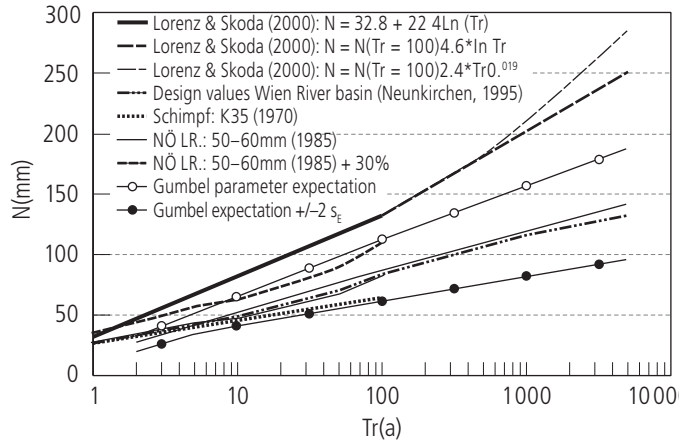

Figure 3.1. Design values of 6-hour storm depth in Vienna River catchment with fitted model curves according to *Table 3.3*: Expectation +/- 2 S_E.

Table 3.1. Expected annual 6-hour storm depth.

	Lorenz (2002)	Schimpf (1970)	Lower Austria (1985)
N(Tr = 1a)	32.8 mm	27 mm	28.51 mm

3.2 Stochastic Hydraulic Model—Parameters

3.2.1 Design Storm Depth Parameters

The storm depth is modeled by a Gumbel distribution. The required parameters for this distribution are the mean value and standard deviation S_N. Parameters have to be estimated from design recommendations; these consist of few N(Tr) points for the given storm duration, as the underlying record series are not available. The parameters were estimated by manually fitted curves in *Figure 3.1*, and through the discussion regarding the accuracy of design values in the preceding sections. *Table 3.1* gives an orientation for the parameter estimation.

The assumed parameters are N_m = 29.44 mm and S_N = 16.74 mm, indicated by the circle-marked lines in *Figure 3.1*. The extrapolated values represent expectations based on potentially erratic data and a limited sample size; therefore a measure of uncertainty is developed. For a numerical solution, the basic variable S_x is estimated by $E(S_N)$ in order to express S_E. *Table 3.2* and *Figure 3.1* show the total scattering and its components.

Table 3.2 exhibits expected values N_Tr, data scattering S_E due to limited sample size n, and raw data error. It also shows ranges at 95.4 percent confidence interval. For modeling purposes, similar results can be achieved in a simplified way by

Table 3.2. Estimated parameters and Gumbel statistics.

GUMBEL	Estimated parameter		Mean	Standard dev.	Assumption			
	Mean value	N_m (mm)	29.44		Constant			
	Standard deviation	S_N (mm)	16.74	3.125	Normal distributed: 95.4% between 10.5 and 23 mm			
	Sample size	n	80		1901–1980 series			
	Reduced mean value	Y_η	0.5569					
	Reduced standard dev.	$S_{Y\eta}$	1.1938	Standard dev. due to limited sample	data error	both		

P_M = 1-1/Tr	Tr(a)	Y_{Tr}	K	N_{Tr} (mm)	S_E(Tr) (mm)	K(Tr)σ_{mN} (mm)	σ_{Nm} = ($S_E{}^2$+ (K σ_{nM})^2)$^{1/2}$	Nm(S_{Nm}) +2σ_{Nm} (mm)	Nm(S_{Nm}) -2 σ_{Nm} (mm)
0.5	2	0.3665	–0.1595	26.77	1.7	–0.5	1.79	30.36	23.18
0.8	5	1.4999	0.7899	42.67	3.0	2.5	3.89	50.46	34.88
0.9	10	2.2504	1.4186	53.20	4.1	4.4	6.05	65.30	41.10
0.98	50	3.9019	2.8020	76.37	6.7	8.8	11.03	98.43	54.31
0.99	100	4.6001	3.3869	86.17	7.8	10.6	13.17	112.50	59.84
0.999	1,000	6.9073	5.3194	118.54	11.6	16.6	20.26	159.05	78.03
0.9998	5,000	8.5171	6.6679	141.13	14.2	20.8	25.22	191.56	90.69
0.9999	10,000	9.2103	7.2486	150.85	15.3	22.7	27.35	205.56	96.14

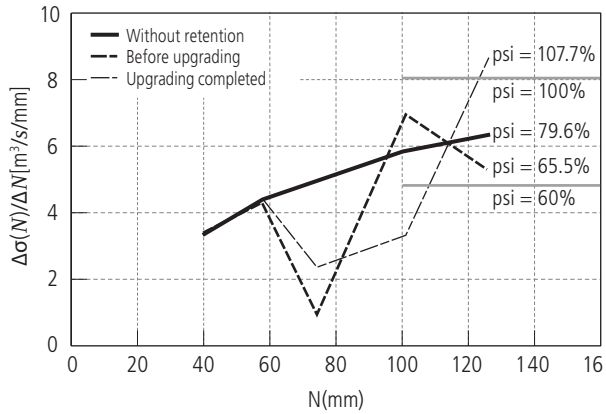

Figure 3.2. Storm depth–peak discharge transfer curve for the 6-hour rainfall in the rural Vienna River watershed at the Halterbach node.

representing all uncertainties by the standard error $S_{E(Tr)}$. To achieve corresponding plausible scattering, the underlying sample size is reduced to n = 30. The mean value is estimated at 29.44 mm and the standard deviation is chosen manually at 15.7 mm.

The standard error S_E obtained in *Table 3.3* corresponds to 12, 15, and 16 percent of the expected N_{Tr} of the 10, 100, and 1,000 year events, respectively. Nobilis (1990) estimated a standard deviation of 15 percent of the expected extreme rainfall in Austria. DWD (1997) and Skoda (2003) report similar deviations for Germany and 24-hour rainfall in Lower Austria. As the expectations and confidence intervals of the simplified and detailed uncertainty investigation diverge by less than 1 mm and 5 mm, respectively, the simplified approach is implemented in the hydraulic FORTRAN model developed by Faber and Nachtnebel (2003).

3.2.2 Rainfall-Runoff Transfer

The concept for transferring the storm depth into a rural and an urban contribution was discussed in section 3.1. The storm depth-peak discharge relations implemented in this study are mainly derived from the calibrated catchment model for the rural river basin (Neukirchen, 1995) and from a hydrodynamic urban runoff model, which can handle the conduit network when it is loaded over its capacity (Neukirchen, 2000). As neither project provides enough information on some of the discrete events and the underlying models are not available, the continuous curves comprise assumed data points beyond the conventional design calculations and use linear interpolation. The assumed data points were obtained via the rain depth-peak discharge gradient in *Figure 3.2*.

Table 3.3. Estimated Gumbel parameters and extrapolated rain-depth values.

GUMBEL	Parameter						
	Mean value		N_m (mm)	29.44			
	Standard deviation		S_N (mm)	15.7			
	Sample size		n	30			
	Reduced mean value		Y_η	0.5362			
	Reduced standard dev.		$S_{Y\eta}$	1.1124			
P_M = 1-1/Tr	Tr(a)	Y_{Tr}	K	N_{Tr} (mm)	S_E (mm)	N_{Tr}+2S_E (mm)	N_{Tr}-2S_E (mm)
0.5	2	0.3665	–0.1525	27.05	2.65	32.34	21.75
0.8	5	1.4999	0.8664	43.04	4.81	52.66	33.43
0.9	10	2.2504	1.5410	53.63	6.64	66.92	40.35
0.98	50	3.9019	3.0257	76.94	10.92	98.79	55.10
0.99	100	4.6001	3.6533	86.80	12.77	112.34	61.26
0.997	300	5.7021	4.6439	102.35	15.70	133.76	70.94
0.999	1,000	6.9073	5.7273	119.36	18.93	157.22	81.50
0.9998	5,000	8.5171	7.1745	142.08	23.25	188.58	95.58
0.9999	10,000	9.2103	7.7976	151.86	25.12	202.09	101.63

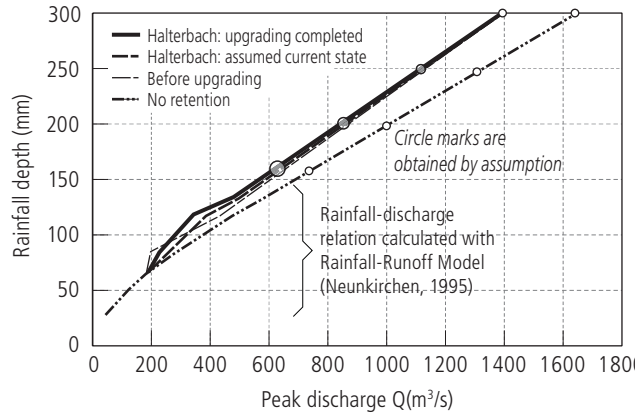

Figure 3.3. Gradient DQ(N)/DN from rainfall-runoff simulations (Neukirchen, 1995) with runoff coefficients y(N).

The rain depth-peak discharge curve at the Halterbach node, the lowest node of the hydrologic model, for the system state without retention basins is extended by a peak runoff coefficient of y = 79.6 percent (*Figure 3.2* and *Figure 3.3*). This value is assumed constant because of the steady precipitation losses of saturated soils. The magnitude corresponding to DQ/DN = 6.38 m^3/s per additional 1 mm precipitation in 6-hour in the 173 km^2 watershed is determined as the peak flow increase between the simulated 118 and 134 mm/6-hour scenario. The rain depth-discharge curve at Halterbach for the system state before the retention basin adaptation is extended by a peak runoff coefficient of 65.5 percent. This number is smaller than the previous one because of the natural storage in the uncontrolled reservoirs. It refers to DQ/DN = 5.25 m^3/s,mm in 6 which is the peak discharge increase between the simulated 118 and 134 mm/6-hour scenario. The curves representing the completed adaptation and the assumed current state at Halterbach are supposed to converge to the natural storage relation when rain depths become much larger than the designed controllable capacity.

The relations for the urban runoff were extended by using the mean peak runoff coefficient of the 118 to 134 mm segment of the five curves indicated in *Figure 3.4*. Because of the dependency of rural and urban rainfall, only a reduced amount of the urban runoff accounts for the design events of given return periods. The numbers in *Figure 3.4*, resulting from hydrodynamic urban rainfall-runoff simulations and the decrease to 70 percent are established by Neukirchen (2000). *Table 3.4* shows a summary of the raindepth-peak discharge relations used for the rural and the urban catchment and their design return periods.

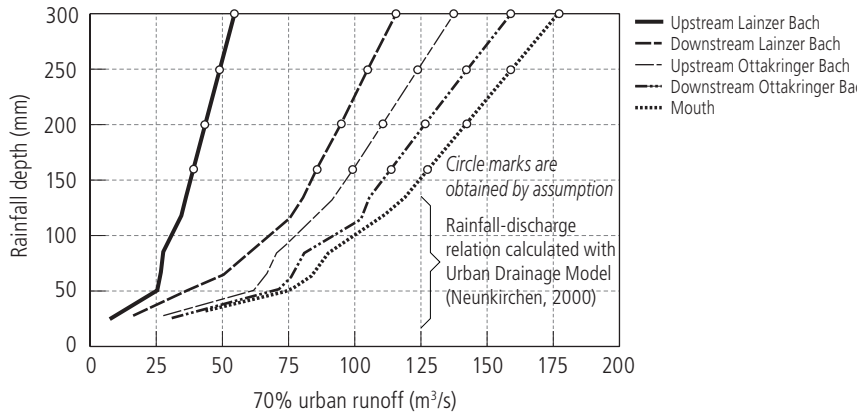

Figure 3.4. Transfer curves of storm depth and reduced discharge for the urban Vienna River catchment and the 6-hour rainfall. Curves represent different river stations.

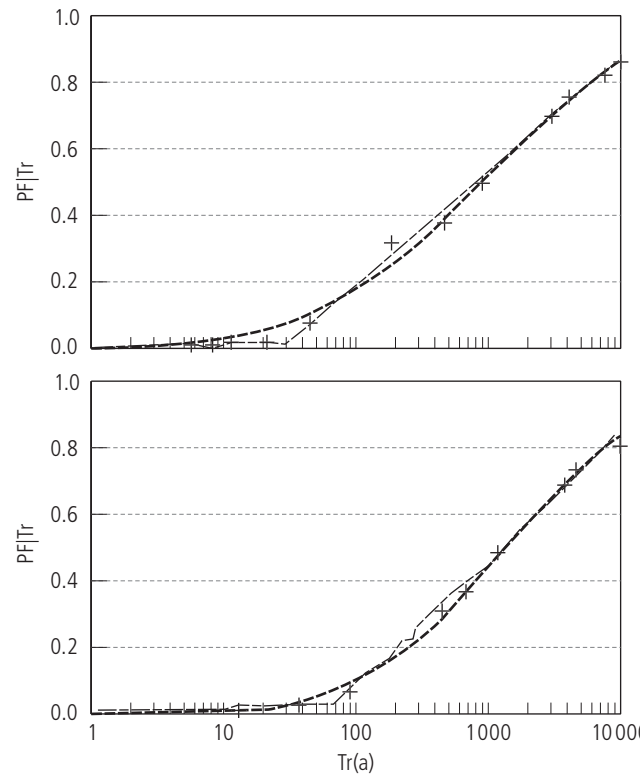

Figure 3.5. Conditional probability of failure for the current and the projected state of the flood control reservoirs. Crosses denote simulated data points; curves are fitted to obtain continuous functions, a) Logarithmic, b) Lognormal.

Table 3.4. Summary of 6-hour rain depth-peak discharge reation, 17 October 2008.

		Discharge rural catchment				Discharge urban catchment				
Return period Tr(a)	Rain-fall depth (mm)	No retention	Before upgrading	Assumed current	Completed upgrading	Upstream Lainzer Bach Km 8.24	Down-stream Lainzer Bach Km 8.24	Upstream Ottakr. Bach Km 2.4	Down-stream Ottakr. Bach Km 2.4	Month Km 0
1	28	44	43	42	43	8	16	28	31	35
10	51	122	120	119	119	25	37	62	71	77
30	65	184	180	179	180	27	52	67	77	85
100	84	278	197	252	226	28	62	71	81	91
1000	118	477	434	379	340	35	76	87	103	112
5000	134	579	518	495	478	37	81	92	106	119
	160	745	655	640	630	40	87	100	115	128
	200	1,000	865	865	865	44	95	111	127	142
	250	1,319	1,127	1,127	1,127	49	105	125	143	160
	300	1,637	1,390	1,390	1,390	55	116	139	159	178

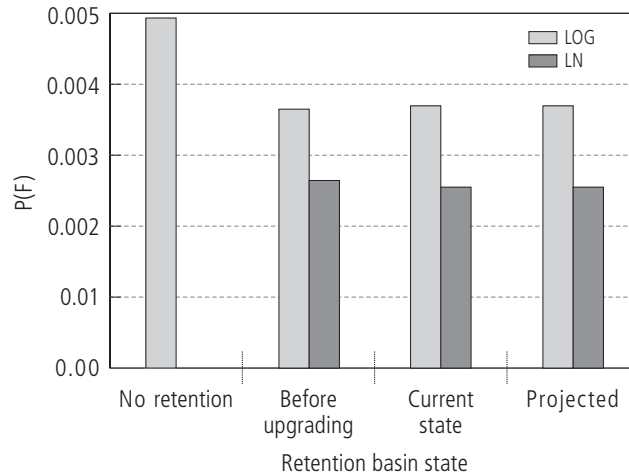

Figure 3.6. Total probability of failure for different structural and operational retention basin states.

3.2.3 Probability of Failure

The hydrologic/hydraulic simulations for the conditional probability of failure indicated in *Figure 3.5* cover scenarios of 12 return periods Tr and different states of the flood control reservoirs. The difference of a small shift in the fitted failure curves stems from activating the Wienerwaldsee reservoir for active flood storage.

The failure curves in *Figure 3.5* are a three-branched logarithmic (LOG) and a cumulative log-normal function (LN) which are processed with the total probability concept into the probability of failure. The small deviation of P(F) of the state before the upgrading of the Auhof and Mauerbach retention schemes and the current state can be explained by hydrologic simplifications, the crude underlying data assumptions, and the unintended smoothing effect of curve fitting. *Figure 3.6* shows the total probability of failure for different structural and operational retention basin states.

4
Damage Assessment Model Development

The objective of Chapter 4 is to develop an empirically justified approach to modeling financial losses from subway flooding. Here, we consider only direct tangible damage via a two-pronged approach. The first part is to examine past instances of catastrophic flooding of subways. The second is to examine and extend analytical approximations used by Neukirchen (1994), based on expert judgment, to estimate the losses from a catastrophic flood. These approaches are then compared and harmonized to develop an empirically justified model for estimating the damage. A major advantage of the review of past cases is that an approximation of the uncertainty in the results can be developed by looking at the damage ranges in past major floods.

It is clear that damage to a subway system would be a function of many variables, such as:

- Length of track flooded; level of standing water in the stations and along the rails;
- Duration of the inundation (e.g., electrical systems may be designed to withstand short periods of rain but would fail when completely submerged for periods of hours or days);
- Velocity of the water (e.g., a slow rise in water level may not damage ballasted track, whereas a high velocity current may scour ballast and foundations);
- Amount of warning time available for mitigation measures to be taken (e.g., operating protection systems, installing additional pumping capacity, removing high-value equipment such as computers, etc.);
- Composition of the floodwater (e.g., salt water, silt-laden water, etc.); and
- Others not identified above.

Unfortunately, there are no data available to adequately characterize the quantitative relationship between all these factors and the damage expected after a flood. The situation is similar in this regard to the long-standing practices in potential flood damage assessment, which can use only a subset of all possible contributory factors. In this report, an approach to damage estimation will be used that is conceptually similar to that of the use of depth-damage curves for flood damage assessment (Penning-Roswell *et al.* 1977; N'Jai *et al.*, 1990; Davis and Skaggs, 1992). The basic approach to estimating the flood damage during flooding of the subway

is to assume that there is a relationship between the length of track flooded and the resulting direct damage, as carried out by Neukirchen (1994). This relationship can be expressed as follows:

$$DD = \alpha \cdot L, \tag{4.1}$$

where DD is the direct damage, L is the length of track flooded, and α is the relationship between the two variables.

As we were unable to obtain adequate empirical data on the effects of other factors, and as we wished to connect the output of the hydraulic model to the damage estimation model, we evaluated the impact of the magnitude of the flooding by introducing a modifying factor to adjust the damage estimates produced by Equation 4.1. The modifying factor is defined as a function of the rate (m/s) of overflowing water.

$$\beta = f(Q_{\text{overflowing}}), \tag{4.2}$$

where β is the damage multiplier and $Q_{\text{overflowing}}$ is the rate at which water enters the subway system because of overtopping or wall failure.

As there was no information available to evaluate this function, it was implemented in the model by the use of a constrained engineering judgment. It was assumed that the results of Equation 4.1 represent reasonable worst-case damage and their modifying factor is defined on the interval (0.1). For convenience, we choose an exponential function of the form

$$\beta = 1 - e^{-\lambda Q_{\text{overflowing}}}. \tag{4.3}$$

This functional form has the advantage of being continuously differentiable and of rising to close to the maximum at a rate controlled by the constant lambda. To evaluate a conventional value for lambda, previous analyses by Neukirchen (1994) were evaluated on the basis of engineering judgment. It was presumed that the platforms are inundated quite quickly after water begins flowing into the subway channel. We presume that damage begins to be incurred at a rate of 20 m^3/s and that an overflowing water discharge of 63 m^3/s is sufficient to lead to significant damage in the subway system. The water level in downstream subway stations was modeled for this flow rate, and it was shown that the water level would inundate platforms by significant amounts (0.5–2 m). Based on this, we presume that a level close to the "maximum" level of damage (damage percentages corresponding to the complete inundations experienced by Taipei, Prague, and Boston: see below) are reached at flow rates slightly higher than 63 m^3/s. We therefore choose lambda to yield 50 percent of the maximum at a flow rate of 20 m^3/s.

The rather simplistic manner in which this factor is treated should not be taken as an indication that it is trivial. This factor is very important in the integrated assessment in that it is the point of linkage between the output of the hydraulic model (Faber and Nachtnebel, 2003) and the damage model. As will become apparent, the results of our model are not particularly sensitive to the functional form of this factor. This is because of the nature of the floods faced by the system under study. Flash floods which result in rapid rises in water level to a level that results in maximum damage tend to reduce this to a binary 0/1 variable. This may not be the case in other systems in which the system reliability may attenuate flows or in which the elements at risk have the ability to absorb exposure to the hazard before reaching 100 percent failure. Any application of the approach presented in this report would require a careful consideration of the appropriate form for this factor.

The final form of the damage equation is therefore

$$DD = \alpha(1 - e^{-\lambda Q_{\text{overflowing}}})L. \tag{4.4}$$

It is important to note that this procedure yields only a crude approximation of the actual damages. The use of such a simple approximation inevitably introduces large uncertainties in the resulting calculated damage. However, there are insufficient data to adequately quantify the results of the other factors. The examination of actual case studies and the explicit evaluation of the uncertainties are thus a critical part of any analysis of this kind.

With this approach in mind, we turn our attention to the development of an appropriate value for alpha and its attendant uncertainties. We will do this by a review of case studies and by examination of the approach implemented by Neukirchen (1994).

4.1 Case Studies

A review of news reports was carried out to identify cases of flooding in subways. There have been a number of cases of flooded subways reported in the last decade. In December 1992 a powerful storm near New York City resulted in coastal flooding that inundated the Hoboken Terminal of the Port Authority Trans-Hudson (PATH) Corporation. Approximately 1 km of the PATH tunnel was flooded (Beardsley, 1993). In June 1999 heavy rainfall resulted in the inundation of several subways in the city of Fukuoka, Japan, because of the sudden overtopping of the Mikasa River (Toda and Inoue, 2002). On 17 December 1999 the subway system in Caracas (Venezuela) was shut down as a result of flooding (Jones, 1999). Several days of rain in Chile in June 2000 shut down the subway systems in Santiago and Valparaiso (UPI, 2000). However, damage from these cases was not reported.

There have been at least four cases in the past decade where floods were reported to have caused direct damage amounting to more than €10m (in repair costs) and service outages of more than a week. Two of the most severe cases were during the course of this study. These cases (Boston, October 1996; Seoul, May 1998; Taipei, September 2001; and Prague, August 2002) will be described below.

4.1.1 Boston, Massachusetts, United States

The Massachusetts Bay Transport Authority operates four rapid transit lines of 100 km in length in the metropolitan Boston area known locally as the "T." One of these, which includes the oldest subway system in the United States, is the 40-km-long Green Line (so named because it runs along the park system was designed by Frederick Law Olmstead and is known as the "Emerald Necklace" of Boston).

On the weekend of 19–20 October 1996 a powerful storm system delivered over 250 mm of rain in Massachusetts over a period of two days. The rainfall caused a tributary of the Charles River known as the Muddy River to overflow its banks near its confluence with the Charles River. This, combined with the backing-up of the local drainage systems due to the high level of the Muddy River, caused floodwater to enter the subway system between the Kenmore Square and Hynes Convention Center/ICA stops (see *Figure 4.1*). Most of the damage was associated with the 53,000 m^3 of water that filled the Kenmore Square Station to a depth of over 7 m. Other less-flooded stations included Symphony, Prudential, Hynes, Copley, and Arlington. The total length of track flooded was approximately 2–3 km (Brown, 1996a, 1996b; CDM 2001; Moore and Chiasson 1996; Mercurio, 2002).

The design standard of the Boston metro was not reported, although the storm was reported to be an approximately 200-year event. Damage was quite extensive. Damaged items included track switch motors, signaling systems, power distribution systems, tracks, and escalators. Much of the system was restored to operation within a week, although signaling and track switching were carried out manually for some time because of the loss of the electrical and communication systems. No deaths or injuries were reported. The total damage was estimated as possibly exceeding $10m, and the total cost of upgrades to the signaling system was over $30m. Some of the repair and upgrade costs were to be financed by the U.S. federal government through the Federal Emergency Management Agency (Brown 1996a, 1996b; Mercurio, 2002).

An interesting aspect of the Kenmore Square flooding is the failure of a portable flood-barrier system installed after a catastrophic flood in 1962 (Mercurio, 2002; Moore, 1997). Although the slots for a barrier had been installed, the boards used to block the system could not be located in time to prevent the floodwater from entering the station. Efforts to put sandbags in place failed, as they had in 1962. The revised operating plan calls for provisions to adequately secure the boards

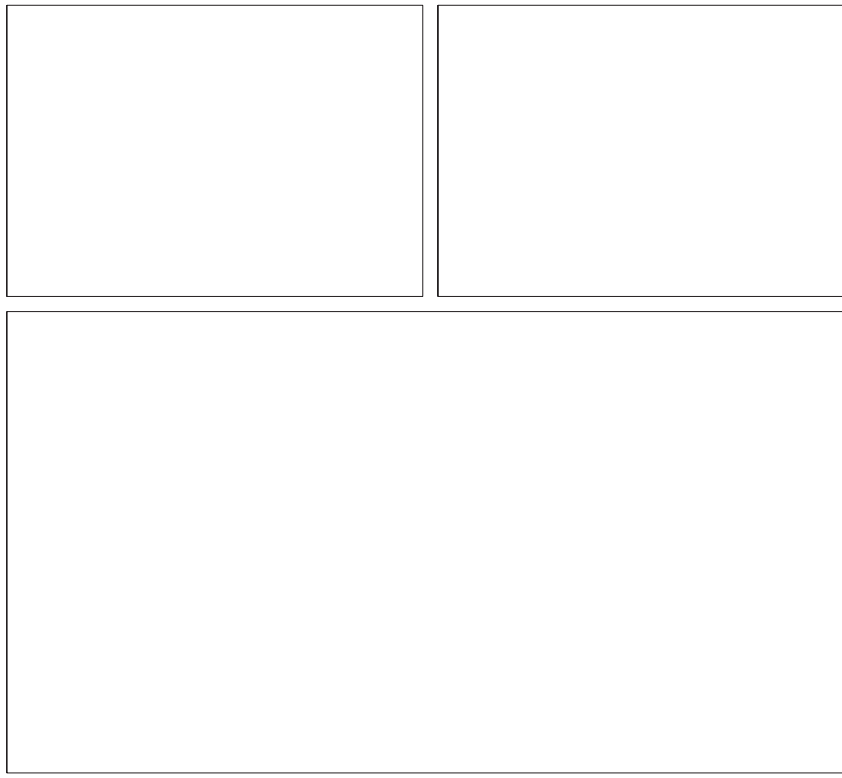

Figure 4.1. Images from flooded Kenmore Square Station.
Photo Credits: WBZ-TV/CBS, WLVI-TV/WB, John Tlumacki (*Boston Globe*).

used to complete the flood barrier, including keeping the boards "under lock and key near the tunnel entrance." The temporary system has been installed on four different occasions since the 1996 floods (Mercurio, 2002).

4.1.2 Seoul, South Korea

Subway Line 7, owned and operated by the Seoul Metropolitan Rapid Transit Corporation, links northeast and southwest Seoul. Construction on Line 7 began in 1994 and was completed in 2000 at a total cost of 868.4 billion won (approximately €800m). The total line comprises 42 stations over a distance of 45 km between the Jangam and Onsu Stations.

A review of press reports yielded relatively few data on the flooding that damaged the line on 2 May 1998. The flooding occurred when retaining walls installed at a construction site on Line 6 built along the Chungnang Stream were breached at 7:30 A.M. during heavy rainfall. The water flowed into Taenung Station on

Line 7 nine minutes later, and proceeded to inundate 11 stations over a length of approximately 11 km with approximately 800,000 m^3 of water. The primary damage was to flooded electric facilities and communication systems. The damage was reported to amount to around 45 billion won (approximately €41.5m). Line 7 was completely out of operation for nine days and was operated at reduced capacity for a further 35 days. The line suffered a decline in ridership of approximately 40 percent (from 500,000 to 300,000 commuters per day) as a result of the reduced capacity.

4.1.3 Taipei, Taiwan

The Taipei Rapid Transit Corporation (TRTC), a joint stock company financed primarily (74 percent) by the Taipei City government, operates six subway lines with a total of 66.7 km of track in the Taiwanese capital city of Taipei (http://www.trtc.com.tw/).

On 16–17 September 2001 Typhoon Nari produced 425 mm of rain in Taipei over a 24-hour period, causing the worst flooding in over 400 years (Chang, 2001). The rains caused extensive flooding of the metro, resulting in the suspension of operation of all subway lines with the exception of the elevated Mucha line (Hsu, 2001). The heavy rains flooded the control center in the basement of the Taipei main station, Kunyang Station, and damaged the "third rail" between the Pannan and Longshan Temple Station on the Pannan line. The flooding of the main railway station occurred 12 hours after the flooding of Kunyang Station. The floodwaters entered Kunyang station through a 6 m^2 hole in the basement of the Chunghsiao-Fuhsing station. This hole apparently should have been closed after completion of construction, but this was overlooked by the contractor (Chuang, 2001) (see *Figure 4.2*). Attempts to sandbag the high point in the line at the Yung Chun station were unsuccessful, and the floodwaters entered the Taipei Main Station at 11:45 A.M. on 17 September. The Municipal Rapid Transit (MRT) control station is located on the third lower level of the Taipei main station, and the computer servers and power supply are located on the fourth lower level. By 2:00 P.M. the floodwaters from the main railway line had also entered the Taipei main station. By late afternoon the control center had to be abandoned. Approximately 30 percent of the computers and screens and all of the power supplies and cables, were lost (Kearns, 2001).

The line between Kuting and Nanshihchiao was reopened on 20 September, and the north–south Tamsui–Hsientien line was back in limited operation on 1 October, with the exception of the Shuanlien stop and the Taipei main station. The Panchiao–Nankang line between Hsinpu and Hsimen was restored to operation on 14 October, with the Hsiaonanmen extension opening on 17 October. By 14 October the system was back up to 58 percent of its pre-typhoon daily average of 900,000 passengers

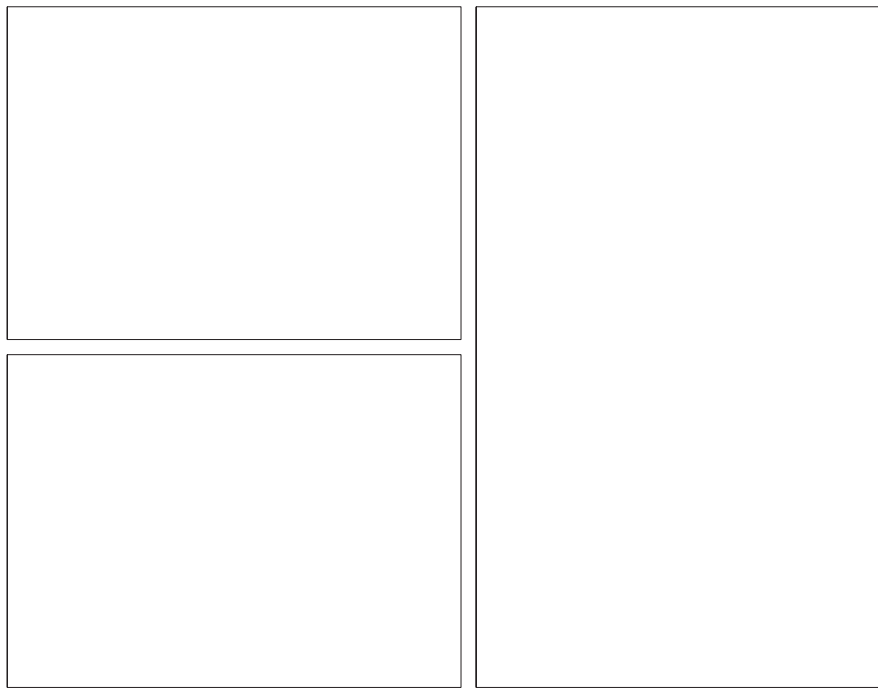

Figure 4.2. Cleanup and repair work on the MRT.
Photo credits: George Tsorng, *Taipei Times*.

per day. The line between Hsimen and Chunghsiao-Fuhsing was reopened on 27 October (Shu-Ling, 2001).

Typhoon Nari, which was a 200-year flood event, exceeded the design standard for flood protection of the Taipei metro. According to Kuo Tsai-ming, deputy director of the TRTC, the most affected systems were "communications equipment, escalators, fire safety equipment, the drainage system, and the wire and ventilation systems installed in the ceiling" (Shu-Ling, 2001). Another report indicates that the repair of the electrical systems was "by far the most daunting task" (Chou, 2001). No deaths or injuries were reported as a result of the subway flooding, although approximately 100 people were killed during the typhoon, mainly as a result of mudslides in the north of Taiwan.

Reports of the estimated direct repair costs for the flooded subway ranged between €66 and 140m (2–4 billion new Taiwan dollars [NT$]) (Kearns, 2001; Surenkok, 2001). In a final report on the total repair bill, this amount was lowered to NT$53m because of cost savings associated with "donations of construction materials and reduced prices from companies not wanting to be seen as making a profit from the typhoon's aftermath" (Shu-Ling, 2001). Funding for repairs was sought

from the municipal Department of Rapid Transit Systems, which tried to raise funding from both the central government and through "austerity measures" conducted in other municipal bureaus and departments (Shu-Ling, 2001). Insurance was not available, as the system was insured only against fire and lightning damage. According to Lee Po-Wen, chairman of the TRTC, the system was not insured against typhoons because of the high annual premium costs of €3.3m, or NT$100m per year (Kearns, 2001).

4.1.4 Prague, Czech Republic

The Prague metro, built in the 1970s and 1980s and operated by the Prague Public Transit Co. Inc., consists of three lines covering 50 km with 51 stations (ww.dp-praha.cz). Daily ridership is approximately 1.2m. As the system was also designed to serve as a fallout shelter, many stations were built with steel doors that would seal off the stations in the event of either flood or nuclear attack (Krushelnycky, 2002).

In August 2002 there were two exceptionally heavy periods of rain in the Bohemian basin due to a slow-moving tropical depression. The first occurred between 6 and 7 August. The second occurred between 11 and 13 August (www.praha-mesto.cz/povoden). In Prague, the Vltava River began to rise on 12 August. On 14 August the river rose rapidly and overflowed its banks (Kikuchi and Sasaki, 2002). The low-lying Karlin district was the most severely affected. Although 1 m high barricades were erected, the water overflowed the barricades, entering the Florenc, Krizikova, Invalidovna, and Palmovka stations on the B Line in the Karlin district and the Nadrazi Holesovice subway/train station on the C Line (Metrostav, 2002). Because of the depth of the subway lines, water cascaded through the tunnels, flooding approximately 17 stations (see *Figures 4.3* and *4.4*) over a distance of approximately 20 km. Although the flooding appeared first on the B Line, the underlying A Line was flooded when a wall collapsed in Mustek station, which is on both the A and B Lines. One station (Florenc) was reported to be flooded to a depth of 35 m, with two trains trapped on the tracks (Carey, 2002). Over one million m^3 of water were pumped out of the system (Konviser, 2002). The return period of the water levels in the Vltava River was estimated to correspond to a 500-year flow. The peak flow rate during the flood was estimated as 5,300 m^3/s, which compares to a annual average flow of 145 m^3/s and a 100-year return flow of 3,700 m^3 per second (http://www.praha-mesto.cz/povoden/).

The metro was at least partially insured by Ceska Kooperativa (Insurance Letter, 2002). Approximately €100m of a European Investment Bank loan was earmarked for repair costs to the metro (CAN, 2002). The loan was for 30 years with a 7-year grace period (EIB, 2002). There was considerable controversy surrounding the flooding of the metro. It was reported that the emergency door in the

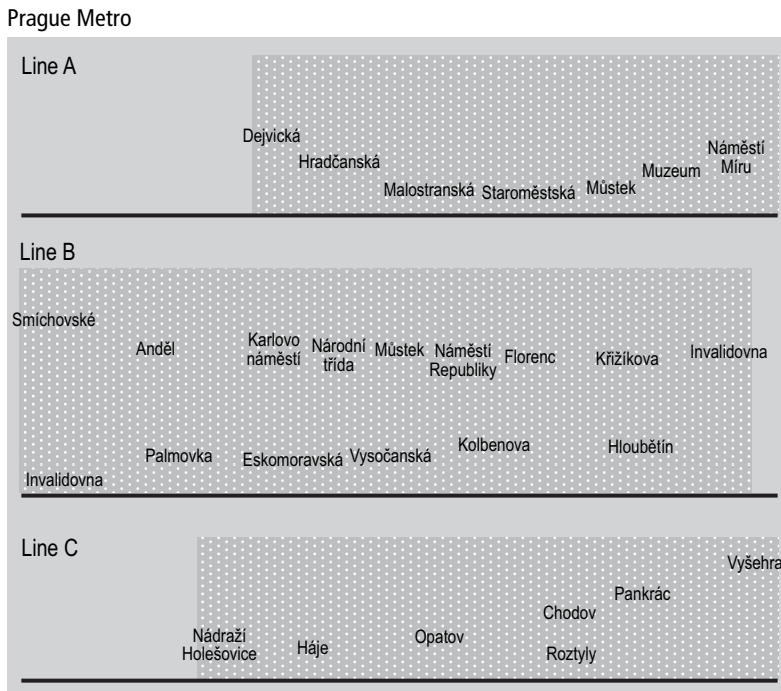

Figure 4.3. Extent of flooding in the Prague metro.
Photos from http://metro.mysteria.cz/povoden.swf.

Invalidovna station failed, which caused flooding in other stations. A complicating factor appears to have been that the metro was kept running as the waters rose, as forecasts had predicted that flood peaks would be considerably lower than those that actually occurred.

4.1.5 Summary

A summary of the damage resulting from flooding on subways is given in *Table 4.1*.

Damage was reported to be primarily associated with electrical/electronic components such as power-supply systems, communications and signaling, escalators, ventilation, etc. Systems were typically completely out of operation for weeks to months and were operated on the basis of temporary measures (manual signaling, etc.) for up to several months. Although there was significant loss of life during the events in Taiwan and South Korea, none of this was reported to be due to flooding on the subway.[1] The deaths reported during these events were primarily associated

[1]However, Toda and Inoue (2002) report that an employee of a restaurant located in an underground space died when trapped by the floodwater during the 1999 Fukuoka subway flood in Japan.

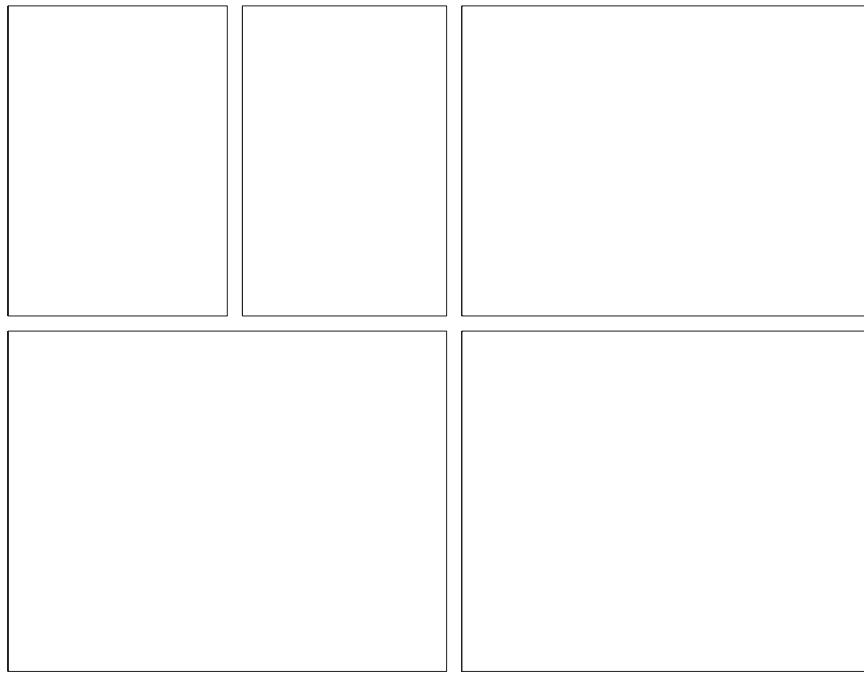

Figure 4.4. Damage in the Prague metro.
Photos from http://tom.vlakpage.cz/index.htm.

Table 4.1. Summary of reported damage in subway flooding incidents.

	Boston, 1996	Seoul, 1998	Taipei, 2001	Prague, 2002
Total cost (€m)	n/a	790*	15,000**	n/a
Total repair cost per km (€m)	n/a	18	∼180	n/a
Km track flooded	02/03/03	11	9–12	15–20
Volume of water (thousand m^3)	53	800	(n/r)	>1,000
Reported flood damage	∼10	40	60–140	66–240
Computed damage per km	1.3–4	∼3.6	0.9–12	4.44–16

*Line 7 only
**Entire system (86 km)

with mudslides, people drowning in swollen rivers, and electrocution from damaged electrical equipment. A common feature reported in all of these episodes was that human error was a contributory, and, in some cases, a major factor. Errors ranged from overly optimistic hydraulic forecasts to incomplete or inadequate construction methods and the failure to install or implement protective action. We note that an evaluation of the reliability of any active system requiring human input or control should include the reliability of the operators. For some protective systems,

Table 4.2. Range of length flooded/damage ratios.

	Boston 1996				Seoul 1998	Taipei 2001				Prague 2002					
Length flooded (km)	2	2	3	3	11	12	12	9	9	20	20	20	15	15	15
Repair cost (€m)	10	40	10	40	35	140	53	140	53	66	180	240	66	180	240
alpha	5.0	20	3.3	13	3.2	12	4.4	16	5.9	3.3	9.0	12	4.4	12	16

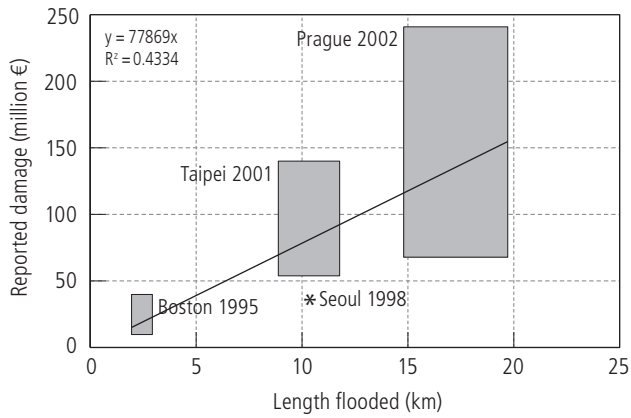

Figure 4.5. Relationship between reported damage and length flooded.

especially those requiring a high degree of reliability, human error may turn out to be the most significant limiting factor in the reliability of the system.

With this information, we may estimate alpha on the basis of a statistical analysis of the rather limited data. To estimate the damage factor, a full factorial design on track length flooded and damage estimates was used to generate all possible combinations of damage reported and track length flooded (see *Table 4.2*).

A simple analysis of these values yields a mean of 9.4 and a range from 3.2 to 20. However, to avoid artificially weighting the cases where there were additional estimates (e.g., Prague), synthetic data points were generated, for example, by taking the arithmetic average of the length flooded and the repair costs. An appropriate number of these synthetic centroids was used (five for Seoul and two for Boston and Taipei) to ensure that all cases were equally weighted. A simple arithmetic average is then 8.1.

A regression was performed to evaluate alpha for the overall dataset and is shown in *Figure 4.5*.

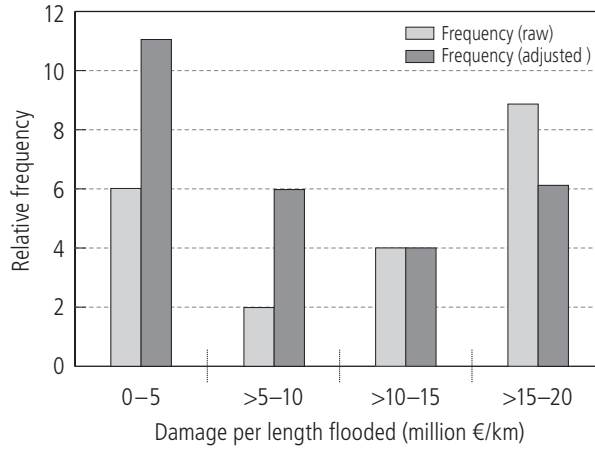

Figure 4.6. Distribution of length/damage ratios.

To estimate the range, we examined a frequency distribution, as shown in *Figure 4.6*.

4.2 Analytical/Cost-Estimation Approach

A second approach is to decompose the subway system into major systems (e.g., track, communication systems, power systems, etc.) and estimate the percentage of damage to the different systems as a result of flooding. This approach is similar to that developed in the FLAIR report (N'Jai *et al.*, 1990) to develop synthetic depth damage curves. If the linear cost of these systems (cost per kilometer as constructed) is known, the appropriate percentages can be multiplied by replacement cost to yield a total damage per length.

In Neukirchen (1994), the damage estimation makes the assumption that the damages could be estimated using a range of 10 percent of the construction costs and 15–20 percent of the electrical costs. As-built costs for subway systems in the United States are shown in *Table 4.3*.

Assuming that stations are located at intervals of approximately 1 km, the total cost of at-grade systems averages $9m/km and ranges from $8–42m/km, whereas the average total cost of subway systems is $48m/km and ranges from $25–120mr/km. Electrical systems comprise approximately 38 percent of the total systems and guideway cost for at-grade systems but only 9 percent for subway systems (presumably reflecting the larger component due to excavation costs). Assuming that these ratios can also be used to characterize the ratio of electrical installation/total installation costs of stations, we find that the electrical components are approximately $2.4m for subway systems as against $3.1m for at-grade systems).

Table 4.3. Reported costs of subway components ($m).

Component	Median	Average	St. dev	Range
Systems*	1.9	2.4	1.2	1.4–5.4
At-grade components				
At grade-ballast guideway	1.7	3.9	5.4	1.2–17.9
At-grade center platform station	9.3	9.0	5.1	4–19
At-grade side platform station	7.4	7.2	0.5	7–7.6
Underground components				
Underground guideway	21	24	11	16–52
Subway center platform station	29	28	13	8–59
Subway side platform station	24	24	3	20–27

*Systems represent primarily electrical and electronic components
Source: Laver and Schneck (1996)

One would, a priori, expect these to be similar. By way of comparison, it appears that the Vienna metro is rather expensive. The overall estimated construction cost was given as ranging from €44–145m/km, with the estimated construction cost of the U4 between Ober St. Veit and Kettenbrückengasse (roughly speaking, an at-grade system) given as €58m/km.[2] This is above the range reported by Laver and Schneck (1996) for the United States. The reason could be the use of different exchange rates and also the lack of inclusion of soft cand special costs, such as land acquisition, utility relocations, and various engineering design and management costs. In addition, labor and tax costs may also vary significantly between Austria and the United States.

If we assume that the electrical components comprise approximately 40 percent of the cost of at-grade systems, that the damage to electrical systems is approximately 15–20 percent of construction costs, and that damage to construction represents approximately 10 percent of construction costs, then we obtain a damaged fraction ranging from 11–14 percent of construction costs for at-grade systems and 10–11 percent for subway systems. Using these ranges, and applying these values to the ranges reported above in Laver and Schneck (1996) for at-grade systems, we obtain a range of $0.9–6m/km. Application to subway systems yields $2.8–17m/km. One can perform a similar exercise for costs associated with the Vienna metro (*Table 4.4*).

[2]The rates were originally given in Austrian Schillings, which were pegged at 13.7603 ÖS per euro in 1999. As the euro did not exist in 1993, when these estimates were provided, the cost is converted at the official rate adopted when the euro was adopted. The range was given as 600m to 2 billion Schillings per km, with the cost on the U4 between Ober St. Veit and Kettenbrückengasse as €800m per km.

Table 4.4. Ranges of damage per kilometer flooded: Method 2.

		Damage %	Total costs
At-grade	$8–42m	10–14%	$0.8–6m
Subway	$25–120m	10–11%	$2.5–13m
Vienna, at-grade	€58m	10–14%	€5.8–8.1m
Vienna, subway	€44–145m	10–11%	€4.4–16m

Table 4.5. Adopted values for alpha and beta for use in Equation 4.4.

Parameter	Value from [21]	Value in this study
Damage per length of track flooded (α)	7	U(1,20)
Damage multiplier (β)	1*	1-exp($-\lambda$Q)

*Implicit: damage was defined at 63 m^3/s.

4.3 Summary

The data from the empirical studies suggest that the values for alpha could range from €3–20m/km of track flooded, with a most likely value of around 5. The results from the engineering estimation yield estimates of between 1 and 16, with the most likely value being between 5 and 12. Considering the many uncertainties, we believe this to be relatively good agreement, given that these estimates were developed using independent methods.

In light of these examinations, we have defined the values of alpha and beta according to *Table 4.5*. It is felt that these represent a reasonable estimate of the uncertainty in the potential damage, as the range is supported by two independent lines of evidence. Subjectively, it is believed that the use of these values will result in slightly conservative (high) estimates of the damage. The data drawn from case studies may be subject to selection bias (i.e., episodes resulting in extensive damage tend to result in more news coverage than those resulting in minimal damage). The analytical estimates may be biased by the potentially high as-built costs of the Vienna subway. However, this conservative approach is not expected to be a major factor and is judged to be well within the bounds of the intervals given. Furthermore, sensitivity studies can be performed to examine the impact of this possibly conservative approach.

The basic damage equation is therefore largely a function of two stochastic variables, alpha and Q. The distribution of alpha, which has a simple distributional form, was the subject of Chapter 4. The distribution of Q was based on the hydraulic simulation model, as discussed previously. This is a nonstandard distribution, suggesting the use of numerical techniques. The way in which this equation is implemented is the subject of the next chapter.

5

Abstraction Methodology and Implementation

The objective of this chapter is to discuss the way in which a model was constructed to tie together the analyses described in Chapters 3 and 4. Chapter 3 corresponds to the "scientific" or "hazard" module discussed in the introduction, and Chapter 4 to the "engineering" or loss computation module. As discussed, each of those analyses provides a set of distributional inputs. This chapter discusses the final step, namely, the integrating module. This corresponds to the "insurance coverage" module discussed previously. However, the integrating module need not focus on insurance coverage. This approach can also be used to illustrate the effect of different mitigation measures on absorbed damage.

This chapter therefore attempts to fulfill three objectives. The first is to illustrate a method for dealing with both epistemic and aleatory uncertainty using a risk curve. The second is to create appropriate model abstractions. As the analysis provided in Faber and Nachtnebel (2003) and summarized in Chapter 3 was very detailed, there was a need to create a "reduced form" of the analyses contained in that report. Running the type of physical simulation analyses carried out in Chapter 3 is computationally prohibitive, as the simulation must consider hundreds or thousands of simulations of different possible values. In such cases, a reduced form may be substituted for the more complex model. The goal of the reduced or abstracted model is to capture the salient elements of the more complex results (cf. Morgan and Henrion 1991, p. 215). The third and final goal of this chapter is to introduce and define the different hypothetical structural and non-structural mitigation measures considered in this case study. Two structural measures (detention basins and portable flood barriers) were considered in conjunction with three financial measures (reserve funds, borrowing, and insurance). The way in which these were abstracted and parameterized will also be discussed in this chapter. The basic approach to developing the risk curve was adapted from Ermolieva *et al.* (2001) and is as follows:

1. Identify a planning period of interest (PPI) corresponding the time frame of concern of the decision maker.

2. Assume that a severe storm of an arbitrary magnitude occurs within the PPI and compute the a priori likelihood of that storm based upon a known rainfall-probability distribution such as a Gumbel distribution. Repeat this process multiple

times to produce a set of rainfall-probability pairs. To increase computational efficiency, sample only from events that are likely to cause damage. For example, as damage is not expected in storms with recurrence intervals of less than 100 years, only storms with recurrence periods exceeding this value are considered. It should be noted that this introduces a conditional probability; namely, we are sampling from a subset of all possible storms and must therefore apply the appropriate probability correction to convert the conditional probabilities computed in the model to absolute probabilities.

3. Transform the rainfall, using an appropriate rainfall-runoff relationship, to discharge into the Vienna River. Determine the amount of water entering the subway system as a result of this rainfall. This step corresponds to the computation of water levels in a more traditional flood risk assessment concerned with damage to structures in a floodplain, such as was implemented in Ermolieva *et al.* (2001). The effects of ex ante structural mitigation measures, which influence the level of water entering the system, are considered at this point. The set of rainfall-probability pairs has now been transformed into a set of overflowing water-probability pairs.

4a. Determine the direct tangible damage resulting from the overflowing water. The set of rainfall-probability pairs has now been transformed into a set of direct damage-probability pairs. Plot the sets of damage/probability pairs on the risk curve described previously. If parameter values were sampled from distributions representing epistemic uncertainty in the preceding calculations, a scatterplot will be generated.

4b. To provide a clear representation of the relationships, produce curves rather than scatterplots by taking subsets corresponding to specified probability intervals and computing the mean or fractiles of the distributions. This represents a conditional probability distribution representing the epistemic uncertainty in damage, given that an event falling within a specified probability band (e.g., the 100-year flood) occurs. Note that this distribution may not be a normal distribution, so use of the standard deviation to determine confidence intervals is suspect, unless it has been verified that the conditional distributions are in fact normal distributions. This can be done formally or simply and quickly by plotting the conditional frequency histogram and ensuring that the distribution is not skewed or overly broad/narrow. The simplest way to generate this is simply to compute means or fractiles directly from the sample.[1] This was the approach chosen here.

5. Estimate the impact of non-structural mitigation measures such as insurance or reserve funds on the total pre- and post-disaster costs incurred to manage the flood. This is done by estimating the extent to which the losses can be compensated for from a reserve fund or an insurance policy, and if the losses cannot be fully

[1]The careful reader might note that the way in which the estimator is computed and its potential error may also be a function of the distribution. We do not deal with this problem in this analysis.

covered, obtain a loan to cover the costs. The premiums paid before the event are counted as costs, as are the interest payments made on any loans taken out after the event.

These steps are described in more detail below. Details of the algorithms used are given in this section.

5.1 Model Abstraction: Flood Hazard Analysis

The hazard analysis was developed from the analysis discussed in Chapter 3 and in extensive detail in Faber and Nachtnebel (2003). It became clear from the discussions and review of the analyses that the uncertainty in the rainfall—particularly for rare events—was a major driver of the uncertainty in the likelihood of catastrophic floods. It was therefore deemed desirable to evaluate this directly within the model and separate the problem of system failure into two components:

1) Determination of the distribution of rainfall and runoff in the river, with the attendant uncertainties; and

2) Determination of the conditional likelihood and magnitude of system failure that occurs at different levels of runoff.

The algorithm chosen to do this was introduced previously. A more detailed description follows.

5.1.1 Rainfall Determination

The first step is to sample from a probability distribution describing the peak 6-hour rainfall. This can be done either by sampling the rainfall and then determining the probability of occurrence from the appropriate probability distribution or by sampling a probability and then determining the associated rainfall. For the purposes of computational efficiency, we selected a procedure that provided increased sampling of low-probability events. A variant of importance sampling was chosen to provide even coverage of the tails of the distribution by sampling over the negative log of the probability from a uniform distribution. The rainfall corresponding to the selected probability was then determined. Based on the analyses in Chapter 3, the probability of the selected rainfall was presumed to follow a Gumbel Type I distribution. The Gumbel Type I distribution is defined by the Cumulative Distribution Function (CDF) given (Beyer, 1968) as:

$$F(n) = exp(-exp(= \tfrac{n-\alpha}{\beta})), \tag{5.1a}$$

The mean and variance of this distribution are given by:

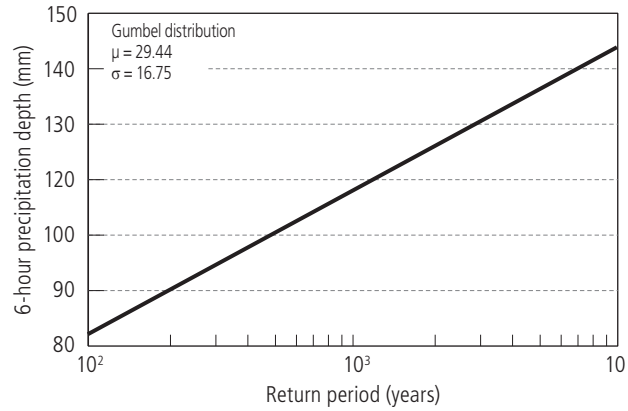

Figure 5.1. Rain depth as a function of return period.

$$\begin{aligned} \mu_n &= \alpha + 0.5772\beta \\ \sigma_n^2 &= \frac{\pi^2\beta^2}{6} \end{aligned} \tag{5.1b}$$

Given an exceedance probability p, the rainfall to which it corresponds can also therefore be solved.

$$n = \alpha - \beta \ln(-\ln(p)) \tag{5.2}$$

The resulting set of (n,p) pairs defines the probabilistic rainfall-recurrence relationship. The results for the Gumbel distribution with a mean value of 29.44 and a standard deviation of 16.75 are illustrated in *Figure 5.1*.

5.1.2 Flow Rate Determination

The next step is to determine the flow rate of the Vienna River at Km 4 resulting from the sampled rainfalls. This problem was discussed in Chapter 3. Detention basins are installed upstream, and these function by modifying the downstream flow rate. They do this by accumulating water while the river is rising, thereby moderating the rise in water levels downstream and then releasing the water levels after the flood peak has passed. At some point, however, the basins may become full and lose their ability to store water for later discharge. As discussed by Faber and Nachtnebel (2003), modifications are being carried out to give operators more control over the filling and emptying of the basins.

Therefore, the downstream discharge is a function of both the peak rainfall and the basin state. This was examined by the use of a detailed rainfall-runoff model. As the incorporation of the detailed model is computationally prohibitive, a reduced-form model is used that determines the discharge at Vienna River Km 4

Table 5.1. Rainfall-runoff relations at Vienna River Km 4 as a function of basin state.

	Peak Discharge at Vienna River, Km 4 (m^3/s)			
6-hour Rainfall (mm)	No basins	Non-upgraded	Assumed current	Upgrades complete
0	0	0	0	0
28	69	68	66	68
51	177	175	175	174
65	247	243	242	243
84	346	265	320	294
118	561	518	463	424
134	668	607	584	567
160	841	751	737	726
200	1106	971	971	971
300	1770	1522	1522	1522

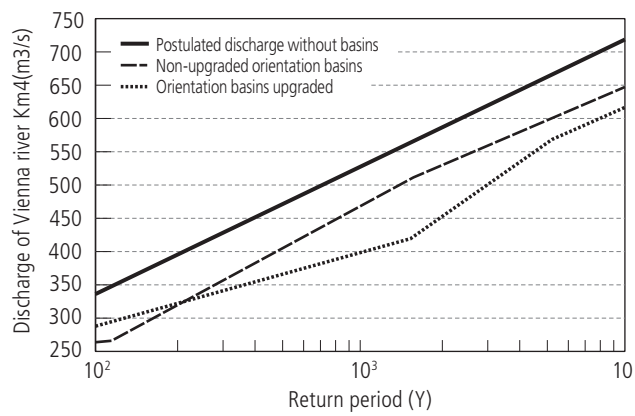

Figure 5.2. Rainfall-runoff relations at Vienna River Km 4 as a function of basin state.

corresponding to the sampled rainfall by the use of the lookup tables given in Chapter 3 and reproduced here in *Table 5.1*. These lookup tables simulate the effect of the retention basins in one of four possible states: no retention basins, non-upgraded retention basins, upgrades to Auhof-Mauerbach retention basins only (the assumed current condition), and completed upgrades on all retention basins.

The resulting deterministic discharge exceedance curves shown in *Figure 5.2* are based on the rainfall-return period plot shown above.

We note that this approach implies that there is no uncertainty associated with the response of the detention basins. The computed uncertainty in the discharge is simply the transformation of the uncertainty in the rainfall. A more complete

analysis might include the effect of the uncertainty in the rainfall-runoff model, developed by running the rainfall-runoff model with the rainfall as a constant value, with the other parameters being allowed to vary stochastically. However, as we believe that the uncertainties in the rainfall are likely to dominate the uncertainties introduced by the detention basins, and as the model is intended to be for illustrative purposes only, we simply include the deterministic lookup tables.

5.1.3 Overflow Determination

The final step is to determine the amount of water flowing into the subway. Based on examination of the results in Faber (2003) and in Chapter 3, it was hypothesized that the flow rate of overflowing water could be roughly estimated from the flow rate in the main channel. This hypothesis was generated by the observation that the likelihood of the failure of the system appeared to be correlated with the probability that the flow exceeded some critical level. Furthermore, it was assumed that flow in the U4 can be represented by the difference between the critical level of flow and the flow in the channel. The observation that the failure probability seemed to track flow exceedance probability suggests that the probability of failure increases dramatically once some "critical" flow is exceeded. This hypothesis appears valid based on an examination of the system and on inspection of the results in Faber and Nachtnebel (2003). A failure that results in the release of water to the U4 occurs when the discharge into the Vienna River exceeds the given threshold, resulting either in overtopping of the floodwall or collapse of the floodwall due to either foundation scouring or hydrostatic pressure. It is clear that an overtopping failure is largely a function of the flow rate in the channel and that the uncertainties are largely those associated with the channel geometry, wall height, and roughness coefficients. As this is a channelized river with a well characterized geometry, it is not thought that these contribute substantially to the uncertainty in the water flow rate at which overtopping is expected. Similarly, erosive failure and wall collapse is largely a function of the computed shear at the channel bed and the shear strength of the invert. There is likely to be more uncertainty in these parameters. It was determined that a failure leading to overflowing of the U4 occurs at a discharge of approximately 530 m^3/s. Because of the uncertainties in the resistance parameters of the floodwall, however, this is not a fixed value but is represented by a probability distribution. In this simulation the "critical" discharge is modeled as a normal distribution with mean 530 and standard deviation of 10 m^3/s. This implies that the failure of the basin could occur with a 5 percent probability at a flow rate of 510 m^3/s and would be almost (95 percent) certain to occur once flows in the main channel exceeded 550 m^3/s.

$$Q_{U4} = Q_{CRITICAL} - Q_{VRK4}; \quad Q_{VRK4} = f(N) \tag{5.3}$$

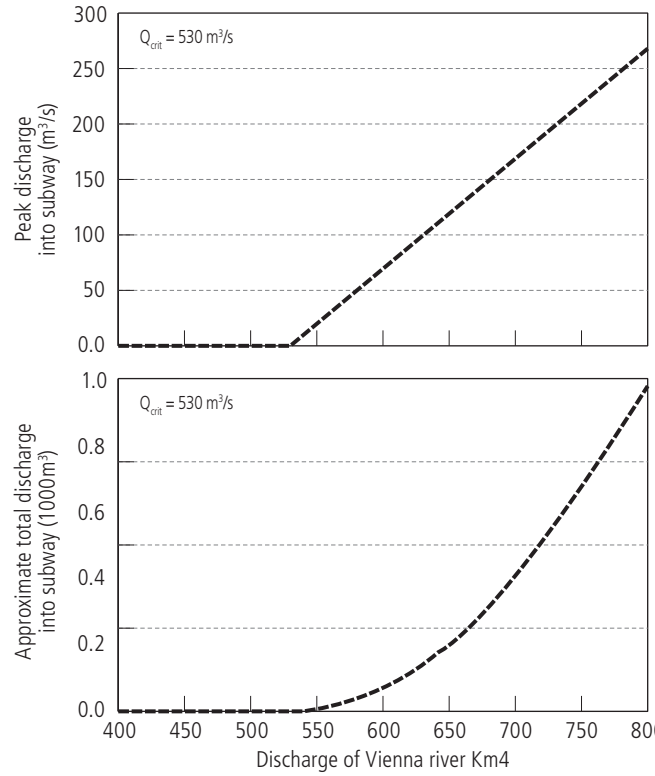

Figure 5.3. Estimation of overflowing water. Upper: approximate peak overflow rate. Lower: approximate total overflowing volume.

Figure 5.3 illustrates the relationship between the estimated peak discharge and the flow into the subway terrace with a Q_CRIT of 530 m^3/s. Also shown is an indication of the total volume of water discharged into the terrace. Although the hydrograph was not computed, this plot was produced by approximating the peak of the hydrograph as a triangle and assuming that the duration of the flooding over the critical discharge is proportional to the difference between the peak discharge and the critical discharge. For this curve, it was assumed that a peak discharge of 730 m^3/s would result in a period of three hours above the critical discharge of 530 m^3/s.

It is important to note that this distribution is a rough approximation used to abstract the reliability assessment provided by Faber and Nachtnebel (2003). If this analysis were to be extended, it would be desirable to conduct a more detailed examination of this conditional failure probability distribution. However, for the purposes of illustration, we will proceed with this rough approximation. Provided that the “critical level of flow” hypothesis is valid, the model could easily be up-

dated simply by changing the parameters of the distribution for the critical level of flow value. It is believed that the second approximation, that of computing the discharge in the subway as the difference between the runoff and the critical level of flow, is a reasonable assumption for flows below that necessary for the water level in the terrace to equal that of the main channel. At higher flows, this would be an overestimation, as a portion of the flow would be carried into the main channel. For collapse failures, this could be a significant underestimation. Once the collapse occurred, a significant amount of channel flow might be diverted into the subway terrace. However, because of the way in which the damage function to the subway is defined, these are not critical. We also note that this approach captures the characteristic that the protective system is a hard-fail rather than soft-fail system. In other words, the system of detention basins and masonry floodwalls provides a very high level of protection up to a certain river flow rate. However, once that system fails, the level of damage can be expected to rise rapidly. This is in contrast to a soft-fail system such as flood hardening, which would increase the ability of the system to withstand inundation.[2]

5.2 Damage Assessment

As discussed previously, we consider the damage to be a function of the length of the track flooded. Model abstraction is not needed for this part of the analysis, as the damage estimation technique was sufficiently simple as to be computationally inexpensive, and it was developed with implementation in the catastrophe model in mind. Estimation of the physical damage requires two parameters: the length of track flooded and the damage per length flooded.

5.2.1 Length Flooded

It is assumed that the subway consists of two sections. One section is not protected by a floodgate and is inundated whenever there is a flood (although the damage may be equal to zero; see below for the definition of the damage multiplier). This section is approximately 7.5 km long from the location where the U4 crosses the Vienna River at Km 10.6[3] to the portable flood barriers installed at the Grosse Einwölbung at approximately Km 3.1. It is conservatively assumed that the inundation can occur at any point along the section. This assumption is conservative, as the most

[2]This is not intended as a critique of the well-designed flood protection system in place. Implementation of flood hardening for the Vienna subway may be overly expensive, infeasible, or even impossible. The point is to illustrate the financial characteristics of different mitigation alternatives and combinations of these.

[3]The point of a likely first inundation was reported (Neukirchen, 1994) to be located at Braunschweiggasse at Km 8.6. This would yield a distance of 5.5 km for the unprotected reach.

likely point for flooding to occur is just prior to installation of the portable flood barrier. A better distribution would therefore be positively skewed, making flooding to shorter track lengths more likely than flooding to longer track lengths A more detailed model might consider the conditional probability of flooding and explicitly model failure probabilities at each location, generating a conditional probability distribution of the length flooded. Such an analysis was not performed, however, and the length flooded in this section was therefore modeled as a uniform random variable U(0,7.5) to determine the length of unprotected track flooded.

The other section is protected by a floodgate. If the floodgate works, none of the section is flooded. If the floodgate fails, all of this section is flooded. However, the length of track that is protected by the floodgate is not precisely known, as the entire system was not modeled. Because of the lack of detailed analyses, this was treated as an epistemic uncertainty and was modeled as a random variable with an upper and lower bound. Potential upper and lower bounds on the lengths at risk were estimated. We take, as a minimum, that the U4 would be flooded as far as the outlet into the Donaukanal for a total inundated stretch of 3.1 km. As water entering Karlsplatz station could provide a point of entry for water into the U1 line, we assume that the U1 would be flooded, at a minimum, between Südtirolerplatz and Reumannplatz, for a total distance of 2.9 km. To set an upper bound, we presume that the maximum stretch of the U1 that could be flooded would be between Reumannplatz and Vorgartenstrasse, for a maximum inundation potential of 6.5 km for the U1. Water entering either Wien Mitte station via the U4 or Stephansplatz station via the U1 could result in flooding of the U3. We take, at a minimum, flooding of the U3 between Burggasse and Schlachthausgasse for a total of 4.4 km flooded. To set an upper bound, we assume that the U3 could be flooded as far as Simmering, for a total inundation length of 7.7 km. This results in the following upper and lower bounds:

Protected stretch (lower bound): 3.1 km U4 + 2.9 km U1 + 4.4 km U3 = 10.4 km
Protected stretch (upper bound): 3.1 km U4 + 6.5 km U1 + 7.7 km U3 = 17.3 km

We therefore model the length flooded as the sum of a U(0,7.5) and a U(10.4,17.3) distribution.

$$\text{LengthFlooded} = \begin{cases} U(0, 7.5), & \text{FloodgateFailureType} = 0 \\ U(0, 7.5) + U(10.4, 17.3) & \text{FloodgateFailureType} = 1 \end{cases}$$

5.2.2 Damage per Length Flooded

As discussed in Chapter 4, it is assumed that the damage per length flooded (alpha) is a uniform variable ranging from €1–20m per km flooded. As previously noted, there were insufficient data to establish an empirically or theoretically grounded

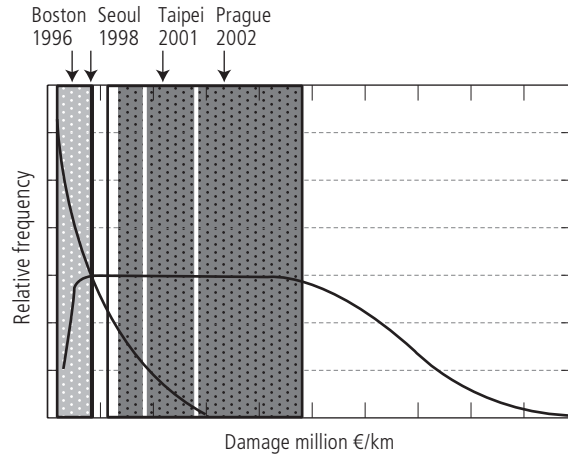

Figure 5.4. Comparison of the synthetic conditional damage distribution for Vienna with case study reports.

relationship between overflowing water and damage. However, it was clear that at low flows (which we define as 5–10 m^3/s) the damage would be slight, and that damage would increase quickly as the pumping and drainage capacity of the subway was overloaded, quickly reaching the maximum potential damage. As discussed in Chapter 4, an exponential form was chosen for mathematical convenience to represent the relationship between overflowing water and percentage of damage. To reflect the sharp rise in damage caused by overflowing water, an exponential function discussed in Chapter 4 was chosen. The value of lambda was chosen to give damage of 50 percent at a flow of 20 m^3/s.

$$\lambda = \frac{\ln(2)}{20\frac{m^3}{S}} = 0.35 \tag{5.4}$$

Figure 5.4 illustrates the synthetic conditional damage curves and shows how these compare to the ranges of damage reported for catastrophic flooding in similar systems.

For distributional sensitivity analyses, an alternative variant explored was the use of normal rather than uniform distributions to estimate the damage. In this variant, the variable representing the length of the protected areas of track was modeled as a normal distribution with mean 13.85 and standard deviation of 3.45. The distribution was truncated at zero to ensure that no negative values were obtained. Likewise, the damage function was modeled as a normal distribution with mean 10 and standard deviation 5, and was again truncated at zero to ensure no negative damage.

The loss of revenue associated with foregone fares was also considered. To obtain a rough order-of-magnitude estimate of this effect, we assume that the service interruption is also a function of the length of track flooded. Based on the experience of past inundations, we take this value as 5 days per km of track flooded. In 2001 the total subway ridership was approximately 400m passengers (Wiener Linien, 2002). Dividing this number by 365 days per year and assuming that the U4 carries approximately 20 percent of the passenger load, we obtain a daily ridership on the U4 of approximately 200,000 rides. At a ride cost of €2, we can derive a total fare loss of approximately €2m per km flooded. As this is only a small part of the maximum total potential damage, we assume this to already be subsumed within the damage estimates. A more detailed analysis might be able to explore this in more detail by examining the effect on revenue of planned outages while tracks are closed for normal maintenance. The exercise discussed here was simply a quick examination of the potential relative contribution to losses from foregone fares and repair costs.

5.3 Financial Parameters

In this section, we will introduce and discuss the implementation of a number of different potential financial mitigation measures. Direct damage is the input to the financial module. Two ex ante financing measures, insurance and a reserve fund, are considered. One ex post financing measures, borrowing, is considered. Budgetary diversion would be simple to include, but was not implemented in this version of the model. The methods for computing financial parameters follows Ermolieva (2001) and Mechler and Pflug (2002), and the nomenclature of the model parameters follows Mechler and Pflug (2002). We note that inflationary risks are not computed explicitly. All amounts are computed in real terms. This introduces a bias. In particular, insurance is not adjusted for potential changes in inflation, whereas inflationary risks associated with investment assets are included implicitly by the use of a real rather than nominal rate of return. If the insurance contract is denominated in a stable currency, such as dollars, euros, or Swiss francs, this may not be a major issue. However, if the insurance contract is denominated in a potentially unstable currency, and adequate contractual safeguards are not maintained, there could be considerable inflationary risks in the value of the insurance contract.

5.3.1 Determine the Timing of the First Severe Event

An important parameter in examining the impact of different financial measures such as insurance or reserve funds is the arrival time of the first event, as this will determine the extent to which a reserve fund has accumulated funds or for how long premiums have been paid. The arrival time of the first occurrence of an event that

can occur in any year with constant probability p is given by a geometric distribution (Beyer 1968).

$$P(t = \tau) = f(\tau) = p(1-p)^{\tau-1} \tag{5.5}$$

It can be shown that, by expanding the terms in a binomial expansion, this approaches a uniform distribution with

$$P(t = \tau | \tau < T) + f(\tau) = \frac{P(t = \tau)}{P(\tau < T)} = \frac{p(1-p)^{\tau-1}}{1-(1-p)^{T-1}} \cong \frac{1}{T-1} \tag{5.6}$$

as pT $<<$ 1. The arrival time is therefore modeled as a uniform distribution.

5.3.2 Insurance

Insurance can be simulated as either proportional insurance or as excess of loss insurance, or both. However, the model is limited in that it is currently possible to define only one layer. The following parameters are used to characterize insurance:

- The attachment point, or "deductible," of the insurance. One hundred percent of all losses below the attachment point are borne by the policyholder.
- The proportion of losses within the insured layer borne by the policyholder. Setting this value to 1 causes insurance to be inactive (i.e., if the policyholder bears 100 percent of the losses, then the insurer pays no claims).

The claims are computed as a proportion to the total loss exceeding the attachment point. However, to define the upper limit of the layer, the claim payments are capped by an exit point.

- The exit point, or "cap," of the insurance. This is the maximum claim payment by the insurer.

Claims are therefore computed by the following relation:

Claim=min(Exit Point,(1- Proportion)*max(0, Damage-Attachment Point)).

The resulting relationship between claim payments, retained losses, and damage is illustrated in *Figure 5.5*, which shows the effect of an attachment point of €10m in direct damages, a 20 percent coinsurance proportion, and an exit point of €100m in claims.

There are two possibilities for determining risk premiums. One would be to define the premiums as being equal to the expected claims. In essence, this assumes that insurance is "costless." In reality, the costs of insurance are non-zero, because

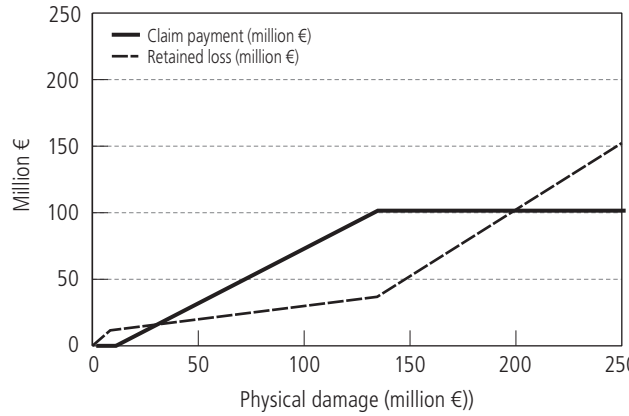

Figure 5.5. Structure of insurance.

of the need for administrative costs, profits, and risk premiums. An assumption of costless insurance may be reasonable for a risk-neutral public insurance program that incurs only administrative costs. For private insurance, the costs are likely to be higher because of the need for profit and the charging of a risk premium. In Mechler and Plug (2002), the functional form of the premium loading factor (PLF) is simply PLF=1+0.03Tr. We compute premium payments in a fashion similar to Mechler and Pflug by charging a risk premium for low-probability events. However, the functional form identified above raises the questions of which return period to use to compute the premium loading factor: the return period of the underlying event or the return period of the loss. The use of the return period of the event is equivalent to the highly conservative assumption that variability in the insurers' losses is dominated by the variability in this particular policy. The return period of the insurers' losses is probably the appropriate parameter to use. However, this requires data on the full portfolio of the insurer. A well structured insurance portfolio would not allow itself to be exposed to such an extent, and the risks of the flood would be spread among risks associated with the other policies issued by the insurer. We therefore charge the risk premium a constant, user-defined premium loading factor. We note that for a company underwriting with significant catastrophic risks, combining the different catastrophes into an integrated catastrophe model may be an efficient way of determining an appropriate premium loading factor. However, such an analysis must be a part of future work. The expected claim payments are simply the probability-weighted claim payments. The use of a risk premium adds the premium loading factor, such that the annual premiums are simply taken to be the expected claims adjusted by the PLF. The accumulated insurance reserve is simply the accumulated premiums minus the claim payment at the time of the catastrophe. If the premiums collected are sufficient to cover the claims,

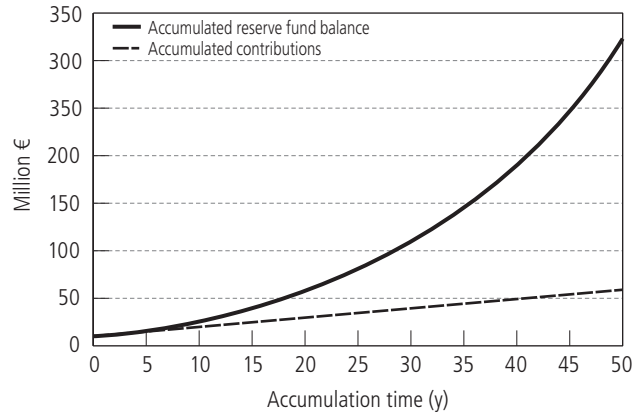

Figure 5.6. Accumulation of reserve funds.

the insurance reserve is positive and the premiums have been "overpaid." If the premiums collected are insufficient to cover the claims, then the insurance reserve is negative and the claims are "underpaid." The losses retained by the policyholder are simply the damage minus the claims.

Retained Loss=max (0, Damage - Claims).

5.3.3 Reserve Fund

We assume that the reserve fund is invested in a relatively safe security, such as bonds. The reserve fund has two components: a one-time initial investment and a constant annual payment.

Accumulated Funds = Initial Reserve Fund*((1+Yield)$^{\tau}$)
+Annual Payment*(((1+Yield$^{\tau}$))-1)/Yield;

The growth of the reserve fund with a €10m initial contribution, a €1m annual contribution, and a 5 percent rate of interest is shown in *Figure 5.6*.

The difference between the contribution and the balance represents the benefit of the reserve fund. It can be seen that benefit is quite small for short time horizons (<10 years), but increases significantly thereafter because of compounding.

A significant methodological question is the "cost" of the reserve fund, a question related to the "cost" of capital. This is a difficult question, discussed at length in Kielholz (2000). Typically, this is evaluated by measuring the opportunity cost of investing in a safe investment versus a more profitable but more volatile investment such as equities. The equity premium might therefore be used to determine the "cost" of the capital. However, this can be misleading. Equities are typically considered to be more volatile and thus to carry a higher downside risk than bonds

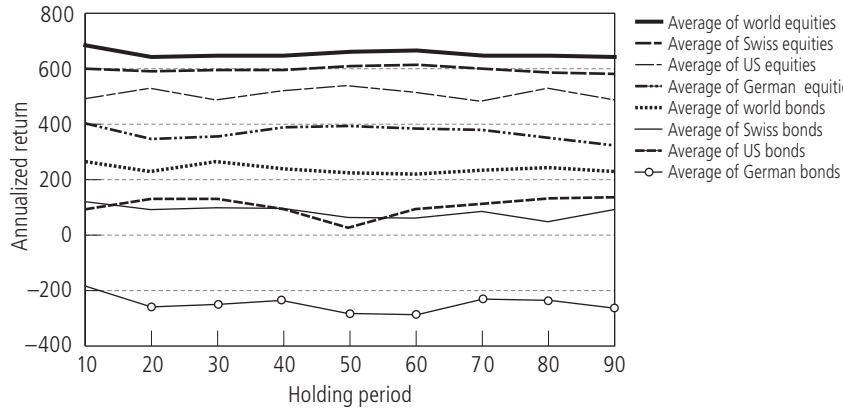

Figure 5.7. Real returns to equities and bonds: Average return as a function of holding period (adapted from Dimson *et al.*, 2002).

(whether this is true or not when measured in real terms, depending upon the holding period, is not clear). A probabilistic assessment might show that there is a significant probability that the equity premium is, in fact, negative, as would be the case if equities underperformed bonds (as has occurred several times over the past century). In this case, there may actually be a "negative" cost associated with holding the funds in a reserve fund. Essentially, one might inadvertently profit from a forced investment in a less volatile investment. One need only consider the financial history of the last several years to provide an illustration of such a phenomenon.

As we have chosen to integrate financial uncertainties with structural uncertainties, we have modeled the yield of the reserve fund as a random variable. Information on the potential uncertainties of investment yield can be obtained from Dimson *et al.* (2002), who present data on the performance of bonds and equities over a century from many different markets. As the uncertainties in yields are expected to be a function of how long the investments are held, we illustrate the concept of equity premium in *Figure 5.7* showing the real (inflation-adjusted) rate of returns to bonds and equities in two markets with relatively good records over the past century (Switzerland and the United States), a developed economy that has suffered two period of devastating inflation (Germany), as well as world aggregate values.

It is clear that, on average, equities outperform bonds. Swiss equities have provided a fairly stable 5 percent real rate of return when held for periods of 10 years or more, in comparison with typical bond returns of less than 3 percent. The traditional argument for holding a reserve fund in bonds rather than equities is that bonds are less volatile than equities and carry less downside risk. In other words, money invested in bonds is expected to be safer and more likely to be available

when needed than the same amount invested in bonds would be. We can explore this hypothesis by examining the volatility of these same instruments, which we define as the standard deviation of the rates of return. The results are shown below.

The impression that bonds are much safer than equities does not appear to be valid when inflation is taken into account and the volatility of real rather than nominal rates of return is examined. We can see that, in general, bonds are only slightly less volatile than equities when inflation is taken into account. This is because the variability in inflation becomes a controlling factor when the other uncertainties are made low. We note that if countries experiencing significant disruptions (e.g., Germany) are included, bonds can even have negative average yields with high volatility.

We wish to acknowledge that there is much work that has been done in this field and that this is only a very simple approach. However, it does illustrate that the trade-off between yield and volatility in the choice of an investment is not simple. In this paper, we have taken an approach that emphasizes this point by investing the reserve fund in a "conservative" equity, thereby emphasizing that the opportunity costs of a fund are sensitively dependent on the choice of the baseline used for determining the value of the foregone alternative. Furthermore, if it is assumed that the performance of a reserve fund is not affected by the occurrence of a flood, one may decide that the low base probability of a flood offsets the potential for low returns. The value of the catastrophe model is precisely that it allows such trade-offs to be made explicitly and to be examined.

We have therefore chosen to have our hypothetical reserve fund invested in a "safe" equity. We take this equity as having a real rate of return characterized by an average yield of 5 percent and a standard deviation given by the regression $\sigma = \tau^{-0.7889}$.

The comparison between the synthetic yields that we have generated and the observed performance of Swiss equities is shown below. As the uncertainty in yields can be quite large for short holding periods (less than 10 years[4]), we show both linear and log scales.

5.3.4 Borrowing

We implement post-disaster borrowing with an extremely simple model. The cost of a loan is simply the difference between the amount borrowed and the amount repaid, and is a standard computation shown below. We take the period to be a

[4]This relation would not be expected to hold true for very short periods, as the range of potential returns starting in any given year would be constrained and thus not as dramatic as shown here. The inaccuracy induced by the use of this relationship is substantially mitigated by the low level of compounding over shorter periods in relation to longer periods. However, a more rigorous treatment of the uncertainty in yields would be necessary if this study were to be applied to short planning periods.

fixed 30 years and assume that the average loan interest rate of 4 percent (real), and allow the interest rate to be an uncertain random variable that can range between 2 and 6 percent real at a 2σ confidence level. We note that these are unfavorable terms, albeit not unreasonably so. An agent of the Austrian government, given a good credit standing and alternative financial resources, would probably not be required to pay such rates or to amortize the loan over such a long period. These values are chosen somewhat arbitrarily but are intended to emphasize the fact that borrowing is also a mitigation measure with substantial costs and that the decision not to mitigate may be an implicit decision to assume a loan at whatever terms may be obtainable if a disaster occurs.

6
Results

In this chapter we will examine the consequences of implementing a number of different mitigation measures using the simple integrated model we have constructed. These mitigation measures are built up from combinations of remedial alternatives. The goal of this section is to examine the impact of selected decisions regarding mitigation of flood risks to the subway. There are, in principle, a number of possible alternatives. Two of the structural alternatives that are being implemented, as discussed previously, are upgrades to the detention basins and installation of a portable flood barrier at the entrances to the underground portions of the U4.

6.1 Structural Measures

A no-action alternative was considered to establish a base case. The no-action alternative essentially considers the pre-1990 conditions. It was presumed that the detention basins are in place but that no measures are taken to allow operability. The flood basins fill and empty passively. The masonry floodwall is also assumed to be in place. It is assumed that if damage occurs, the losses will be covered by a loan. As discussed in the previous chapter, the loan is assumed to be a 30-year loan with a real interest rate of between 2 and 6 percent. It is assumed that unlimited credit is available. Alternative 1 is the installation of a portable flood barrier at the openings to the covered sections of the metro. The effect of these flood barriers is to limit inundation of downstream reaches. As these systems can be expected to have a reliability of less than 100 percent, it is assumed that they have a failure-on-demand rate of 25 percent. In other words, they are assumed to fail only once in every four events requiring their installation. As there is no empirical or theoretical basis for this assumption, the effect of the reliability of these flood barriers on the results will be examined. Furthermore, it is assumed that the installation of these systems costs €100,000 and €10,000 per year to maintain. This is simply an estimate of the costs associated with two person-months of design services and two person-months of construction and testing costs, combined with a materials cost of €50,000. Annual operating costs (inspection, testing, and occasional repair) are assumed to be 10 percent of installation costs. These costs can be specified by the user. Alternative 2 comprises upgrading the basins to allow controlled filling and release of floodwater. The system is discussed in more detail in Faber (2003). This system, coupled with a real-time flood-forecasting system, is currently being

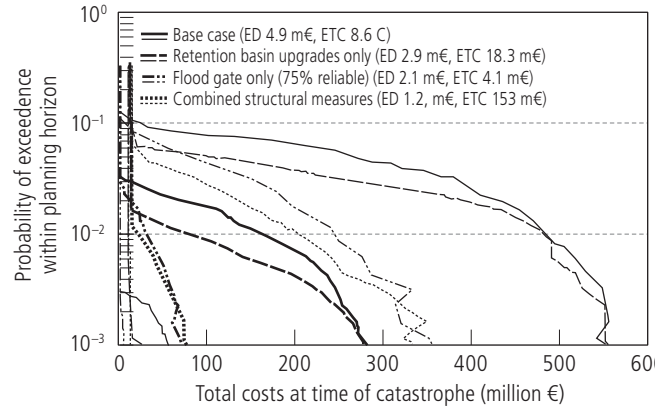

Figure 6.1. Examination of structural alternatives.

installed to increase the level of protection against extremely rare floods. The costs for this alternative are based on Neukirchen (1994), who reported an estimate of €8m and operating costs that are expected to be 1 percent of installation costs. To emphasize the fact that this is an illustrative example, we have rounded this value up to €10m. The combined scenario represents the combination of portable flood barriers and detention basin upgrades. The results of these scenarios are shown in *Figure 6.1*.

For the base-case, no-action alternative, it can be seen that over a 50-year period, there is approximately a 3 percent chance that damage could be incurred. However, because of the uncertainty in the rainfall, the range in annual probabilities in which damage might be incurred would range between 0.3 percent (at a 10 percent confidence level) and 15 percent (with a 90 percent confidence level). The expected damage over this period is approximately €5m. Because of loan servicing costs, the expected total costs are higher and amount to €8.6m. However, examination of the curve illustrates the problem of using an expected damage in this case. The distribution of damage is not a single mode distribution. Instead, it essentially represents a combination of a large ($\sim$97 percent) chance of no damage and a small chance of very great damage. The expected value does not represent a central tendency of this distribution. The risk curve illustrates this by demonstrating that while the chance of damage above zero is approximately 3 percent (on average), the chance that damage is greater than €100m is approximately 1 percent.

We once again note that this is not a realistic scenario for the city of Vienna. Structural mitigation measures are being installed. More significantly, a variety of other measures would likely be available to cover the repair costs. These could include diversion and contributions from the city or federal government. If a loan is required, the interest rate is not expected to be as high as that assumed here

(particularly if it was covered by a bond issue) or the term to be so long. However, these financial parameters may be more reasonable for a city in the developing world with fewer financial resources, a poorer credit rating, and no plans for structural mitigation measures. It is important to keep in mind that, as discussed in the Introduction, this is an illustrative study. It is not intended to provide concrete policy recommendations for the city of Vienna without there being considerable improvements in the data and extensive consultation with decision makers to develop realistic alternatives.

Examination of Alternative 1 reveals that, relative to the base case, the floodgate does not alter the probability at which damage will start to occur. The probability of damage exceeding zero is about 3 percent, unchanged from the base case. What does change is the damage at lower probabilities. The probability that the damage is limited to less than €50m are lowered to approximately 0.5 percent, and the chance of damages exceeding €100m is considerably less than 0.1 percent. The expected damage from this case is approximately €2 ME, with expected total costs of €4.1m. The plot clearly shows that the primary role of the floodgate is to limit rather than prevent damage. Using the expanded concept of risk, we can say that the floodgate primarily addresses the consequences of an event rather than the probability. If risk is defined simply as system failure without distinction between large failure and small failure, the floodgates are ineffective. However, it is clear from the plot that the floodgates do have a major impact in limiting the damage and may be able to limit damage to an "affordable" level.

Examination of Alternative 2 shows that, as intended, the upgraded detention basins lower the probability at which damage will start to occur. The expected probability of damage exceeding 0 drops from 3 percent to slightly over 1.5 percent. However, once damage occurs, it is catastrophic. This is because a storm large enough to overwhelm the detention capacity of the basins would cause major damage to an unprotected subway system. Furthermore, construction and operation costs must be added to the catastrophic costs to yield the total cost of dealing with flooding. This means that there is a 100 percent chance that total costs will exceed €10m, and there is a 1 percent chance that total costs will exceed €100m. The expected damage is reduced from €5m to €3m, but the expected total costs increase from €8.6 to 18m. The plot clearly shows that the primary role of the detention basins is to prevent rather than limit damage. From a risk-analytic perspective, we can identify this as a measure that primarily affects the probability of an event. If risk is defined simply as avoiding adverse consequences at all costs, this alternative would not be considered acceptable. However, it is clear from the graph that the basins do have a significant effect on the likelihood of damage being incurred. If a decision maker is unconcerned with potential damage below a certain level of likelihood, this type of alternative may be appropriate.

Finally, the combined alternative captures some of the desirable elements of the single approach, albeit at the cost of also including some of the drawbacks. Damage is limited by the floodgate and its likelihood is reduced by the detention basins. In addition, the uncertainty surrounding the losses is decreased. The expected damage is reduced to €1.2m, with a very low probability that the damage will exceed €100m. The expected total cost is approximately €16m.

6.2 Financial Measures

The first financial measure to be considered is insurance. The structure of a potential insurance policy was discussed in a previous chapter. Here, we set up a hypothetical insurance policy. The insurance policy variables are all decision variables, so there is no basis for selecting any particular set of combinations without knowing the decision maker's preference. In this case, we choose to have a €10m deductible, a 10 percent coinsurance rate, a €500m claims cap, and a premium loading factor of 100 percent (meaning that premiums are collected which are expected to be double the expected value of the claims, reflecting the risks borne by the insurer in offering a policy against such a catastrophic event). The results are shown below. For purposes of comparison, a 1,000 percent premium loading factor is also shown (reflecting a premium set to be equal to 10 times the expected claims payment, which is illustrative of a highly risk-averse or poorly diversified insurer).

The second financial mechanism is a reserve fund, the structure of which was discussed above. Again, many of the policy variables are decision variables, so there is no basis for selecting any particular set of combinations without knowing the decision maker's preference. In this case, we have chosen a set of variables to mimic the costs of the more expensive structural measure by assuming a one-time investment of €10m and an annual contribution of €0.1m. We assume that these funds are invested in a "safe" equity, which we benchmark as similar to the performance of Swiss equities. We note that the investment of the reserve fund in equities rather than bonds technically eliminates the cost of this option. The real costs would be those associated with lack of liquidity, which are beyond the scope of this analysis. The combined financial alternative represents a strategy mixing an insurance policy with a €10m deductible, a €500m cap, and a 20 percent coinsurance rate, with a reserve fund comprising a one-time initial contribution of €1m and an annual contribution of €10,000. The computed annual premiums are similar to those of the pure case at €150,000 (slightly lower because of the higher coinsurance rate), and the expected total costs are € minus 2m, representing the possibility that a profit is expected on the basis of no flood occurring and on the profit being taken from the interest accumulated over 50 years on the reserve fund. The results of these simulations are shown in *Figure 6.2*.

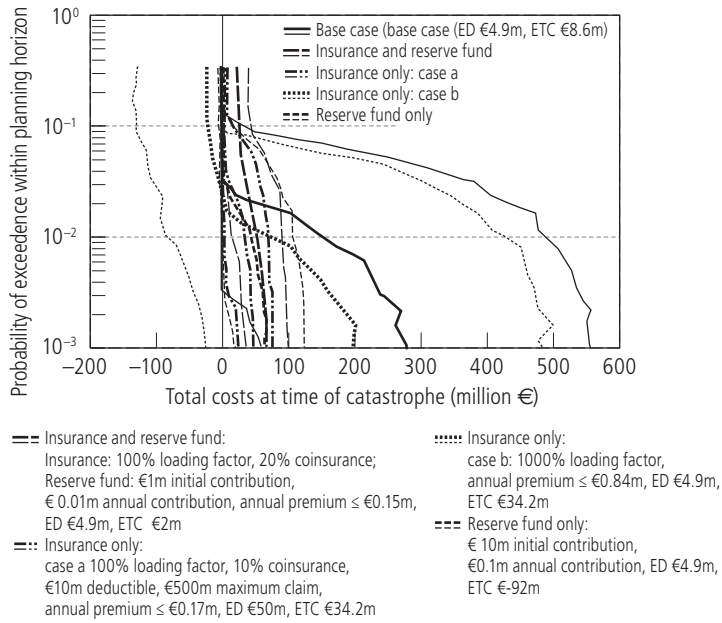

Figure 6.2. Financial measures.

In the insurance-only scenario, the expected damage is unchanged (as expected) from the base case, and premiums of €170,000 per year are computed using this premium loading factor. The expected total costs, including premium payment up to the time of the catastrophe, are €8m. With the higher (and probably more likely) premium loading factor of 1,000 percent, the premiums are close to €1m per year and the expected total costs are therefore quite high, at €34m. However, it can be seen that insurance has a remarkably similar effect (from a purely financial perspective) to a floodgate. Upon reflection, the reason for this is clear. Insurance is intended to limit rather than prevent losses. It can do this quite effectively. Examination of the uncertainty bands also shows the role of insurance as an uncertainty-reducing mechanism. In comparison with floodgates, the insurance policy reduces the uncertainty quite effectively (by passing it on to the insurer in the form of a contract). However, this case also illustrates the drawback to insurance, which is that it can be an expensive option if the event fails to happen, with the costs being sensitively dependent upon the premium loading factor. Another significant factor, which is not illustrated by this plot, is the risk that the insurer may withdraw coverage. If a structural measure is put in place, the decision maker retains more control over the mitigation option. If an insurer withdraws coverage or goes bankrupt, then the policy holder is placed back in the position from which he/she started with no benefit from the policy and no future protection.

The reserve fund reveals a somewhat startling feature in comparison with the other alternatives. It is clear that it does nothing (in common with all financial measures) to reduce damage. What it does do is to shift the loss curve to the extent that damage can be compensated for from accumulated funds. The fund also mitigates the effect of loan costs in the sense that funds taken from the reserve fund do not accrue interest penalties. This lowers the probability of costs exceeding zero to something slightly greater than 1.5 percent. However, costs can still be quite high, having a 0.8 percent likelihood of exceeding €100m. On the other hand, there is a strong chance that the flood will not happen and that, ultimately, the interest on the reserve fund can be either taken as a profit or invested in other loss-reduction mechanisms. For this case study, this effect is dominant because there is a significant chance of no disaster occurring at all over the time period concerned. In this case, the interest earned on the invested funds represents a profit. This illustrates the importance of the concept of risk as including potentially positive outcomes as well as negative outcomes. Even if an event occurs, the accumulated funds may be able to cover the costs if the event is not exceptionally severe. It can be seen that the probability of uncovered losses exceeding zero also drop because there is a significant probability that the accumulated funds will be large enough to cover the losses. A somewhat hidden but significant feature is that the loss-reduction properties of a reserve fund are amplified by the avoidance of high interest costs. By lowering the principal outstanding on a potential loan, the reserve fund is able to avoid loan costs. However, the catastrophic loss-limiting functions of this mechanism are very limited. For an organization facing potentially ruinous losses, the reserve fund does not eliminate its exposure in the way that an insurance policy might. Another significant contrast with insurance is that a reserve fund not only does not reduce uncertainties, but that it can even increase them (albeit often in a positive direction). Finally, two drawbacks not illustrated by this plot are the time dependency of the protection offered and the political risk that the fund will be diverted to other uses rather than being allowed to accrue interest. As a long time period of interest was chosen, there is a significant chance of accruing a large balance in the reserve fund. If a short time period was chosen (say, 10 years), the results might look quite different.

A clear feature of the combined financial alternative is that it brings together the low uncertainty of the insurance policy with the profit-generating possibilities of the reserve fund, a point illustrated by the graph. The benefit of a highly loaded insurance policy, on the other hand, would not be as high, although the reserve fund might be designed to offset some of the losses associated with premium payments. Of course, this combined alternative is subject to the same non-quantified risks discussed for the single solutions. An attractive element of this combination, however, is the possibility of an immediate risk reduction by the purchase of an insurance policy that takes effect upon purchase. The accumulated funds in the reserve fund

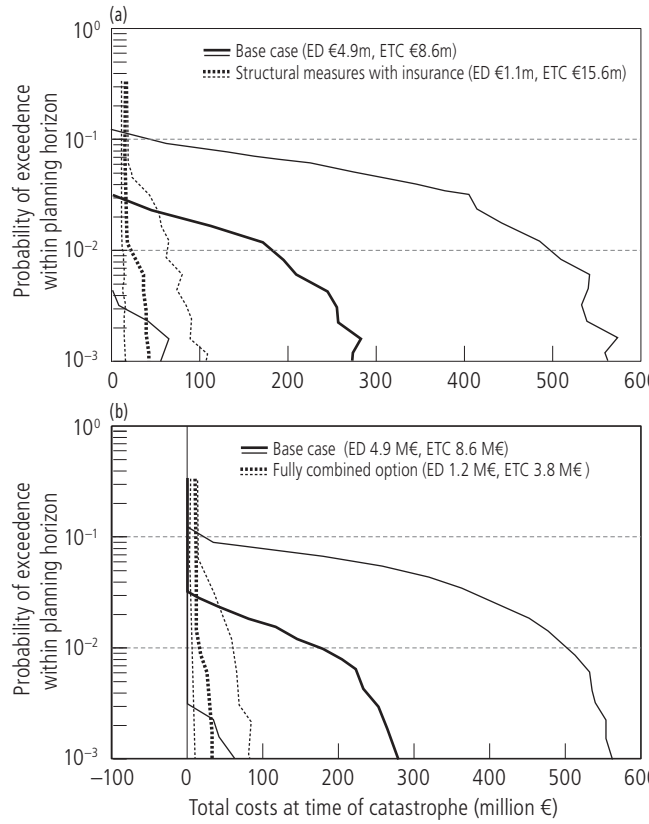

Figure 6.3. Total costs at time of catastrophe (million €): (a) structural measures and insurance; (b) mixed scenarios.

can help to offset the risk that the insurer may choose to withdraw coverage at some point in the future, as sufficient funds may have accumulated by that point to cover any possible catastrophe.

6.3 Mixed Measures

A final set of three scenarios combines structural measures with both financial measures, singly and in full combination. This comprises a scenario combining structural measures with insurance (*Figure 6.3a*) and a scenario combining structural measures and both financial measures. For this alternative, we combine the structural measures with an insurance policy, as defined above, with a 100 percent premium loading factor.

For the structural + insurance scenario, the computed premiums are only €30,000 (provided, of course, that an insurer would be willing to offer insurance at that rate) (*Figure 6.3b*). The installation of the floodgate and the upgrading of the detention basins have lowered the expected claims, allowing reduced insurance premiums. The expected damage is €1.1m per year, with expected total costs of €16m. An important aspect of this alternative over the purely structural combined alternative is that the uncertainty has been significantly reduced. Expected total costs are similar, possibly reflecting the fact that the savings in loan servicing costs of exceptionally large loans will offset the cost of insurance. The real appeal lies in the fully combined scenario. The expected damage is now €1.1m, leading to annual premiums of €30,000. The expected total costs are €3.5m, down from approximately €8.6m in the base case. Potential total costs are limited to well under €50m, and even considering uncertainty, are not expected to rise above €100m, regardless of the size of the flood. We can see that this approach blends all of the approaches, to yield a solution in which the advantages of each solution offset many of the disadvantages of the single solutions. The inclusion of insurance offsets the uncertainties associated with the other options. The inclusion of a modest reserve fund helps to avoid the potential for lost funds associated with construction of a structural measure that may never be called upon to function. The detention basin upgrades and installation of the flood barrier reduce expected claims to the point that insurance premiums are modest. Inclusion of a sensitivity analysis shows that even if the premium loading factor is increased to 1,000 percent (premiums = 10 x expected claims), the premiums are still only €160,000 and the total costs are approximately the same as in the no-action alternative, with the potential losses still being drastically reduced.

7
Discussion and Conclusions

Our primary conclusion is that the implementation of a concept of risk that integrates the different technical perspectives on risk into a unified framework is feasible and yields valuable insights into the nature of the protection provided by different mitigation alternatives. This implementation of an integrated concept of risk is achieved by identifying a clear assessment variable (total ex ante and ex post costs of mitigating flood damage) and expressing the probability distribution of this variable under different mitigation scenarios using a stochastic complementary cumulative distribution function or "risk curve." This approach provides considerable additional relevant information to a decision maker. It also allows the problem to be structured in such a way that it provides a clearer indication of the advantages and disadvantages of different mitigation options. This has been demonstrated by examining a current problem faced by decision makers and using, to the maximum extent possible, accurate and relevant data. We further note that the results highlight the fact that the advantages and disadvantages of a particular proposed mitigation option are complex and cannot always be reduced to a single-valued metric such as expected benefit or system reliability, as is typical of the actuarial and probabilistic approach, respectively. However, technical approaches need not rely on a single-valued metric. The portrayal of losses in terms of a stochastic risk curve, rather than a single-valued metric, provides considerable additional information without an undue level of complexity. For disciplines focused on the concept of risk as primarily probability (e.g., probability of suffering a financing gap or probability of system failure), we note that consequences matter. A failure that results in only minor damage or a financial option that results in only a minor financing gap is significantly different from one that results in catastrophic damage or an uncloseable financing gap, even if that failure or that gap is slightly more likely. The use of a risk curve can distinguish these and allow informed decisions. For analysts whose studies typically focus on expected values that combine probability and consequence into a single metric, we note that some options appear to be oriented toward the reduction of epistemic uncertainty. For example, a decision maker who is highly averse to uncertainty may consider insurance as a viable option, given that the fundamental nature of insurance is to transform an uncertain large loss into a certain smaller loss. As in any decision problem, the decision maker must be aware of his/her goals and constraints and not allow the analytical tools of the component disciplines to define the problem for him.

A second finding of the study is that although structural (loss-preventing) and financial (loss-spreading) mitigation measures may have significantly different characteristics, they may still be examined in a consistent way if an appropriate measure of risk can be identified. This is closely connected to the use of a broader conception of risk that identifies the strengths and weaknesses of different mitigation measures. Understanding the comparative strengths and weaknesses of different instruments can assist in the design of a system in which the advantages of some measures are used to offset the disadvantages of others, thereby reducing and controlling the risks. For example, the explicit treatment of epistemic and aleatory uncertainty allowed clarification of the different characteristics of reserve funds versus insurance. In this case, the reserve fund served to reduce (or even offset) the cost of ex post borrowing, although it provided essentially no protection against very large events and did not reduce the uncertainty in the loss curve. The effect of the reserve fund was to shift the risk curve in a beneficial direction at all probability levels. On the other hand, insurance provided protection against the relatively larger and less likely losses and reduced the uncertainty associated with the large events. The effect of the floodgate was similar to that of insurance in that losses from very rare events were reduced; however, insurance was clearly more effective at reducing the uncertainty of large losses at the expense of increasing costs. Both of these were quite different from the type of protection provided by the detention basins, which served to reduce the probability of losses but were subject to considerable uncertainty about the losses when the capacity of the basins could be overwhelmed by beyond design-basis storms. The synergistic effects of combined measures were apparent in that the use of structural measures assisted in mitigating the major drawback of insurance (the high cost) by reducing expected losses, while the insurance policy managed the residual uncertainty associated with the structural measures. Moreover, the effect of a reserve fund was enhanced when combined with loss-reduction techniques that extended the potential for accumulating adequate reserve funds. In this case, we were able to demonstrate that by using plausible values and realistic options drawn from a real flood-risk-management problem, considerable reduction in the total cost of mitigating flood damage may be achieved through a combination of structural measures with financial measures.

Several methodological issues arose during the course of the study. One was that integrating inputs from several disciplines into a single analysis, not surprisingly, can be challenging in practice. Even in the course of an integrated study, the proper way to link the output of the hydraulic model to the damage model was not clear. Although a solution was found at the end, the study may have looked quite different if the approach ultimately adopted had been used at the outset.[1] This

[1] It should be noted that this is one of the benefits of performing such a study in an academic rather than a consulting framework. Consulting studies typically do not have the luxury of implementing major model revisions during the course of the analysis. The consulting team must start with a clear

is mainly because of the different approaches to conceptualizing the risk analysis problem in the contributing disciplines. It is incumbent on analysts in such studies to understand the assumptions, limitations, and data requirements of the interfacing disciplines sufficiently to be able to communicate effectively. However, this suggests that integration is not simply a process of completing the component analyses and then combining them at the end. Considerable communication is required throughout the process to ensure that the necessary learning processes occur. Academic studies can help in this regard by providing templates and examples of how such integration might take place. Another issue that arose late in the study were the challenges to quantifying the "cost" of a reserve fund in a probabilistic way. The concept of opportunity cost, which is a traditional approach in cost-benefit analyses, is a simple concept in deterministic terms but considerably more complex to implement in probabilistic terms, when the "cost" can be negative. Finally, we note that we have approached the treatment of epistemic uncertainty in financial parameters from a very empirical, theoretical, engineering-oriented perspective, as the background of the primary authors is largely an engineering one. Our approach to uncertainty was quite consistent with what Renn (1992) has observed as the dominant technical paradigm of using relative frequencies (observed or modeled) as a way of specifying probabilities. Considerable improvements may be obtained by treating financial uncertainties using tools that are more widely accepted within the financial community.

There were significant limitations in this study; these suggest areas where considerable improvement could be made to the approach presented here. Although there are certainly many areas for improvement, it is the authors' opinion that the two major technical limitations of this study are the lack of specific accounting for the time preference of losses (i.e., no discounting) and the lack of a more thorough investigation of the "cost" of a reserve fund. An appropriate method of discounting for this problem was not identified. It was felt that the standard engineering cost-estimation approach of geometric discounting was inappropriate[2] because of the relatively long time horizons used. Use of even a moderate discount rate would tend to obscure the impact of large events occurring more than a few decades in the future. However, it is precisely these rare, costly, and infrequent events with

analytical approach before data are collected and simulations performed, or the study will quickly run over budget and over schedule. At worst, the consulting study may be delayed to the point that it cannot be used for a decision that must be made quickly.

[2]On the other hand, it was realized that if the losses are associated with replacement of items with a value that depreciates because of wear and obsolescence, and would be replaced or renewed on a regular basis with or without a flood, then high discount rates may be quite appropriate. In this case, the effect of a flood would be more related to the issue of cash flow, and an alternate metric (such as maximum annual cost rather than total cost incurred) might be more appropriate. This highlights the need to fully understand the objectives and goals of the decision maker before conducting an applied analysis.

which we are concerned. The decision not to discount was an explicit decision on the part of the lead author of this report. A major improvement to this study would be an examination of alternative methods for discounting future losses from catastrophic events. Moreover, as previously discussed, a full examination of the "cost" of a reserve fund in the context of a study that includes epistemic uncertainty was not carried out. The difficulties of applying the concept of opportunity cost for valuing the cost of a reserve fund were not fully appreciated at the outset and did not become apparent until the study was nearing completion. We also note that we have made no attempt at optimization in this analysis, largely because optimization requires a clear statement of the goals to be achieved and the constraints that are faced. Rather than hypothesize about what these might be, we consider that such parameters are best developed in consultation with the decision makers.

We may return at this point to Renn's discussion of the limitations of technical risk analyses. He identifies four major criticisms of the technical perspectives on risk: first, what people perceive as an undesirable effect depends on their values and preferences; second, the interactions between human activities and consequences are uniquely more complex than the average probabilities used in technical risk analyses are able to capture; third, the institutional structure of managing and controlling risks is prone to organizational failures and deficits which may increase the actual risk; and fourth, the numerical combination of magnitude and probabilities assumes equal weight for both components. On the other hand, he asserts that the the narrowness of this approach contains both its weakness and its strength and that the exclusion of social context and meaning from technical risk analysis provides an abstraction that enhances the intersubjective validity of the results but at the price of neglecting the social processing of risk.

Although these criticisms are well taken, we believe that it is also useful to distinguish between fundamental weaknesses and applied weakness. Several of the criticisms of technical risk analyses do not appear to be fundamental to quantitative simulation modeling. In particular, this study has addressed the fourth weakness and demonstrated that this is a problem more in the application than in the fundamental approach of technical analyses. The use of single-valued metrics that numerically combine probability and consequences are not necessary for the conduct of a technical risk analysis. On the other hand, we do recognize that the use of single-valued metrics is extremely common in practice. Overcoming this applied weakness will not be a trivial task. Several of the other criticisms, namely, that different individuals may value negative outcomes differently and that the institutional measures are subject to organizational failures, can also be partially addressed by improvements in the application of simulation techniques by developing models capable of quantifying the outcomes of concern to different stakeholders and by including terms for human or organizational failure. However, as quantification is a fundamental aspect of simulation modeling, these concerns probably cannot be

completely addressed within a technical framework. In some cases, the nature of the problem may be such that quantitative analysis is simply not the best tool for managing risk.

However, the virtue of exercises such as these is that they allow the impact of different potential goals and constraints to be examined systematically—at least to the extent to which the concerns of different stakeholders can be quantified. The value of such flexibility may become particularly apparent in situations where multiple stakeholders, with different objectives and constraints, must negotiate to determine a jointly acceptable solution. This advantage is hinted at by Walker (1997), and it is precisely this aspect of catastrophe modeling that is explored within the Tisza River study by Ekenberg *et al.* (2003) and Brouwers (2003). Approaches to scenario construction and goal/constraint identification within a negotiated environment are being pursued within the Risk, Modeling, and Society Project at IIASA. Furthermore, the optimization techniques explored by Ermoliev *et al.* (2000) and Ermolieva *et al.* (2001) may allow the use of integrated models in a close-to-real-time environment during meetings and negotiations. Evaluation of the characteristics of alternative financial instruments are being pursued by Mechler and Pflug (2002).[3] It is hoped that this study will contribute to the goals of the project by demonstrating an integrative framework that includes multiple forms of uncertainty, clarifies the characteristics of different mitigation alternatives, and deals with both structural and financial mitigation options on a consistent basis. It remains to future work to weave together the disparate strands of full treatment of uncertainty, integration of spatially explicit structural and non-structural mitigation options, fast optimization, and stakeholder negotiation to achieve the integrative possibilities that are now only potential in this type of analysis.

[3]The model is described fully in Mechler *et al.* (2006) and Hochrainer (2006).

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