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Development of a Catastrophe Model for Managing the Risks of Urban Flash Flooding in Vienna

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Abstract

This report provides a case study examining flood risks in the city of Vienna. The purpose is to illustrate an interdisciplinary approach to flood risk analysis, combining hydrological flood risk assessment and simulation modeling with the finances of flood risk management.

Three scenarios were preliminarily identified for analysis: catastrophic flooding on a major European river (the Danube) that flows through Vienna; storm flooding due to failure of storm drainage systems; and flash flooding of a small tributary (the Vienna River) that flows into the Danube. Our initial efforts revealed that the Vienna River flash flooding scenario was a credible, significant, and tractable problem for analysis. The wealth of data available also made this scenario a useful test case for developing and illustrating interdisciplinary work, which is a significant aspect of the project activity. The focus of this report is, therefore, on the flash-flooding scenario. This report does not include discussion of the other scenarios, as they were not completed in an interdisciplinary fashion either because of lack of adequate data and models for all aspects of an interdisciplinary study, or because there were judged to be non-credible and therefore of limited use as an illustrative example.

In the course of developing an interdisciplinary approach to examining catastrophic flood risks, we found that the concept of risk used in flood management varied subtly but significantly between the disciplines contributing to the study. An important result of this study is the integration of these different disciplinary concepts of risk within a single interdisciplinary analysis. A fuller accounting for uncertainty in a way that is consistent between the component disciplines, and the appropriate distinction between various different types of uncertainty, form a second major aspect of the study. Our primary finding is that an approach that integrates perspectives on risk characteristic of the different technical disciplines contributing to this study is feasible and that it provides a useful framework for comparing the characteristics of different mitigation strategies. The results of simulations suggest alternatives for combining different mitigation measures such that the characteristics of different components of an overall strategy complement each other to lower total costs and to reduce both the likelihood and the uncertainties of catastrophic financial losses.

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Development of a Catastrophe Model for Managing the Risks of Urban Flash Flooding in Vienna

1. Introduction and Theoretical Background

The purpose of this report is to illustrate an interdisciplinary approach to flood risk analysis, combining hydrological flood risk assessment and simulation modeling with the finances of flood risk management. We do this by examining flood risks in the city of Vienna together with some alternatives for mitigation of the damages caused by flooding.

In the course of developing an interdisciplinary approach to examining catastrophic flood risks, we found that the concept of risk used in flood management varied subtly but significantly between the disciplines contributing to the study. However, although the differences in usage may appear subtle, the way in which the term "risk" is conceptualized (for example, as probability, as consequence, as expected value, etc.) can significantly affect the way in which the analysis produced within a particular discipline is structured. More importantly, it can significantly affect conclusions about recommended courses of actions, particularly when a decision-maker is faced with choosing from among very different options developed on the basis of analyses prepared within different disciplinary frameworks. This can arise, for example, when attempting to decide whether to implement a structural approach (e.g., raising the height of river levees) or a financial approach (e.g., transferring 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 between different disciplines.

This project has been carried out within the framework of catastrophe model development. We will spend some effort in this section to introduce the reader to different concepts of risk that arise within catastrophe modeling. We will first discuss a 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 remainder, and majority, of the report (Chapters 2-5) lays out a concrete implementation of these ideas in a case study examining the 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, for managing the risks of flooding in Vienna and elsewhere. Our results build on on-going work at BOKU and IIASA on the development and use of models in the management of catastrophic risks (Amendola et al. 2000; Brouwers 2003; Ekenberg et al 2003; Ermoliev et al. 2000; Ermolieva, 1997; 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, catastrophes, and spatial and temporal scales.

As in any analysis, we have operated under significant constraints, some external and some self imposed. A self-imposed constraint is that it is not our goal in this analysis to attempt to provide and implement a 'true' definition of the term "risk" or "uncertainty". It is not clear if such a task is even possible. Neither do we attempt to include all possible concepts of risk within our larger analysis, although we do attempt 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. We do not intend to propose a canonical definition that can fit any situation. Our intention is only to clarify the way in which we have used these terms, and to show how a slightly broader conception allows integration across different technical (hydraulic and financial) disciplines. Such an integration yields, in turn, the ability to produce meaningful comparisons of very different flood mitigation alternatives. In addition, external constraints on the availability of resources and data over the course of the study restrict the usefulness of this analysis as a direct input into policy decisions regarding flooding for the city of Vienna. The study was not commissioned to provide such input. 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 policy makers, this study lacks several critical elements of a decision support study. Quality assurance and quality control (QA/QC) reviews of data and codes were not undertaken. A review of the legal and regulatory requirements for a decision was not performed. These aspects often impose significant legal and scheduling constraints on the analyst, and together with the budgetary and time constraints typical of applied analyses, hinder the exploration of alternate approaches to structuring and evaluating problems. We do hope to raise some interesting questions and suggest some possible courses of action should similar situations arise elsewhere. We are grateful to have had the opportunity to explore a very applied problem with the freedom to address issues and make decisions in the way that seemed most appropriate from an intellectual perspective rather than being forced to follow pre-defined approaches because of external constraints.

1.1 Concepts of Risk

The way risks are understood, analyzed, and quantified varies widely depending upon what type of system is under consideration. 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. However, evaluation of insurance and financial mechanisms for spreading and covering flood consequences implies financial representation of risks. According to Renn, the technical perspective of risks comprises 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). A goal of this study is the integration of these typically distinct approaches within the technical perspective. According to Covello and Merkhofer (1993) "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" (need page number). This definition is appropriate for our purposes since it offers fruitful opportunities for integrating the differing technical perspectives. Although largely consistent with the concept of risk used within the financial community, there are differences. Financial experts, extending 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 respectively 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 that the possibilities of unexpected positive outcomes are also included. The distinction is relevant to the extent that a policy oriented towards "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 includes the probability/consequence distinction and the (implicit) full conception of uncertainty advocated by Covello and Merkhofer, but broadens consequences to include upside risks as well as downside risks. We emphasize that the psychological dimensions, such as the aversion that individuals might have for certain types of risk, or 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 a 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. For example, in safety engineering the risks are associated with the reliability of a construction and probability of its failure. In this case, risks are endogenous on decisions. Traditionally, measures are directed towards increasing safety with less emphasis put on the estimation of the consequences of potential failure. This approach focuses on probability of occurrence as a measure of risk. A scenario to be avoided is identified (e.g., destructive flooding, release of radioactivity from an nuclear reactor, etc.) and the "risk" is the probability of occurrence of the adverse event. Typical examples of this paradigm include traditional

[1] According to White and co-workers (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."

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 hundred year flood (i.e., the probability of flooding in any year should be less than 1%). Other disciplines have focused on the magnitude of the adverse consequences as a measure of risk, most frequently keeping 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 exposures to hazardous chemicals in the workplace or in the environment such that no adverse effects are expected, without explicit regard 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 six hour period) and developing an empirical distribution. The functional form of the distribution can be determined based on statistical principles, or can be assigned based upon engineering judgment. The statistical problem is then using 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 or by limitations on the ability to collect samples. In many locations, reliable historical records may only cover a period of several decades. Even if

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[2] The magnitude or severity of a rainstorm is often defined as the amount of rainfall averaged over a specific period of time. Because 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 six 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.

it were, measuring peak rainfall or river flow during a storm is subject to error. There is also no guarantee that climatic conditions generating the rainfall or land use patterns that affect 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 in fact 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 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 the period of centuries or of millennia are dramatically larger than estimating the magnitude of storms that recur over the 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, or a storm that is expected to occur once every decade, may vary only over a few tens of millimeters and may be managed by simply adding an appropriate design margin onto an engineered design. In the United States, the use of a safety margin on levee heights of three feet (approximately 1 meter) was just such a consideration (National Research Council, 2000). However, when attempting to protect against storms that recur over period of millennia, the range of 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 might end up installing a system in which most of the costs were directed at ensuring that the design margin was sufficiently large. On the other hand, if one simply used 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 was much larger than the best estimate.

However, once effective measures are taken to protect against the more frequent floods, it is precisely these rare and uncertain floods that may now pose the majority of the risk to the affected populations. The decision maker is therefore in a quandary with pitfalls on either side. If the true likelihood of a particular severe flood is quite high and no mitigation efforts are undertaken, massive damages 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 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). In addition to the costs and benefits of different mitigation measures, the reliability of the mitigation measures is therefore 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 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 to structure these uncertainties in such a way that the decision maker can see the results of a decision and to what extent the losses and attendant uncertainties change under different decisions.

The approach we have chosen uses a "risk curve" or CCDF (complementary cumulative distribution function) to characterize the risk. A single CCDF plots the magnitude of an event on the horizontal axis vs. 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 exceedence 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 because 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 damages from one resulting in high damages, 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 a severe consequence vs. 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

Evaluation of measures to deal with catastrophes is challenging. It combines available historical data with various types of models. Traditional statistical and actuarial models are usually insufficient to represent the full range of potential damages a catastrophe may inflict on a location in the future. There are several reasons for this. The first is the intrinsic uncertainty in when a catastrophe may strike. Catastrophes are rare events that may occur immediately or may not occur for a thousand years. There is typically a lack of historical data on the occurrence of events in a particular location, though the data may be available on at larger spatial scales (e.g., regional or national scales). Thus, in our case assessment of risk, analyses are based on catastrophe modeling to gain additional information on the range of plausible future outcomes.

The catastrophe models being examined and developed within the 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 incorporate 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) modeling and PSA approaches have historically been used

[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.

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

1) The technique: Catastrophe modeling makes use of simulations rather than purely historical actuarial data for purposes of estimating probabilities and outcomes. One of the main reasons for developing a catastrophe model is that there is not enough historical data for actuarial estimates. One must therefore generate data 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 sub-models. 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 (a) allows the development of a model by interdisciplinary teams and (b) allows, 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 for the model to be developed by interdisciplinary teams allows the inclusion of the relevant expertise without requiring that all members of the team be experts in all disciplines represented in the model. The important element is that all members of the team should have an understanding of how to properly interpret the output of the submodels and all should understand the ultimate use of the model. The ability to implement computationally inexpensive reduced form models - referred to as "catastrophe generators" by Ermoliev and co-workers (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, it does not focus solely on the probability (e.g., the reliability of a system) of failure. In contrast to many actuarial methods, it does not

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

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. 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), catastrophe 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, and 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 policy impacts of options for a nation-wide insurance program. This proved useful at a stakeholder workshop, where the local residents, insurance companies and the central government reached a consensus on a policy direction. (See Ermolieva et al. 2001; Ekenberg et al. 2003; Brouwers et al. 2003; Linnerooth-Bayer and Väri, forthcoming).

1.4 Catastrophe Modeling and Uncertainty

Catastrophic risks are low-probability, high-consequences events. Often, stemming from the low probability, they are plagued by major uncertainties. One lesser-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 models do not yet exist that can synthesize accurate predictions of rainfalls, windspeeds, or other climactic phenomenon with detailed resolution across the full range of spatial-temporal scales (e.g., from global scale to scales on the order of square kilometers and from annual scales to hourly scales) necessary for accurate risk analyses. When the possibility of climate change is taken into account, the epistemic uncertainties increase dramatically. Petak and Atkisson 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" (p. 186). This statement remains as true today as when it was written twenty 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 for the success of insurance programs because of the effect that 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 in 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 a long experience in risk analysis techniques for dealing with uncertainty, and that 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 of Guy Carpenter, 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 range of results would be and how different approaches to a problem could lead to different outcomes" (Venter, 2003). 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) and Covello and Merkofer (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 Rhine River, see Merz et al. (2002). In addition, there is a long experience in producing the so-called robust strategies that do not require precise estimation of all uncertainties and risks. Instead, robust solutions yield solutions against the vast majority of uncertainties without the need for a precise evaluation of all all sources of uncertainty (see, for example, Ermolieva et al. 1997, Ermoliev et al. 2000, Amendola et al. 2000, Digas 1998, Baranov 2003).

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 towards 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 governmental treasuries. These distributions of impacts might be the basis for either negotiation, optimization, or both.

To realize these advantages, is it 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 and co-workers (2000) for the case of insurers, illustrating how catastrophe modeling can lead to improved policies on the part of insurers on their coverages 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 Tisza study (see Linnerooth-Bayer and Väri, forthcoming; Ermolieva et al. 2001; Ekenberg and co-workers, 2003) and earthquake risks management (see Amendola et al. 2000, Ermoliev et al. 2000, Baranov 2003) examined the use of catastrophe models in the negotiations between stakeholders (including citizens, local and national government officials, engineers, and insurers) dealing with flood risks on the Tisza River and with policy relevant discussions of earthquake risks management for insurance legislation in Italy and Russia. 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, thereby extending traditional engineering-based approaches to flood risk management to integrate 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, or "risk curve", we illustrate (1) an information rich approach to deal 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 case studies 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 to provide an integrated catastrophe model. This model is then 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 and the results of options integrating both structural and non-structural measures. Both epistemic and aleatory uncertainty are handled explicitly throughout. The report concludes with a discussion of the insights gained 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 on Figure 2.1, the river flows through some of the most densely populated districts of the city. The most exposed infrastructure is located along an over eight km long reach, namely, the subway line, which is constructed in an open section at the right river bank, and the main roads on both sides. From a hydrological viewpoint, flood hazards from the Vienna River are critical due to the large amounts 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.

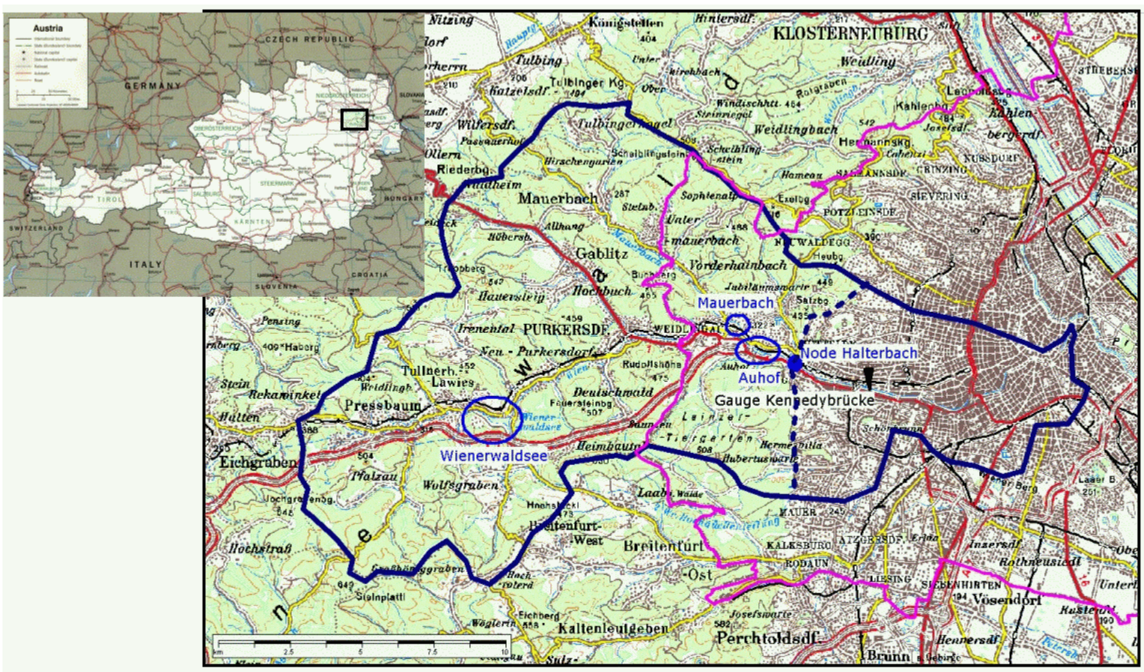

Figure 2.1: Vienna River watershed Map: ÖK 200, BEV (1999)

Indicated in Figure 2.1 are:

Watershed with rural (173 km²) and urban character (57 km²) dark blue
Outlet of the rural catchment for rainfall runoff modeling: Node Halterbach light blue
Flood retention reservoirs: Auhof, Mauerbach and Wienerwaldsee light blue
Gauge Kennedybrücke in the urban river reach black
Border Vienna - Lower Austria pink

The current design of the 12 km urban reach is a stonework and concrete bed and the construction of tunnelled river reaches. This system was constructed between 1895 to 1915 in parallel with the construction of the city railway. Two sections of 0.375 and

2.156 km were tunnelled. 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 hydraulic 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 tunnelled sections, lateral waves of +/- 0.75 meters at 5.5 - 6.5 m/s mean velocity (Grzywienski, 1965) 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. These storms are typically of several hours to days duration and generate flooding due to the fast watershed responses. Even low hills and mountains can intensify storm events in comparison to plain areas by regeneration of convective cells (Kelsch, 2001). The orograhpically 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 exceedence probability[5]. Applications of design rainfall data in flood protection and urban hydrology often employ rain yield or rain depth relations. Intensity-Depth-Frequency (IDF) curves are developed for specified regions from fitting mostly exponential functions to recorded rainfall aggregates of partial series. Modeling of very rare storms employs design values developed from local records or regionalized data. These numbers represent conservative estimates of expected values and the parametrical uncertainty is 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 4.5). It is unclear to what relative extent climate change, measurement errors, data processing and extrapolation uncertainties have contributed to this increase. According to the Vienna hydrographical service (Pekarek, 1998) the precipitation characteristics and recording and analysing methods have changed in the latest years so that currently return periods cannot be assigned to recently monitored extreme storms. A re-evaluation of the Schimpf criteria and design data, which were widely used in Austria since the early seventies, is recommended by the author. These criteria would imply that the 48 hours rain-depth of 240mm measured in the hills west of the city (K35-criterion) in July 1997 exceeded a 1000-year event. There are also concerns about the accuracy of the extrapolation of the Lower Austrian series 1901 - 1980 (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

[5] A simple way to determine the annual exceedence probability is to count the number of years that the flood exceeded a certain level and divide that by the total number of observations. In other words, a flood with an annual exceedence probability of 10% is that magnitude of flood that is equaled or exceeded in one out of every ten years of observation. It may then be referred to as the “ten-year” flood, although this may be misleading in that

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 e.g. by increasing the raw data by a certain amount. Design values for the greater region around the Vienna River basin were published from the following authors, mainly for and from Hohe Warte data. For completeness reasons, publications which are not directly relevant for this investigation are also listed.

Steinhauser (1957): Data of 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 & Schimpf (1965) assigns the K35 criterion to the western Viennese 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 Federal government (Lower Austria, 1985): This publication uses the 1901-1980 series and recommends design values up to 48 hours and a exceedence probability of 0.01 for different zones. The western Viennese hills and the Vienna River catchment are located in the region of 50 - 60 mm mean extreme daily precipitation, where 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 by 20 - 40 % 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 & Adam (1992): Design recommendations for conduits in Vienna are based on the maximum annual 15 minutes-rainfall intensity with 105 l/s/ha south-west of River Danube and 90 l/s,ha north-east. These values are developed from the 1901-1955 series (Steinhauser, 1957). Other rain durations and return periods are usually obtained by employing the Reinhold (1935 & 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 & Skoda (2000): Design rainfall is calculated by OKM (Orographic Convective Model; Lorenz & Skoda, 2000; HAÖ, 2003) using partial series of the ÖKOSTRA project (In the city of Vienna, only the Hohe Warte series is long enough) and a meteorological prediction model for convective storms with orographic influence. Lorenz & Skoda corrected the measurement error by a 5% increase of raw data. The orographic influence is accounted by incorporating a 1.5 km raster elevation model. Durations range from five minutes to 12 hours and return periods from 0.5 to 100 years. The authors recommend two formulas for return periods larger than 100 years and a re-evaluation of their results when improved convective models and a larger rainfall database are available. Electronic data was obtained from HZB via MA 45. These model data are available for entire Austria and is presently recommended in Lower Austria for durations up to 3 hours and return periods 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 lower limits of maximum convective precipitation inside an area of 6 x 6 km.

Lower Austrian Federal government (Lower Austria, 2001): A review of the Lower Austrian 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 to 1999 series of Hohe Warte.

The increase of the design values over time based on observations is evident by comparing Steinhauser (1957), Auer (1989) and ÖKLIM (2001). Higher values due to a different model approach are obtained by Lorenz & Skoda (2000). 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 the spatial variability.

For establishing the design rainfall amounts for flood investigations in Vienna River basin and protection reservoir adaptation, an extrapolation from the Lower Austrian series (1901-1980) and Schimpf's data was performed by Neukirchen (1995), as indicated in Figure 2.2. Both of these analyses were reassessed and it was concluded that the storm depths were underestimated. Figure 2.2 comprises the 30 % increased values from the Lower Austrian series 1901 - 1980. It also shows the values proposed by Lorenz & 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. Due to the orographic influence, the numbers for the rural Vienna River basin (which are not available) might be even larger, but they are currently reevaluated for annual probabilities smaller than 0.01 and durations of more than three hours.

For this study, it is assumed that reliable values fall between the design values and the Lorenz & Skoda figures, but 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.

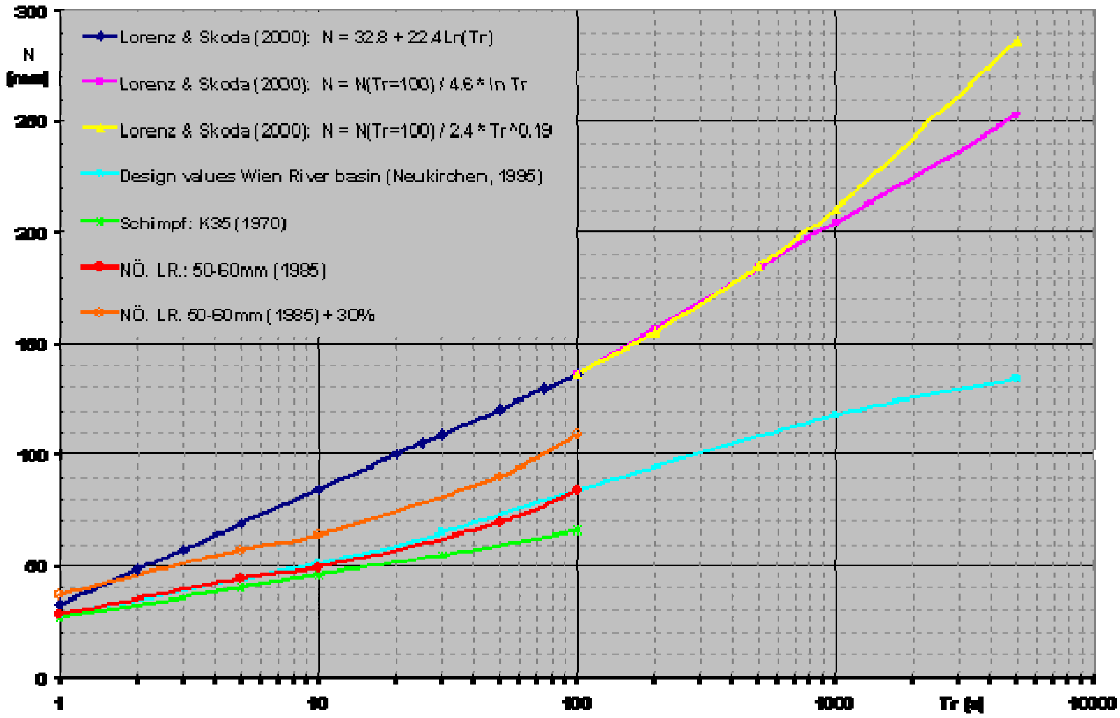

Figure 2.2: Comparison of 6 hours point design rainfall in the rural Vienna River catchment.

As rainfall of a larger areal extension has a smaller intensity as 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 km²) and the entire Vienna River catchment (230 km²), areal reduction factors of 95 to 80 % are found in Maniak (1988), Gutknecht (1982) and Lorenz & Skoda (2000). As this reduction applies to all point rainfall design values in the same way, it is neglected in the project.

The Vienna River has a mean annual flow, based on data from 1981 to 1999, of 1.16 m³/s (HZB, 1999). The maximum discharge was estimated for the 18 May 1851 event with 600 m³/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 gauge Kennedybrücke 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.

Table 2.1: Estimates of peak discharges during significant floods at gauge Kennedybrücke, km 7.65

Peak discharge [m³/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)

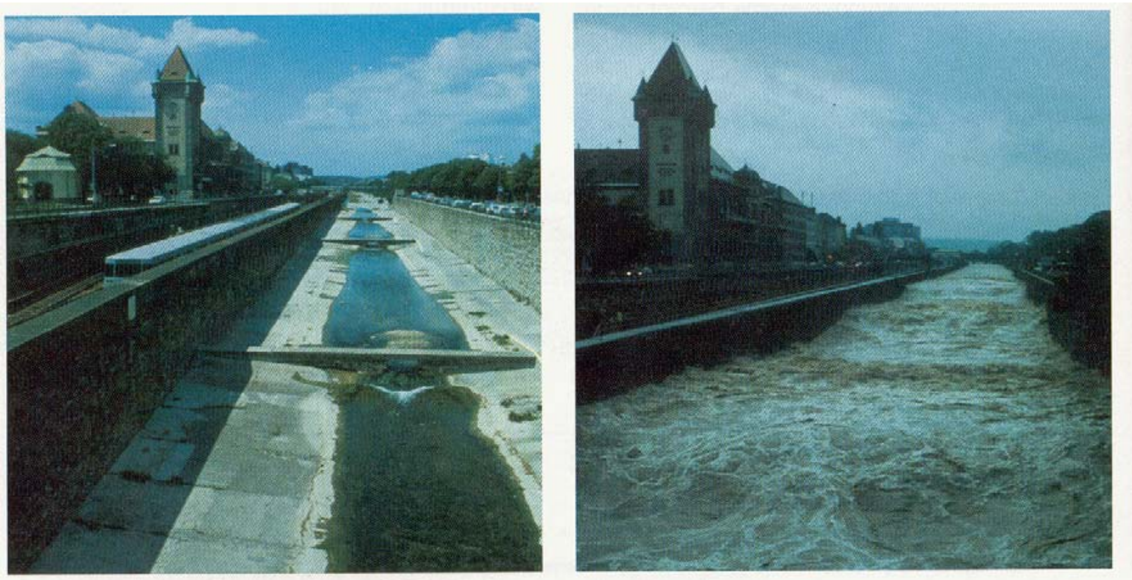

Figure 2.3: Vienna River at km 8 during normal flow conditions and during the 1975 flood (Source: Gewässerschutzbericht 2002, BMLFUW)

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 is missing. As the available gauge series are not very long and 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 storm water runoff is estimated and added along the river. It is assumed that the reoccurrence periods of rainfall and discharge are equal. Catchment models were established by Neukirchen (1985) with a simplified flood control basin performance estimation, 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 of 1991. The largest peak discharge and volume at the city's entrance were calculated for the six hours storm. The urban runoff contribution is calculated with a rainfall-runoff and hydrodynamic transport model (data e.g. 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 subway line U4 at the right embankment. For 7.5 km, it is situated mostly in open sections beside the river before it enters the underground track. A partition wall protects the subway line from floods. Portable flood barriers can be installed in two locations in order to prevent the overflowing water amounts from being conveyed to underground sections of the line that include major subway junctions. These emergency measures were installed recently and require a 6-hour lead time for installation. At the left embankment main roads are located, together with densely populated areas. Various service pipes are placed under the road embankments.

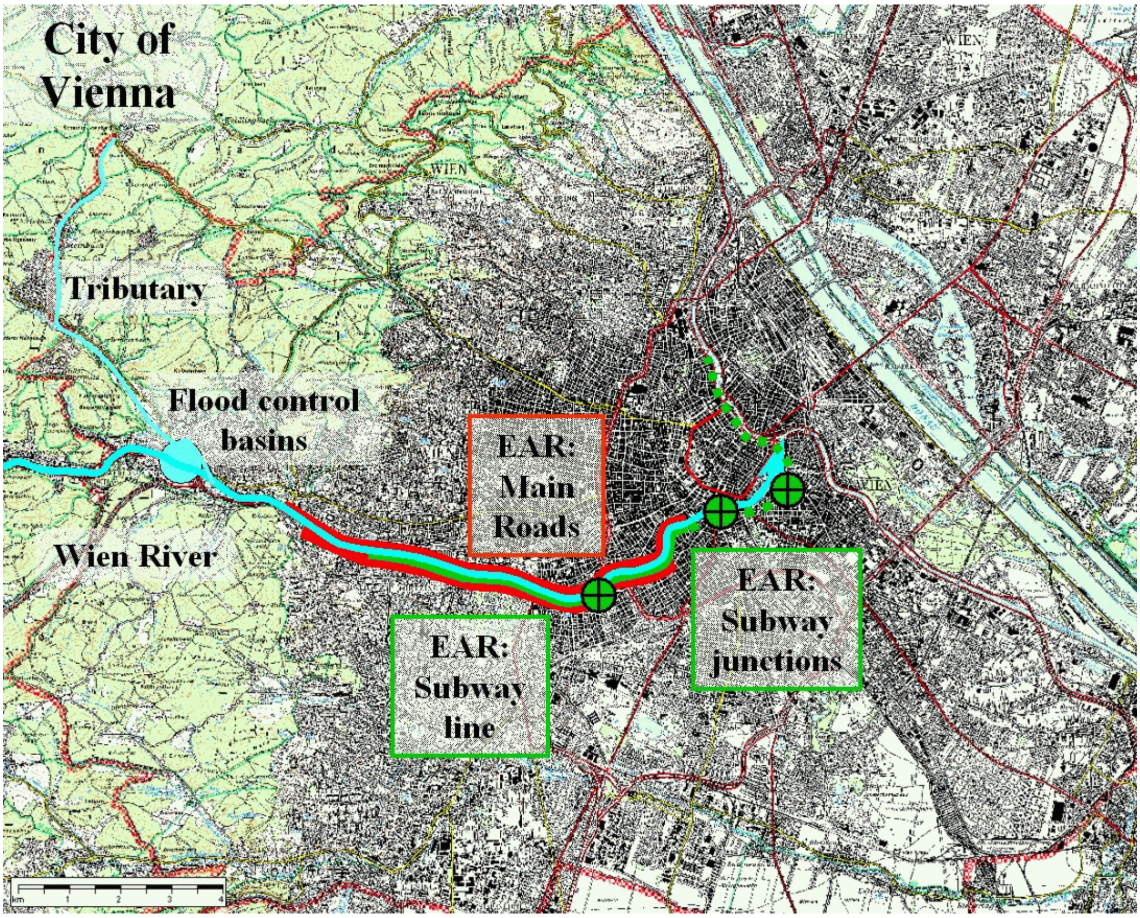

Figure 2.4a:

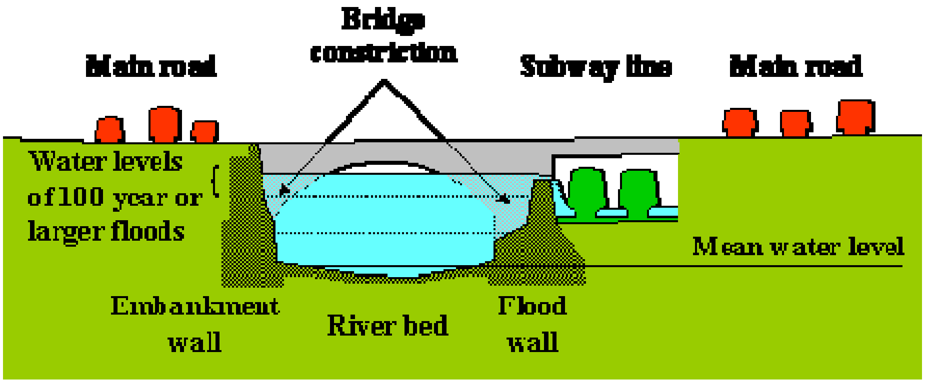

Figure 2.4b:

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

The construction of the first city railway along Vienna River was started in 1894, and opened for the public 1898. It was closed down in 1918 and re-opened as an electric line in 1926. The stepwise reconstruction into the current transport system 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 flood wall. 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 case 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 Längenfeldgasse, Karlsplatz and Landstrasse where the tunnels of nearly all connected lines are inundated.

In addition, about one kilometer downstream the Auhof basins, local inundation of both embankment roads may occur.

Another failure mechanism is wall collapse. The masonry subway partition wall was constructed about 100 years ago and subsequently restored. During floods, it is subjected to hydrostatic and dynamic horizontal water forces and in unfavorable cases also to pore water pressure acting in the wall joints and fissures. Considering the wall geometry of bends in plan view, the strength depends also 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 embankment wall on either the left or right banks.

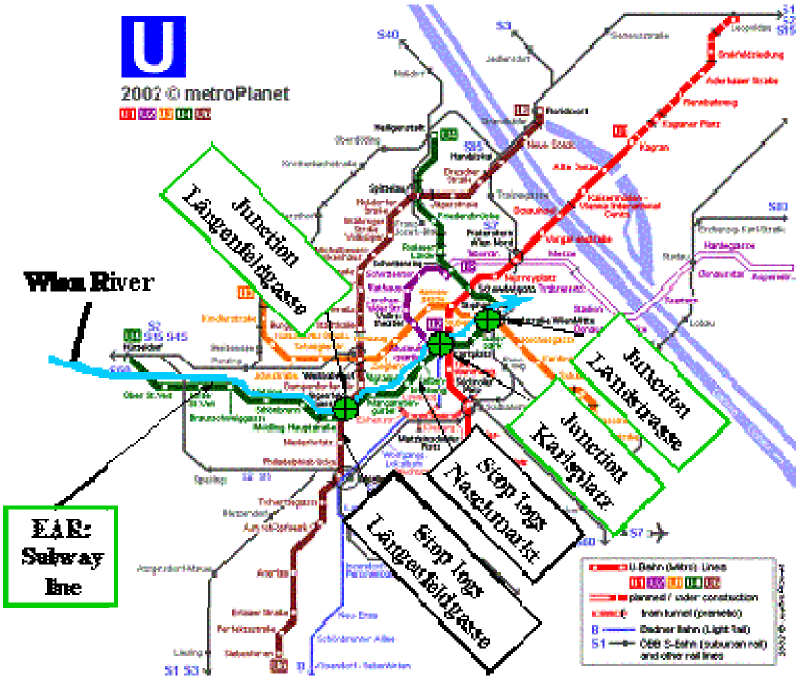

Figure 2.5: Subway network and protection measures. Map: *http://www.metropla.net/eu/vie/wien.htm*

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 a wall failure leading to severe damage on the left embankment and in consequence 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 this occurs 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 an improved ecological and recreational character for the river, a suite of 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. In order to improve flood-carrying capacity of the channelized river and to improve water quality, the study further proposes a large urban storm water bypass channel below the current riverbed. Urban storm water discharges can reach up to 200 m3/s at the mouth of Vienna River in extreme cases (Bauer, 1993; MA 45, 1996). This goal of the project is to reduce the 1000-year design flood of the rural river basin from its original (pre-1990) value of 475 m3/s to 380 m3/s. The entire urban storm water will be conveyed in a bypass channel located in the current riverbed. In addition, a forecast-based runoff model for reservoir control will be installed and the retention schemes (Figure 5.1) shall be adapted. The Mauerbach and

Auhof schemes have been rehabilitated to serve ecological and recreational purposes in addition to their flood-protection role. The re-design 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 will focus on rainfall forecast for the real time controlled basin operation and the implementation of a warning and basin operation system.

The current situation (March 2003) of 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 constructed state of 1900. According to a critical analysis in the eighties, the retention basin performance was found insufficient for adequate protection requirements as very large hydrograph peaks such the generated 100- and 1000-year events pass the flood control basins without considerable reduction of the flood peak (IWHW, 1988). This was due to an insufficient storage volume and control capacity, causing premature basin filling of the Auhof reservoirs by tributaries of the adjacent hills and by the increasing branch of the Vienna River hydrograph.

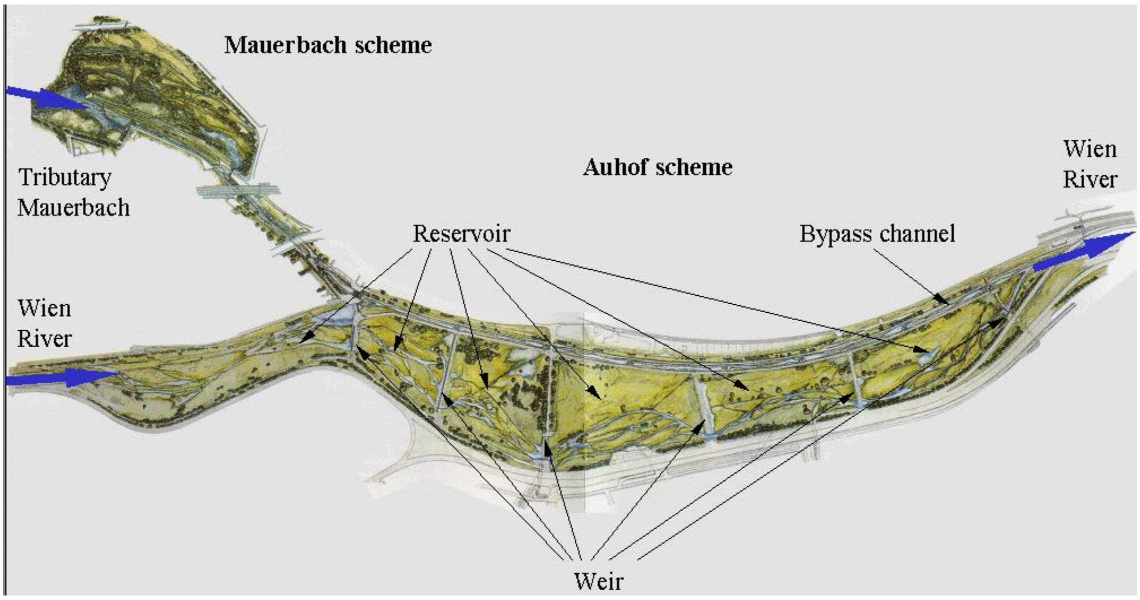

Figure 2.6: Auhof and Mauerbach retention schemes (MA 45, 1996)

The Auhof flood storage system consists of an upstream basin distributing the discharge into the bypass channel or the storage cascade consisting of five basins. During upgrading works, completed in 2001, the weir crests were partly heightened and hydraulic steel structures were upgraded for adaptive control purposes. The landscape of the basins was re-designed under an ecological viewpoint. The Mauerbach basins consist of a distribution basin and a storage basin. Similar changes to those implemented in Auhof were also conducted at Mauerbach reservoir and were completed in 2001.

The Wienerwaldsee is an artificial reservoir with a 13,5 m high barrage that was constructed in 1894 for drinking water provision of demand peaks and emergencies of up to 24.000 m3 per day (Bauer, 1993). Plans have been drawn up to 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 have not been started. Some of the reasons for this is the fact that other drinking water sources will take over its capacity in 2005, and the further utilization of Wienerwaldsee is therefore not clear. The options of selling the basin to the adjacent Lower Austrian communities or using the basin purely for flood protection purposes have been broadly 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
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 3 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 2.7. The remaining discharge in Vienna River in m3/s is colored blue. The lower profile exhibits the influence of the urban storm water bypass channel. A detailed description of structural and operational basin states can be found in Bauer (1993), MA 45 (1996) and Neukirchen (1995; 1996; 1997).

Table 2.2: Projected retention basin storage capacity along 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

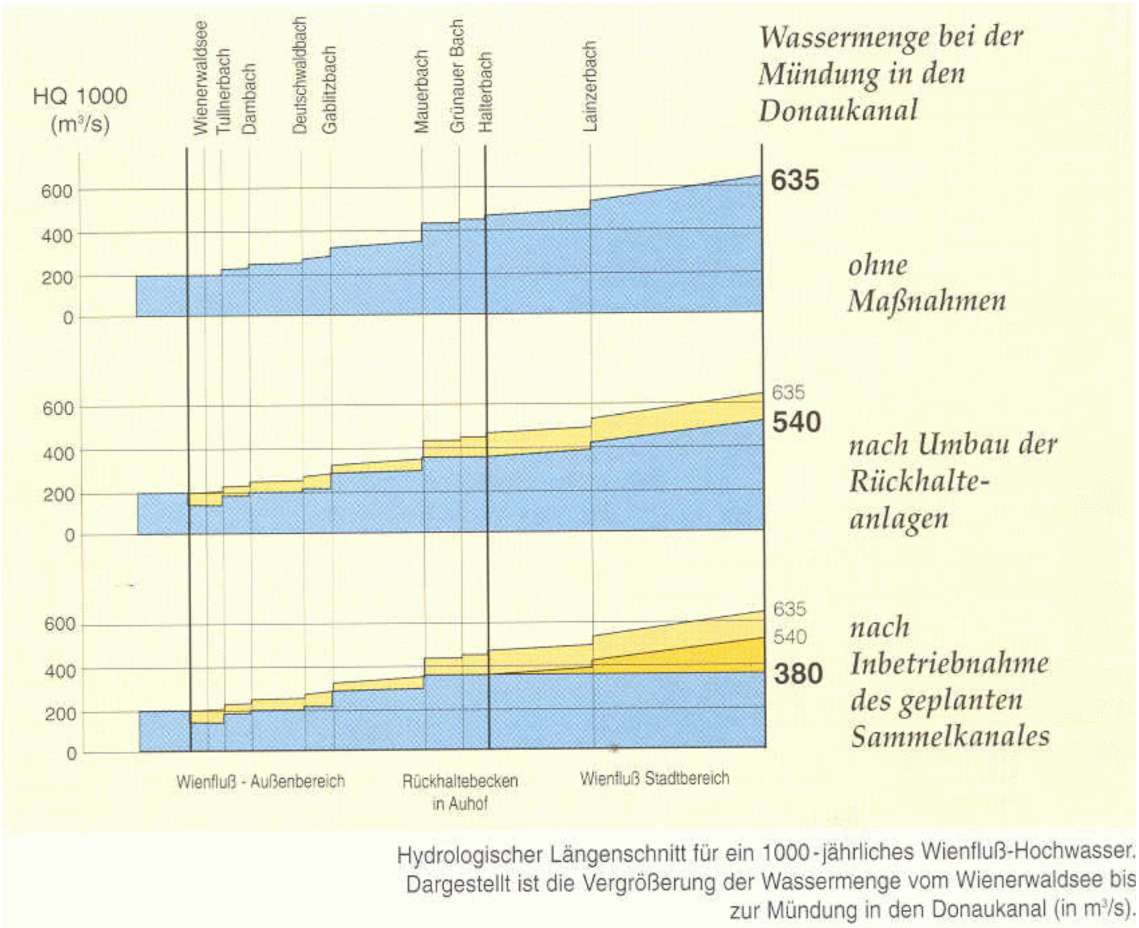

Figure 2.7: Hydrological profile of the 1.000-year design peak discharge (MA 45,1996)

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). Technical details of the data and models used can be found in that reference. The objective of the hydrologic and hydraulic analysis was to come up with an estimate of the frequency of failure of the investigated protection system 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

For modeling the watershed hydrology, the rainfall depth that was 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 1D-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 in order 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 flood wall, scouring of the river bed and collapsing 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, indicated in Figure 3.2 a) is fitted to the data points in order 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 or equal to a defined value x indicated by the return period Tr=t:

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

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.

$$\boldsymbol{P}(\boldsymbol{X} = \boldsymbol{x}_\tau) = \boldsymbol{P}(\boldsymbol{T}_r = \boldsymbol{\tau}) \cong \frac{1}{\boldsymbol{\tau}} - \frac{1}{\boldsymbol{\tau}+1} = \frac{1}{\boldsymbol{\tau}(\boldsymbol{\tau}+1)} \tag{3.2}$$

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 & 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 flood wall, the critical river bed material’s shear stress and the scour depth and centre 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.

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 sN. Parameters have to be estimated from design recommendations, consisting 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 the discussion on the accuracy of design values the preceding sections. Table 3.1 gives an orientation for the parameter estimation:

Table 3.1: Expected annual six hour storm depth

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

The assumed parameters are = 29.44 mm and sN = 16.75 mm, indicated by the black 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 sx is estimated by E(sN) in order to express sE. Table 6.4 and Figure 3.1 show the total scattering and its components.

GUMBEL	Estimated parameter			Mean	Standard dev.	Assumption			
	Mean value	N_m [mm]		29.44		Constant			
	Standard deviation	s_N [mm]		[illegible]	[illegible]	Normal distributed: 95.4% between [illegible] and [illegible] mm			
	Sample size	n		[illegible]		[illegible] Series			
	Reduced mean value	y_n		[illegible]					
	Reduced standard deviation	s_{yn}		[illegible]	Standard deviation due to				
					limited sample	data error	both		
$P_u = 1-1/Tr$	Tr [a]	y_{Tr}	K	N_{Tr} [mm]	$s_E(T_r)$ [mm]	$K(T_r) \cdot s_{sN}$ [mm]	s_{NTr} = $(s_E^2+(K s_{sN})^2)^{1/2}$	$N_{Tr}+2s_{NTr}$ [mm]	$N_{Tr}-2s_{NTr}$ [mm]
0.5	2	[illegible]	[illegible]	[illegible]	1.7	[illegible]	[illegible]	[illegible]	[illegible]
0.8	5	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.9	10	[illegible]	[illegible]	[illegible]	4.1	[illegible]	[illegible]	[illegible]	[illegible]
0.98	50	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.99	100	4.6001	[illegible]	[illegible]	[illegible]	[illegible]	13.17	[illegible]	[illegible]
0.999	1000	[illegible]	[illegible]	[illegible]	11.6	[illegible]	[illegible]	[illegible]	[illegible]
0.9998	5000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
0.9999	10000	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 3.2: Estimated parameters and Gumbel statistics

Table 3.2 exhibits expected values NTr, data scattering sE due to limited sample size n and raw data error K(Tr)ssN, and sNTr. Curves indicating +/- 2sNTr (95.4% confidence interval) are indicated. For modeling purposes, similar results can be achieved in a simplified way by representing all uncertainties by the standard error sE(Tr). In order to achieve corresponding plausible scattering, the underlying sample size is reduced to n = 30. The mean value is estimated to 29.44 mm, the standard deviation is chosen manually to 15.7 mm.

GUMBEL	Parameter						
	Mean value		N_m [mm]	29.44			
	Standard deviation		s_N [mm]	15.7			
	Sample size		n	30			
	Reduced mean value		y_n	0.5362			
	Reduced standard deviation		s_{yn}	1.1124			
$P_u = 1-1/Tr$	Tr [a]	y_T	K	N_T [mm]	s_E [mm]	$N_T + 2s_E$ [mm]	$N_T - 2s_E$ [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	1000	6.9073	5.7273	119.36	18.93	157.22	81.50
0.9998	5000	8.5171	7.1745	142.08	23.25	188.58	95.58
0.9999	10000	9.2103	7.7976	151.86	25.12	202.09	101.63

Table 3.3: Simplified Gumbel parameters and extrapolated rain-depth values.

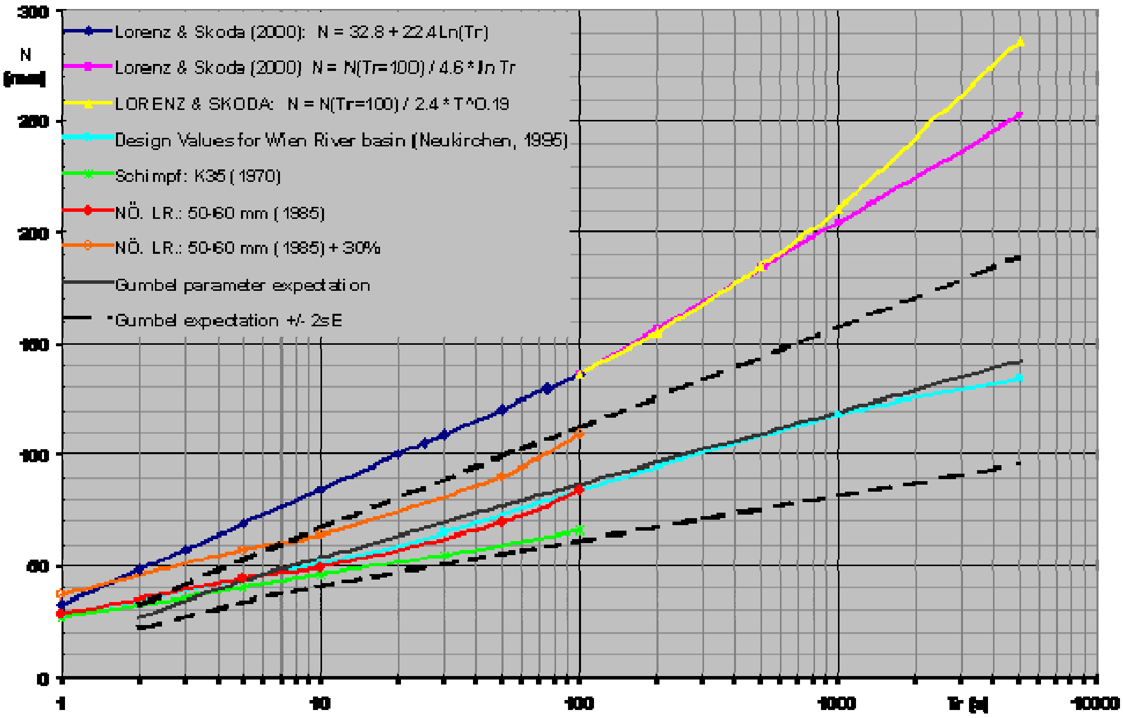

Figure 3.1: Design values of 6 hours storm depth in Vienna River catchment with fitted model curves according to Table 6.5: Expectation +/- 2 sE

The standard error sE obtained in Table 3.3 corresponds to 12, 15 and 16% of the expected NTr of the 10, 100 and 1,000 year event respectively. Nobilis (1990) estimated a standard deviation of 15% of the expected extreme rainfall in Austria. DWD (1997) and Skoda (2003) report similar deviations for Germany and 24-rainfall in Lower Austria. As the expectations and confidence intervals of the simplified and the detailed uncertainty investigation diverge by less than 1 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 a preceding section. The storm depth-peak discharge relations implemented in this study are mostly 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 both projects provide barely information on some 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 6.6 presents a summary of the employed quantities of the six hours storm depth and the rural and urban peak discharges.

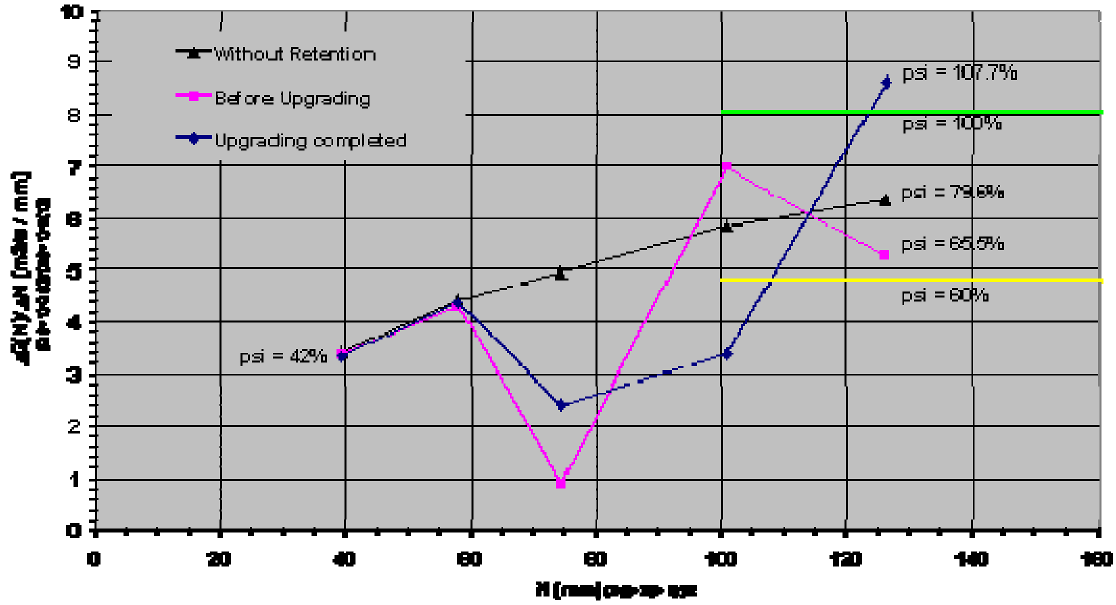

Figure 3.2: Storm depth-peak discharge transfer curve for the 6 hours rainfall in the rural Vienna River watershed at node Halterbach

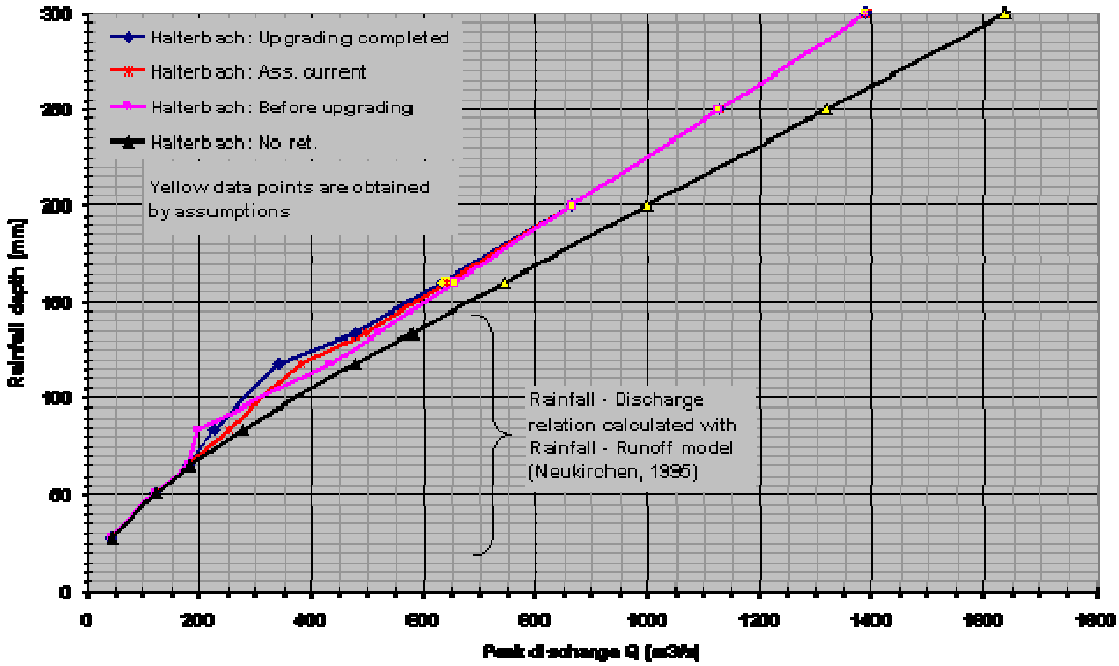

Figure 3.3: Gradient DQ(N)/DN from rainfall-runoff simulations (Neukirchen, 1995) with runoff coefficients y(N)

The rain depth-peak discharge curve at node Halterbach, 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 % (v. Figure 3.2 and Figure 3.3). This value is assumed constant due the steady precipitation losses of saturated soils. The magnitude corresponding to DQ/DN = 6,38 m3/s per additional 1 mm precipitation in 6 hours in the 173 km2 watershed is determined as the peak flow increase between the simulated 118 and 134 mm/6h scenario. The rain depth-discharge curve at Halterbach for the system state before retention basin adaptation is extended by a peak runoff coefficient of 65.5%. This number is smaller than the former due to the natural storage in the uncontrolled reservoirs. It refers to DQ/DN = 5.25 m3/s,mm in 6 which is the peak discharge increase between the simulated 118 and 134 mm/6h 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.

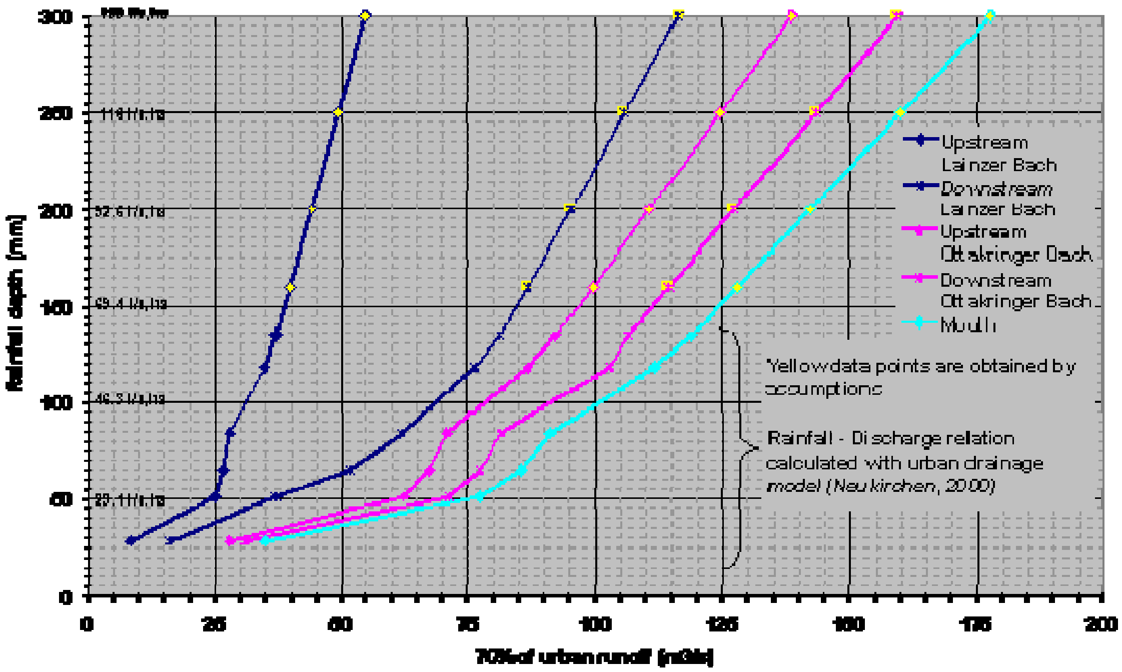

Figure 3.4: Transfer curves of storm depth and reduced discharge for the urban Vienna River catchment and the 6 hours rainfall. Curves represent different river stations.

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. Due to 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% are established by Neukirchen (2000). Table 3.4 shows a summary of the employed rain-depth - peak discharge relations for the rural and the urban catchment and their design return periods.

[illegible]	[illegible]	[illegible]				[illegible]				
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

Table 3.4: Summary of 6 hours rain depth - peak discharge relation

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 Wienerwaldsee reservoir for active flood storage.

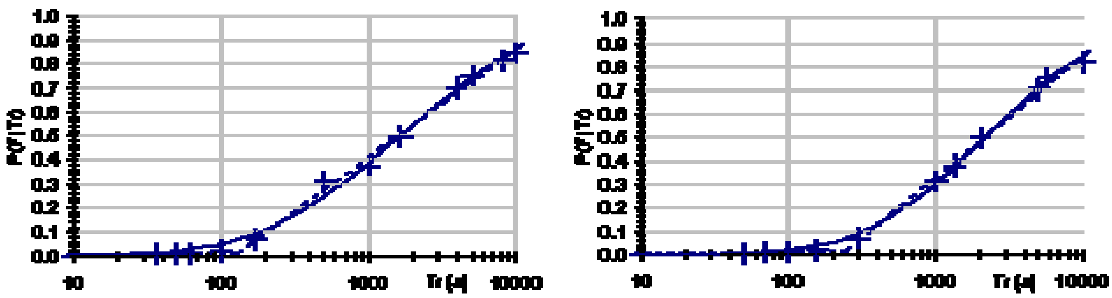


Figure 3.5: Conditional probability of failure for the current (l.) and the projected state of the flood control reservoirs. Crosses denote simulated data points; curves are fitte to obtain continuous functions. A) Logarithmic. B) Lognormal

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 upgrading the retention schemes Auhof and Mauerbach and the current state can be explained by hydrologic simplifications, the crude underlying data assumptions and by the unintended smoothing effect of curve fitting.

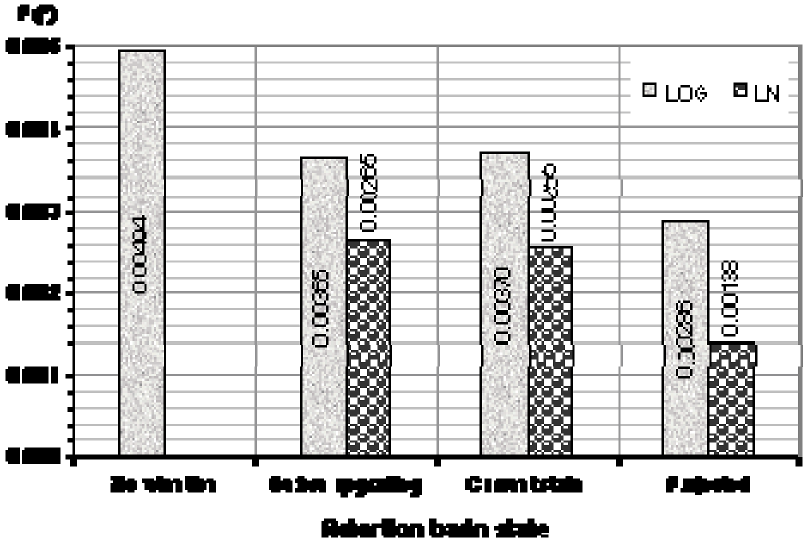


Figure 3.6: Total probability of failure for different structural and operational retention basin states

4. Damage Assessment Model Development

The objective of Chapter 4 is to develop an empirically justified approach to modeling financial losses from flooding of subways. Here, we only consider direct tangible damages. This is carried out via a two-pronged approach. The first approach is to examine past instances of catastrophic flooding of subways. The second approach is to examine and extend analytical approximations that were employed 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 damages. 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 ranges of damages in previous major floods.

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

- the length of track flooded;
- the level of standing water in the stations and along the rails;
- the 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);
- the 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.);
- the amount of warning time available to take mitigation measures (e.g., operating protection systems, installing additional pumping capacity, removing high-value equipment such as computers, etc.);
- The composition of the floodwaters (e.g., salt water, silt-laden water, etc.):
- Others not identified above.

Unfortunately, data do not exist to adequately characterize the quantitative relationship between all of 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 is able to use only a subset of all possible contributory factors to estimate flood damage. 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 presume that there is a relationship between the length of track flooded and the resulting direct damages, as carried out by Neukirchen (1994). This relationship can be expressed as follows:

$$DD = \alpha \cdot L \qquad (4.1)$$

where

DD is the direct damage,

L is the length of track flooded, and

α is the relationship between the two variables.

Because we were unable to obtain adequate empirical data on the effect of other factors, and because we wanted 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^3/s) of overflowing water.

$$\beta = f\left(Q_{overflowing}\right) \tag{4.2}$$

where

β is the damage multiplier, and

$Q_{overflowing}$ is the rate at which water enters the subway system due to overtopping or wall failure

Because 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 presumed that the results of Equation 4.1 represent reasonable worst case damages and the modifying factor is defined on the interval (0,1). For convenience, we choose an exponential function of the form

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

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. In order 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 rather quickly inundated after water begins flowing into the subway channel. We presume that damages begin to be incurred at a rate of 20 m^3/s and that an overflowing water discharge of 63 m3/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 upon 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) are reached at flow rates slightly higher than 63 m3/s. We therefore choose lambda to yield 50% 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 this 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 due to the nature of the floods faced by the system under study. Flash floods which result in rapid rises in water level to the level resulting 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% failure. 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

$$DD = \alpha\left(1 - e^{-\lambda Q_{overflowing}}\right)L \qquad (4.4)$$

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

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 on 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 Corp (PATH). 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, due to the sudden overtopping of the Mikasa River (Toda and Inoue, 2002). On 17 December 1999, the subway system in Caracas (Venezuela) was shutdown 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, damages from these cases were not reported.

There have been at least four cases in the past decade where floods were reported to have caused direct damages (repair costs) of greater than 10 million euro and service outages of more than a week. Two of the most severe cases occurred 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, USA

The Massachusetts Bay Transport Authority operates four rapid transit lines comprising 100 km 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 Green Line (so named because it runs along the park system designed by Frederick Law Olmstead known as the "Emerald Necklace" of Boston).

On the weekend of October 19-20 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 junction with the Charles River. This, combined with the backing-up of the local drainage systems due to the high river stage in the Muddy River, caused floodwaters to enter the subway system between the Kenmore Square and Hynes Convention Center / ICA stop. The majority of the damages were associated with the 53,000 m^3 of water that filled the Kenmore Square Station to a depth of over seven meters. Other less-flooded stations included Symphony, Prudential, Hynes, Copley, and Arlington. The total length of track flooded was approximately 2-3 km. [Brown, 1996 a, 1996b; CDM 2001; Moore and Chiasson 1996; Mercurio 2002].

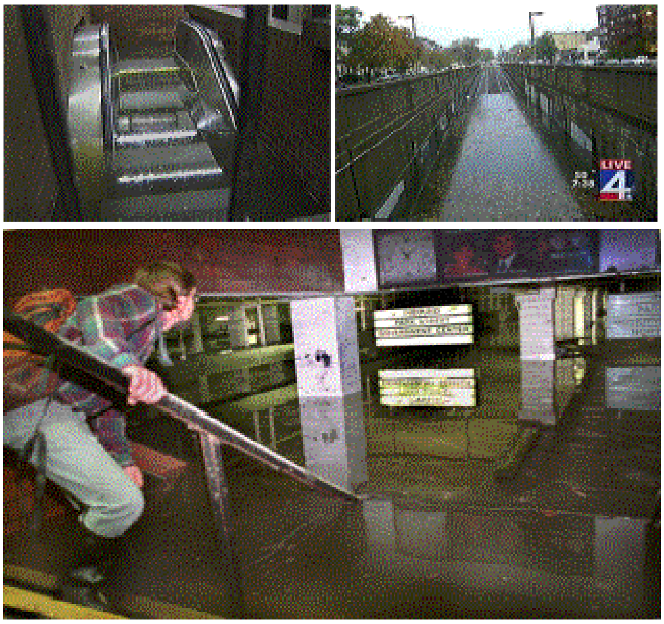


Figure 4.1: Images from flooded Kenmore Square Station. Photo Credits: WBZ-TV/CBS, WLVI-TV/WB, John Tlumacki (Boston Globe)

The design flood 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 signalling and track switching was done manually for some time due to the loss of the electrical and communication systems. No deaths or injuries were reported. The total damage was estimated to possibly exceed $10 million, and the total cost of upgrades to the signaling system was over $30 million. A portion of the repair and upgrade costs were to be financed by the 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 floodbarrier system that had been 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 floodwaters from entering the station. Although sandbags were places to try to prevent waters from entering the station, the efforts failed, as they had in 1962. The revised operating plan calls for provisions to adequately secure the boards used to complete the floodbarrier, 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 Seven, 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 800 million euro). The total line comprises 42 stations over a distance of 45 km between the Jangam and Onsu Stations (Korea Times, 2000).

A review of press reports yielded relatively little data on the flooding that damaged the line on May 2, 1998. The flooding occurred when retaining walls installed at a construction site on subway line six installed along the Chungnang Stream were breached at 7:30 in the morning during a heavy rainfall. The water flowed into the Taenung Station on line seven nine minutes later, and proceeded to inundate eleven 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 damages were reported to amount to approximately 45 billion won (approximately $35 million). Line seven 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% (from 500,000 to 300,000 commuters per day) as a result of the reduced capacity. (Korea Times 1998a, 1998b).

4.1.3 Taipei, Taiwan

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

On September 16-17 2001, Typhoon Nari produced 425 mm of rain over Taipei , causing the worst flood 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, the 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 twelve hours after the flooding of the Kunyang Station. The floodwaters entered at the Kunyang station and through a 6 m^2 hole in the basement of the Chunghsaio-Fuhsing station. The hole in the basement of the Chunghsiao-Fuhsing station apparently was an overlooked construction item, as the hole should have been closed when construction was completed. However, the contractor had failed to fill in the opening as required (Chuang 2001). Attempts to sandbag the high point in the line at the YungChun station were unsuccessful, and the floodwaters entered the Taipei Main Station at 11:45 AM on September 17. The MRT Control Station is located in the third lower level of the Taipei main station, and the computer servers and power supply are located on the fourth lower level. By 1400 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% of the computers and screens were lost, and all of the power supplies and cables (Kearns 2001).

The line between Kuting and Nanshihchiao was reopened on September 20, and the north-south Tamsui-Hsientien line was back in limited operation on October 1 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 October 14, with the Hsiaonanmen extension opening on October 17. By October 14, the system was up to 58% of its pre-typhoon daily average of 900,000 passengers per day. The line between Hsimen and Chunghsiao-Fuhsing was reopened on October 27 [Shu-Ling, 2001].

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

The design standard for flood protection of the Taipei metro was a 200-year flood event, which was exceeded by Typhoon Nari. According Kuo Tsai-ming, deputy director of the TRTC, the most affected systems were "communications equipments, 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 persons were killed during the typhoon, mainly as a results of mudslides in the north of Taiwan.

Reports of the estimated direct repair costs for the flooded subway ranged between €66-140 million (NT$2-4 billion) damage (Kearns, 2001; Surenkok 2001). A final report on the total repair bill was lowered to $53 million, due to cost savings associated with "donations of construction materials and reduced prices from companies not wanting to be seen making a profit from the typhoon's aftermath" (ref). Funding for repairs were sought from the municipal Department of Rapid Transit Systems, which sought to raise such funds from both the central government as well as from "austerity measures" from other municipal bureaus and departments (Shu-Ling, 2001). Insurance was not in place, as the system is insured only against fire and lightning damage. According to Lee Po-Wen, chairman of the TRTC, the system was not insured against typhoons due to the high annual premium costs of €3.3 million, or NT$100 million 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 fifty kilometers with 51 stations (ww.dp-praha.cz). Daily ridership is approximately 1.2 million. Because the system was also designed to serve as a fallout shelter, many stations were built with steel doors designed to seal off the stations in the event of either floods or nuclear attack (Krushelnycky, 2002).

In August 2002, the Bohemian basin received two exceptionally heavy periods of rainfall as a result of a slow moving tropical depression. The first occurred between August 6 and 7. The second period occurred between August 11 and 13 (www.praha-mesto.cz/povoden). In Prague, the Vltava river began to rise on August 12. On August 14, the river rose rapidly and overflowed its banks (Kikuchi and Sasaki 2002). The low-lying Karlin district was the most severely affected. Although barricades were erected, the water level exceeded the 1 m height of the barricades, and entered into the Florenc, Krizikova, Invalidovna, and Palmovka stations on the B line in the Karlin district and into 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 seventeen stations (see figure 4.3) 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 the Mustek Station, which is common to both the A and B lines. One station (Florenc) was reported to be inundated to a depth of 35 m, with two trains remaining on the tracks (Carey 2002). Over one million m^3 of water was pumped out of the system (Konviser 2002). The return period of the water levels in the Vltava were estimated to correspond to 500 year flow. The peak flow rate during the flood was estimated as 5,300 m3/s, which compares to a annual average flow of 145 m3/s and a 100 year return flow of 3,700m3 per second (http://www.praha-mesto.cz/povoden/)

The metro was at least partially insured by Ceska Kooperativa (Insurance Letter, 2002). Approximately 100 million of a European Investment Bank loan was earmarked for repair costs to the metro (CNA 2002). The loan was a thirty year loan with a seven year grace period (EIB 2002). There was considerable controversy surrounding the flooding of the metro. It was reported that the emergency door in the Invalidovna station failed, flooding the other stations. A complicating factor appears to be that the metro was kept running as the waters rose, due to forecasts which predicted flood peaks considerably lower than those actually observed.

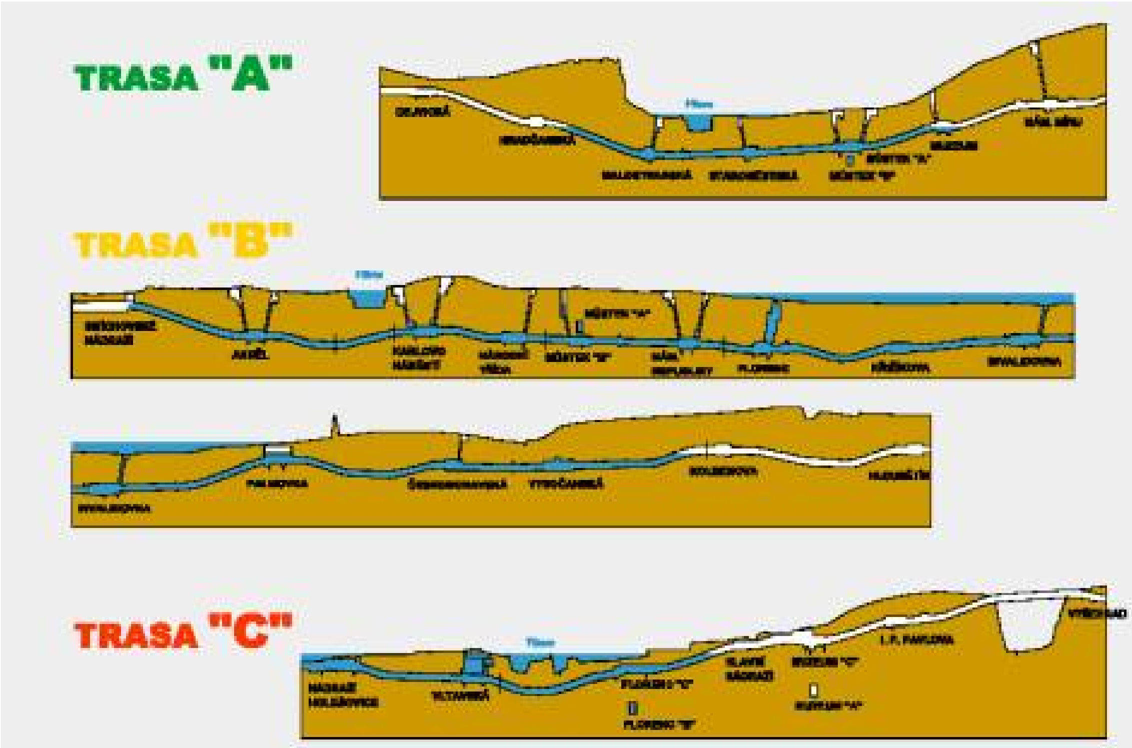


Figure 4.3: Extent of Flooding in Prague Metro (source: http://metro.mysteria.cz/povoden.swf)

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

4.1.5 Summary

A summary of the damages resulting from flooding on subways is given below.

Table 4.1: Summary of Reported Damages in Subway Flooding Incidents

M€	Boston, 1996	Seoul, 1998	Taipei, 2001	Prague, 2002
Total System Cost	N/r	790*	15,000**	N/r
Total Construction cost per km	N/r	18	~180	N/r
Km Track Flooded	02/03/03	11	9 - 12	15-20
Amount of water (thousand m³)	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)

Damages were 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 were reported to be due to flooding on the subway[6]. Reported deaths during these events were primarily associated with mudslides, drowning in swollen rivers, and electrocution from damaged electrical equipment. A common feature in all of these reported episodes was that human errors were contributory factors, and were major factors in some cases. These errors ranged from overly optimistic hydraulic forecasts to incomplete or inadequate construction methods and the failure to install or implement protective actions. We note that an evaluation of the reliability of any active system requiring human input or control should include a reliability of the operators. For some protective systems, 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. In order 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. This results in the following table:

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

A simple analysis of these values yields a mean of 9.4 and a range from 3.2 to 20. However, in order to avoid artificially weighting the cases where there were additional estimates (e.g., Prague), synthetic data points were generated as by taking the arithmetic

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

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 data set and shown in Figure 4.5.

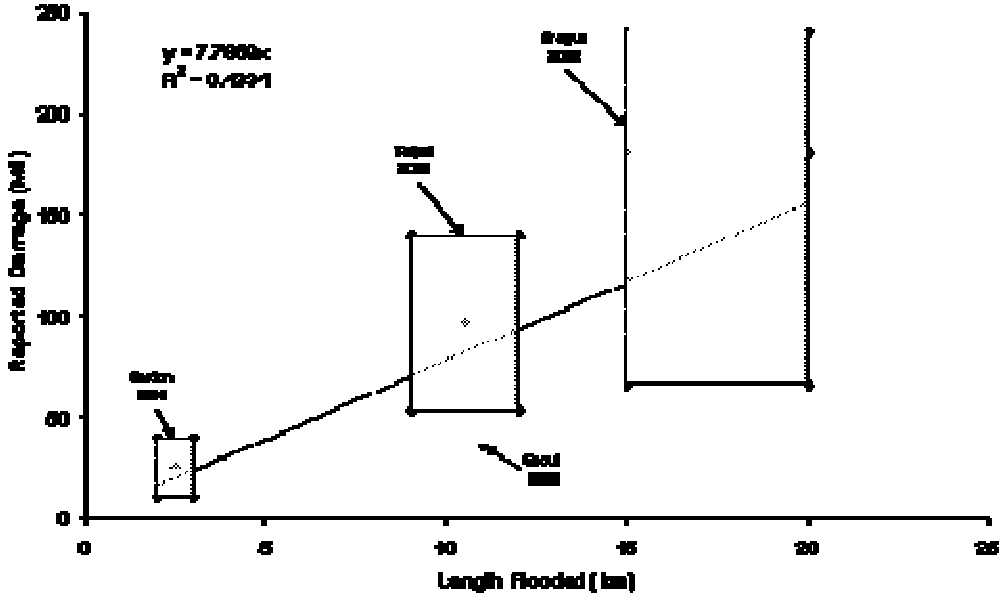


Figure 4.5: Relationship between Reported Damages and Length Flooded

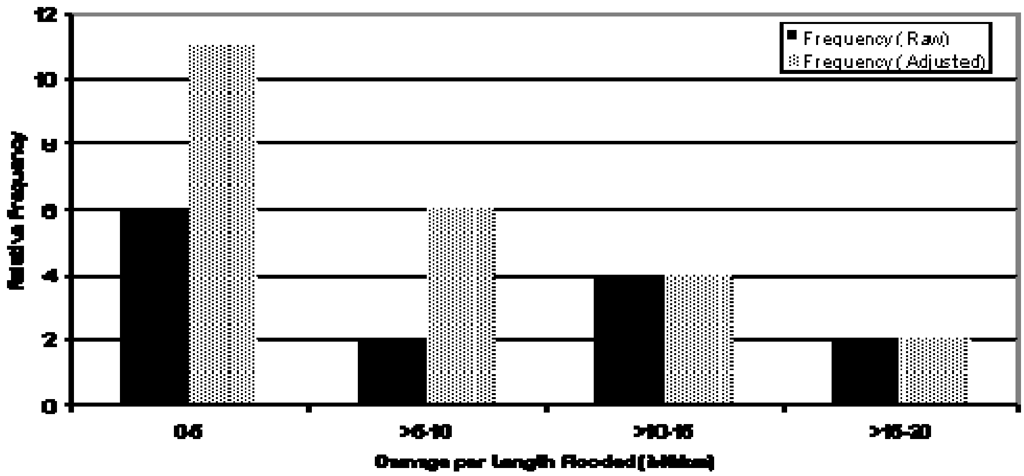


Figure 4.6: Distribution of Length/Damage Ratios

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 percent damage to the different systems as a result of inundation. This approach is similar to the approach developed in the FLAIR report (N'Jai et al. 1990) and others 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% of the construction costs and 15-20% of the electrical costs. According to (Laver and Schneck 1996), as-built costs for for subway systems in the US are as follows:

Table 4.3: Reported Costs of Subway Components (M$) from Laver and Schneck (1996)

Component	Median	Average	Stdev	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

Assuming that stations are located at intervals of approximately 1 km, the total cost of at grade systems average 9M$ per km and range from 8-42 M$ per km, whereas the average total cost of subway systems are 48 M$ per km and range from 25-120 M$ per km. Electrical systems comprise approximately 38% of the total systems and guideway cost for at grade systems but only 9% for subway systems (presumably reflecting the larger component due to excavation costs). Presuming 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 vs 3.1 M$ for at grade systems). 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 44-145 M€ per km with the estimated construction cost of the U4 between Ober St. Veit and Kettenbruckengasse (roughly speaking, an at-grade system) given as 58 M€ per km[7]. This is above the range reported by Laver and Schneck (1996) for the United States. The reason could be due to inappropriate exchange rates and also to the lack of inclusion of soft-costs and special costs, such as land acquisition, utility relocations, and various engineering design and

[7] The rates were originally given in schillings, which were pegged at at 13.7603 öS per euro in 1999. Because 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 600 million to 2 billion schillings per km, with the cost on the U4 between Ober St. Veit and Kettenbruckengasse as 800 M€ per km.

management costs. In addition, labor and tax costs may also vary significantly between Austria and the US.

If we assume that the electrical components comprise approximately 40% of the cost of at-grade systems, and that the damages to electrical systems are approximately 15-20% of construction costs and damages to constructions are approximately 10% of construction costs, then we obtain a damaged fraction ranging from 11-14% of construction costs for at grade systems and 10-11% 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 0.9-6 M$ per km. Application to subway systems yields 2.8-17 M$ per km. One can perform a similar exercise for costs associated with the Vienna metro.

Table 4.4: Ranges of Damage per Kilometer flooded, Method 2

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

C. Summary

The data from the empirical studies suggests that the values for alpha could range from 3-20 M€ per km of track flooded, with a most likely value around 5. The results from the engineering estimation yield estimates between 1 and 16, with a most likely value between 5 and 12. Considering the manifold uncertainties, we consider this to be relatively good agreement given that these estimations 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 episodes resulting in minimal damage). The analytical estimates may be biased by the potentially high as-built costs of the Vienna subway. However, this conservatism 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 possible conservatism.

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(-λQ)

*Implicit: damages were defined at 63 m^3/s.

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 upon the hydraulic simulation model, as discussed previously. This is a non standard distribution, thereby suggesting the use of numerical techniques. The way in which this equation is implemented comprises 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 corresponds to the "engineering" or loss computation module. As discussed, each of those analyses provide 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 damages.

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 goal is to create appropriate model abstractions. Because 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 of the sort carried out in Chapter 3 is computationally prohibitive when 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 (1990) 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. In order to increase computational efficiency, only sample from events that are likely to cause damage. For example, because damages are not expected at storms with recurrence intervals of less than one hundred years, only storms with recurrence periods exceeding this value are considered. It should be noted that this introduces a conditional probablility; 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 in 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 damages 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. One has now transformed the set of rainfall-probability pairs to a set of overflowing water-probability pairs.

4. Determine the direct tangible damages resulting from the overflowing water. One has now transformed the set of rainfall-probability pairs to 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.

4a. 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 damages 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 one has 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 these is simply to compute means or fractiles directly from the sample[8]. That 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 to what extent the losses can be compensated from a reserve fund or an insurance policy, and if the losses cannot be fully covered, obtain a loan to cover the costs. The premia 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 as a result of 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 catastrophic floods. It was therefore desired to evaluate this directly within the model and separate 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.

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

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 six 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 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 CDF given (Beyer 1968) as

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

The mean and variance of this distribution are given by

$$\mu_n = \alpha + 0.5772\beta$$
$$\sigma_n^2 = \frac{\pi^2\beta^2}{6} \qquad (5.1b)$$

Given an exceedence probability p, one can also therefore solve for the rainfall to which it corresponds.

$$n = \alpha - \beta \ln(-\ln(p)) \qquad (5.2)$$

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.

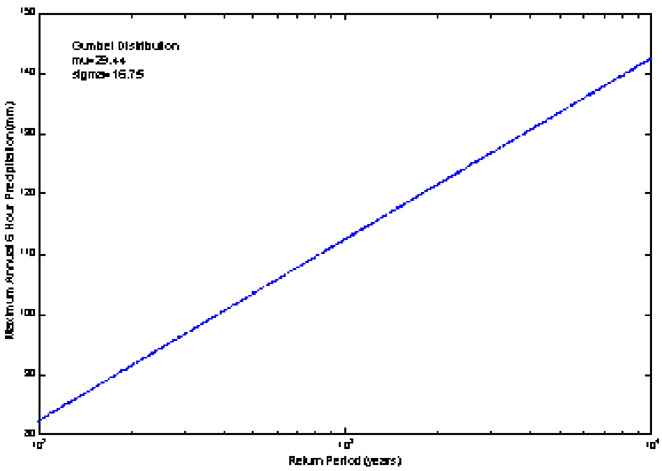


Figure 5.1: Rain depth as a function of return period

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. Because the incorporation of the detailed model is computationally prohibitive, a reduced form model is used that determines the discharge at Vienna River Km 4 corresponding to the sampled rainfall by the use of the lookup tables given in Chapter 3 and reproduced here. 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.

Table 5.1: Rainfall-Runoff Relations at Vienna River Km 4 as a function of Basin State

Six Hour Rainfall (mm)	Peak Discharge at Vienna River Km 4 (m^3/s)			
	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

The resulting deterministic discharge exceedence curves are shown in Figure 5.2 based upon 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 and the other parameters allowed to vary stochastically. However, because we believe that the uncertainties in the rainfall are likely to dominate the uncertainties introduced by the detention basins, and because the model is intended to be an illustrative model, we simply include the deterministic lookup tables.

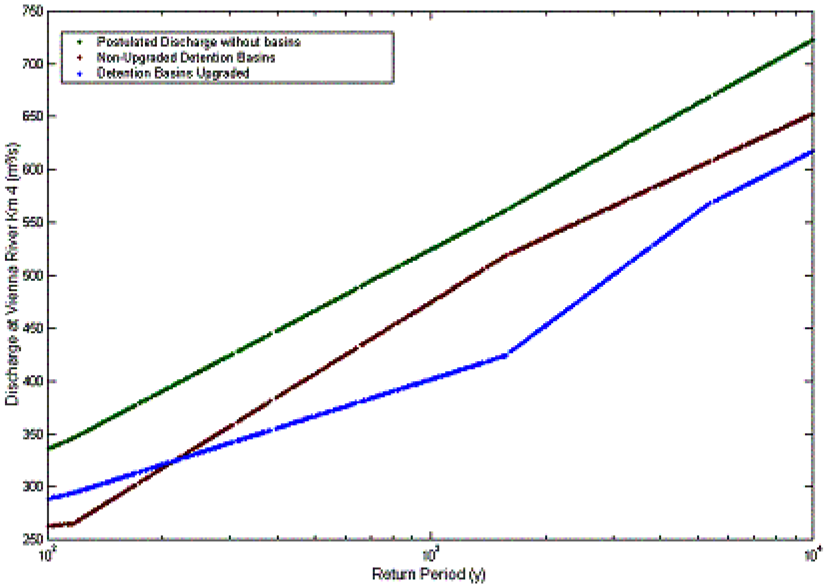


Figure 5.2: Rainfall-Runoff Relations at Vienna River Km 4 as a function of Basin State

5.1.3 Overflow Determination

The final step is to determine the amount of water flowing into the subway. Based upon examination of the results in Faber 2003 and in Chapter 3, it was hypothesized that the flow rate of overflowing water could be could be roughly estimated from the flow rate in the main channel. This hypothesis was generated by the observation that the likelihood of 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 exceedence probability suggests that the probability of failure increases dramatically once some "critical" flow is exceeded. This hypothesis appears valid based upon examination of the system and upon 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 in the Vienna River exceeds some threshold amount resulting either in overtopping of the floodwall or collapse of the floodwall due either to foundation scouring or hydrostatic pressure. It is clear that an overtopping failure is largely a function of the flow rate in the channel, and the uncertainties are largely those associated with the channel geometry, wall height, and roughness coefficients. Because 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 m³/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 m³/s. This implies that the failure of the basin could occur with a five percent probability at a flow rate of 510 m³/s and would be almost certain (95% likely) to occur once flows in the main channel exceeded 550 m³/s.

$$Q_{U4} = Q_{CRITICAL} - Q_{VRK4}; \qquad (5.3)$$

Figure 5.3 illustrates the relationship between the estimated peak discharge and the flow into the subway terrace with a Q_CRIT of 530 m³/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 m³/s would result in a period of three hours above the critical discharge of 530 m³/s.

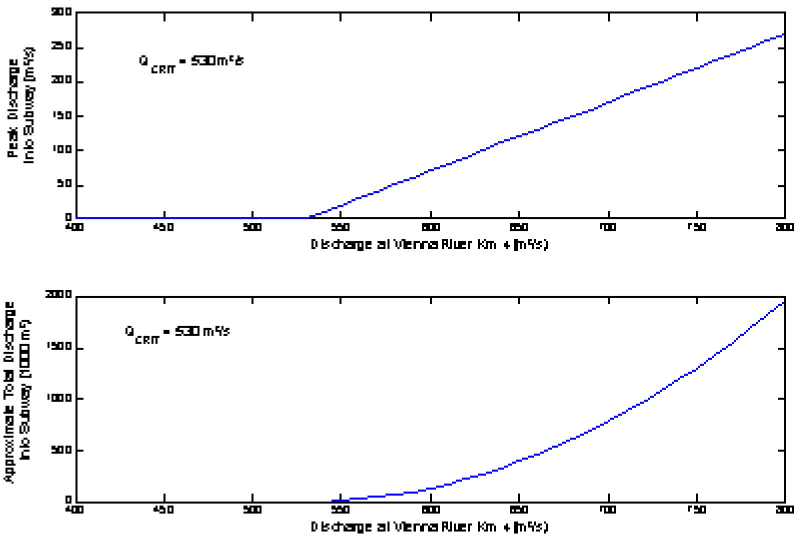


Figure 5.3: Estimation of Overflowing Water. Upper: Approximate peak overflow rate. Lower: Approximate total overflowing volume

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 purposes of illustration, we will proceed with this rough approximation. Provided that the "critical level of flow" hypothesis is valid, the model could be easily updated 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 overestimate, as a portion of the flow would be carried in 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 a soft-fail system. In other words, the system of detention basins and masonry floodwalls provides a very high level of protection up until 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[9].

5.2 Damage Assessment

As discussed previously, we consider the damage to be a function of the length of track that is flooded. Model abstraction is not needed for this portion of the analysis, as the damage estimation technique was sufficiently simple that it is not computationally expensive, and it was developed with implementation in the catastrophe model in mind. Estimation of the physical damage requires two parameters: the length of track that is 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 in every case if there is a flood (although the damages 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[10] to the portable flood barriers installed at the Große Einwolbung 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 likely point for flooding to occur is just before the portable flood barrier is installed. A better distribution would therefore be positively skewed, making shorter track lengths flooded more likely than longer track lengths flooded. 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.

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

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

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 until the outlet into the Donaukanal, for a total inundated stretch of 3.1 km. Because water entering the Karlsplatz station could provide a point of entry of 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 the Wien Mitte station via the U4 or the Stephansplatz station via the U1 could result in inundation of the U3. We take, at a minimum, flooding of the U3 between Burgasse and Schlachthausgasse for a total of 4.4. km flooded. To set an upper bound, we presume that the U3 could be inundated 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.

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

5.2.2 Damage per Length Flooded

As discussed in Chapter 4, it is assumed that the damage per length flooded (alpha) is presumed to be a uniform variable ranging from 1 to 20 M€ per km flooded. As previously noted, there was insufficient data to establish an empirically or theoretically grounded relationship between overflowing water and damage. However, it was clear that at low flows (which we define as 5-10 m³/s) the damage would be slight, but that damages would rise quickly as the pumping and drainage capacity of the subway was overloaded and would quickly reach 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 percent damage. In order to reflect the sharp rise of damages with overflowing water, an exponential function discussed in Chapter 4 was chosen. The value of lambda was chosen to give a 50% damage at a flow of 20 m³/s.

$$\lambda = \frac{\ln(2)}{20\,\frac{m^3}{s}} = 0.35 \qquad (5.4)$$

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

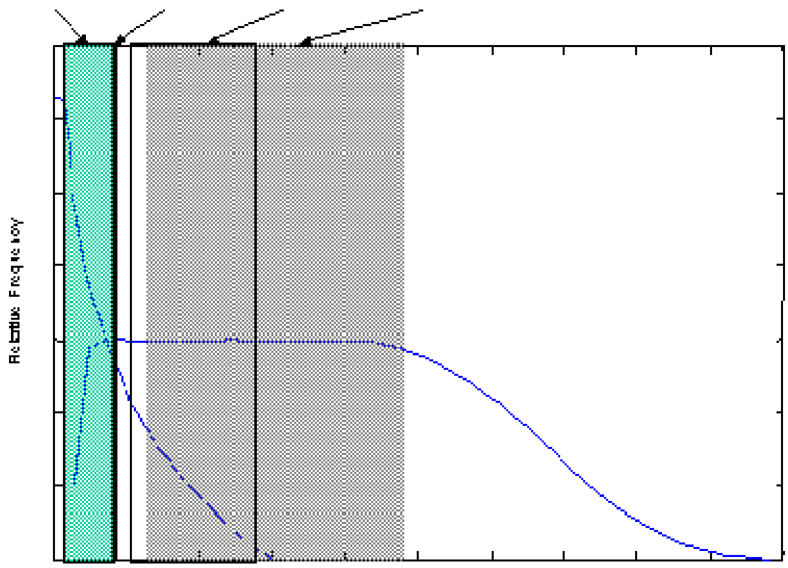


Figure 5.4: Comparison of the synthetic conditional damage distribution for Vienna with case study reports

For distributional sensitivity analyses, an alternative variant explored was to use normal distributions rather than uniform distributions to estimate the damages. 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 damages. The results are shown in Figure 5.5.

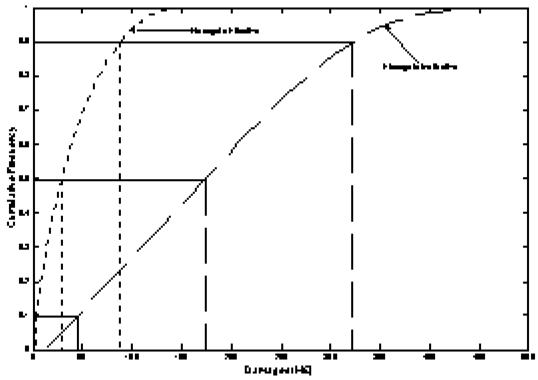
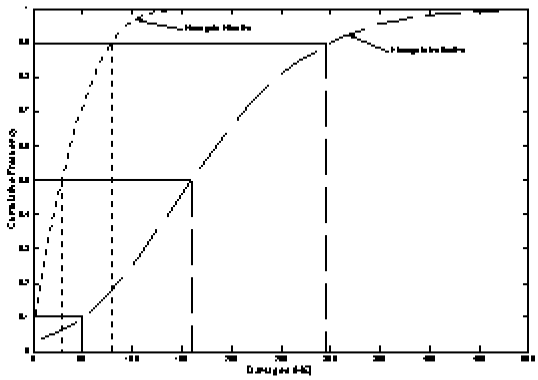


Figure 5.5a:



Figue 5.5b;

Figure 5.5: Effect of Distributional Forms. A) Uniform B) Normal

The loss of revenues associated with foregone fares was also considered. In order 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 upon the experience of past inundations, we take this value as five days per km track flooded. In 2001, the total subway ridership was approximately 400 million passengers (Wiener Linien, 2002). Dividing this number by 365 days per year and assuming that the U4 carries approximately 20% of the passenger load, we obtain a daily ridership on the U4 of approximately 200,000 rides. At a ride cost of 2€ per ride, we can derive a total fare loss of approximately 2 M€ per km flooded. Because this is only a small part of the maximum total potential damage, we presume that this is already subsumed within the damage estimates. A more detailed analysis might be able to explore this in more detail by examining the effect on revenues 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 of lost 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 damages are 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 a dollar, euro, or swiss franc, 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, then 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 to what extent a reserve fund has accumulated funds or for how long have premia 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, by expanding the terms in a binomial expansion, that this approaches a uniform distribution with

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

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 currently 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. 100% of all losses below the attachment point are borne by the policyholder.
- The proportion of losses within the insured layer that is 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, in order 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 damages is illustrated by an example figure showing the effect of an attachment point of 10M€ in direct damages, a 20% co-insurance proportion, and an exit point of 100M€ in claims as shown in Figure 5.6.

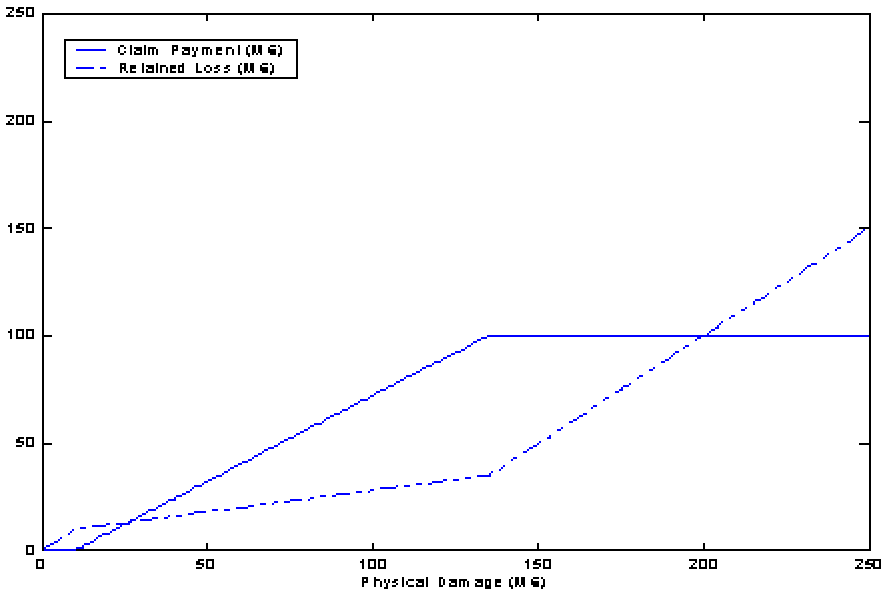


Figure 5.6: Structure of Insurance

There are two possibilities for determining risk premia. One would be to define the premiums to be equal to the expected claims. In essence, this assumes that insurance is "costless". In reality, the costs of insurance are non-zero, due to the need for administrative costs, profits, and risk premia. 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 due to the need for profit and the charging of a risk premium. In Mechler and Plug (2002), the functional form of the premium loading factor is simply PLF=1+0.03Tr. We compute premium payments in a fashion similar to Mechler and Pflug by charging a risk premium is charged 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, namely, 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 are dominated by the variability in this particular policy. The return period of the insurer's 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 signficant catastrophic risks, the integration of the different catastrophes into an integrated catastrophe model may be an efficient way to determine 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 premia 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 collected premiums are sufficient to cover the claims, the insurance reserve is positive and the premiums have been "overpaid". If the collected premiums 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 damages minus the claims.

Retained Loss=max(0,Damages - Claims);

5.3.3 Reserve Fund

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

Accumulated Funds=Initial Reserve Fund*((1+Yield)$^{\tau}$)

+AnnualPayment*(((1+Yield $^{\tau}$)-1)/Yield;

The growth of the reserve fund with a 10M€ initial contribution, a 1M€ annual contribution, and a 5% rate of interest is shown in Figure 5.7.

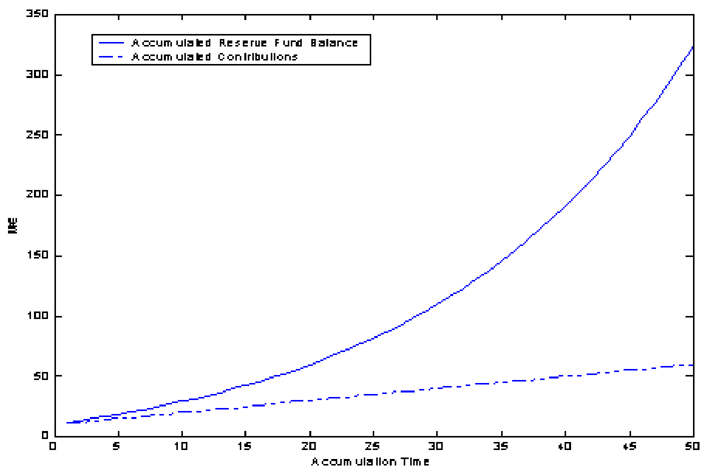


Figure 5.7: Accumulation of Reserve Funds

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 due to compounding.

A significant methodological questions 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 vs 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 carry a higher downside risk than bonds (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 under performed bonds (as 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 less volatile investment. One need only consider the financial history of the last several years to provide an illustration of such a phenomenon.

Because 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 and co workers (2002), who present data on the performance of bonds and equities over a century from many different markets. Because 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 the figure below 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 US), one developed economy that suffered two period of devastating inflation (Germany), as well as the world aggregate values.

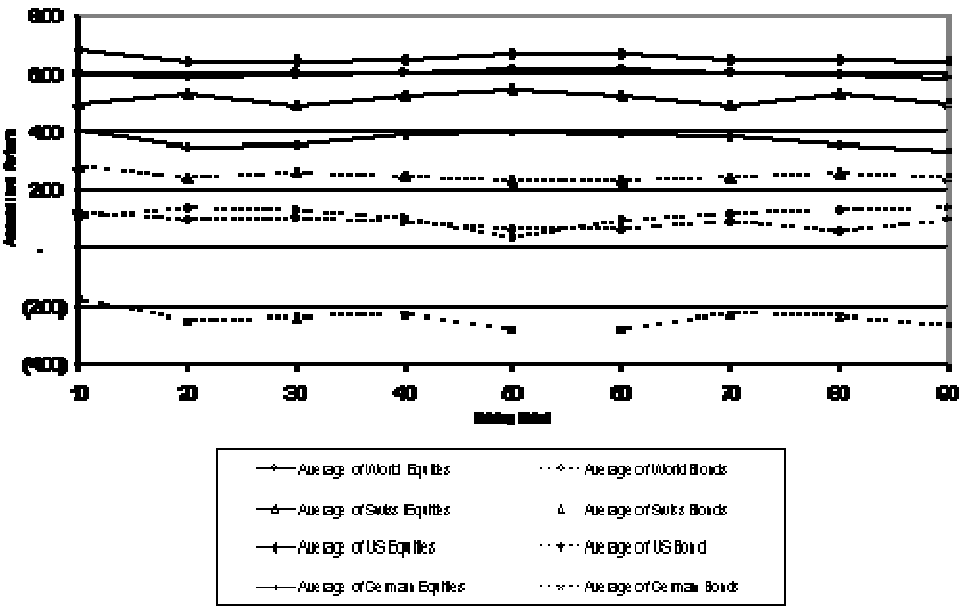


Figure 5.8: Real Returns to Equities and Bonds: Average Return as a function of Holding Period (adapted from Dimson et al. 2002)

It is clear that on average, equities outperform bonds. Swiss equities have provided a fairly stable 5% real rate of return when held for periods of ten years or more, in comparison with typical bond returns of less than 3%. 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, it is expected that money invested in bonds is safer and more likely to be available when needed than would the same amount invested in bonds. We can explore this hypothesis by examining the volatility these same instruments, which we define as the standard deviation of the rates of return. The results are shown below.

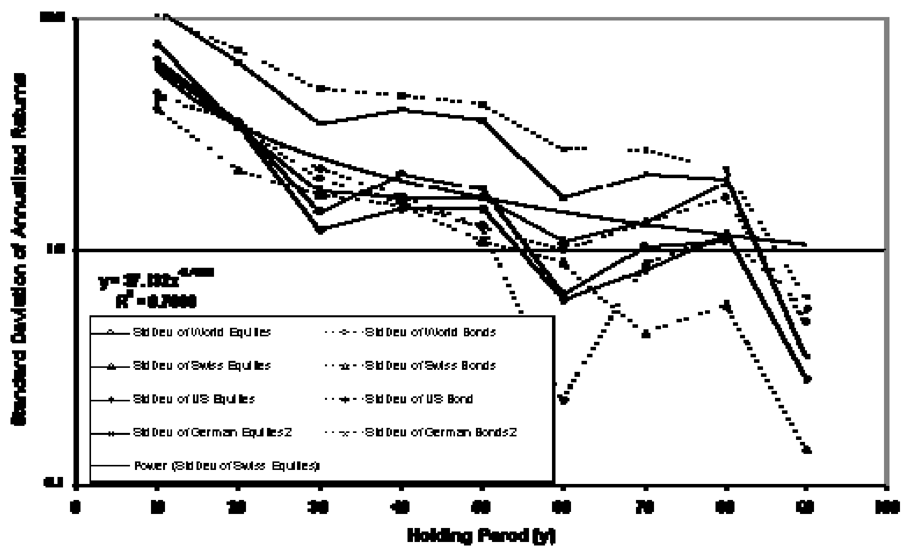


Figure 5.9: Real Returns to Equities and Bonds: Standard Deviation of Return as a function of Holding Period (adapted from Dimson et al. 2002)

The impression that bonds are much safer than equities does not appear to be valid when inflation is taken into account by examining the volatility of real rather than nominal rates of return. 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 tradeoff between yield and volatility in the choice of an investment interest 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 upon the choice of a 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 is that it allows such tradeoffs to be made explicitly and 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% and a standard deviation given by the regression relation illustrated in Figure 5.9, namely, $\sigma = \tau^{-0.7889}$.

The comparison between the synthetic yields that we have generated and the observed performance of Swiss equities is shown below. Both because the uncertainty in yields can be quite large for short holding periods (less than 10 years[11]), we show both linear and log scales.

[11] This relation would not be expected to hold true for very short periods, as the range of potential returns starting at any given year would be constrained and would not be 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 necessars if this study were to be applied to short planning periods.

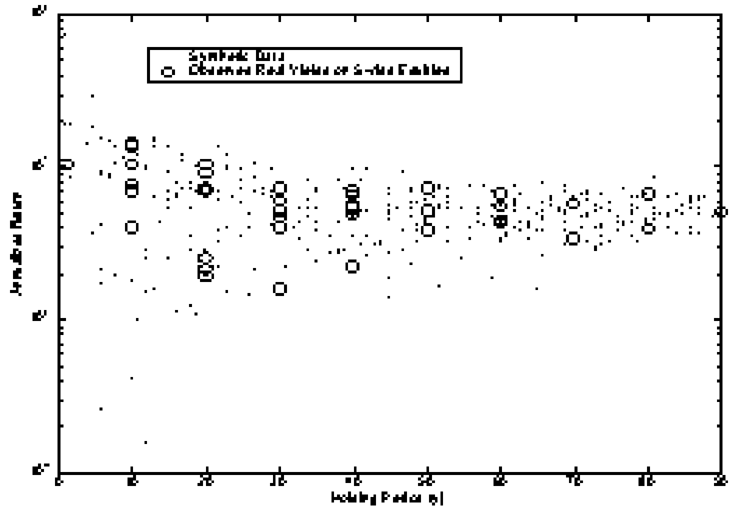


Figure 5.10a:

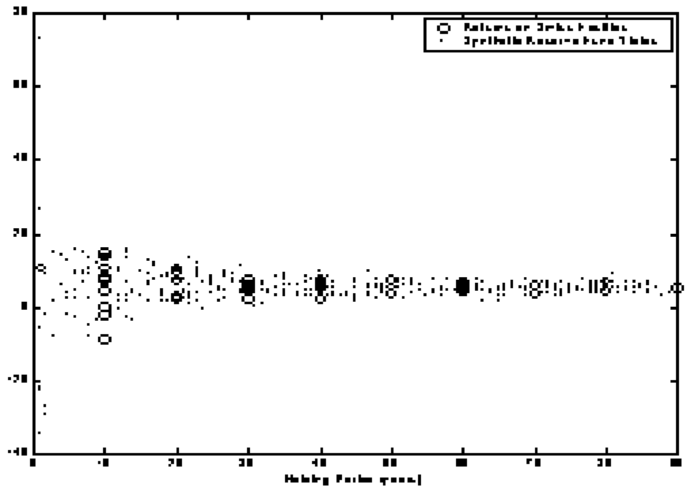


Figure 5.10b:

Figure 5.10: Comparison of Real Returns to Synthetic Equities to Historical Real Returns to Swiss Equities (adapted from Dimson et al. 2002)

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 fixed thirty years and assume that the average loan interest rate of 4% (real), and allow the interest rate to be an uncertain random variable that can range between 2%-6% real at a 2σ confidence level. We note that these are unfavorable terms, albeit not unreasonably so. An agent of the Austrian government, given the good credit standing and alternate financial resources, would probably not be required to pay such rates nor would have 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 shall 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 the 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 considers essentially the pre-1990 condition. It was presumed that the detention basins are in place, but no measures are taken to allow operability. The floodbasins fill and empty passively. The masonry floodwall is also assumed to be in place. It is assumed that if damages occur, the losses will be covered by a loan. As discussed in the previous chapter, the loan is assumed to be a thirty-year loan with a real interest rate between 2-6%. 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 (see figure below). The effect of these flood barriers is to limit inundation of downstream reaches. Because these systems can be expected to have a reliability of less than 100%, it is assumed that these systems have a failure-on-demand rate of 25%. In other words, these systems are assumed to fail only once in every four event requiring their installation. Because 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 E100,000 and costs E10,000 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 E50,000. Annual operating costs. (inspection, testing, and occasional repair) are assumed to be 10% of installation costs. These costs can be specified by the user. Alternative 2 comprises upgrading of the basins to allow controlled filling and release of floodwaters. The system is discussed in more detail in Faber (2003). This system, coupled with a real-time flood forecasting system, is currently being 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 E8 million and operating costs that are expected to be 1% of installation costs. To emphasize the fact that this is an illustrative example, we have rounded this value up to E10 million. The combined scenario represents the combination of portable flood barriers and detention basin upgrades. The results of these scenarios are shown below.

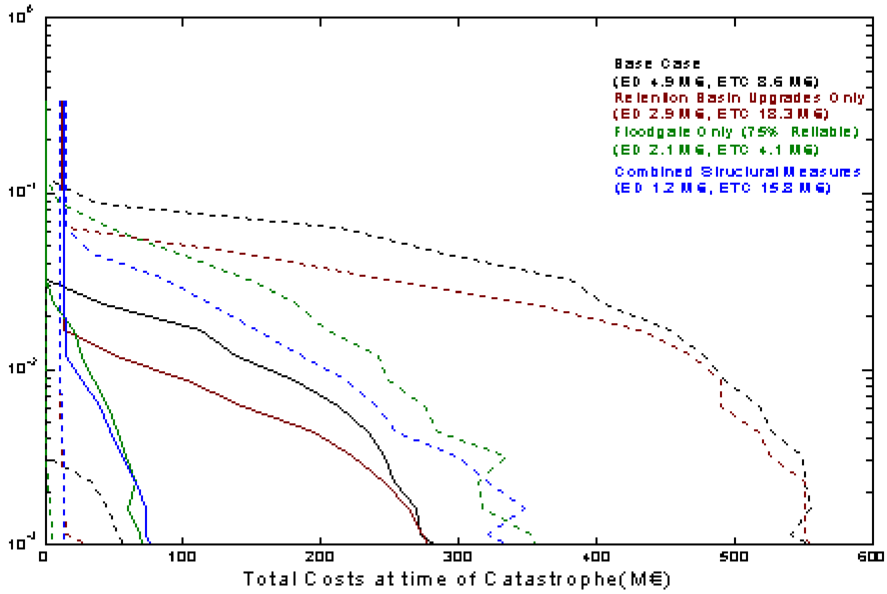


Figure 6.1: Examination of Structural Alternatives

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

We once again note that this is not a realistic scenario for the city of Vienna. The 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 was required, it is not expected that the interest rate would be as high as that assumed here (particularly if it was covered by a bond issue) or that the term would have 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 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 damages will start to occur. The probability of damage exceeding zero is about 3%, unchanged from the base case. What does change are the damages at lower probabilities. The probability that the damages are limited to less than 50 ME are lowered to approximately 0.5%, and the chance of damages exceeding 100 ME are considerably less than 0.1%. The expected damage from this case is approximately 2 ME, with expected total costs of 4.1 ME. 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 damages and may be able to limit damages to an "affordable" level.

Examination of Alternative 2 shows that, as intended, the upgraded detention basins lower the probability at which damages will start to occur. The expected probability of damage exceeding 0 drops from 3% to slightly over 1.5%. 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% chance that total costs exceed 10 ME, and there is a 1% chance that total costs will exceed 100 ME. The expected damages are reduced from 5 to 3 ME, but the expected total costs increase from 8.6 to 18 ME. 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 affects primarily 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 incurring damages. If a decision-maker is unconcerned with potential damages 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 including some of the drawbacks as well. Damages are limited by the floodgate and their likelihood is reduced by the detention basins. In addition, the uncertainty surrounding the losses is decreased. The expected damages are reduced to 1.2 ME, with a very low probability that the damages will exceed 100 ME. The expected total costs are approximately 16 ME

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. We set up a hypothetical insurance policy here. The insurance policy variables are all decision variables, so there is no basis for selecting any particular set of combinations without knowing the decision-makers preference. In this case, we choose to have a €10 million deductible, a 10% coinsurance rate, a €500 million claims cap, and a premium loading factor of 100% (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). For purposes of comparison, a 1000% premium loading factor is also shown (reflecting a premium set to be equal to ten times the expected claims payment, illustrative of a highly risk-averse or poorly diversified insurer).

The second financial mechanism is that of a reserve fund. The structure is discussed previously. 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-makers 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 €10 million and an annual contribution of E0.1 million. We presume 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 the costs 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 10 M€ deductible, a 500 M€ cap, and a 20% coinsurance rate with a reserve fund comprising a one-time initial contribution of 1 million and an annual contribution of €10,000. The computed annual premiums are similar to those of the pure case at €150,000 (slightly lower due to the higher coinsurance rate), and the expected total costs are -2 ME, representing the possibility that a profit is expected on the basis of no flood occurring and the profit being taken from the interest accumulated over fifty years on the reserve fund.

The results of these simulations are shown in Figure 6.2. In the insurance only scenario, the expected damage is unchanged (as expected) from the base case, and premiums of 170K per year are computed using this premium loading factor. The expected total costs, including premium payment up until the time of the catastrophe, are 8 ME. With the higher (and probably more likely) premium loading factor of 1000%, the premiums are close to a million euro per year and the expected total costs are therefore quite high, at 34 ME. However, it can be seen that insurance has an effect remarkably similar (from a purely financial perspective) to that of the 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 the floodgate, 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 doesn't happen, and the costs are sensitively dependent upon the premium loading factor. Another significant factor, that 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 they started with no benefit from the policy and no future protection.

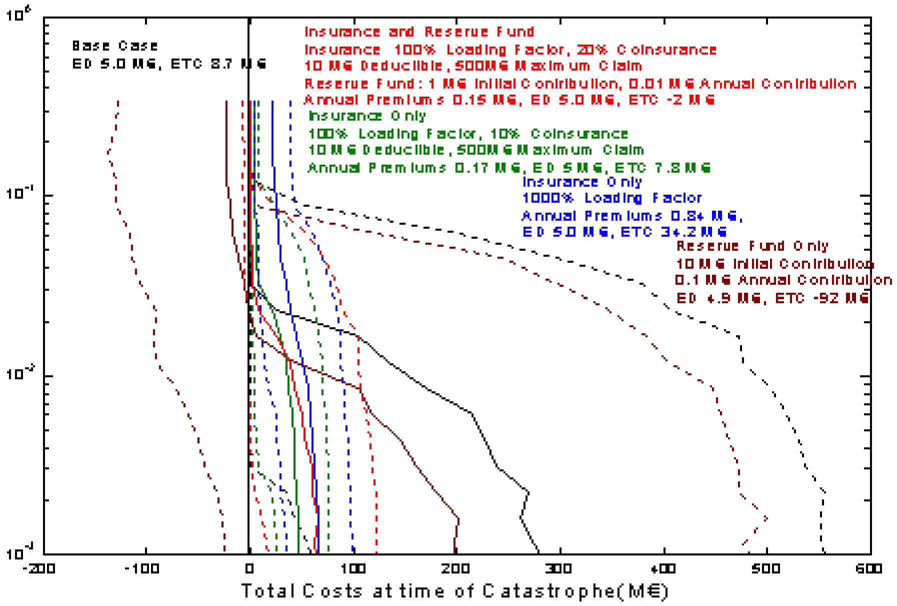


Figure 6.2: Financial Measures

The reserve fund reveals a somewhat startling feature in comparison to the other alternatives. It is clear that it does nothing to reduce damages (in common with all financial measures). What it does do is shift the loss curve to the extent that damages can be compensated from accumulated funds. The fund also mitigates the effect of loan costs to the extent 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%. However, costs can still be quite high, with a 0.8% likelihood of costs exceeding 100 ME. On the other hand, there is a major chance that the flood will not happen and that the interest on the reserve fund can eventually 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 of interest. 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 adequate 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 avert 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 their 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, it can even increase them (albeit often in a positive direction). Finally, a drawback that is not illustrated by this plot is 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. Because a long time period of interest was chosen, there is a significant chance of accruing large balances 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 combines 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 so 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 can help to offset the risk that the insurer will 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 combined structural measures with both financial measures singly and in full combination. This comprised a scenario combining structural measures with insurance, 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% premium loading factor.

For the scenario of structural+insurance, the computed premia are only 30K (provided, of course, that an insurer would be willing to offer insurance at that rate). The installation of the floodgate and the upgrading of the detention basins has lowered the expected claims, allowing lowered insurance premiums. Expected damages are 1.1 M€ per year, with expected total costs of 16 M€. 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 savings in loan servicing costs of exceptionally large loans offsetting the cost of insurance. The real interest lies in the fully combined scenario. The expected damages are now 1.1 M€, leading to annual premia of 30K€. The expected total costs are 3.5 M€, down from approximately 8.6 M€ in the base case. Potential total costs are limited to well under 50

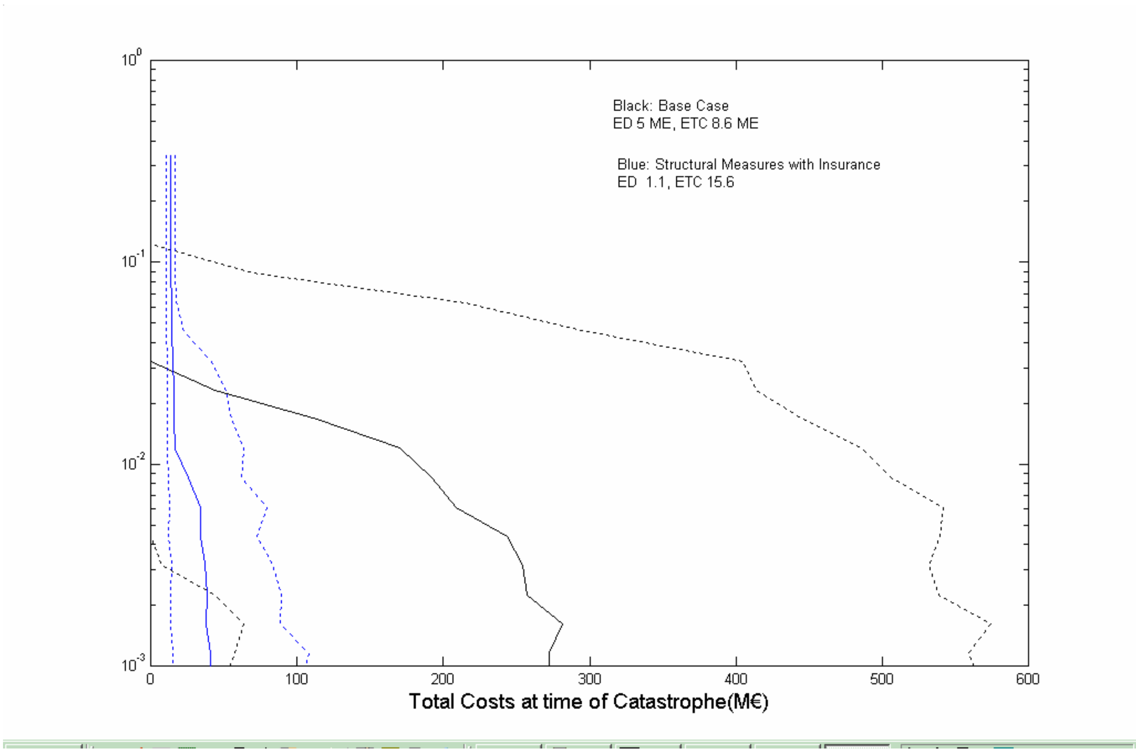


Figure 6.3a*:*

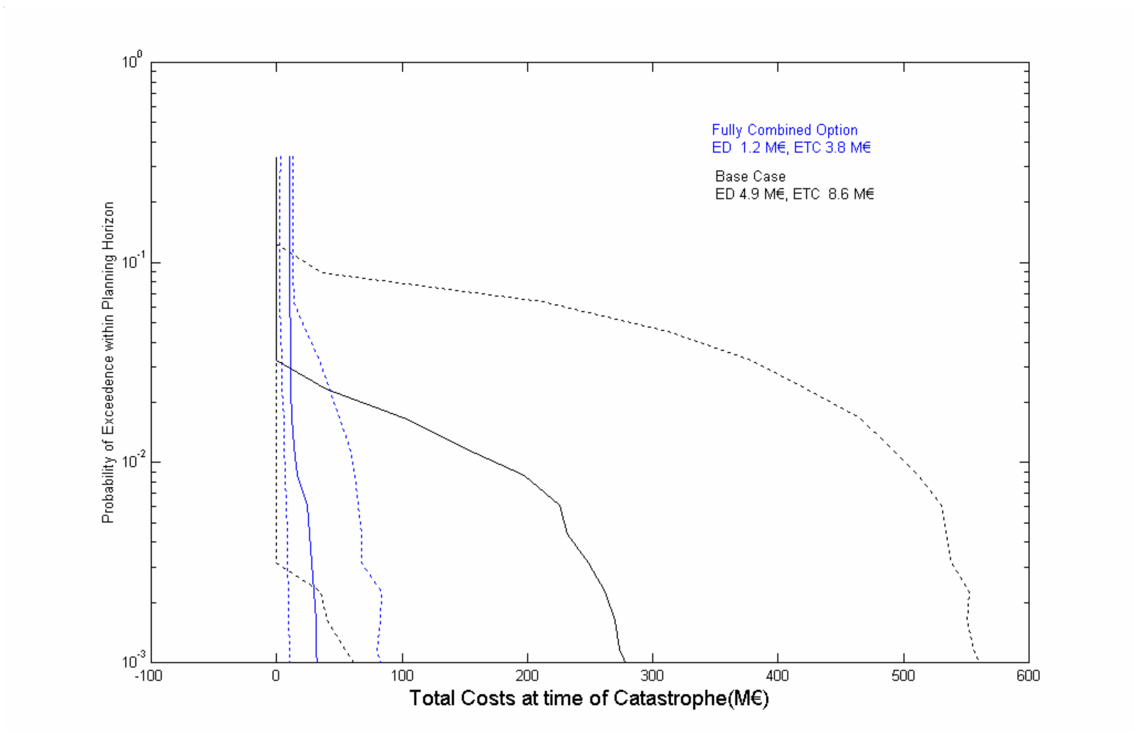


Figure 6.3b*:*

Figure 6.3: Mixed Scenarios

M€, and even considering uncertainty, are not expected to rise above 100 M€ 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 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 premia are modest. Inclusion of a sensitivity analysis shows that even if the premium loading factor is increased to 1000% (premiums = 10x expected claims), the premia are still only 160 KE and the total costs are approximately the same as the no action alternative, with the potential losses still 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 structuring of the problem in such a way as to provide 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 an actuarial approach and a 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 in terms of 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 the probability of system failure), we note that consequences matter. A failure that results in only minor damages or a financial option that results in only a minor financing gap is significantly different than one which results in catastrophic damages or an uncloseable financing gap, even if that failure or the 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 towards the reduction of epistemic uncertainty. For example, a decision maker that 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 their goals and constraints and not allow the analytical tools of the component disciplines to define the problem for them.

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 with 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 other measures, thereby reducing and controlling the risks. For example, the explicit treatment of epistemic and aleatory uncertainty allowed a clarification of the different characteristics of reserve funds vs 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 was subject to considerably 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. Also, 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 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 by combining structural measures with financial measures.

Several methodological issues arose during the course of the study. One is 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 eventually adopted had been used at the outset[12]. This is due in large part to the different approaches in conceptualizing the risk analysis problem in the contributing disciplines. It is incumbent on the analysts in such studies to understand the assumptions, limitations, and data requirements of the interfacing disciplines sufficiently that they may 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 occur. Another issue that arose late in the study is that there are 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 is considerably more complex to implement in probabilistic terms, when the a “cost” can be negative. Finally, we note that we have approached the treatment of epistemic uncertainty in financial parameters from a very empirical, atheoretical, engineering-oriented perspective, as the background of the primary authors is largely an engineering background. Our approach to uncertainty was

[12] 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 analytical approach before data is collected and simulations performed, or else the study wil 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.

quite consistent with what Renn has observed as the dominant technical paradigm of using "relative frequencies (observed or modeled) as a means to specify 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 which suggest areas where considerable improvement could be made in the approach presented herein. Although there are certainly many areas for improvement, it is the authors' opinion that the two major technical limitations of this study include 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[13], due to 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 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 of this study would be an examination of alternate methods for discounting future losses from catastrophic events. Also, 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 in 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 more complex and unique 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*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. 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 prices of neglecting the social processing of risk."

[13] On the other hand, it was realized that if the losses are associated with replacement of items with a value that depreciates due to wear and obsolence, 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.

We believe that these criticisms are well taken, but 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 to 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 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, because quantification is a fundamental aspect of simulation modeling, these concerns can probably not 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, at least to the extent to which the concerns of different stakeholders can be quantified, the virtue of exercises such as these is that they allow the impact of different potential goals and constraints to be examined systematically. 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 being 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. Furthermore, the optimization techniques being explored by Ermoliev et al. (2000) and Ermolieva et al. (2001) may allow the use of integrated models in a close to realtime environment during meetings and negotiations. Evaluation of the characteristics of alternative financial instruments are being pursued by Mechler and Pflug (2002). It is hoped that this study can 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.

8. References

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