GROUNDWATER CONTAMINATION FROM WASTE-MANAGEMENT SITES: THE INTERACTION BETWEEN RISK-BASED ENGINEERING DESIGN AND REGULATORY POLICY

By

JOEL WARREN MASSMANN

B.S.C.E., The Ohio State University, 1980
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Department of GEOLOGICAL SCIENCES

The University of British Columbia
1956 Main Mall
Vancouver, Canada
V6T 1Y3

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ABSTRACT

This dissertation puts in place a risk-cost-benefit analysis for waste management facilities that explicitly recognizes the adversarial relationship that exists in a regulated market economy between the owner-operator of the facility and the government regulatory agency under whose terms the facility must be licensed. The risk-cost-benefit analysis is set up from the perspective of the owner-operator. It can be used directly by the owner-operator to assess alternative design strategies. It can also be used by the regulatory agency to assess alternative regulatory policies, but only in an indirect manner, by examining the response of an owner-operator to the stimuli of various policies. The objective function is written in terms of a discounted stream of benefits, costs, and risks over an engineering time horizon. Benefits are in terms of revenues for services provided; costs are those of construction and operation of the facility. Risk is defined as the expected cost associated with failure, with failure defined as a groundwater contamination event that violates the licensing requirements set forth by the regulatory agency. Failure requires a breach of the containment structure and contaminant migration through the hydrogeological environment to a compliance surface. Reliability theory is used to estimate the probability of breaching and Monte Carlo finite-element simulations are used to simulate advective contaminant transport. The hydraulic conductivity values in the hydrogeological environment are defined stochastically. The probability of failure is reduced by the presence of a monitoring network established by the owner-operator. The level of reduction in the probability of failure can be calculated from the stochastic contaminant transport simulations. While the framework is quite general, the development in this dissertation is specifically suited for a landfill in which the primary design feature is one or more synthetic liners and in which contamination is brought about by the release of a single, nonreactive species in an advective, steady-state, horizontal flow field. The risk cost benefit analysis is applied to 1) an assessment of the relative worth of alternative containment-construction activities, site-investigation activities, and monitoring activities available to the owner-operator, 2) an assessment of alternative policy options available to the regulatory agency, and 3) two case histories. Sensitivity analyses designed to address the first issue show that the allocation of resources by the owner-operator is sensitive to the stochastic parameters that describe the hydraulic conductivity field at a site. For the cases analyzed, the installation of a dense monitoring network is of less value to the owner-operator than a more conservative containment design. Sensitivity analyses designed to address the second issue suggest that from a regulatory perspective, design standards should be more effective than performance standards in reducing risk, and design specifications on the containment structure should be more effective than those on the monitoring network. Performance bonds posted before construction have a greater potential to influence design than prospective penalties to be imposed at the time of failure. Siting on low-conductivity deposits is a more effective method of risk reduction than any form of regulatory influence. Results of the case histories indicate that the methodology can be successfully applied at field sites, and that the risks associated with groundwater contamination may be small when compared to the owner-operators' benefits and costs.

Graduate Supervisor: R. Allan Freeze
Department of Geological Sciences

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1. INTRODUCTION

In recent years, problems associated with groundwater contamination have received much attention from the public, from the media, and from many segments of our local, state, and federal governments. Particular scrutiny has been focused on contamination from landfills, impoundments, storage tanks, and other waste-management facilities, in part because they are usually located in urban areas and therefore the consequences of their contaminations can be especially grim, and in part because there is often an identifiable party responsible for these types of point sources.

As an outgrowth of these attentions, a flurry of legislative and regulatory activities dealing with groundwater contamination from waste-management facilities has been undertaken in the past decade [Office of Technology Assessment (OTA), 1984]. Two general approaches have been adopted in developing these regulations. The first approach is to develop regulations based upon available technologies. The second approach is to develop regulations based upon the management of risks. The approach based upon technologies, although relatively easy to administer, results in sometimes arbitrary and inflexible regulations that are always susceptible to obsolescence because of rapid technological developments. The approach based upon risk management, although more flexible, results in regulations that may be very difficult to administer. William D. Ruckelshaus, who twice served as administrator for the U.S. Environmental Protection Agency, from

1970-73 and from 1983-1985, discusses the two approaches (Ruckelshaus, 1986):

> "It's much more difficult to administer a law which mandates you to look at the environmental values that you're protecting based on the action that you take - as opposed to saying that if you have this kind of a discharge, you must put this kind of a technology on to reduce it. That's a simple thing to administer. But just because it's easy doesn't mean it's good public policy. And I think we sometimes sacrifice, for administrative ease, good sense. Justifying public or private expenditures on the basis of benefits received is a much tougher administrative assignment. Nevertheless, that's the assignment that we ought to give to people.
>
> "There's a lot of understanding among the people who have been in this arena for years of the need for adopting an approach that emphasizes the management of risks, that deals with them in their scientific reality, and that doesn't believe that by setting unrealistic goals and deadlines we're really going to accomplish very much."

In either case, whether technology-based or risk-based, the resulting regulations have become part of a complex, and often controversial, system of physical, economic, and social processes that include engineering designs, hydrogeological environments, free-market economies, and ethical and political decisions.

In this dissertation, a framework for assessing the interrelationships among the various aspects of this system is developed. The foundation for this framework is the contention that groundwater contamination from waste-management facilities takes place within an adversarial environment in which the objective of an owner-operator to maintain profitability may be in direct conflict with the objectives of a regulatory agency established to address the safety and environmental concerns of society. Although the approach used is generally applicable to a variety of waste management scenarios, the present study is concerned with the design, operation, and regulation of new landfills in which the primary mechanism of failure involves a breach of containment across engineered barriers and the migration of contaminants through the hydrogeological environment. It is assumed that primary method of containment is synthetic membranes. The effects of leachate collection systems and other engineering activities are not included in the analysis of the containment structure. The assumptions are made that the siting process has been completed prior to our analysis and that the facilities will be placed in unconsolidated, permeable deposits so that the influence of advection will far outweigh that of dispersion, diffusion, and retardation. It is also assumed that flow occurs in a saturated, two-dimensional, horizontal aquifer. With these assumptions, the contaminant travel times can be estimated with some degree of confidence using computer models.

Two risk-cost-benefit objective functions can be developed: one for the assessment of alternative engineering designs by the owner-operator, and one for the assessment of alternative regulatory policies by the regulatory agency. The goal for each of the two participants is to maximize their respective objective functions by minimizing risks and costs while maximizing benefits. The risks for both parties are defined as the expected costs of failure, where failure is defined as the event a plume of contaminated groundwater reaches a compliance point or surface during a compliance period. The compliance points and periods are assumed to be specified by the regulatory agency. The costs for the owner-operator are construction and operation costs and the benefits are primarily revenues for services provided. The costs for the regulatory agency are principally administrative costs while the benefits are those associated with the preservation of clean water.

The merits of alternative design strategies can be directly assessed by examining how the owner-operator's objective function is influenced by various system designs. The design alternatives available to the owner-operator revolve around the possible trade-offs between: 1) exploration activities, 2) design features of the containment structure, 3) installation of monitoring networks, and 4) possible remedial actions.

In theory, the merits of alternative regulatory strategies can be assessed by examining how the regulatory agency's objective function is influenced by various regulatory policies. In practice, however, the cost and benefit terms in the regulatory agency's objective function are extremely difficult to quantify in economic units. An alternative approach for assessing regulatory strategies is to use the owner-operator's objective function to examine how owner-operators might respond to various regulatory stimuli. By carrying out such an exercise, an indirect comparison of the worth of various regulatory policy options can be provided. The policy alternatives available to the regulatory agency include 1) performance standards and/or design standards, 2) compliance locations and monitoring requirements, 3) penalties for violation of licensing provisions, 4) litigation procedures, and 5) procedures for remedial actions.

The development of the owner-operator's objective function requires an explicit quantification of both the consequences of failure and the probabilities of failure for each of the design or regulatory strategies under consideration. The techniques that are used to estimate the failure probabilities incorporate: a) the application of reliability theory for modeling the performance of engineered barriers, b) a geostatistical description of the hydrogeological environment, c) numerical simulation of advective contaminant migration paths in the hydrogeological environment, d) a stochastic interpretation of predictive uncertainties from the numerical simulations, and e) a Bayesian approach to updating estimates on the basis of additional data.

The remainder of this chapter is devoted to a general discussion of the approach used to assess alternative design and regulatory strategies. Chapter 2 describes techniques for selecting the best design or regulatory strategy from a group of alternatives. The development of the owner-operator's and regulatory agency's objective functions is described in Chapter 3. The techniques for estimating failure probabilities are presented in Chapters 4 through 6. Chapter 7 presents results from sensitivity studies for a hypothetical landfill and Chapter 8 describes two case studies. Conclusions are summarized in Chapter 9.

1.1 Emphasis on Design Rather than Remedial Issues

This dissertation differs from other recent studies that have addressed regulatory issues associated with groundwater contamination [Raucher, 1983, 1984; Sharefkin, Schecter, and Kneese, 1984; Schecter, 1985] in that it emphasizes issues associated with the design of new facilities rather than those associated with remedial actions at facilities that have already failed.

There will be no letup in the need for new facilities. OTA [1984] recently updated the 1977 EPA estimates of the annual amounts of waste generated in the United States alone. They estimated an inflow of 0.81 billion gallons per year of industrial waste to chemical landfills, between 180 and 280 million tons per year of nonhazardous industrial and municipal waste to sanitary landfills, and 3.7 million cubic yards per year of low- level radioactive waste.

There is considerable evidence in the literature concerning remedial action to suggest that aquifer restoration or contaminant containment is very difficult and expensive. Raucher [1984] concluded that restoration will be supported by cost/benefit analysis in very few cases, and he presented two cases where it was not. In addition, restoration is not very successful. In the case- history survey carried out by Burman et al. [in press], cleanup was successful in only 16% of the cases studied, and it was totally ineffective in 46% of the cases. In view of these lines of evidence, it appears that improved design of facilities is particularly important. It is in the societal interest to prevent contamination events rather than to try to cope with them after they occur.

1.2 The Adversarial Environment

A second difference between this analysis and other recent work lies in the explicit recognition of the adversarial relationship that exists in a regulated free-enterprise economy between the owner-operator of a waste-management facility and the governmental regulatory agency under whose terms the facility must be licensed. "Adversarial" does not necessarily mean "combative" but simply that the objectives of each party are different and may, in some sense, be in conflict. The basic argument for an adversarial treatment is as follows:

1. Waste management in North America takes place within a mixed free-market, welfare-state economy.
2. Waste-management facilities are usually operated by private entrepreneurs as part of the free-market economy. The objective of the owner-operator of a waste-management facility is to provide a necessary service at a profit. He needs to achieve an acceptable long-term rate of return on his investment.
3. The health and safety and the aesthetic desires of the public are protected by the government as part of the welfare economy through establishment of regulatory agencies governed by legislation. The objective of the regulatory agency is to set regulatory policy and to put in place licensing, monitoring, and enforcement procedures that will reduce the number of failures of waste-management facilities to a level at which the consequences

to society are politically acceptable.

4. Waste-management facilities are designed by engineering firms hired by owner-operators. Design engineers must design the facilities under the direction of the owner-operator such that (a) the operation of the facility will lead to an acceptable rate of return for the owner-operator under current and future economic conditions and (b) the facility will meet all licensing and regulatory criteria.

5. Regulatory policies are designed by government agencies established by elected legislators. Regulatory officials must design policies under the direction of legislators such that (a) they protect the health and safety of the public and fulfill its aesthetic desires and (b) they allow for the existence of a healthy economic climate in the waste-management industry and in the industries that generate the waste. The first of these objectives is widely recognized; the second is often forgotten. As pointed out by Rothermal [1983], the only alternatives to a profitable waste-management industry are (a) curtailment of production of the goods that produce the waste with attendant loss of the benefits that occur from the goods or (b) illegal waste disposal with its greater attendant costs to society.

Two points should be made with regard to the arguments presented above. First, it is recognized that some waste-management facilities are not run as part of the free-market economy. However, the owners of these facilities are usually municipal governments, and the regulatory agencies are usually set up by state or federal regulation. For such facilities, the same dichotomy of objectives and the same adversarial framework apparently will exist. Even for high-level nuclear waste disposal in the United States, the development of repositories is in the hands of one federal agency and regulation is in the hands of another. For all such facilities, there seems little loss in generality in treating the owner-operators as free-market firms and the regulatory bodies as government agencies.

The second point concerns the role of the design engineer in protecting public safety. In the present study, this entire responsibility is placed in the hands of the regulatory agency and it is assumed that design engineers will not concern themselves with this issue if an adequate regulatory system is in place. It is believed that this is an accurate reflection of exiting practice, but this is in no way meant to disparage the social consciences of design engineers. Engineers function under a code of ethics. In the absence of regulations, the design engineer would presumably prepare designs for a waste-management facility that were in keeping with his interpretation of the code of ethics. In the present study, however, it is assumed that the regulations in place are considered adequate and that the design engineer will feel he has satisfied his ethical obligations if he meets the regulatory requirements.

1.3 Approaches to Engineering Design

The approach adopted in this dissertation for examining regulatory and design strategies is to use an objective function which compares the net present value of future benefits, costs, and risks for each design and regulatory strategy under consideration. In order to calculate the risk terms in the objective function, it is necessary to estimate probabilities of failure for the waste management facilities. Geotechnical and hydrogeological engineers have not traditionally used this approach to design.

In the past, design engineers working on geotechnical problems have tended to view themselves foremost as protectors of the welfare and safety of the public [Baecher et al, 1980]. Evidence of this viewpoint includes the first item in the list of "Fundamental Canons" in the ASCE Code of Ethics, which states that "Engineers shall hold paramount the safety, health, and welfare of the public in the performance of their professional duties" [Firmage, 1980]. W. W. Moore, co-founder of the geotechnical consulting firm of Dames and Moore, supports this approach to design when he states that "Consulting engineers have the responsibility of providing their clients with engineered products, whether they are designs and publications or reports containing recommendations, that will meet the clients' needs and protect the health, safety, and welfare of the general public" [American Council of Engineering Consultants, 1982].

One of the principal design tools used in this "protector-of-the-public" approach is the safety factor, which is usually defined as a ratio of some type of capacity or strength measure to some type of demand or load measure. Appropriate values for these safety factors are based upon precedence and "accepted engineering standards" as described in the ASCE's "Guidelines to Practice" [Firmage, 1980]. The actual values used depend upon the type of problem being analyzed, the type of materials that are used, the type of loadings expected, the consequences of unsatisfactory performance, and company or consulting firm policies.

As examples, acceptable factors of safety might range from 1.3 for slope stability problems to 3.0 for foundation bearing capacity analyses, with factors for retaining walls somewhere between the two [Harr, 1977]. All three problems essentially rely upon predictions of stresses in soil masses, yet the safety factors are quite different. The dependence of the safety factors on material properties is evidenced by the practice of using safety factors of 1.1 for the slope stability of rockfill embankments and 1.5 for earth embankments [de Mello, 1977]. Dependence on the consequences of poor performance is indicated by the design of upstream embankment slopes for dams with lower factors of safety than downstream slopes.

Some of the advantages of using safety factors as an approach to design are that they fit comfortably within the deterministic analyses traditionally used by the geotechnical and civil engineering professions; they can be communicated among engineers relatively easily; and they are "hard" standards that a design either meets or does not meet. Perhaps more importantly, however, the safety factor approach does not require that the consequences of failure be explicitly defined, quantified, or even discussed. The reluctance to consider failures on the part of the design engineer who views himself as a protector of the public is perhaps understandable.

The danger of this approach is that designers may incur very high expenditures in design and construction to protect projects from unknown conditions. These high expenditures can cause inefficiencies both in the way resources are allocated among projects and in the way resources are allocated among activities within a single project. The first case, which can be termed an inter-project inefficiency, is due to the finite amount of resource that can be allocated to public projects. Monies spent on conservative designs in one sector of engineering activities must out of financial necessity result in less conservative designs in some other sectors. Projects designed by geotechnical and structural engineers appear to be more conservative than other engineering works. This conservatism is indicated by comparing the low risk of death due to failures of civil works such as dams, bridges, or buildings with the much higher risk of deaths due to failures in engineering designs associated with automobile transportation or industrial manufacturing [Baecher et al, 1980]. This imbalance in conservativeness is evidence that inter-project inefficiencies exist.

The second type of inefficiency, which can be termed intra-project inefficiency, is also due to finite resources; in this case the finite amount specified for a given project. For geotechnical engineering projects, the resources must be allocated among activities that can generally be classified as site investigation, design/construction, monitoring, and remedial action. Geotechnical engineers have traditionally made these allocations in a discrete sequence of events. The engineer first develops a site investigation strategy; next he designs and constructs the facility; he then plans a monitoring system to check the facility's performance; and lastly, he determines remedial actions to correct unsatisfactory performance [Peck, 1969].

Trade-offs exist between the levels of effort expended in each of these categories. As an extreme example, if a site investigation is not performed, the engineer must assume the most hostile situation exists and must design and construct the facility to be tolerant of these worst-case conditions. Alternatively, an elaborate site-investigation program can be implemented. If this investigation indicates conditions considerably better than the worst, savings can be realized in the design and construction stage. However, if the investigation shows that conditions are similar to the most hostile, the engineer has gained little from the site- investigation program. Trade-offs also exist between design and monitoring efforts. It can be argued that a conservative design requires less monitoring than a less-conservative design to achieve the same level of confidence in performance. The site investigation, design/construction, and monitoring activities should also depend upon the kinds of remedial actions that can be taken in the event that the performance of the facility proves unsatisfactory.

If an improvement in performance can be gained by shifting resources from one of these activities to another, then an intra-project inefficiency exists in the current allocation. This study will concentrate on these intra-project inefficiencies in the context of the design of waste-management facilities. An attempt is made to determine: 1) whether such inefficiencies exist, 2) if they do exist, whether they are large enough to be important, and 3) if they are important, how they might be eliminated.

In order to fulfill these goals, it is first necessary to re-evaluate the role of the design engineer. Baecher et al [1980] contend that a better role than that of protector of the public safety would be that of a balancer of risks of failure against the costs of reducing these risks. The risks are functions of the probabilities of failure and the consequences of failure. The risk balancing role therefore requires 1) an explicit acceptance of the possibility of failure, 2) the adoption of probabilistic analyses in lieu of the more traditional deterministic approaches, 3) the explicit incorporation of engineering economics into the design process, 4) an attempt to quantify the uncertainties inherent in engineering analysis, and 5) an attempt to quantify the consequences of failure in terms of economic and life losses.

It should be noted that there are many approaches to design that can be taken by geotechnical engineers other than the "protector-of- the-public" and the "risk-balancer" philosophies presented above. Many, perhaps most, approaches can best be described as a hybrid of the safety factor and risk balancing techniques.

An example of such a hybrid is presented in the Uniform Building Code [1982] used by structural engineers. Factors of safety are still used, but the loads and strengths are multiplied by weights that depend upon uncertainties and consequences of failure. For example, if the structure is classified as essential, such as a hospital or a fire station, the expected demand load for earthquakes is multiplied by 1.5 before it is entered into the safety factor. If the structure is non-essential but will be occupied by more than 300 people, the demand load is multiplied by 1.15. For all other structures, the safety factor is determined directly from demand loads. With this technique, measures related to the consequences of failure can be directly incorporated into the safety factors. Although this type of approach can be applied in geotechnical design, it does not appear to be useful for studying the problem of intra-project inefficiencies.

Another approach that is increasing in popularity and that might be described as hybrid views safety factors as random variables rather than deterministic quantities. The probability of failure is simply defined as the probability that the safety factor is less than one. The problem with this approach is that traditional safety factors are nominal or conditional values rather than absolute values [de Mello, 1977]. They depend upon or are conditioned by the mode of failure that is assumed, the model that is applied, the material properties that are used, and the solution technique employed. The probability that the factor of safety is less than one is therefore not the absolute probability of failure, but is rather the probability of failure due to the mechanism that is modeled, given that the materials behave as assumed. Modeling the safety factor as probabilistic is an improvement, but some applications in the past have tended to over-simplify problems, have attached undue confidences to the probabilities, and have attempted to solve problems not amenable to this type of analysis.

2. TECHNIQUES FOR SELECTING DESIGN AND REGULATORY STRATEGIES

The overall objectives of the analyses described in this dissertation are 1) to compare alternative design strategies available to owners and operators of waste management facilities and 2) to compare alternative regulatory strategies available to agencies established to address safety and environmental concerns of society. In this chapter, a decision analysis framework is constructed for comparing and selecting alternatives. The overall decision structure is developed in Section 2.1. The values used to measure uncertainty and consequences are described in Sections 2.2 and 2.3. Various criteria for making decisions are presented in Section 2.4. Section 2.5 describes techniques to predict the value of perfect and imperfect information.

2.1 Decision Analysis

Formal decision analysis as applied to engineering design is defined as a framework for selecting the best alternative from a set of alternative designs for an engineering system. A less formal and perhaps more descriptive definition is given by Keeney [1984]: "decision analysis is a formalization of common sense for decision problems which are too complex for informal common sense." The intent of the approach is to break down complicated decision problems into smaller, and therefore simpler parts whose separate analyses can be combined to provide a solution to the whole problem. The methodology is described in many texts, including Lindley [1971], de Neufville and Stafford [1971], and Raiffa and Schlaifer [1970].

A decision problem exists whenever there is a choice between at least two alternative courses of action that result in different consequences. Decisions can be described with four components: decision variables, state variables, consequences, and constraints. Decision variables define the list of possible alternatives available to the decision maker; consequences are the final result of the decision. It is often the case that a number of possible consequences may result for each alternative or decision variable that is selected, and the actual consequence that occurs depends upon variables that are beyond the control of the decision maker. These variables, which are often uncertain, are termed state variables. In addition, many decision problems are constrained in that certain alternatives are unacceptable since they may result in consequences that are unacceptable no matter how unlikely their occurrence.

The first step in any decision problem is to draw up lists of decision variables and state variables that are exclusive and exhaustive. Exclusiveness demands that only one variable from each list occur. This means that only one decision variable can be selected and that the variable describing the state of nature can take on only one value. Exhaustiveness demands that all possible variables be included in the lists. Exclusiveness can usually be obtained by carefully defining the decision and state variables. Exhaustiveness, which is perhaps the most crucial requirement in the entire decision analysis, is often impossible to insure.

Once the lists of variables have been made, the second step in the analysis is to identify the consequences associated with each pair of decision and state variables. These consequences should be expressed in units that are commensurable with one another insofar as practicable. The final step in the decision is to identify a criterion for selecting the most desirable decision variable from the list of alternatives.

The relationships between decision variables, state variables, and consequences can be illustrated using either decision trees or decision matrices. An example decision matrix is presented in Figure 2.1. The decision variables are listed along the rows and the state variables are listed along the columns. The

Decision Variables	State Variables			
	S_1	S_2	S_3	S_4
a_1	C_{11}	C_{12}	C_{13}	C_{14}
a_2	C_{21}	C_{22}	C_{23}	C_{24}
a_3	C_{31}	C_{32}	C_{33}	C_{34}

Figure 2.1 - Example Decision Matrix

consequences are listed at the intersections. Constraints can be represented by cross-hatching unacceptable consequences.

For problems that involve sequential or multiple decisions, decision matrices can become cumbersome. An alternative approach for illustrating the relationships between decision variables, state variables, and consequences is to use a decision tree, as shown in Figure 2.2. Decision trees grow horizontally from the left to the right - the trunk is to the left and the branches are to the right. The points where branches split are termed nodes. There are two types of nodes: decision nodes and state nodes. At decision nodes, which are denoted as squares, the analyst makes the choice. At state nodes, which are denoted as circles, nature makes the choice. For sequential decisions the two types of nodes alternate from left to right. At the end or terminal of each branch are the consequences.

Although the decision tree presented in Figure 2.2 indicates that decision variables, state variables, and consequences are sets of discrete values, the approach is not limited to these situations. Decision problems with variables and consequences that are continuous can also be assessed. Rather than branches, the set of continuous parameters are denoted with fans, shown in Figure 2.3.

For analyses in which the state variables are known with certainty, decision making becomes an optimization problem, and techniques such as mathematical programming may be used to obtain

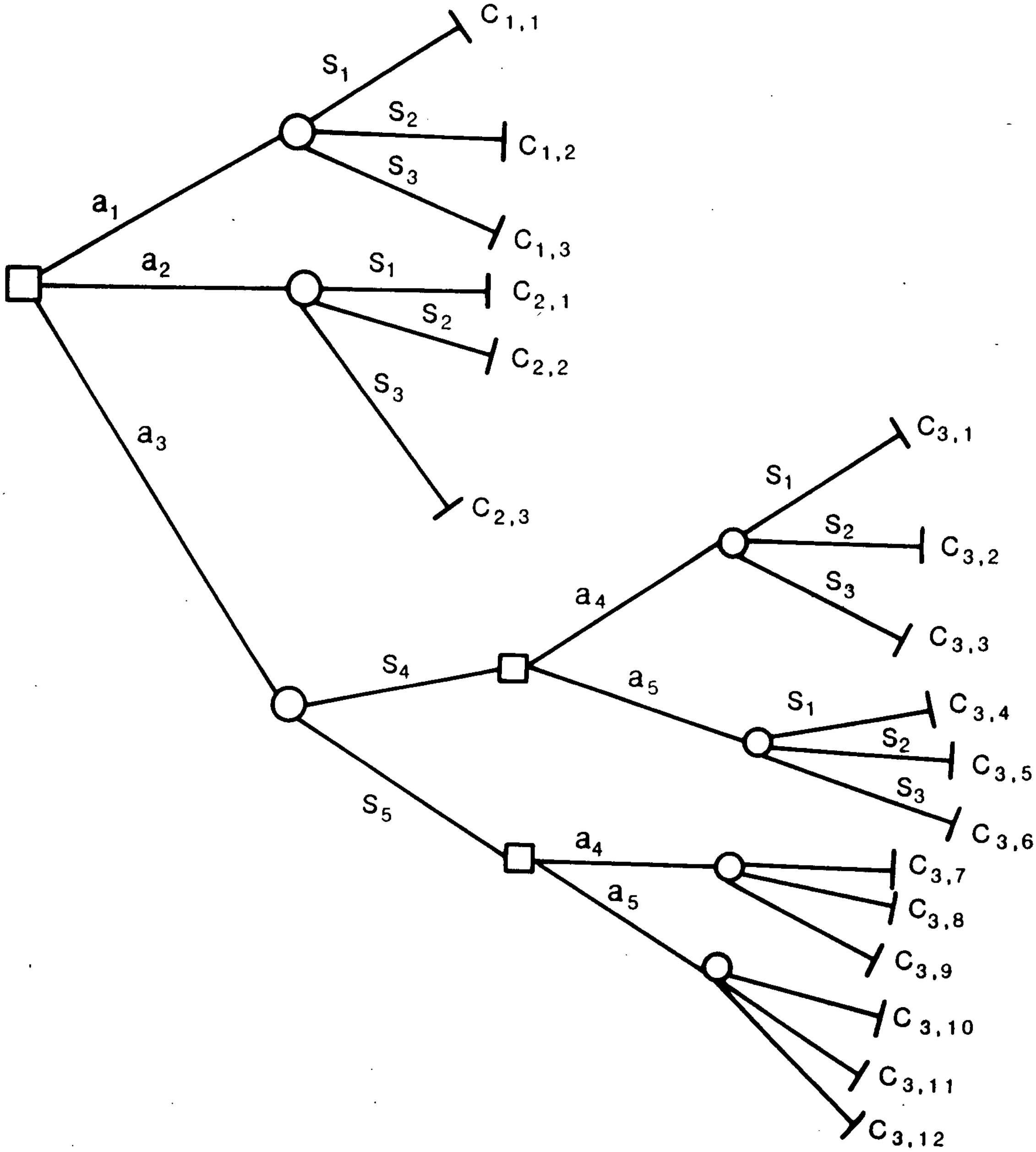


Figure 2.2 - Example Decision Tree for Discrete Variables

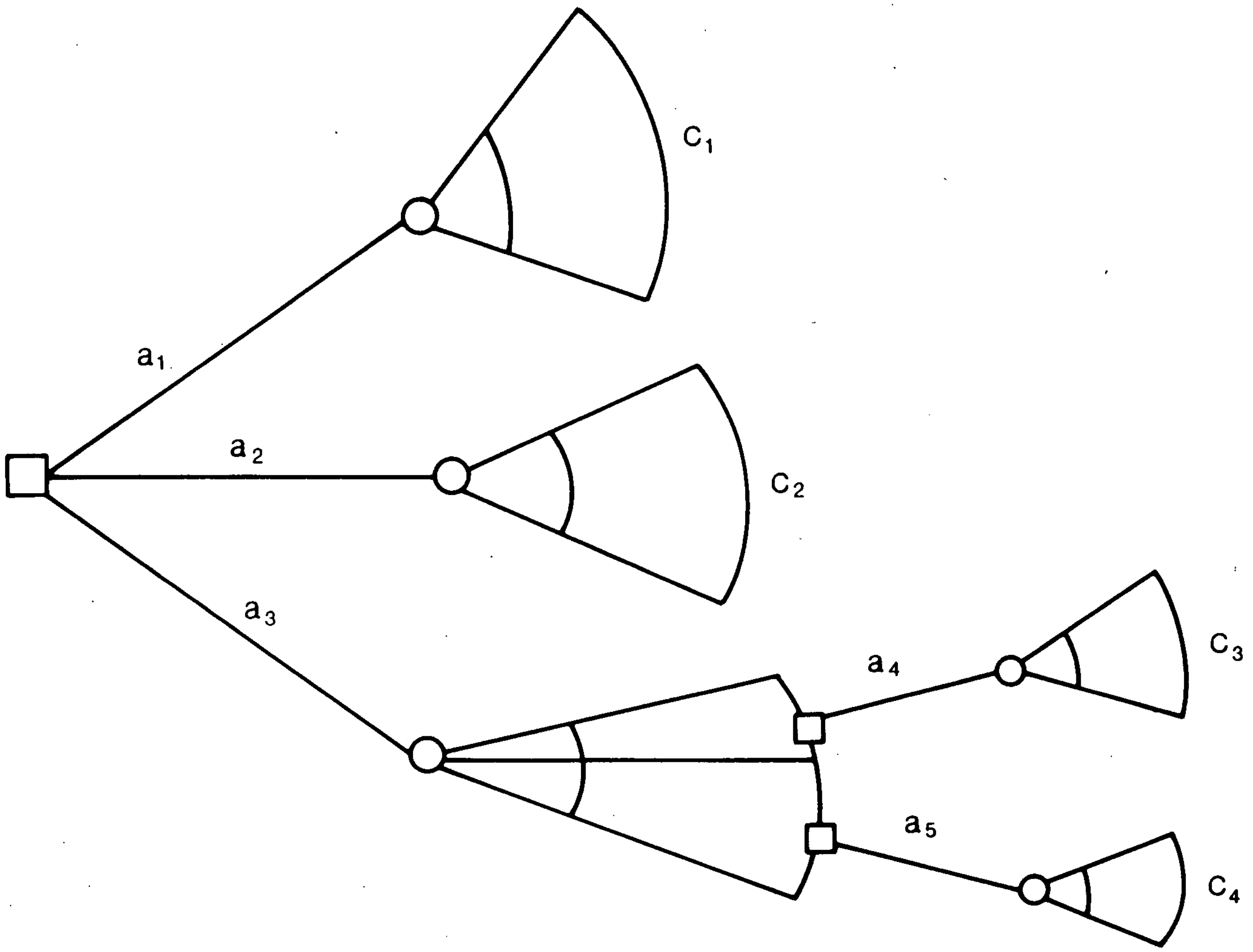


Figure 2.3 - Example Decision Tree for Continuous Variables

a solution [cf. Stark and Nicholls, 1972; Bryson and Ho, 1975]. Although such problems may be quite difficult to solve, they represent the limiting case of decisions with perfect information. Optimization techniques such as chance-constrained linear programming can also be used to assess decision problems with uncertainties, but these techniques are limited to very specialized cases. For the analyses of the waste management facility that is presented in this dissertation, the degree of precision offered by optimization techniques is not felt to be warranted, especially in light of the complexities and limitations associated with these techniques.

2.2 Measuring Uncertainty

The consequences that result when a decision variable is selected depend upon the state variables, which are often uncertain. To assess alternative actions, it is necessary to quantify the degree of uncertainty associated with these state variables by assigning probabilities to their occurrence. These probabilities, which are generally subjectively determined, present one of the major difficulties in assessing decisions.

2.2.1 Probability Interpretations

> -"The probable is what usually happens"
> Aristotle, c.a. 300 B.C.

Almost everyone is agreed on what the purely mathematical properties of probability are. Virtually all controversy centers on questions of interpreting the generally accepted concepts of probability, that is, of determining the extra-mathematical properties of probability. Although there appear to be as many interpretations as there are interpretors, it is convenient, or at least expedient, to follow L. J. Savage's [1954] convention of three views on the interpretation of probability:

1. Objectivistic view

Holders of the objectivistic view assert that probability only has meaning for independently repeated random events and that the magnitude of probability that applies can be obtained only by observating repetitions of the event. This view was held by the early developers of probability theory. Included among these are

Bernoulli in the 18th century, Gauss and Venn in the 19th century, and Fisher and von Mises in the early part of the 20th century [Good, 1954].

2. Necessary View

Those who hold the necessary view contend that probability measures the extent to which one set of propositions, out of logical necessity and apart from human opinion, confirms the truth of another set of propositions. This view essentially holds probability as an extension of logic. This interpretation has been developed and defended by Keynes [1921], Jeffreys [1939], and Carnap [1950].

3. Personalistic View

The personalistic interpretation of probability is that it is a measure of the confidence that a particular individual has in the truth of a particular proposition. This interpretation differs from the necessary view in that it assumes that two people faced with the same evidence may assign different confidence levels to the truth of the same proposition, and both may be "rational", "coherent", or "reasonable." Good [1950], Savage [1954], and de Finetti [1970] are defenders of the personalistic interpretation.

2.2.2 Probability and Geotechnical Decision Analysis

-"In the long run we shall all be dead."
J. M. Keynes, 1921

According to the objectivistic or frequentist point of view, probabilities can only be obtained by observing a "large number" of outcomes of independently repeated random trials. In a very strict sense, the planet Earth is the outcome of a single repeated trial and any aspect of its geology can therefore not be assigned an objective probability. More general views assert that the processes that developed the Earth's geology were random and independent and that many aspects of geologic deposits are therefore independent and repeated. Regardless of such philosophic, and perhaps esoteric, arguments, the holders of the objectivistic view contend that it is not reasonable to assign probabilities to the truth of propositions. This view therefore has little applicability for decision analysis.

The necessary and personalistic views are very similar, the only difference being the assumption in the necessary view that two individuals in the same situation, having the same tastes and supplied with the same information, will act in the same way. We will adopt the more general personalistic view and will assume that two such individuals may act differently, and still both act "reasonably."

The personalistic view may at first appear to be a very anarchistic, undisciplined approach. If everyone is allowed to choose their own probabilities, what will prevent these probabilities from degenerating into very qualitative and descriptive animals? The key to the personalistic view is that it demands that individuals behave reasonably, rationally, or coherently. Instead of using the descriptors reasonable or rational, which are plagued with a multitude of interpretations, coherency is used. Coherency will be given a very precise, mathematical definition in the next section. Let it suffice to say that coherency, with the laws of probability, will temper these potentially unruly personal probabilities.

2.2.3 The Basis of Personalistic Probability

The origin of the calculus of probability can be traced to Pascal, who in the 17th century solved the first mathematically non-trivial problems. The first book on the subject was published during the 17th century by Huygens [Good, 1959]. Nearly all of the laws and relationships developed during the following three centuries were based upon the objectivistic interpretation of probability. It wasn't until the 20th century that the personalistic view began to gain momentum. The dilemma that faced proponents of personal probability was that of proving that the laws of probability developed during the previous 300 years applied to their viewpoint as well as to the objectivistic viewpoint. Lack of such proof would have required a virtual re-invention of the probability wheel.

In 1954, L. J. Savage published "Foundations of Statistics," which helped to provide the needed proof. Savage developed and proved seven theorems needed to link the personalistic interpretation of probability to the calculus of probability developed under the objectivistic interpretation. His arguments essentially defend the assumption that there is only one form of uncertainty. Two important consequences of this assumption are comparability of events and coherency [Lindley, 1971].

Comparability of events requires that if E and F are any two uncertain events, then either a) E is more likely than F, b) F is more likely than E, or c) E and F are equally likely. Coherency requires that if G is a third uncertain event, and if E is more likely than F, and F more likely than G, then E is more likely than G. Comparability and coherency imply a unique value for the uncertainty of all events. In mathematical jargon, they guarantee the "existence" of personal probabilities. They also unlock the toolbox of probability calculus and allow the laws of probability to be applied to non-repeatable, uncertain events.

It is very difficult to prove or disprove comparability, and the notion is much discussed in philosophical circles. It is often argued that some events can have their probabilities quantified numerically and others cannot. The former group, termed statistical events, are events capable of infinite, or at least extensive, repetition. The latter group, called non-statistical events, are essentially unique. The decision maker is most often concerned with nonstatistical events. Although comparability cannot be explicitly proven, it is used in practice by such "professions" as bookmakers and insurers and will be accepted without further argument in this study.

The coherency requirement is much easier to prove. The argument is as follows [Lindley, 1971]. Assume a person feels E is more likely than F, F is more likely than G, and G is more likely than E. The person is not coherent in the mathematical sense. Further, assume the person is offered two of the events and is asked to "buy" the event he feels is most likely to occur. If the event he has chosen does in fact occur, the person is rewarded a prize. With such a system, the person would buy E if offered E and F. If he has E and is then offered G or E, he would buy G. Finally, when he has G and is offered F, he would buy F. To repeat the cycle, when he has F and is offered E, he would buy E. This cycle could be repeated indefinitely. The incoherent person is thus a perpetual money-making machine.

Savage's work has in no way quieted the arguments as to whether there is any room for personal probabilities in the very objective disciplines of mathematics, science, and engineering. It is often argued that "the notion of personalistic probability belongs to the field of psychology and has no place in applied statistics" [Jaynes, 1968]. The truth of the situation, however, is that if the tenets of personal probability are rejected, there is simply no way to incorporate prior information into probabilistic analyses except in the unique case where the prior information consists of frequency data. Attempts have been made to include non-statistical information in prior probabilities by using probability functions based upon maximum entropy formulations [Tribus, 1970; Jaynes, 1968], but these developments are limited to very specific random processes and to very specific types of prior information. Why throw away information, sometimes the only information available, because it cannot be expressed in a precise mathematical formulation?

As previously mentioned, if one accepts the assumption that there is only one form of uncertainty and accepts the resulting consequences of comparability and coherency, the laws and calculus of probability can be applied to the personalistic viewpoint of probability. These laws, though fully applicable, have somewhat different interpretations in the personalistic approach than they do in the objectivistic approach. These different interpretations are briefly discussed in the following section.

2.2.4 Personalistic Interpretations of the Laws of Probability

The most fundamental interpretation of personal probabilities is that they are quantifications of degrees of belief. These probabilities are necessarily conditional and in their most general form should be written as

$$Pr(e/H) = \text{Probability associated with event "e" given all conditions "H"} \tag{2.1}$$

The parameter "H" is the set of conditional descriptors. It includes all the information, biases, and prejudices which affect our degrees of belief. Different people will have different H descriptors and may therefore assign different probabilities to the same event. The probabilities will change as either the description of the uncertain event changes or as the conditions "H" change. For coherent people, the form of this change is governed by the laws of probability.

The complete probability calculus has as its foundation three simple laws:

1) If e and f are exclusive, uncertain events, then:

$$Pr(e \text{ or } f)=Pr(e)+Pr(f) \tag{2.2}$$

2) If e and f are uncertain events, then:

$$Pr(e \text{ and } f)=Pr(e)Pr(f/e) \tag{2.3}$$

3) If e and f are exclusive and exhaustive events, then the propability of any uncertain event g is:

$$Pr(g)=Pr(g/e)Pr(e)+Pr(g/f)Pr(f) \tag{2.4}$$

The first law states when degrees of belief can be added, the second law states when they can be multiplied. The third law, which is perhaps the most widely used, states that degrees of belief can be determined by breaking an event down into smaller parts that may be analyzed separately.

These three probability laws have two principal applications in the personalistic interpretation of probability. Firstly, they provide a means for checking the coherency of probability assessments. Secondly, they can be used to calculate coherent probabilities from those already available. These laws will be used throughout this dissertation, often implicitly, to calculate probabilities of random variables or events that are difficult to assess from random variables or events that are more naturally estimated by engineers or hydrogeologists.

2.2.5 Estimating Subjective Probabilities

The previous sections have discussed how the subjective interpretation of probability is the only interpretation that is meaningful when it comes to decision making. The underlying basis of this subjective or personalistic interpretation is that probabilities represent degrees of belief. In this section, two aspects of these degrees of belief are briefly discussed. The first part of the discussion describes some of the biases and difficulties that people have in formulating their own degrees of belief. The second part describes procedures used to extract or quantify these degrees of belief once a person has made them.

The process of formulating and extracting individual judgment about uncertain events is termed probability encoding. A fairly extensive literature exists describing both the way people formulate probabilities and the best ways for determining or eliciting these probabilities. Much of this literature has been developed by researchers working in the area of cognitive psychology. Review papers are given by Hogarth, [1975] and by Spetzler and Stael von Holstein [1975].

In general, people appear to assess uncertainty in a manner analogous to the way they assess distance [Spetzler and Stael von Holstein, 1975]. Just as with distances, people are better at assessing uncertainties for some type of variables than for others. Models are often used to determine how uncertainties in parameters that are more easily assessed translate into uncertainties in parameters that might be needed in decisions. For example, uncertainties in hydraulic conductivities are generally easier to estimate than uncertainties in groundwater velocity. Computer models are used to translate the hydraulic conductivity uncertainties into velocity uncertainties needed to make decisions.

There are two ways of defining a "good" probability assessor. Substantive goodness refers to the knowledge which the assessor has regarding the subject matter of concern. Normative goodness refers to the ability of the assessor to express his opinions in probabilistic forms. For example, in a task involving properties of geologic materials, a hydrogeologist could be expected to possess substantive, but not necessarily normative goodness; for statisticians faced with the same task, one would expect the contrary [Hogarth, 1975]. The relative importance of the two types of goodness have been studied in some detail in the areas of meteorology and stock-market forecasting [Winkler, 1967; Stael von Holstein, 1971]. The results showed that substantive experts achieved "scores" only slightly higher than normative experts. The authors note, however, that the experimental conditions were not representative of "real-world" conditions. They also note that the abilities of the substantive experts improved dramatically if they received feedback as to the accuracy of their judgments.

Regardless of the degree of substantive skills that a person has, there are often conscious or subconscious discrepancies between a person's assessment of probabilities and an accurate description of his actual underlying judgment. These discrepancies are termed biases. The existance of biases has been documented primarily from laboratory experiments. Examples are listed below [Hogarth, 1975].

* Small probabilities are often over-estimated and large probabilities are often underestimated.
* People generally estimate probability distributions that are "tighter" or have less variance than actual distributions.
* People often assign higher variances to variables with higher mean values.
* Recently-obtained information is given more weight than is warranted over older information.
* Events that are desired are given higher probabilities of occurrence than events that are not desired.
* People are generally conservative in that additional information does not change probabilities as much as it should.
* People tend to assume probability distributions that are shaped like the normal distribution.
* When asked to generate random numbers, subjects generally show a tendency towards too few repetitions and too many alternations.

The techniques used to encode or elicit subjective probabilities can limit these biases. The classifications of probability encoding techniques are summarized in Table 2.1. These classifications are based upon the way in which questions are asked and the way in which responses are given. Subjects can be asked to assign probabilities when given values, to assign values when given probabilities, or to assign both. The subject can respond either directly by giving a number or indirectly by choosing between simple alternatives.

If the indirect response approach is used, the alternatives can be defined in two ways. The first way is with respect to the uncertain quantity and an external reference event. For example, the subject can be asked which he feels is more likely: that the average hydraulic conductivity is between 0.01 and 0.1 or that three flips of a coin will result in three heads. The second way that alternatives can be defined is with respect to two ranges of the value scale for the uncertain quantity. For example, the subject can be asked which he feels is more likely: that the average hydraulic conductivity is between 0.01 and 0.1 or that it is between 1.0 and 100.0. This latter technique, though somewhat easier to use, often results in distributions that are too tight.

2.2.6 Bayes Theorem and the Effects of Measurements

An obvious approach to solving decision problems with uncertainty is to attempt to remove the uncertainty. Unfortunately, it is very rarely practicable to remove all uncertainty. However, it is often feasible to reduce the uncertainty by obtaining information. This information is used to modify or update

Table 2.1 - Classification of Probability Encoding Techniques

	Response Mode		
	Indirect		Direct
Inquiry Mode	External Reference	Internal Reference	
Assign probabilities to fixed values	Probability Comparisons	Relative Likelihoods	Cumulative Probability
Assign values to fixed probabilities	Probability Comparisons	Interval Technique	Fractiles
Assign both values and probabilities	__________	__________	Graph Drawing

subjective probabilities using Bayes theorem. This theorem, which is one of the oldest results of probability theory, provides the connection between subjective probabilities and objective data.

In general terms, Bayes theorem is expressed in terms of "initial" or "prior" probabilities, "final" or "posterior" probabilities, and "likelihoods." The prior probability of an hypothesis is its probability before some experiment is performed. The posterior probability is the probability after the experiment is performed. These two probabilities are generally different. The likelihood of an hypothesis is the probability, given that hypothesis, of the actual result of the experiment.

Assume a decision maker has prior probabilities $Pr(S_1)$, $Pr(S_2)$,...$Pr(S_N)$ for uncertain events. Let X_k denote the outcome of the experiment and let $Pr(S_1/X_k)$,$Pr(S_2/X_k)$...$Pr(S_N/X_k)$ be the revised or posterior probabilities. From the second law of probability:

$$Pr(S_j \text{ and } X_k) = Pr(S_j)Pr(X_k/S_j) \quad (2.5)$$

$$Pr(X_k \text{ and } S_j) = Pr(X_k)Pr(S_j/X_k) \quad (2.6)$$

Combining equations 2.5 and 2.6 gives Bayes theorem:

$$Pr(S_j/X_k) = Pr(X_k/S_j)Pr(S_j)/Pr(X_k) \quad (2.7)$$

where

$Pr(S_j)$ = Prior probability

$Pr(S_j/X_k)$ = Posterior probability

$Pr(X_k/S_j)$ = Likelihood given S_j

Bayes theorem states how we ought to learn. It says how our beliefs, expressed as prior probabilities, should be modified by information described by likelihoods. As mentioned in the previous section, experiments have shown that people who do not use Bayes theorem often do not change their prior probabilities as much as Bayes theorem would recommend.

2.3 Measuring Consequences

2.3.1 Consequences in Monetary Units

In comparing alternatives, it is necessary to express the consequences of the alternatives in numbers that are commensurable with one another insofar as practicable. For many engineering decisions, monetary units are the only units that meet this requirement [Grant, Ireson, and Leavenworth, 1982].

One of the earliest applications of economic analyses to engineering projects involved selecting locations for railways [Wellington, 1877]. Engineering economics was popularized during the 1930's and 1940's when benefit-cost ratios were first used on a wide scale basis to evaluate flood control projects. The Flood Control Act of June 22, 1936 introduced the approach [Grant, Ireson, and Leavenworth, 1982]:

> "It is hereby recognized that...the Federal Government should improve or participate in the improvement of navigable waters or their tributaries, including watersheds thereof, for flood-control purposes if the benefits to whomsoever they may accrue are in excess of the estimated costs, and if the lives and social security of people are otherwise adversely affected."

Economic evaluations of benefits and costs were later used to justify highways and freeways during the 1950's and 1960's and nuclear power plants during the 1960's and 1970's. During these times the approach also gained popularity in state, local, and even private projects.

Assigning economic values to consequences is by no means a trivial task. Many consequences may have aesthetic, emotional, or other non-economic components that are very difficult to assign monetary values. These non-economic components are often ignored because of these difficulties.

2.3.2 Consequences in Utility Units

Once the consequences of alternative actions or decisions are expressed in monetary units, it would seem a trivial matter to simply select the alternative whose consequence has the maximum economic value. This would be the case if the outcome of each alternative is known with certainty. However, because of the uncertainty in state variables discussed in the previous section, the outcome of a given action is not completely known and the decision process is not as straight-forward or simple as it first appears.

As an example, consider the simple decision tree shown on Figure 2.4. If alternative A is selected, the decision maker is guaranteed $1,000. If alternative B is selected, the decision maker has a probability p of receiving $10,000 and a probability 1-p of losing $5,000. The alternative that the decision maker selects depends upon the value of p. If the decision maker uses what is termed an "expected value" approach, he will select the sure $1000 if p is less than 0.4. If p is equal to 0.4, the

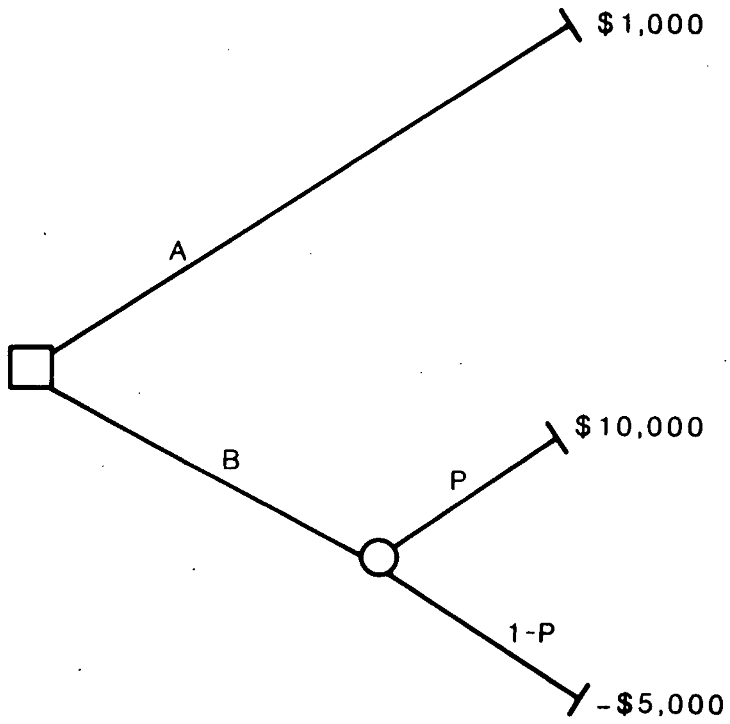


Figure 2.4 - Decision Tree When Consequence of One Alternative is Known

lottery is "fair" in that the expected winnings equal the investment:

$$\$1000 = p(\$10{,}000) + (1-p)(-\$5000) \qquad (2.8)$$

$$p = 0.4$$

When relatively large sums of money are involved, most individual will choose the sure gain and shy away from the lottery, unless the chances of winning are higher than the probabilty that makes the lottery fair. This behaviour is termed risk averseness.

Utility functions are used to quantify the degree of risk averseness that an individual exhibits. These function curves can be generated by assigning a utility function value of 1.0 to the most desirable consequence and a value of 0.0 to the least desirable consequence. All other consequences will then have utility function values in the range between 0.0 and 1.0. The utility function values of the other consequences are determined by finding values of p for which the decision maker views the lottery and the sure gain as equally attractive. For example, the decision maker may select the sure $1000 if p is less than 0.5. If p is greater than 0.5, he will choose the lottery. The $1000 is assigned a utility function value of 0.5. This process can be repeated for a number of different consequences between -$5,000 and $10,000. The resulting curve is termed the utility function.

An example utility function is shown in Figure 2.5. The sigmoid

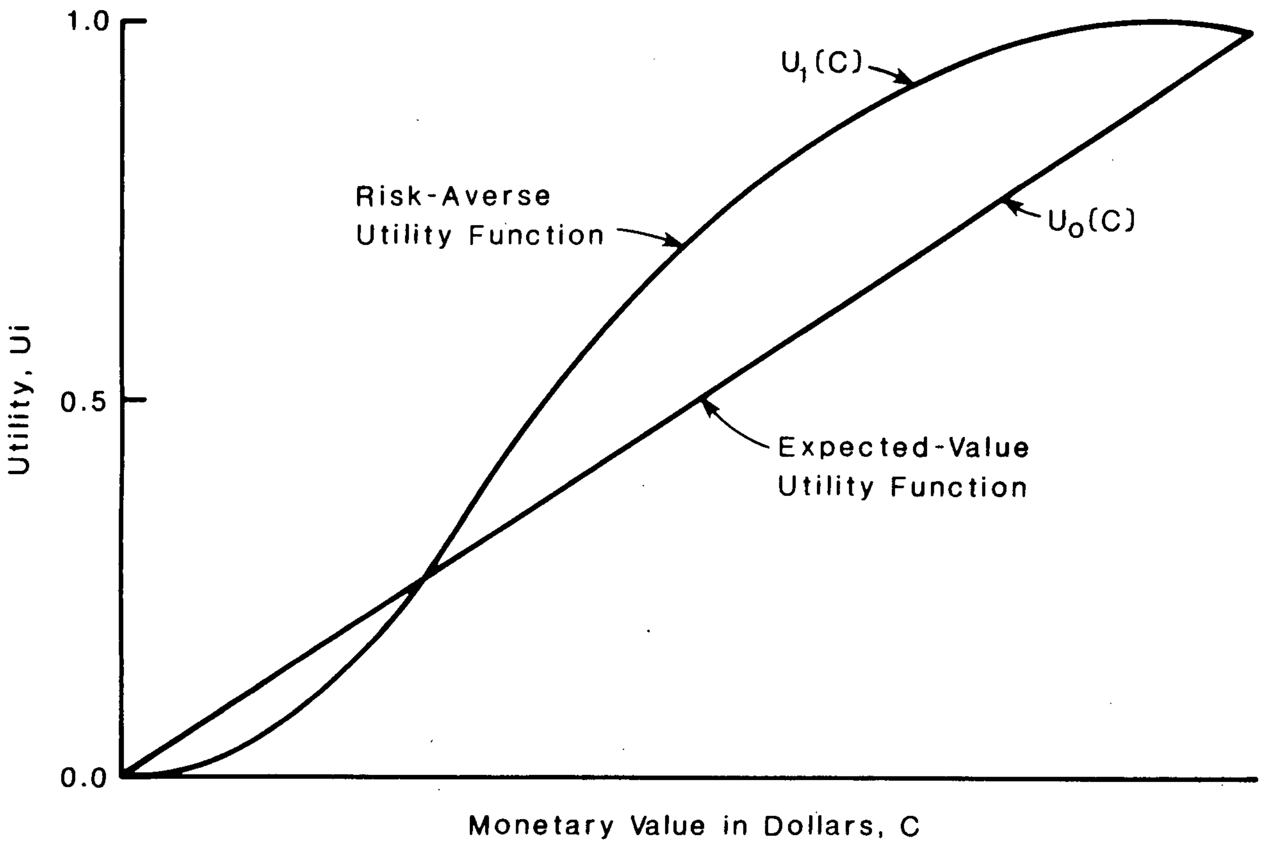


Figure 2.5 - Example Utility Curve

shape shown on this figure is typical of utility function. The concave portion of the curve to the left indicates that people are generally willing to make small investments in "unfair" lotteries. Evidence of this behaviour includes the popularity of government-sponsored lotteries in which a one- or two-dollar ticket may win a several million dollar payoff. People are willing to play these lotteries even though they are unfair since the expected winnings are less than the investments.

The linear middle portion of the curve indicates an expected value or non-risk-averse behaviour. In this range of investment, people are willing to play fair lotteries in which the expected winnings are equal to the investment. This type of behaviour is typical of companies or individuals whose net worth is large when compared to the gains or losses of the different consequences.

The convex portion of the curve on the right indicates risk-averse behaviour. In this range of investment, people will play lotteries only if the expected winnings are greater than the investment. The nearly-horizontal asymptote shows that the attractiveness of incremental gains decreases as an individual's capital increases.

The two utility functions presented on Figure 2.5 can be used to define a normalized utility function:

$$\gamma(C) = u_i(C)/u_0(C) \tag{2.9}$$

where $u_i = u_0$ for the expected value approach, and $u_i = u_1$ for the risk-averse approach. For expected value behaviour, the normalized utility function equals 1 for all monetary values; for risk averse behaviour it is greater than 1 for all monetary values. The advantage of the normalized utility function is that risk averseness can be incorporated into decisions by simply multiplying the consequence in terms of dollars by the normalized utility function:

$$U_{ij} = \gamma(C_{ij})C_{ij} \tag{2.10}$$

The value U_{ij} is termed the utility associated with consequence C_{ij}.

2.4 Decision Criteria

The final step in a decision analysis is to identify a criterion for selecting the most desirable decision variable from the list of alternatives. A number of different criteria with varying degrees of complexity and conservatism have been suggested [Lindley, 1971; Harr, 1977]. Several of the more popular criteria are listed below.

2.4.1 Maxi-Min Criterion

The maxi-min criterion is based on a pessimistic outlook that focuses attention on the least desirable state variable. The consequence with the smallest utility or monetary value is identified for each alternative decision variable. The decision variable with the largest minimum consequence is then selected. For the example shown in Table 2.2a, the maximin criterion would recommend alternative A_3, since its minimum consequence of \$1500 is the largest of all the minimum consequences.

The advantage of this procedure is its simplicity in that probability assignments are not required. The disadvantage is that it is very conservative and does not consider all possible consequences.

2.4.2 Mini-Max Criterion

The mini-max criterion is based upon losses or regret. The regret associated with a decision variable is defined as the difference between the most desirable consequence for each state

Table 2.2 - Example Comparison of Decision Criteria

Decision Variables	State Variables S_1	S_2	S_3	Minimum Consequence	Expected Consequence
A_1	0	8,000	10,000	0	7,800
A_2	2,000	6,000	1,000	1,000	4,100
A_3	10,000	7,000	1,500	1,500	5,650
Probability:	0.1	0.6	0.3		

Table 2.2a - Matrix of Consequences

Decision Variables	State Variables S_1	S_2	S_3	Maximum Regret
A_1	10,000	0	0	10,000
A_2	8,000	2,000	9,000	9,000
A_3	0	1,000	8,500	8,500

Table 2.2b - Matrix of Regrets

of nature and the consequence that would result if the decision variable had been chosen. For example, suppose state variable S_1 in Table 2.2a occurs. The largest payoff for S_1 is \$10000. If S_1 occurred and decision variable A_3 had been selected, the decision maker would have zero regret. On the other hand, if decision variable A_2 had been selected, he would have a regret of \$10000 - \$2000 = \$8000.

Table 2.2b summarizes regrets for each combination of decision variable and state variable. The procedure used with the mini-max criterion is to select the decision variable whose maximum regret is a minimum. For the example shown in Table 2.2b, decision variable A_3 would be selected on the basis of the mini-max criterion.

This procedure is less conservative than the maxi-min procedure described earlier. Probability assessments are not required and all consequences are considered.

2.4.3 Maximum Likelihood Criterion

The maximum likelihood criterion only considers those consequences associated with the most likely state variable. The decision variable is chosen which gives the most desirable consequence for the most likely state variable. For the example given in Table 2.2a, the maximum likelihood criterion would identify decision variable A_1.

Although this procedure does not require numerical values for probabilities or utilities, is does consider uncertainties in a non-quantitative manner. The disadvantage is that it does not consider consequences associated with any state variables other than the most likely.

2.4.4 Maximum Expected Utility Criterion

None of the criteria discussed above makes complete use of all the information that is included in the probabilities and consequences. The maxi-min criterion considers only the minimum consequence and none of the probabilities; the mini-max criterion considers all the consequences but none of the probabilities; and the maximum-likelihood criterion considers only the consequences associated with the most-likely state variable.

The procedure for using the maximum expected utility criterion involves selecting the alternative whose summation of consequences times probabilities is a maximum. For the probabilities listed for the example given in Table 2.2a, this criterion would identify A_1 as the preferred alternative.

The maximum expected utility criterion uses all the probabilities and all the consequences to arrive at a decision. Rigorous theoretical developments show that the criterion is unbiased and results in "coherent" decisions [Lindley, 1971]. The maximum expected utility criterion will be used for the decision analyses that will be included in this study.

2.5 The Expected Value of Information

Bayes theorem, as discussed in Section 2.2.6, provides a mechanism for using additional information or data to modify or update subjective prior probabilities. Although additional information can be either pleasing or displeasing in terms of its attractiveness to decision makers, additional information always has some value. The desirability of collecting information depends on the relationship between the expected value of the information and the costs that would be incurred in collecting it. The techniques that can be used to quantify information value will be discussed in this section.

2.5.1 The Expected Value of Perfect Information

In the absence of additional information, the decision variable that will be selected is the one with the maximum expected utility. The expected utility of decision variable A_i is given by:

$$\overline{U_i} = \sum_{j=1}^{N} U_{ij} Pr(S_j) \qquad (2.11)$$

where

$\overline{U_i}$ = Expected utility of decision variable A_i

U_{ij} = Utility of consequence C_{ij}

$Pr(S_j)$ = Probability of state variable S_j

If we had perfect information, we would select the decision variable that has the maximum utility for the state variable that occurs. The expected utility for decision variable A_i with perfect information is given by:

$$\overline{U}_{max} = \sum_{j=1}^{N} (\max U_{ij}) Pr(S_j) \qquad (2.12)$$

where

$\overline{U}_{max}$ = Expected utility with perfect information

$\max U_{ij}$ = Maximum utility for state variable S_j

The expected value of perfect information is the difference between the expected utility with perfect information and the expected utility with no additional information:

$$V_{max} = \overline{U}_{max} - \text{Maximum of } \overline{U_i} \qquad (2.13)$$

where

V_{max} = Expected value of perfect information

As an example, consider the simple decision matrix shown in Table 2.3a. Without additional information, the decision maker would choose decision variable A_1 since it has the maximum expected utility:

$$U_1 = (\$5200).5 + (\$4900).5 = \$5050 \qquad (2.14a)$$

$$U_2 = (\$5000).5 + (\$5000).5 = \$5000 \qquad (2.14b)$$

If we had perfect information, we would choose U_1 if state variable S_1 occurs and we would choose U_2 if state variable S_2 occurs. The expected utility with perfect information is given by:

Table 2.3 - Example Expected Value of Information

Decision Variables	State Variables S_1	S_2	Expected Value
A_1	$5200	$4900	$5050
A_2	$5000	$5000	$5000
Prior Probability:	0.5	0.5	

Table 2.3a - Consequence Matrix

Decision Variables	State Variables S_1	S_2	Expected Regret
A_1	$0	$100	$50
A_2	$200	$0	$100
Prior Probability:	0.5	0.5	

Table 2.3b - Regret Matrix

State Variable	Outcome of Expirement X_1	X_2
S_1	.75	.25
S_2	.25	.75

Table 2.3c - Likelihoods for Outcomes of Experiment

$$U_{max} = (\$5200).5 + (\$5000).5 = \$5100. \tag{2.15}$$

The expected value of perfect information for this simple decision is given by:

$$V_{max} = \$5100 - \$5050 = \$50 \tag{2.16}$$

A second way of looking at the expected value of perfect information is by using regrets, which were discussed in Section 2.4.2. The regret associated with consequence C_{ij} is defined as the difference between the most desirable consequence for each state of nature and the consequence that would result if the decision variable had been chosen and the state variable had occurred. Regrets for the example decision matrix given in Table 2.3a are summarized in Table 2.3b.

Equation 2.10, which gives the expected value of perfect information, can be rewritten as:

$$V_{max} = \text{Min} \sum_{j=1}^{N} (\max U_{ij} - U_{ij}) Pr(S_j) \tag{2.17}$$

where

$$\max U_{ij} = \text{Maximum utility for state variable } S_j$$

The value in the parentheses in Equation (2.17) is the regret in choosing decision variable A_i if state variable S_j occurs:

$$R_{ij} = (\max U_{ij} - U_{ij}) \tag{2.18}$$

The expected regret for decision variable A_i is given by:

$$\overline{R_i} = \sum_{j=1}^{N} R_{ij}Pr(S_j) \tag{2.19}$$

Combining Equations (2.17) and (2.18) shows that the expected value of perfect information is equal to minimum expected regret:

$$V_{max} = \text{Min} \sum_{j=1}^{N} R_{ij}Pr(S_j) = \text{Min } \overline{R_i} \tag{2.20}$$

For the example given in Table 2.3b, the expected regrets are:

$$\overline{R_1} = (\$0).5 + (\$100).5 = \$50. \tag{2.21a}$$

$$\overline{R_2} = (\$200).5 + (\$0).5 = \$100. \tag{2.21b}$$

The minimum expected regret and the expected value of perfect information is equal to \$50, which is the same as the result determined earlier.

2.5.2 The Value of Imperfect Information

With perfect information, the probabilities associated with different state variables after information has been gathered are either 0 or 1. Unfortunately, information in real-world situations is seldom perfect. The probabilities associated with different state variables are modified by the additional information, but they do not become either 0 or 1. Bayes theorem given by Equation 2.7 specifies how the prior probabilities should be modified:

$$Pr(S_j/X_k) = Pr(X_k/S_j)Pr(S_j)/Pr(X_k) \tag{2.22}$$

where

$Pr(S_j)$ = Probability associated with state variable S_j before additional information

$Pr(S_j/X_k)$ = Probability associated with state variable S_j after information X_k is gathered

$Pr(X_k/S_j)$ = Likelihood of gathering information X_k given state variable S_j.

After additional information has been collected, the decision maker will select the decision variable that maximizes the expected utilities calculated using the posterior probabilities. The expected utility without additional information is given by:

$$\overline{U}_i = \sum_{j=1}^{N} U_{ij}Pr(S_j) \tag{2.23}$$

The expected utility with additional information X_k is given by:

$$\overline{U}_i' = \sum_{j=1}^{N} U_{ij}Pr(S_j/X_k) \tag{2.24}$$

The decision maker knows what to do if he obtains data X_k, namely, to select the maximum expected utility, max $\overline{U}_i'$. What he does not know, however, is which information will arise. The best he can do is calculate an expected, expected utility. The first expectation is with respect to probabilities associated with state variables and the second expectation is with respect to probabilities associated with obtaining additional data. The first expectation is given by:

$$\overline{U}_{ik}' = \sum_{j=1}^{N} U_{ij}Pr(S_j/X_k) \tag{2.25}$$

The second expectation is given by:

$$\overline{\overline{U}}_i' = \overline{U}_{ik}'Pr(X_k) \tag{2.26}$$

The expected value of partial information is given by the difference between the maximum expected utility after additional information has been obtained and the expected utility before additional information is obtained:

$$V_{max} = (\sum_{k=1}^{M} \max \overline{\overline{U}}_{ik}') - \text{Max } \overline{U}_i \tag{2.27}$$

As an example, consider the decision matrix shown in Table 2.3a. Assume that an experiment can be performed that results in one of two outcomes and that the likelihoods of each outcome are equal to those given in Table 2.3c. The first set of expectations with respect to the unknown state variables is given by:

$$\overline{U}_{11}' = (\$5200)(.75) + (\$4900)(.25) = \$5125 \qquad (2.28a)$$

$$\overline{U}_{12}' = (\$5200)(.25) + (\$4900)(.75) = \$4975 \qquad (2.28b)$$

$$\overline{U}_{21}' = (\$5000)(.75) + (\$5000)(.25) = \$5000 \qquad (2.28c)$$

$$\overline{U}_{22}' = (\$5000)(.25) + (\$5000)(.75) = \$5000 \qquad (2.28d)$$

The second set of expectations with respect to the outcome of the experiment is given by:

$$\overline{\overline{U}}_{11}' = (\$5125)(0.5) = \$2562.50 \qquad (2.29a)$$

$$\overline{\overline{U}}_{12}' = (\$4975)(0.5) = \$2487.50 \qquad (2.29b)$$

$$\overline{\overline{U}}_{21}' = (\$5000)(0.5) = \$2500 \qquad (2.29c)$$

$$\overline{\overline{U}}_{22}' = (\$5000)(0.5) = \$2500 \qquad (2.29d)$$

The value of partial information is given by:

$$V_{max} = (\$2562.50 + \$2500) - \$5050 = \$12.50 \qquad (2.30)$$

As a final note, additional information often exhibits decreasing marginal value with increasing sample size, as shown in Figure 2.6. If the sample cost is a linear function of sample size, as shown on Figure 2.6, the maximum warranted sample is the intersection of the two curves. As shown on Figure 2.6, the optimal sample size in terms of an expected benefit to cost ratio, is usually much smaller.

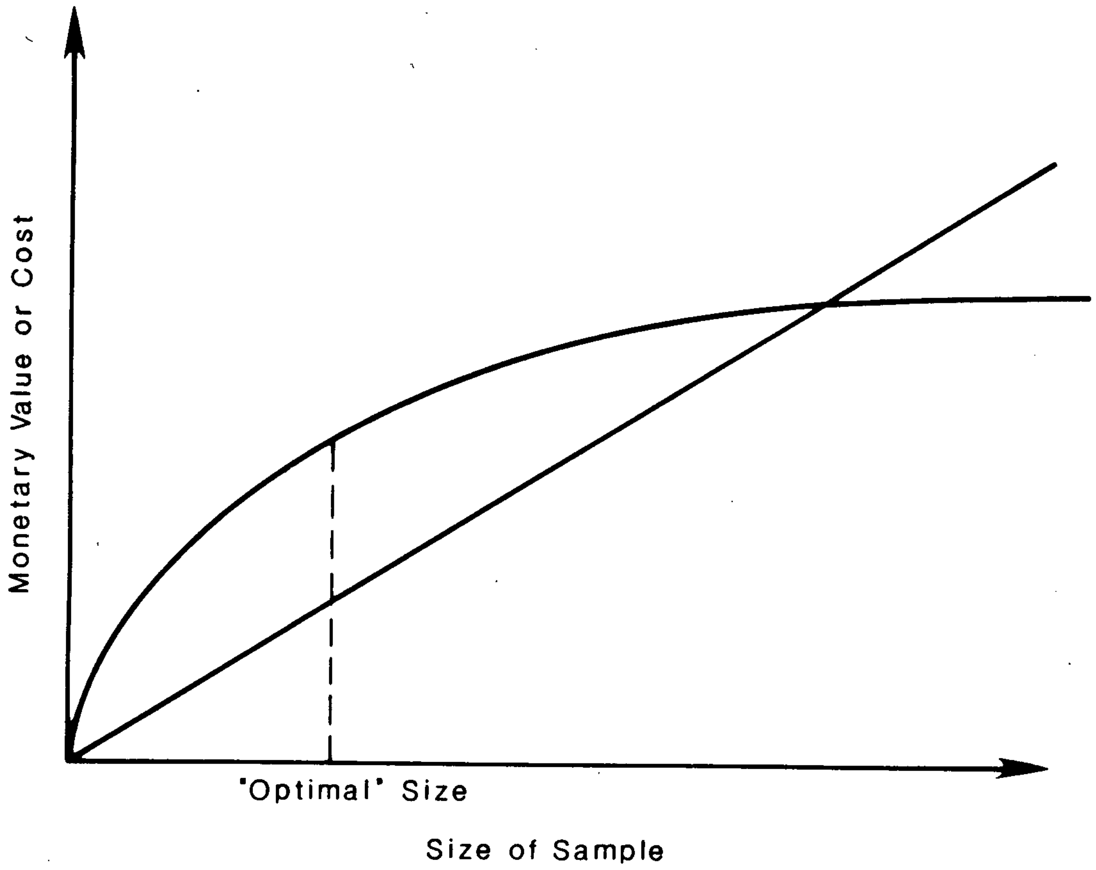


Figure 2.6 - Example Cost and Value-of-Information Curves

3. THE RISK-COST-BENEFIT EQUATION

The decision analysis described in Chapter 2 provides a framework for selecting a course of action from a group of alternatives. The preferred action is the one whose consequence has maximum expected utility. For decisions related to groundwater contamination from waste managemement facilities, the consequences consist of benefits, costs, and risks. In this chapter, an objective function comparing these benefits, costs, and risks is developed for both owner-operators and regulatory agencies. A description of the specific terms included in the objective function and the effects of the time value of money are also presented. The approach used to quantify risks is discussed in some detail.

3.1 Equation Form

One of the more general forms of an objective function for use in a risk-cost-benefit analysis treats the stream of future benefits, costs, and risks in a net present value calculation [Crouch and Wilson, 1982; Mishan, 1976]:

$$\Phi = \sum_{t=0}^{T} [\ B(t) - C(t) - R(t)\ ] / (1+i)^t \qquad (3.1)$$

where

Φ	=	objective function [US$],
t	=	time [yr],
T	=	time horizon [yr],
i	=	discount rate [decimal fraction],
$B(t)$	=	benefits in year t [US$],
$C(t)$	=	costs in year t [US$], and
$R(t)$	=	risks in year t [US$].

The risk, R(t), in Equation (3.1) is defined as the expected cost associated with the probability of failure:

$$R(t) = P_f(t)\ CF(t)\ \gamma(CF) \qquad (3.2)$$

where:

$P_f(t)$	=	probability of failure in year t [decimal fraction],
$CF(t)$	=	costs that would arise due to the

consequences of a failure in year t [US$] and

$\gamma(CF)$ = normalized utility function [decimal fraction]

The utility function allows one to take into account the possible risk-averse tendencies of some decision makers. Figure 3.1 shows a utility function, $U_0(CF)$, that represents the "expected-value" approach, and another, $U_1(CF)$, that represents risk-averse behavior. The normalized utility function used in equation 3.2 is defined as:

$$\gamma(CF) = U_1(CF)/U_0(CF) \qquad (3.3)$$

For the expected-value approach, $\gamma = 1$ for all values of the cost of failure, CF; for the risk-averse behavior, $\gamma > 1$ for all CF. Small owner-operators who do not have a large net worth are the most likely to use a risk-averse utility function. Larger companies are more likely to take an expected value approach. Risk aversion is also influenced by the availability of liability insurance and the perception of the owner-operator as to the likelihood of a government bailout in the event of failure. For social decisions such as those in the hands of regulatory agencies, there are many arguments in the literature [Arrow and Lind, 1970; Fischoff et al., 1981; Baecher et al, 1980] to suggest that there should be no risk aversion.

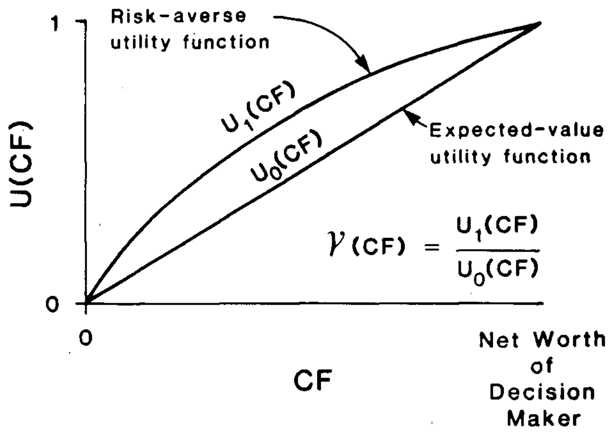


Figure 3.1 - Utility Functions

Related to risk aversion is the phenomenon of risk perception. The risk analysis literature abounds with articles about risk perception [cf. Crouch and Wilson, 1982; Fischoff et al., 1981; Keeney, 1984]. People often do not have a realistic estimate of their risk due to actual or perceived threats. From the point of view of an owner-operator of a waste management facility, his perceived risk is the one he should apply; he is an individual and he has the right to do so. There is a question, however, in social decisions whether a regulatory agency should use its perceptions of risk or act as a conduit for public perceptions.

The objective function given by Equation 3.1 can be used by either the owner-operator or the regulatory agency as long as the variables used are the ones pertinent for the point of view being taken and the problem at hand.

For assessing alternatives by the owner-operator, the costs are the capital costs and operational costs of constructing and operating a waste-management facility. The benefits are primarily in the form of revenues for services provided.

The costs associated with the probability of failure are those that affect his profitability: fines, taxes, or charges levied by the regulatory agency; costs of litigation; costs of remedial action; and the value of any revenues forgone if operations must be curtailed or stopped.

For assessing alternatives by the regulatory agency, the costs are the administrative costs of maintaining the regulatory agency at a level suitable to the particular set of regulations. The benefits to society are primarily those associated with the preservation of clean water. The costs associated with the risk of failure are the costs of remedial action where these are not borne by the owner-operator, the value of the benefits undone by the contamination incident (in the form of reduced groundwater quality), and the societal costs associated with the impairment of human health or the loss of human lives.

The objective function components for both the owner-operator and the regulatory agency will be described in more detail in Sections 3.4 and 3.5. However, before presenting these materials, it is necessary to discuss the value of human life and health and the value of clean water.

3.2 Value of Life and Health

The failure of a waste-management facility may lead to impaired health and the possible loss of lives. In that the probability of failure cannot be reduced to zero, it is impossible in assessing the costs associated with the probability of failure to avoid consideration of the value of human life and health.

In this dissertation, it is assumed that the regulatory agency is responsible for protecting public safety and that design engineers will not concern themselves with this issue if an adequate regulatory system is in place.

There is a very large literature on the dollar value of life [cf. Starr et al., 1976; Jones-Lee, 1976; Fischoff et al., 1983; Landesfeld and Seskin, 1982]. The paper by Fischoff et al. [1981] provides a general summary of the methods that have been proposed to determine the value of a "statistical" life, and the one by Sharefkin et al. [1984] provides a discussion of the issue with particular reference to groundwater contamination incidents. Most methods fall into one of the following classes:

1. The human productivity approach, based on the present worth of future lost earnings.
2. The legal approach, based on court awards for lives lost.
3. The insurance principle, based on the premiums people are willing to pay to avoid increased risk.
4. Implicit valuation based on observable responses to the risk associated with goods and services whose markets are

reasonably well developed.

5. De facto valuation as embedded in government regulations already enacted.

In my opinion, the first two of these methods are ethically unsatisfactory, and quantitative estimates based on the implicit valuations inherent in the other three methods are very difficult to uncover. It seems that decisions on the value of life are somehow outside the bounds of economic analysis and are better decided in the political arena via the democratic process. The approach espoused by Baecher et al. [1980] and Vanmarcke and Bohenblust [1982] for risk-based decision analysis of dam safety is preferred. They recommend avoiding the quagmire of attempting to estimate dollar values for statistical lives by maintaining separate accounts for lives and dollars. With this approach, the cost of human life is not included in the risk term used in the risk-cost-benefit analysis of public policy alternatives. For any given alternative the economic costs and benefits are kept in one account and the lives saved or lives exposed in a separate account. This philosophy can be incorporated by maximizing the objective function for the regulatory agency subject to a constraint of the form:

$$\sum P_f \, L < L_{pa} \tag{3.4}$$

$\sum P_f$ = total probability of failure over the period, $0 < t < T_r$,

L = best estimate of lives lost or exposed by the alternative under assessment, and

L_{pa} = politically acceptable limit of statistical lives at risk.

If L is the same for each alternative in a set of alternatives, as it would be in assessing alternative policies for a given site, the constraint can be simplified to:

$$\sum P_f < (\sum P_f)_{pa} \tag{3.5}$$

where

$(\sum P_f)_{pa}$ = politically acceptable probability of failure.

It is obvious that the politically acceptable probability of failure or the politically acceptable lives at risk cannot be considered as fixed known quantities. They are determined in the crucible of the democratic process through elections, referendums, and public hearings, and under the influence of adversarial lobbies.

This approach of meeting an acceptable societal standard with respect to the possible loss of life is akin to the concept of "acceptable risk," about which there is considerable controversy in the decision-analysis literature. It is clear [Fischoff et al., 1981] that acceptable risk is dependent on values and belief; it is definable only for a well-defined constituency. In practice, acceptable risk is the risk associated with the most acceptable decision; it is not acceptable in any absolute sense. If one accepts these ideas, it is clear that acceptable risk is decided in the political arena and that "acceptable" risk really means "politically acceptable" risk as we have used it above.

3.3 Value of Clean Water

One of the benefits to society of a properly engineered waste-management facility is the preservation of the quality of the groundwater in the aquifer underlying the site. One of the costs to society of the failure of a waste-management facility is the loss of these benefits.

There is considerable literature on the benefits of clean water in the context of surface water but almost none in the context of groundwater. In this section an attempt is made to place in a groundwater perspective some of the ideas developed for surface-water resources by Kneese and Bower [1986] and Howe [1979]. Some beginnings in this direction have also been made in recent papers by Raucher [1984] and Sharefkin [1984].

It appears that there are two senses in which clean groundwater has value. It has value in use as a resource for the current generation, and it has value in storage for future generations. The first of these benefits can be evaluated with the concept of scarcity rent, and the second in terms of preservation benefits.

Economists define rent [Mishan, 1976] as the difference between the benefits generated by a resource in its current use and the minimum sum the resource owner is willing to accept to keep the resource in its current use rather than divert it to an alternative use. The scarcity rent [Howe, 1979] of a renewable but depletable resource is defined as the present value of all future sacrifices associated with the use of a marginal unit of an in situ resource. Under appropriate market conditions, the scarcity rent is equal to the market value of these in situ resources. If an aquifer or a portion of it has an annual optimal yield, Q(t), and if B_{sr} is the unit scarcity rent, the annual benefits of having the water in place for use by the current generation is B_{sr} times Q(t). It is this benefit that is foregone in the event of a failure of a waste-management facility that pollutes the aquifer (or that portion of it that could produce an annual optimal yield, Q(t).)

The concept of preservation benefits for surface water has been described by Kneese and Bower [1968]. They noted that the need to preserve potable water cannot justify particularly high standards of water quality in watercourses because treatment at the point of intake from streams prior to delivery to industrial or municipal water systems is an economically viable approach. They conclude that higher water quality must be justified on the grounds of recreational and preservation benefits, the so-called "intrinsic" benefits of clean water. For groundwater, it is hard to argue on behalf of recreational benefits, but there is evidence that society ascribes large preservation value to groundwater. In fact, as with surface water, it is likely that the intrinsic benefits of groundwater-quality protection outweigh the direct benefits, which at current market rates for water are not particularly large. For example, Raucher [1984] carried out an economic analysis of alternative remedial actions associated with two contamination incidents. He concluded that no form of direct remedial action could be economically justified; the recommended course of action involved locating an alternative source of supply. He did not include intrinsic benefits in his cost-benefit analysis, but he did back-calculate the value that they would have to attain to justify direct remedial action. The numbers he obtained struck him as unreasonably large. However, it must be noted that such large numbers may be representative. The public demand for groundwater cleanup in such cases (as evidenced politically by the passage of the CERCLA, or "Superfund" legislation) implies a large intrinsic value. This public demand may be taken as a strong political statement of the preservation value of clean aquifers.

Greenley et al. [1982] identified three more-or-less separable components of value that can be ascribed to the preservation of clean water.

1. Option value -- the protection of future options for competing usage.
2. Existence value -- a willingness on the part of society to pay for the simple knowledge of the existence of clean water.
3. Bequest value -- the satisfaction society gains from bequeathing a clean natural environment to future generations.

They even presented data on the value of each of these components for surface water, based on questionnaires completed by a random sampling of the public. It is not clear how these values translate to groundwater. They are far less than those implied by Raucher's back-calculation.

Gorelick [1982] and Gorelick and Remson [1982] presented an alternative way of looking at the value of an aquifer as a resource. They noted that an aquifer has value to society both as a source of groundwater and as a receptor of leachates from waste-management sites. They developed techniques for the optimal location and management of leachate sources in an aquifer that is also tapped by wells.

3.4 Equation Components for Owner-Operators

The objective function for assessing alternatives available to the owner-operator is outlined in Table 3.1. In Table 3.1a, the equations for the major components of the objective function are described in some detail. The parameters used in these equations are defined in Table 3.1b. Typical values for the parameters are also presented in Table 3.1b. The values used for the sensitivity studies that are described in Chapter 7 are also included in Table 3.1b.

Figure 3.2 clarifies the time and space framework for the owner-operator's risk-cost-benefit analysis, and summarizes the parameters discussed below. Figure 3.2a shows the time scales used. At time $t=0$ a site exploration and facility design phase is begun. This is followed at time $t=t_{con}$ by a construction phase. The construction phase is in turn followed at time $t=t_{op}$ by the actual operation of the facility. Finally, at time $t=T_O$, the facility is decommissioned.

Figures 3.2b and 3.2c illustrate the spatial framework assumed for the waste management. A breach of the containment structure results in a plume that migrates toward a regulatory compliance surface. A monitoring network may be installed by the owner-operator between the facility and the compliance surface. The concentration profile at the compliance point is also shown on Figure 3.2. At time t^{**}, the concentration at the compliance surface increases as a step function.

Table 3.1 - Risk-Cost-Benefit Analysis for Owner-Operator

A. Equations

Objective Function

$$\Phi_o = \sum_{t=0}^{T_o} [B_o(t) - C_o(t) - R_o(t)]/(1+i_m)^t \qquad \text{(T3.1.1)}$$

Benefits

$$\begin{aligned} B_o(t) &= 0 && 0 < t < t_{op} \\ &= B_I && t_{op} < t < T_o \\ &= B_I + B_{II} && t = T_o \end{aligned}$$

$$B_I = B_R V + B_p r_1 V \qquad \text{(T3.1.2)}$$

$$B_{II} = (C_L A + C_{BP})(1 + i_m)^{T_o} \qquad \text{(T3.1.3)}$$

$$V = Z/(T_o - t_{op}) \qquad \text{(T3.1.4)}$$

Costs: After Tax

$$C_o(t) = C_o'(t) + C_o''(t) \qquad \text{(T3.1.5)}$$

$C_o'(t)$ = before-tax costs in year t

$C_o''(t)$ = income taxes in year t

$$\begin{aligned} &= f[B_o(t) - C_o'(t)] && \text{if } [\quad] > 0 \\ &= && \text{if } [\quad] < 0 \end{aligned}$$

Costs: Before Tax

$$C_o'(t) = C_{cap}(t) + C_{op}(t) \qquad \text{(T3.1.6)}$$

$C_{cap}(t)$ = capital costs in year t

$C_{op}(t)$ = operating costs in year t

Table 3.1 - continued

Costs: Capital

$$
\begin{array}{lll}
C_{cap}(t) & = C_I & t = 0 \\
 & = C_{II} & t = t_{con} \\
 & = C_I + C_{II} + C_{III} & t = t_{op} \\
 & = 0 & \text{all other } t
\end{array}
$$

$$C_I = C_L A + C_S A + C_{BP} \qquad (T3.1.7)$$

$$C_{II} = (C_Y + C_N n) Y_X \qquad (T3.1.8)$$

$$C_{III} = m C_A A \qquad (T3.1.9)$$

$$C_{IV} = (C_Y + C_V l) Y_M \qquad (T3.1.10)$$

$$C_V = C_Q V \qquad (T3.1.11)$$

Costs: Operating

$$
\begin{array}{lll}
C_{op}(t) & = C_{VI} & 0 < t < t_{op} \\
 & = C_{VII} + C_{VIII} & t_{op} < t < T_o
\end{array}
$$

$$C_{VI} = [0.14(\overset{t_{op}}{} C_{cap}(t'))]/t_{op} \qquad (T3.1.12)$$

$$C_{VII} = C_U V + C_E EV + C_T V + 2.08 C_B^{LV} + C_W r_2 V + 0.10 B_2 r_1 V \qquad (T3.1.13)$$

$$C_{VIII} = C_C l Y_M^k \qquad (T3.1.14)$$

$$C_{IX} = C_D A \qquad (T3.1.15)$$

Table 3.1 - continued

Risks: General

$$R_o(t) = R_{comp}(t) + R_{mon}(t) \quad (T3.1.16)$$

$R_{comp}(t)$ = costs associated with risk of plume arrival at compliance surface in year t (failure)

$R_{mon}(t)$ = costs associated with risk of plume arrival at monitoring network in year t (detection)

Risks: Failure

$$R_{comp}(t) = P_f(t)CF_o(t) \quad (CF_o) \quad (T3.1.17)$$

$$P_f'(t) = P_f(t)(1 - P_d) \quad (T3.1.18)$$

$$CF_o(t) = C_P + C_J + C_R + C_G(t) \quad (T3.1.19)$$

$$C_G(t) = \frac{B_{II} +}{(1-i_m)^{To-t}} \sum_{t'=t}^{T_o} \frac{(1 + r_3)[B_I - C_{op}(t)]}{(1 + i_m)^{t'-t}} \quad (T3.1.20)$$

Risks: Detection

$$R_{mon}(t) = P_p'(t)CD_o\gamma(CD_o) \quad (T3.1.21)$$

$$P_p'(t) = P_p(t)P_d \quad (T3.1.22)$$

$$CD_o = aC_R \qquad \text{where } a = X_M/X_S \quad (T3.1.22)$$

Economies of Scale

$$\left[\frac{[C_o'(t)]_A}{[C_o'(t)]_B}\right] = \left[\frac{Z_A}{Z_B}\right]^s \quad (T3.1.23)$$

Table 3.1 - continued

B. Definitions of Parameters

1. Parameters used directly in equations

Parameter	Definition	Unit	Range	Base Case
T_0	Owner-operator's time horizon	yr	10-50	46
t_{con}	Time at which construction begins	yr	1-5	3
t_{op}	Time at which operation begins	yr	2-10	6
i_m	Inflation-free, constant-dollar, market discount rate	decimal fraction	0.05-0.20	0.10
Z	Capacity of landfill	ton	10^5-10^7	4.5×10^5
A	Area of landfill	m^2	10^4-10^6	3.0×10^4
B_R	Charge for waste handled	US$/ton	10-100	90
B_P	Price obtained from recovered products	US$/ton	0-100	50
r_1	Ratio: Recovered products/waste handled	decimal fraction	0-0.10	0.05
f	Income tax rate	decimal fraction	0.48	0.48
Y_X	Total depth of exploratory drilling	m	0-1,000	90
n	Number of measurements of hydraulic conductivity in exploration program per m drilled	m^{-1}	0.1-1.0	0.1
Y_M	Total depth of drilling for installation of monitoring network	m	0-1,000	90
ℓ	Number of monitoring points in monitoring network per m drilled	m	0.1-1.0	0.1
k	Number of samples collected at each monitoring point per year	yr^{-1}	0-52	4
m	Design variable; number of liners in parallel	Integer	0-4	1
E	Energy requirements of facility	MJ/ton	100-200	140
L	Labor requirements of facility	man-h/ton	0.1-0.2	0.1

Table 3.1 - continued

Parameter	Definition	Unit	Range	Base Case
r_2	Ratio: residual waste/waste handled	decimal fraction	0-0.05	0.01
Θ_0	Ratio: postfailure net benefits/ prefailure net benefits	decimal fraction	0-1.0	0.0
X_M	Distance from edge of landfill to compliance surface	m	100-10,000	1000
a	Ratio: distance from center of landfill to monitoring network/ distance to compliance surface	decimal fraction	0-1.0	0.25
C_L	Cost of land	US\$/m^2	0.1-1.0	0.2
C_S	Cost of services	US\$/m^2	$\sim 0.5C_L$	0.1
C_{BP}	Performance bond posted	US\$	$0-10^7$	0.0
C_Y	Cost of drilling and casing	US\$/m	50-150	100
C_N	Cost per in situ hydraulic conductivity measurement	US\$/ meas.	200-2,000	1000
C_A	Cost of synthetic liner	US\$/m^2	2-10	8.0
C_V	Cost of installation of monitoring point	US\$/ mon-point	50-500	400
C_Q	Cost of equipment	US\$/ton	5-10	7.5
C_U	Cost of maintenance and supplies	US\$/ton	0.1-1.0	0.5
C_E	Cost of energy	US\$/MJ	0.002-0.01	0.004
C_T	Cost of preemplacement treatment	US\$/ton	0-100	0.0
C_B	Cost of labor	US\$/man-h	10-20	15
C_W	Cost of disposal of residual waste	US\$/ton	50-150	100
C_C	Cost of chemical analysis of one sample collected from monitoring point	US\$/ sample	50-1,500	300
C_D	Cost of restoration of landfill and decommissioning of facility	US\$/m^2	1-2	1.5

Table 3.1 - continued

C_P	Cost of regulatory fine in event of failure	US$	$0-10^7$	$5x10^6$
C_J	Cost of litigation and damages assessed in case of failure	US$	$0-10^7$	$5x10^6$
C_R	Cost of remedial action in event of failure	US$	$0-10^7$	$5.75x10^6$
ε	Scale factor for economies of scale	decimal fraction	0.8	0.8

2. Parameters used in determination of $P_f(t)$, $P_p(t)$, and P_d

Parameter	Definition	Unit	Range	Base Case
$\bar{t}^*$	Mean breach of time of a single liner	yr	0-100	14
m	Number of liners in parallel	integer	0-4	1
H	Head drop between landfill and compliance surface	m	0-100	8.2
p	Porosity of aquifer	decimal fraction	0.1-0.5	0.2
$\bar{K}$	Mean hydraulic conductivity	m/yr	1-10,000	1500
σ_K	Standard deviation for hydraulic conductivity	m/yr	100-5000	1500
α_x, α_y	Correlation scales for hydraulic conductivity	m	10-1000	300
L_S	Length of breach in containment structure	m	1-100	20

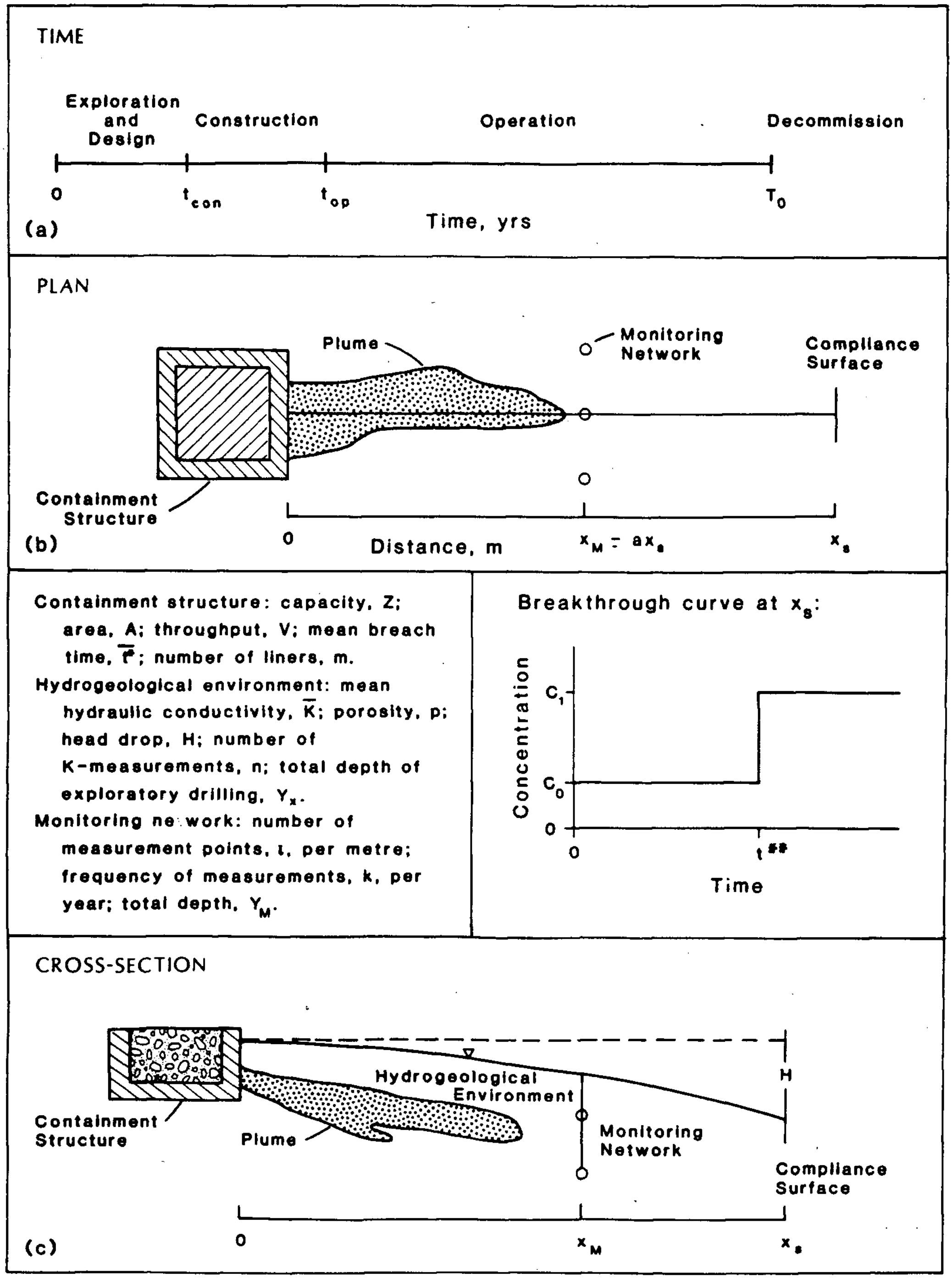


Figure 3.2 - Framework for Owner-Operator's Risk-Cost-Benefit Analysis with Respect to a) Time, b) Plan View, and c) Cross-Section View

The evaluation of economic decisions in terms of net present value requires the specification of a discount rate. For the owner-operator, especially in the case where all or most of his capital investment takes place at or near time zero, it is proper to set the discount rate equal to the current market rate on private borrowing, i_b. Because the interest rate cannot be known with certainty for the future and investment decisions are strongly sensitive to discount rates, it is common to carry out a sensitivity analysis on i_b during the decision process.

Net-present-value calculations must also take into account the influence of the inflation rate, d. There are two approaches: (1) the "current-dollar" approach, which requires the direct inclusion of the inflation rate in the discount factor of the risk-cost-benefit analysis, and the use of a discount rate, $i = i_b$ (2) and the "constant-dollar" approach in which d does not explicitly appear and the discount rate is i_m, where $i_m = i_b - d$ [Grant et al., 1982]. Note that the constant-dollar discount rate, i_m, does not include the inflationary component, so in an inflationary period i_m will be lower than the actual market rate. In this study we have used the constant-dollar approach, without explicit inclusion of d, and with i_b and i_m in the vicinity of 0.10 and 0.05, respectively. All costs, benefits, and risks are calculated in 1980 U.S. dollars, and where the data available to us are from a date other than 1980, they have been converted into 1980 U.S. dollars by using the consumer price index.

Investment decisions may be very sensitive to the time horizon, T_O over which calculations are made. For most representative discount rates, benefits that occur beyond about 20 years into the future have an increasingly negligible contribution to the net present value of a project. Wilson [1981] suggested a time horizon of 10-25 years for waste-management facilities. A few authors have suggested horizons as long as 50 years.

Benefits are derived primarily from the first term of B_I in Table 3.1A which is the revenue received for waste handled; the parameter, V, is the annual throughput (tons/yr). The second term is the revenue received for recovered products. The term, B_{II} represents the return of investment with interest at $t = T_O$ of monies put down at $t = 0$ for land purchase and the posting of a regulatory bond. It is assumed that investment in land bears interest at the same rate as investment in the market. The benefits B_I and B_{II} are the benefits foregone in the event of failure.

Costs include both capital and operating costs, as well as the impact of the income-tax rate, f. The term C_I under capital costs represents the cost of land, services and posting a regulatory bond. The term, C_{II}, is the cost of exploration. The term, C_{III}, is the cost of the containment structure. (Note that if the unit cost of one synthetic liner is C_A, the cost of m identical liners is mC_A). The term, C_{IV}, represents the capital costs of installing a monitoring network. The term, C_V, is the cost of capital equipment.

The operating-cost expressions, C_{VI} and C_{VII}, are based on Wilson's [1981] work. C_{VI} reflects the fact that operating costs during exploration and construction are taken as a percentage of the average capital costs during this period. The terms of C_{VII}, taken in order, represent the cost of (1) maintenance, materials, and supplies; (2) energy; (3) waste treatment; (4) labor; (5) disposal of residual waste; and (6) marketing of recovered products. The term, C_{VIII}, is the cost of collection and analysis of samples from the monitoring network. The term, C_{IX}, is the cost of decommissioning and/or restoring the facility at $t = T_o$.

Wilson [1981] suggested that there are economies of scale associated with the costs of waste-management facilities. If these are to be taken into account, the right-hand side of equation (T3.1.6) should be multiplied by a scale factor such that the before-tax costs, $[C_o'(t)]_A$ and $[C_o'(t)]_B$, for two facilities A and B are related to their capacities, Z_A and Z_B, by equation (T3.1.24). Wilson suggested a value for the scale factor of 0.8.

Taxes are an additional cost item to the owner-operator. There are three major types of taxation -- property taxes, excise taxes, and income taxes -- and the last must be paid on both capital gains and corporate profits. For most decision analyses, the relative values of these taxes are such that the property taxes, excise taxes, and capital gains taxes can be ignored and the tax rate, f, can be taken as the income-tax rate on corporate profits [Dieter, 1983]. In North America, f is approximately 0.5.

Depreciation of capital equipment can produce tax advantages to the owner-operator. However, in our study, no appreciable assets are assumed. In a waste-management facility where the principal design feature involves synthetic liners, depreciable equipment is a minor component of total capital cost, so the benefits of depreciation on taxes can be ignored.

Risks reflect both the costs that would arise in response to plume detection at the monitoring network, CD_o, and at the compliance surface, $CF_o(t)$. The four components of the $CF_o(t)$ term given by equation (T3.1.19) are regulatory penalties, C_P, litigation costs, C_J, remedial costs, C_R, and net benefits reduced ($0 < r_3 < 1$) or foregone ($r_3 = 0$). The CD_o term given by equation (T3.1.23) reflects the cost of remedial actions due to failure, C_R, reduced by a factor, a, that depends on the geometry of the system. For long, narrow plumes, $a = X_M/X_S$ (Figure 3.2b). The γ terms in equations (T3.1.17) and (T3.1.21) represent normalized utility functions of the type illustrated in Figure 3.1; they allow for risk aversion on the part of the owner-operator.

Small owner-operators who do not have a large net worth are the most likely to use a risk-averse utility function. Larger companies are more likely to take an expected-value approach.

Table 3.1B lists each of the parameters appearing in equations (T3.1.1) through (T3.1.24) with definitions and units. The ranges of values have been garnered from a wide variety of economic and technical sources. Among the most valuable sources of cost data are Wilson [1981] and Rishel et al. [1984] for construction costs; Wilson [1981] for operating costs; Everett [1980] and the Office of Technology Assessment [1984] for monitoring, sampling, and analysis costs; and Richel et al. [1984], Raucher [1984] and Sharefkin et al. [1984] for costs of remedial alternatives. Wood [1984] summarized the current range of rates charged for waste handled. The set of values indicated by the base case in the right-hand column of Table 3.1B is used in the sensitivity analyses reported later in the dissertation.

3.5 Equation Components for Regulatory Agencies

The objective function for assessing alternatives available to the regulatory agency is outlined in Table 3.2, with the equations presented in Table 3.2A and the parameters defined in Table 3.2B. The function has the same terms as the owner-operator's function, but the terms have different interpretations and values.

Turning first to Table 3.2A, it can be seen that the objective function for the regulatory agency is written in terms of the social discount rate, i_s, and the regulatory time horizon, T_r.

The selection of a discount rate for decisions in the public sector is more controversial than that for decisions in the private sector. As a lower bound, it is recognized that the social discount rate, i_s, should be at least as large as the risk-free, constant-dollar interest rates, i_g, paid on long-term government bonds. Following Arrow [1965] and McDonald [1981], it is now generally accepted that the social discount rate should be a weighted average of i_b and i_g:

$$i_s = i_b p + i_g (1 - p) \tag{3.6}$$

where p represents the fraction of the cost of the public project that comes at the expense of private investment (usually taken in the range 0.10--0.20) and (1 - p) represents the fraction of costs that come at the expense of private consumption.

Table 3.2 - Risk-Cost-Benefit Analysis for Regulatory Agency

A. Equations

Objective Function

$$r = \sum_{t=0}^{T_r} [B_r(t) - C_r(t) - R_r(t)]/(1+i_s)^t \qquad \text{(T3.2.1)}$$

Constraint

$$\Sigma P_f' \leq (\Sigma P_f)_{pa} \qquad \text{(T3.2.2)}$$

$$\text{where } \Sigma P_f' = \sum_{t=0}^{T_r} P_f'(t)$$

Benefits

$$
\begin{aligned}
B_r(t) &= B_1 && t = 0 \\
&= B_2 && 0 < t < T_r \\
B_1 &= B_{bp} && && \text{(T3.2.3)} \\
B_2 &= (B_{sr} + B_{op} + B_{ex} + B_{be})Q(t) && && \text{(T3.2.4)}
\end{aligned}
$$

Costs

$$
\begin{aligned}
C_r(t) &= C_1 && 0 < t < T_r \\
&= C_1 + C_2 && t = T_o \\
C_1 &= C_a && && \text{(T3.2.5)} \\
C_2 &= B_{bp}(1 + i_m)^{T_o} && && \text{(T3.2.6)}
\end{aligned}
$$

Table 3.2 - continued

Risks

$$R_r(t) = P_f'(t)CF_r(t) \qquad (T3.2.7)$$

$$P_f'(t) = P_f(t)(1-P_d) \qquad (T3.2.8)$$

$$CF_r(t) = P_o(t)C_r + C_j + C_g(t) - B_p - B_j \qquad (T3.2.9)$$

$$C_g(t) = \sum_{t'=t}^{T_r} \frac{(1 - r_3)[B_r(t) - C_r(t)]}{(1 + i_s)^{t'-t}} \qquad (T3.2.10)$$

$$\begin{aligned} P_o(t) &= P_o \qquad 0 < t < T_o \qquad (T3.2.11) \\ &= 1 \qquad t_o < t < T_r \end{aligned}$$

B. Definition of Parameters

Parameter	Definition	Unit
T_r	Regulatory time horizon	yr
T_o	Owner-operator time horizon	yr
i_s	Social discount rate	decimal
$(\sum P_f)_{pa}$	Politically acceptable probability of failure over T_r	decimal
B_{bp}	Regulatory bond posted by owner	US$
B_{sr}	Scarcity rent of groundwater	US$/L
B_{op}	Option value of groundwater	US$/L
B_{ex}	Existence value of groundwater	US$/L
B_{be}	Bequest value of groundwater	US$/L

Table 3.2 - continued

$Q(t)$	Optimal yield of aquifer or portion of aquifer liable to contamination from waste-management facility	L/yr
C_a	Annual administrative cost of operating facility	US$
C_r	Cost of remedial actions borne by regulatory agency	US$
C_j	Litigation costs in event of failure	US$
B_p	Penalties received from owner-operator in event of failure	US$
r_3	Ratio of post-failure to pre-failure net benefits to society	decimal
$P_f(t)$	Probability of failure of waste management facility in absence of monitoring network	decimal
$P_f'(t)$	Probability of failure of waste management facility with monitoring network in place	decimal
P_d	Probability of detection	decimal
P_o	Probability that owner-operator cannot bear remedial costs following a failure	decimal

The regulatory agency's time horizon is likely much longer than the 10-50 years used by owner-operators. The consideration of inter-generational bequest value, which was discussed in section 3.3, demands a time horizon on the order of at least 100 to 200 years. This incompatibility of the time horizons between the owner-operator and the regulatory agency is one of the stumbling blocks preventing the development of effective regulatory policies.

The constraint on the total probability of failure is used to incorporate a limit on the statistical lives at risk from the waste-management facility. The basis for this constraint is discussed in Section 3.2.

The term, B_1, defined in equation (T3.2.3), is the revenue, B_{bp}, received from a regulatory bond posted by the owner-operator at t = 0. The term, B_{bp}, is equal to the term, C_{BP}, in the capital costs listed in Table 3.1.2A for the owner-operator. The first term of B_2 is the scarcity rent of the annual yield, Q(t). The remaining terms collectively represent the preservation benefits.

The term, C_1, defined in equation (T3.2.5) is the annual administrative cost, C_a, of operating the regulatory agency. The term, C_2, defined in equation (T3.2.6) represents the return to the owner-operator at $t = T_o$ of the regulatory bond posted at t = 0, with interest borne at the market rate.

The risks defined in equation (T3.2.7) follow an expected-value approach without risk aversion. The probability of failure in the risk term takes into account improvements afforded by the detection capabilities of the owner-operator's monitoring network. The costs to society of a failure in year t, as indicated in equation (T3.2.9), include remedial costs, C_r, litigation costs, C_j, and benefits foregone, $C_g(t)$, but not the impact on human health, which is treated as a separate account and covered by the constraint. The two negative components in equation (T3.2.9) are the penalties received, B_p, and the damages collected through litigation, B_j.

For ease of presentation, the structure is presented as if only a single waste-management facility were involved. More realistically, the objective function for the regulatory agency would involve the sum of the individual objective functions for all the facilities under its jurisdiction.

Table 3.2B provides a summary of the parameters that appear in the equations presented in Table 3.2A.

3.6 Summary Comparison

Table 3.3 provides a summary comparison of the risk-cost-benefit analyses developed for the owner-operator and the regulatory agency. Of the two, the framework developed for the owner-operator is by far the most valuable. Analyses carried out for the regulatory framework are useful for making back-calculations of the apparent value placed on clean water by society or assessing the impact of bankruptcies on regulatory cleanup costs, but not for direct application to questions of alternative policy because of the sparseness and uncertainty of the available data. Almost all the important factors suffer from this weakness. Howe [1979] noted that data on in-situ scarcity rents are both hard to find and difficult to interpret. Greenley et al.'s [1982] values for preservation benefits may have no applicability to groundwater. Even aquifer yields pose a knotty problem [Freeze and Cherry, 1979]. It is believed that the development of the regulatory risk-cost-benefit framework presented in Table 3.2 is a necessary counterpoint to the framework presented for the owner-operator in Table 3.1, but it is now clear that it is very difficult to work with.

In light of these limitations, only the owner-operator's risk-cost-benefit analysis is used. The formulation is much more specific, and reasonable estimates are available for the input parameters. By selecting the appropriate variable for analysis, it is possible to carry out sensitivity analyses for a wide variety of influences on the system. Referring to the variables

Table 3.3 - Summary Comparison of Risk-Cost-Benefit Analysis for Owner-Operator and Regulatory Agency

Item	Owner-Operator	Regulatory Agency
Objective function, Φ	$\Phi_0 = \sum_{t=0}^{T_0} \frac{1}{(1+i_m)^t} [B_0(t) - C_0(t) - R_0(t)]$	$\Phi_r = \sum_{t=0}^{T_r} \frac{1}{(1+i_s)^t} [B_r(t) -C_r(t) - R_r(t)]$
Discount rate, i	Market discount rate, i_m	Social discount rate, i_s
Time horizon, T	Engineering time horizon, T_0 = 10-50 yr	Social time horizon, T_r = 100-200 yr
Benefits, B(t)	Revenues for service provided	Preservation of clean water
Costs, C(t)	Construction and operation of waste-management facility	Administration of regulatory agency
Risks, R(t) = costs associated with proability of failure	Regulatory penalties, cost of litigation, remedial action, benefits foregone (reduced revenues)	Impairment of human health or loss of human lives, cost of litigation remedial action, benefits foregone (reduced water quality
Probability of failure, $P_f(t)$	Probability of groundwater contamination incident that violates performance standards at compliance surface	Same as for owner-operator
Utility	Risk averse	Expected value
Risk perception	Perception of owner-operator	Perception of public or of regulatory agency
Value of life	Not included in owner-operator's analysis	Must be included in some manner in regulatory analysis
Acceptable risk	Risk associated with alternative that maximizes utility function	Societal acceptable risk; defined politically
Monitoring	Warning of potential failure; network located near source	Enforcement of performance standards; network located at compliance surface

Table 3.3 - continued

Item	Owner-Operator	Regulatory Agency
Remedial action	Avoid further regulatory penalties or litigation and/or bring facility back on-line	Ensure health and safety of public; protect water quality
Decision variables	1. Exploration: number, location, and depth of drill holes; parameters to be measured; number and depth of measurements	1. Location of compliance surface
	2. Containment: number, thickness, and permeability of synthetic liners	2. Design standards: number, type, values
	3. Monitoring: number, location, and depth of measurement points; frequency of measurements; species to be analyzed.	3. Performance standards: species, values
	4. Remedial: containment method: excavation and reburial, grout curtain or slurry trench, hydraulic control; location, design.	4. Monitoring requirements by owner-operator &/or agency: number, location, and depth of measurement points; frequency of measurements; species to be analyzed.
		5. Penalties for violations: fines, bonds posted.
		6. Remedial: do nothing, restoration, containment, avoidance

Note: For ease of presentation, the structure is presented as if only a single waste-management facility were involved. More realistically, the objective function for the regulatory agency would involve the sum of the individual objective functions for all the facilities under its jurisdiction.

defined in Table 3.1B, one can look at sensitivity to:

1. Size: Z, A, V
2. Economic factors: T_O, i_m, B_R,
3. Hydrogeology: K, V_K
4. Exploration program: Y_x, n
5. Design: m
6. Monitoring network: Y_m, a, l, k
7. Remedial action: C_R
8. Regulatory policy: C_p, C_{BP},

The sensitivity analyses can be interpreted directly from the point of view of an owner-operator, but they also have value from the point of view of a regulatory agency. By observing the response of an owner-operator to different regulatory stimuli, an assessment of the worth of alternative regulatory policies in meeting societal objectives can be made.

3.7 Probability of Failure

For reasons prescribed in the previous section, the remainder of this dissertation is concerned primarily with the owner-operator's objective function. The benefit and cost terms of this function can be determined in a relatively straight-forward manner. The problem lies in the risks, $R(t)$; and the crux of the risk problem is to determine how a given design allocation affects the probability of failure, $P_f(t)$.

The failure of a waste-management facility, viewed either from the point of view of the owner-operator or regulatory agency, involves the release to the environment of toxic, radioactive, or otherwise hazardous chemical species. Such a release could be to the atmosphere, to surface water, or to groundwater. This dissertation is concerned only with waste-management facilities that release hazardous leachates to groundwater.

Most waste-management facilities with this potential take the form of landfills, ponds, or subsurface emplacements. The left-hand column of Table 3.4 lists the most common types of facilities. Landfills and waste ponds lead to point sources at the surface; subsurface emplacements lead to point sources at depth.

As summarized in Table 3.4, containment of contaminant sources can be attempted in a variety of ways [Cartwright et al, 1981; Folkes, 1982; Barber and Maris, 1983; Cosler and Snow, 1984; Anderson et al., 1984]. The most common methods involve natural

Table 3.4 - Waste Management Facilities That Release Leachate to Groundwater

	Liner or buffer of natural materials	Synthetic liner	Leachate control system; drains, well, pumps
<u>Landfills</u>			
Sanitary landfills for solid, nonhazardous municipal waste	X	X	X
Chemical landfills for solid and liquid hazardous industrial waste	X	X	X
<u>Waste Ponds</u>			
Sewage lagoons for liquid municipal waste	X	X	X
Tailings ponds for slurried mining wastes	X	X	X
Brine ponds from petroleum recovery and Salt and potash mining	X	X	X
<u>Subsurface Emplacements</u>			
Near-surface buried tanks for liquid industrial and low-level radioactive waste			X
Deep repositories for solid high-level radioactive waste	X		
Deep-well injection of hazardous liquid industrial waste			X

buffers, synthetic liners, leachate control systems, and tanks or canisters. The various design features are often coupled in parallel to provide a "multiple-barrier" system. It is believed that the methods developed in this study are general enough to be applied to any of the combinations that appear in Table 3.4. However, to avoid a constant stream of caveats and asides, the material will be presented in the context of a landfill in which containment is attempted with one or more synthetic liners in parallel. Figure 3.2 illustrates the type of physical system we envisage.

"Failure" is defined as a groundwater contamination incident that violates a performance standard established for the facility under existing regulatory policies. Presumably a failure will be identified by the exceeding of a maximum permissible concentration for a particular chemical species in a regulatory monitoring well located at a compliance point or on a compliance surface. Among the possible compliance surfaces [Domenico and Palciauskas, 1982; LeGrand, 1981] are: (1) the outside boundary of the landfill, impoundment, or container; (2) the boundary of the physical plant of the waste-management facility; (3) the property boundary; or (4) a downstream aquifer, well field, stream, or lake. This study assumes a compliance surface of type (3) or (4) at a distance, x_s, from the edge of the landfill, where x_s is considerably greater than the dimensions of the landfill itself.

Failure requires three conditions. First, the containment structure must be breached. Next, the contaminant plume resulting from the breach must migrate to the compliance surface. Finally, the plume must escape detection by any monitoring system the owner-operator has installed.

For reasons that will become clear in Chapter 5, it is assumed that this migration will take place in the form of a contaminant plume with a steep concentration gradient at its front; that is, a plug flow with $C = C_o$ ahead of the front and $C = C_1$ behind, C_o being the ambient concentration of the particular contaminant species and C_1 its concentration in the plume. If the performance standard for this species lies between C_o and C_1, one can define the probability of failure in terms of the travel time of the contaminant from within the containment structure to the compliance surface. In fact, the probability of failure in year t, $P_f(t)$, is simply the probability that the time until breach of containment plus the travel time of the plume through the hydrogeological environment lies between t and t-1. If t = 0 is defined as the year in which the risk-cost-benefit analysis is carried out and $t = t_{op}$ as the year in which the facility is put into operation, then:

$$P_f(t) = 0 \quad \text{for } (0 < t < t_{op}) \tag{3.7}$$

$$P_f(t) = \Pr[(t'-1) < t^* + t^{**} < t'](1-P_d)$$

$$\text{for } (t_{op} < t < T_o)$$

where: $t' = t - t_{op}$,

$t^* =$ time until breach of containment [yr], and

$t^{**} =$ travel time of plume through hydrogeological environment to compliance surface [yr]

$P_d =$ probability of detection

If we assume that the time until breach of containment and the travel time of the plume are independent, the right side of Equation 3.7 can be rewritten as the product of three terms:

$$P_f(t) = \sum_{t'-1}^{T-t_{op}} Pr(t^*=t')Pr(t^{**}=t-t_{op}-t')(1-P_d) \qquad (3.8)$$

The assumption of independence is valid so long as the breach event does not significantly affect the groundwater flow system. This assumption is appropriate for relatively slow, low-volume leaks.

The three terms on the right side of Equation 3.8 are affected by site-exploration activities, containment-construction activities, and monitoring activities. Containment-construction activities affect the probability associated with breaching, $Pr(t^* = t')$, site-exploration activities affect the probability associated with plume migration, $Pr\ (t^{**} = t-t_{op}-t')$, and monitoring activities affect the probability of plume detection, P_d.

In Chapters 4, 5, and 6, the techniques used to estimate each of these three probability terms are developed.

4. RELIABILITY THEORY AND THE PROBABILITY OF CONTAINMENT BREACHES

The first event that must occur in the sequence leading to failure of a waste management facility is a breaching of the containment structure. Breaching can be caused by many complex and interacting processes. The causes of breaching, which are summarized in Table 4.1, include failures in design, construction, operation, and administration. A more detailed discussion of breaching mechanisms for containment structures consisting of synthetic liners is presented in Folkes [1982] and the U.S. EPA [1983].

The complexity of breaching mechanisms precludes using physically- based approaches for calculating the probabilities of individual breaching modes. Because of this, an empirical approach using time-dependent reliability theory is used to estimate breaching probabilities. This approach does not attempt to distinguish the mechanism of breach; it simply treats the time to the initiation of a contaminant source as a random variable. Physical attributes of the containment structure are included in the analysis, but the actual physical mechanisms of breaching are not considered.

The reliability equations used to describe the containment structure are developed in Section 4.1. The sensitivities of these equations to various input parameters are studied in Section 4.2. Section 4.3 presents a summary of assumptions and conclusions.

Table 4.1 - Causes of Breach of Containment at Waste-management Facilities Utilizing Synthetic Liners

Design Failures	1. Synthetic liners of insufficient permeability or thickness.
	2. Liner failure due to unexpected severity of stress-strain, freeze-thaw, or wet-dry cycles.
	3. Liner failure due to unexpected chemical interactions between liner, waste, and groundwater.
Construction Failures	1. Liner punctured, ripped or otherwise damaged during installation.
	2. Failure of liner construction to meet design specifications.
Operation Failures	1. Liner damage during operation due to compaction, roots, animals, etc.
	2. Failure of leachate control systems due to equipment breakdown or power failure.
Administrative	1. Lack of manpower or capital to carry out construction and operation.
	2. Failure of quality-control programs.
	3. Loss of administrative control due to bankruptcy.

4.1 Modeling the Waste Management Facility

The waste management facility modeled in the present study is illustrated in Figure 4.1. The system consists of one or more units or cells. The wastes in each cell are contained by one or more synthetic liners. Each cell will function so long as at least one liner is functioning and the complete system will function so long as all cells are functioning.

Reliability theories provide a method for predicting system performance as a function of component performance [Ross, 1980]. Consider a system of n components and let:

$$x_i=1 \text{ if component } i \text{ is functioning} \qquad (4.1)$$

$$x_i=0 \text{ if component } i \text{ has failed.}$$

The system can be defined with a state vector:

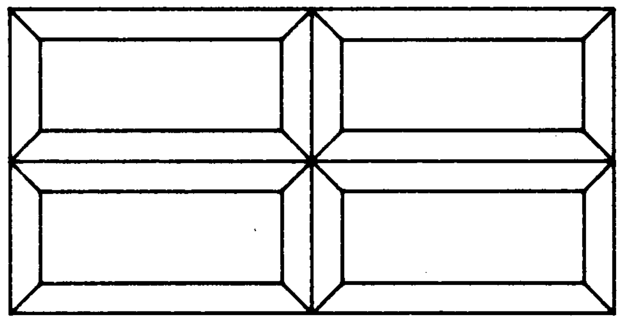
$$x'=(x_1,x_2,........x_n). \qquad (4.2)$$

The performance of the system can be described with a structure function:

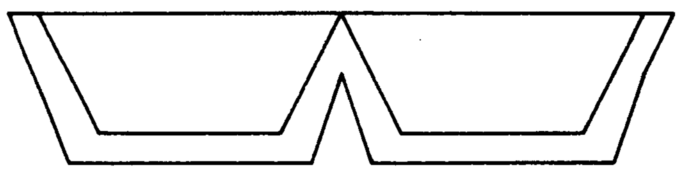
$$S(x')=1 \text{ if system is functioning} \qquad (4.3)$$

$$S(x')=0 \text{ if system has failed.}$$

The structure function reflects the way components are configured. The simplest configurations are the series structure and the parallel structure. For the series structure, which is diagramed in Figure 4.2a, the system will perform only if all components perform. The structure function for the series



a) Plan view



b) Cross-section view

Figure 4.1 - Plan and Cross-Section Views of Waste Management Facility

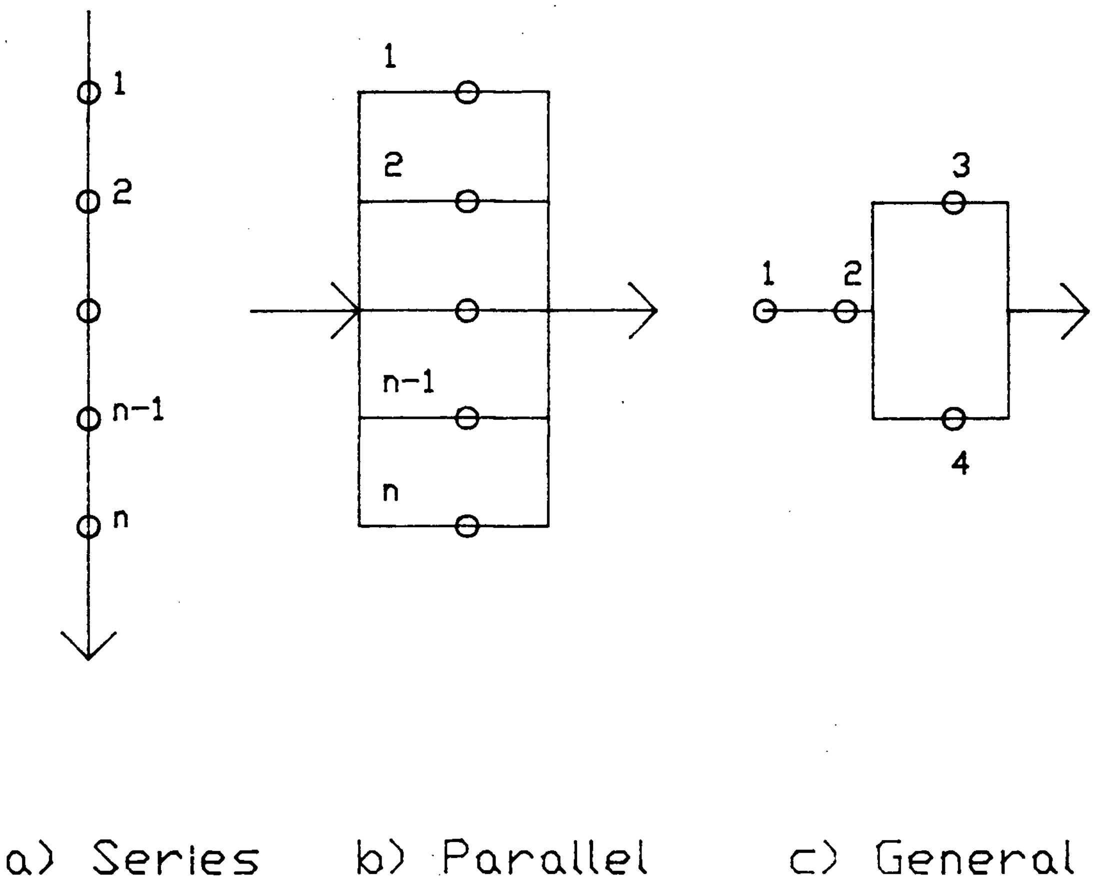


Figure 4.2 - Example System Configurations

configuration is:

$$S(x')=Min(x_1,x_2,.......x_n) \tag{4.4}$$

The system will have failed if $x_i=0$ for any i.

For the parallel structure, diagramed in Figure 4.2b, the system will perform if any of the components perform. The structure function for the parallel configuration is:

$$S(x')=Max(x_1,x_2,.......x_n). \tag{4.5}$$

The system will function if $x_i=1$ for any i.

The more general component configuration, diagramed in Figure 4.2c, consists of a combination of series and parallel structures. Structure functions, though somewhat more complex, can be developed for these more general configurations. For example, the structure function for the configuration shown in Figure 4.2c is

$$S(x')=x_1x_2Max(x_3,x_4). \tag{4.6}$$

The reliability of a system is defined as the probability that the system performs:

$$r=Pr[S(x')=1]. \tag{4.7}$$

If we define p_i as the probability that component i performs, then

$$P_i=Pr[x_i=1]=1-Pr[x_i=0] \tag{4.8}$$

Reliability theories offer techniques for determining the reliability of a system, r, as a function of individual component probabilities, p_i. These functional relationships are prohibitively complex, even for relatively simple systems, unless one assumes components perform independently. The implications of assuming independence for the components of the waste management facility are discussed in Section 4.3.

The reliability function for n independent components in a series configuration is given by:

$$r(p')=\Pr[x_i=1 \text{ for all } i]= \prod_{i=1}^{n} p_i \tag{4.9}$$

For a parallel configuration, the reliability function is given by:

$$r(p')=1-\Pr[x_i=0 \text{ for all } i] = 1 - \prod_{i=1}^{n} p_i \tag{4.10}$$

In the general case, the probability that a component is functioning will be time dependent. If we assume that individual components function for a random length of time and then permanently fail, this time-dependent reliability is:

$$\begin{aligned} p_i(t)&=\Pr[\text{component } i \text{ is functioning at time } t] \\ &=\Pr[\text{lifetime of } i > t] \end{aligned} \tag{4.11}$$

If the cumulative distribution function for the life of component i is denoted by $F_i(t)$, then:

$$P_i(t) = 1 - F_i(t) = \overline{F}_i(t) \tag{4.12}$$

The term $F_i(t)$ is defined as the reliability function of component i. The reliability of a system of n components in a series configuration is obtained by combining equations (4.9) and (4.12):

$$
\begin{aligned}
F_s(t) &= \Pr[\text{system lifetime} > t] \\
&= 1 - F_s(t) \\
&= \prod_{i=1}^{n} F_i(t) \qquad (4.13)
\end{aligned}
$$

For a parallel configuration, the reliability function is obtained by combining equations (4.10) and (4.12):

$$\overline{F}_s(t) = 1-\prod_{i=1}^{n} (1 - \overline{F}_i(t)) \qquad (4.14)$$

For the waste management facility comprised of waste cells and synthetic liners, the distribution function for the complete system can be developed from equations (4.13) and (4.14). To begin, we assume that if a waste cell has more than one liner, these liners are configured in an independent parallel structure, so that the cell will function so long as at least one liner functions. The probability that cell i functions longer than time t is given by:

$$\overline{F}_i(t) = 1 - \prod_{j=1}^{m_i} F_{ji}(t) \qquad (4.15)$$

where

$\overline{F}_i(t)$ = probability the life of cell i is greater than t,

m_i = number of synthetic liners in waste cell i, and

$F_{ji}(t)$ = probability the life of liner j in cell i is less than t.

Next, we assume the cells are configured in an independent series structure so that the complete landfill system will function so long as all cells function. The probability that the system functions longer than time t is given by:

$$\overline{F}_s(t) = \prod_{i=1}^{n} \overline{F}_i(t) \tag{4.16}$$

Combining equations (4.15) and (4.16) gives:

$$\overline{F}_s(t) = \prod_{i=1}^{n} [1 - \prod_{j=1}^{m_i} F_{ji}(t)] \tag{4.17}$$

Equation (4.17) allows the reliability function for the waste management system to be determined from the probability distribution functions for the individual liners. However, estimating these liner distribution functions can be problematic.

A large number of distribution forms have been proposed for liner- type components. One of the more general forms, illustrated in Figure 4.3, is the "human mortality" curve [Stark and Nicholls, 1972). The first part of the curve represents early failures which may result from construction and installation inadequacies. This initial mortality is followed by a period in which failures are due primarily to events that have an equal chance of occurring in any given year. Eventually, failure will occur as a result of "old-age" or wear, represented by the bell-shaped portion of the curve.

The functional form of the probability distribution shown in

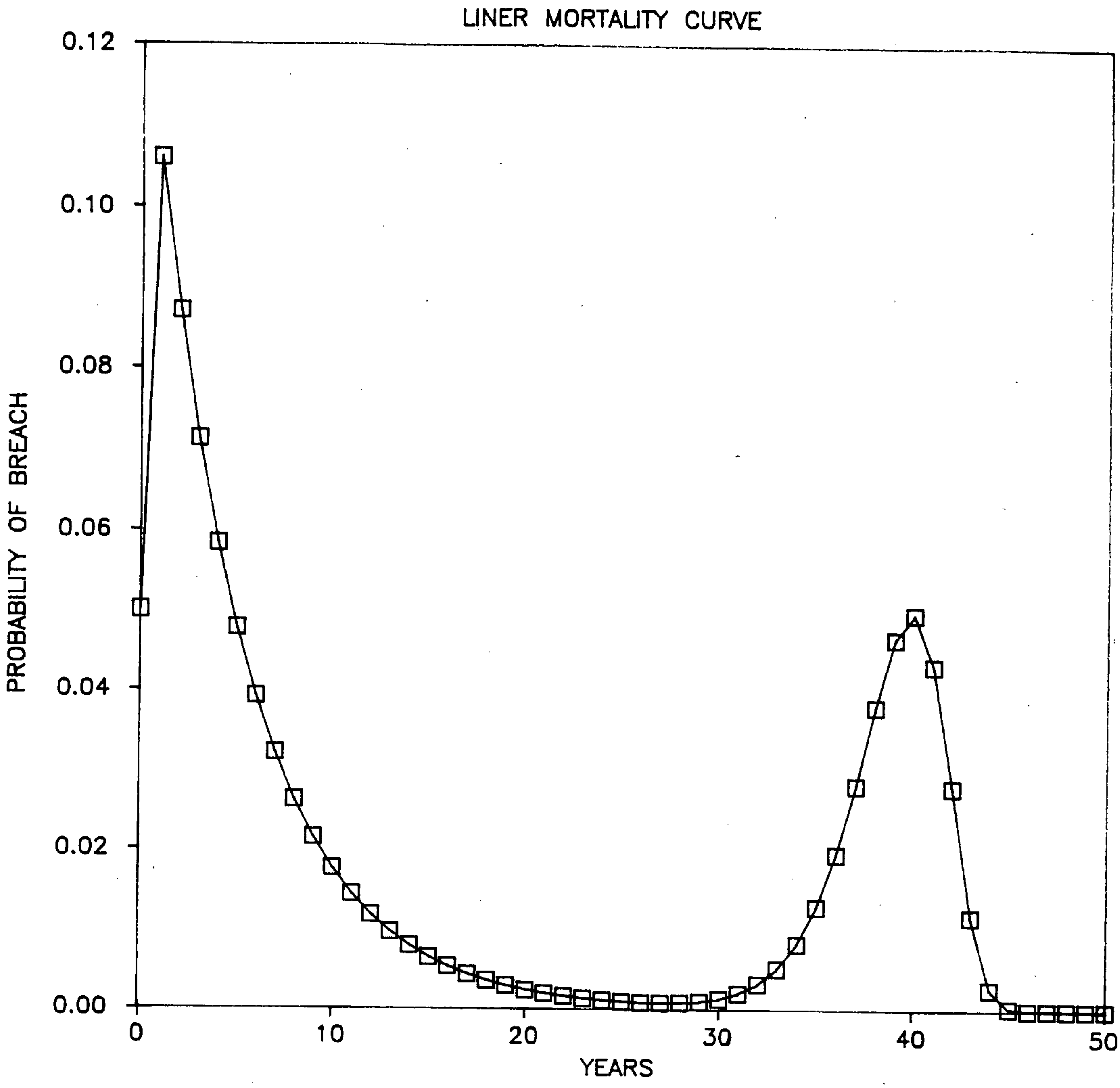


Figure 4.3 - Liner Mortality Curve

Figure 4.3 is [Chu et al, 1983]:

$$f(t) = a\,\delta(t) + b\lambda e^{-\lambda t} + c\alpha\beta(\alpha t)^{\beta-1}e^{(-\alpha t)^{\beta}} \qquad (4.18)$$

where

$\delta(t)$ = Dirac delta function,

λ = rate constant for early breaches,

α = rate constant for late breaches,

β = shape factor for late breaches,

t = time, and

a,b,c = weighting coefficients.

To be a proper probability density function, the integral of equation (4.18) from time zero to infinity must equal one. This condition is guaranteed if the sum of the weighting coefficients equals one [Chu et al, 1983]:

$$a + b + c = 1.0 \qquad (4.19)$$

The first term on the right side of equation (4.18) represents breaches due to manufacturing and installation failures. The second term on the right side of equation (4.18), which represents breaching due to events that have an equal annual probability of occurrence, is the exponential distribution. Example shapes for different values of λ are illustrated in Figure 4.4. The third term on the right side of equation (4.18), which represents breaches due to wear, is the Weibull distribution. Example shapes for different values of α and

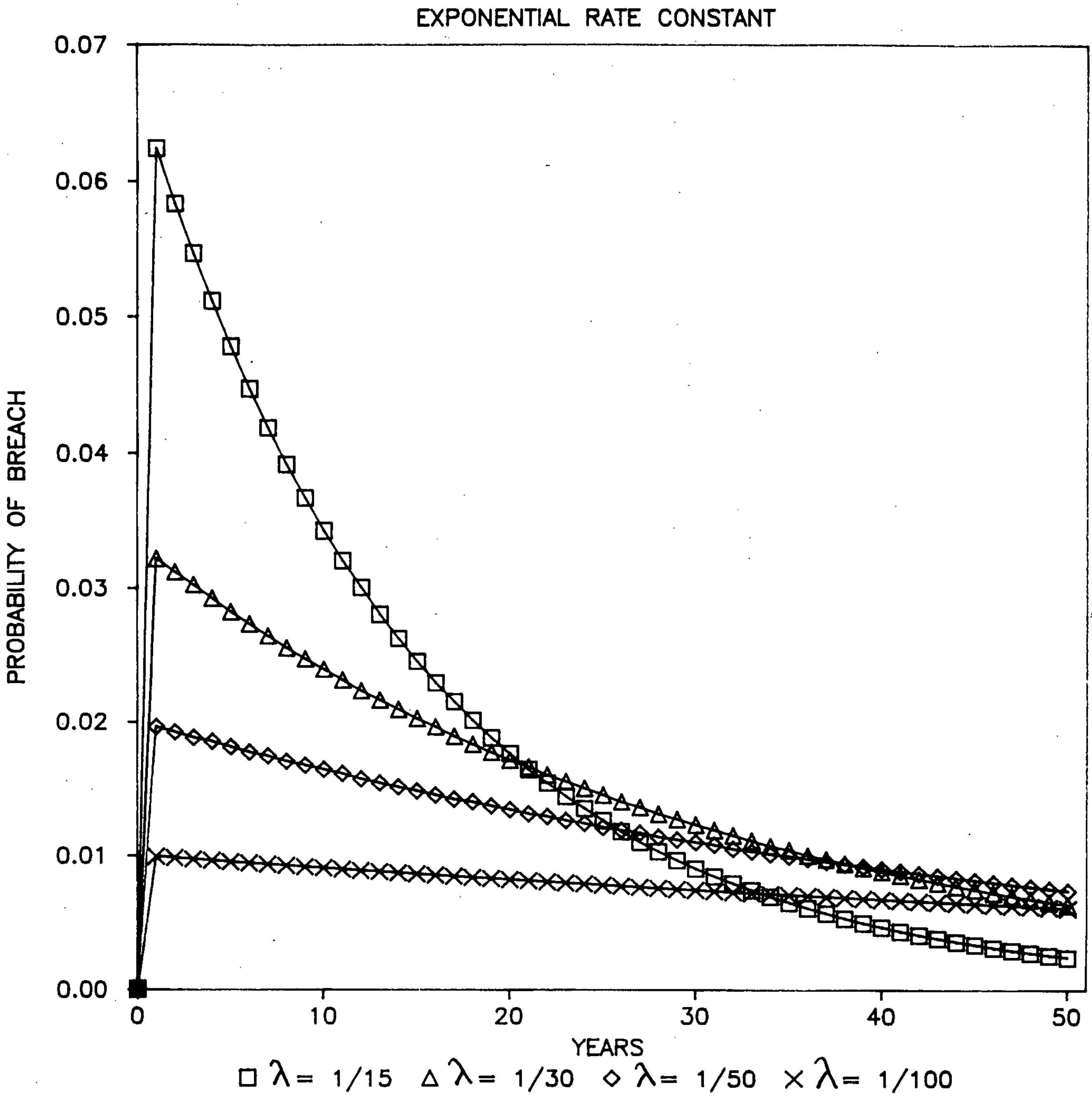


Figure 4.4 - Exponential Distribution with Different Rate Constants

β are shown in Figure 4.5.

The reliability of a liner with a probability density function given by equation (4.18) is:

$$\overline{F}_{ji}(t) = 1 - F_{ji}(t) = 1 - \int_0^t f(t)dt \tag{4.20}$$

$$\overline{F}_{ji}(t) = b_{ji}\exp[-\lambda_{ji}(t-t_i')] + c_{ji}\exp[-\alpha_{ji}(t-t_i')^{\beta_{ji}}]$$
$$\text{for} \quad t > t_i'$$

$$\overline{F}_{ji}(t) = 1 - a_{ji} \qquad \text{for } t = t_i'$$

where

$\overline{F}_{ji}(t)$	=	reliability of liner j in waste cell i,
t_i'	=	year cell i begins operation,
λ_{ji}	=	exponential rate constant for liner j in cell i,
α_{ji}	=	Weibull rate constant for liner j in cell i,
β_{ji}	=	Weibull shape factor for liner j in cell i, and
a_{ji}, b_{ji}, c_{ji}	=	weighting coefficients for liner j in cell i.

The mean or expected value for a random variable with a probability density function given by $f_{ij}(t)$ is defined as

$$\mu_{ji} = \int_0^\infty t f_{ji}(t)dt \tag{4.21}$$

The expected life of a synthetic liner with a probability density function given by equation (4.18) is obtained by combining equations (4.18) and (4.21). The result of the integration is given by the following expression [Stark and Nicholls, 1972]:

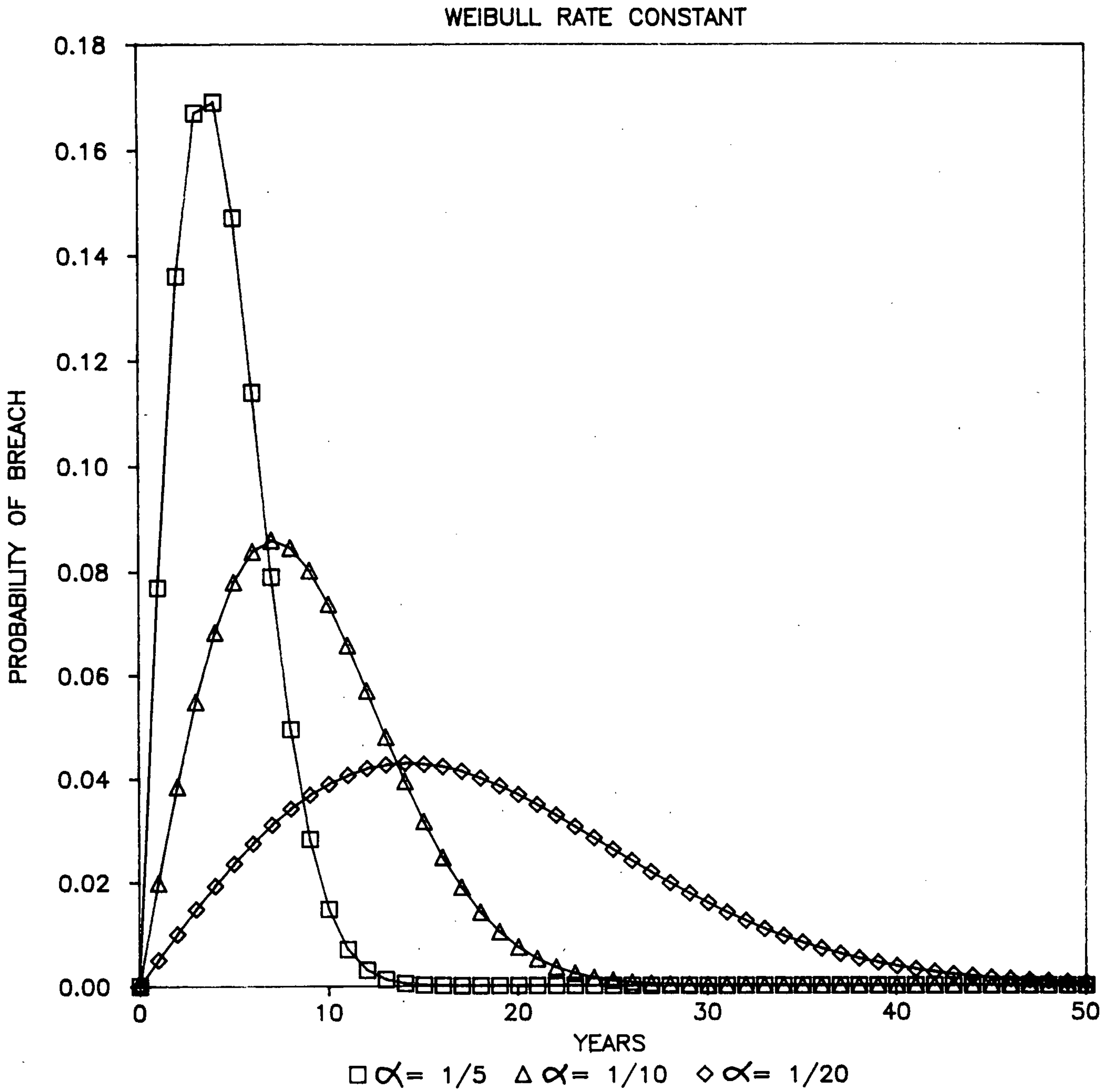


Figure 4.5 - Weibull Distribution with Different Rate Constants

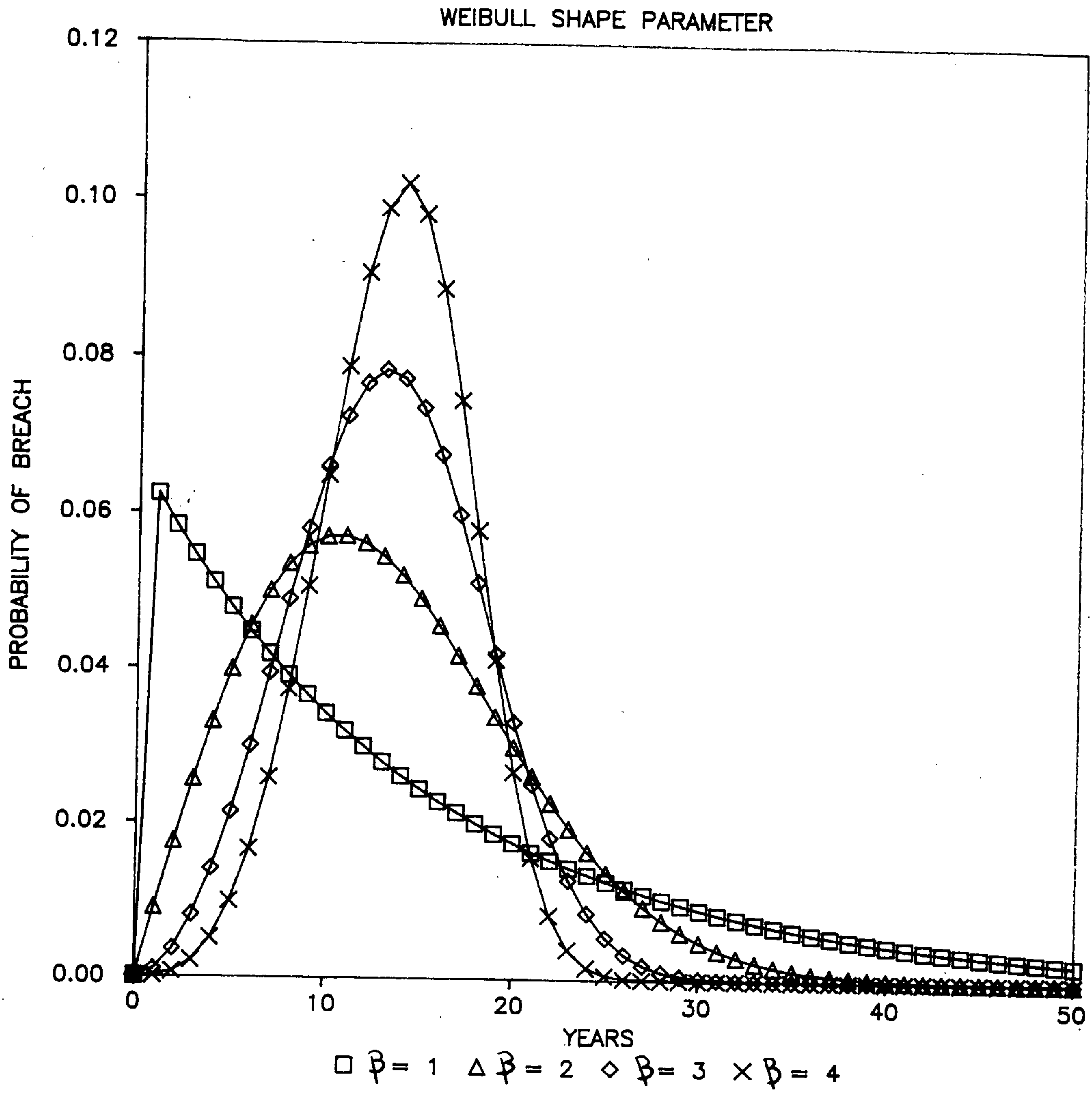


Figure 4.6 - Weibull Distribution with Different Shape Factors

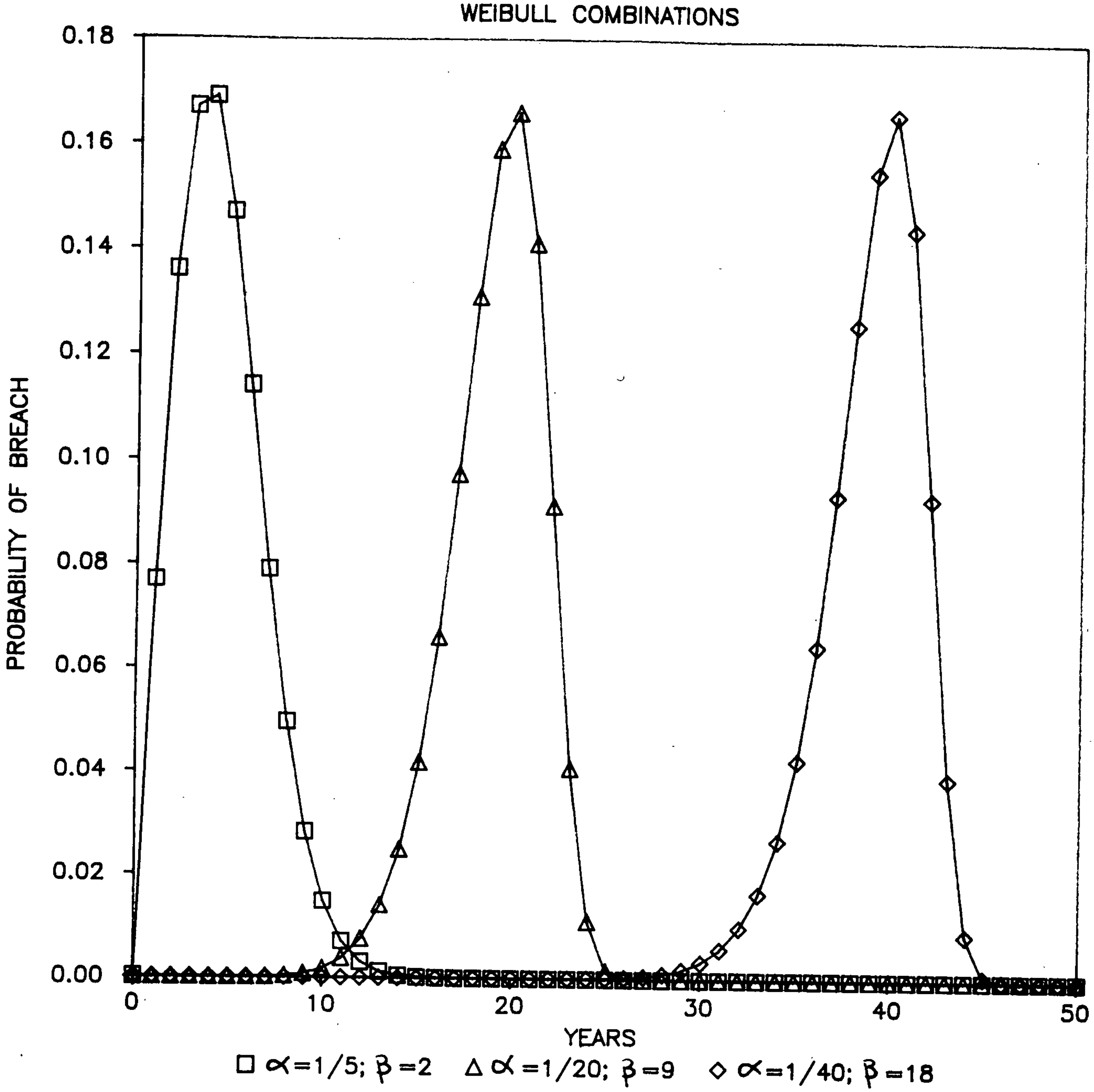


Figure 4.7 - Weibull Distribution with Different Combinations of Rate Constants and Shape Factors

$$\mu_{ji} = (b_{ji}/\lambda_{ji}) + c_{ji}(\Gamma(1+1/\beta_{ij})/\alpha_{ij}) \tag{4.22}$$

where

$\Gamma(\quad) =$ gamma function.

Tables giving values of the gamma function can be found in most mathematical handbooks. For applications in this dissertation, the value of $1+1/\beta_{ij}$ will range between 1 and 2. Table 4.2 presents gamma function values within this range.

Combining equations (4.20) and (4.17) gives the reliability function for the complete waste management system:

$$\overline{F}_s(t) = \prod_{i=1}^{n} [1 - \prod_{j=1}^{m_i} (1 - b_{ji}\exp[-\lambda_{ji}(t-t_i')] - c_{ji}\exp[-(\alpha_{ji}(t-t_i')^{\beta_{ji}}] \qquad t > 0 \tag{4.23}$$

$$\overline{F}_s(t) = \prod_{i=1}^{n} [1 - \prod_{j=1}^{m_i} a_{ji}] \qquad t = 0$$

Equation (4.23) simplifies to the following expression if each liner and cell are assumed to behave identically:

$$\overline{F}_s(t) = [1 - (1-be^{-\lambda t} - ce^{-(\alpha t)^{\beta}})^m]^n \qquad t > 0 \tag{4.24}$$

Table 4.2 - Gamma Function Values for Arguments between 1.0 and 2.0 [after Kreysig, 1983].

a	(a)	a	(a)
1.00	1.0000	1.60	0.8935
1.10	0.9514	1.70	0.9086
1.20	0.9182	1.80	0.9314
1.30	0.8975	1.90	0.9618
1.40	0.8873	2.00	1.0000
1.50	0.8862		

$$\overline{F}_s(t) = (1 - a^m)^n \qquad t = 0$$

The objective function developed in Chapter 2 is in terms of annual benefits, costs, and risks. To calculate annual risks, the annual probability of breaching for the waste management system must be estimated. This probability is given by:

$$\begin{aligned} Pr(t^* = t) &= F_s(t) - F_s(t-1) \\ &= \overline{F}_s(t-1) - \overline{F}_s(t) \end{aligned} \tag{4.25}$$

4.2 Probability of Breaching: Sensitivity Studies

The parameters used in equation (4.23) to describe the waste management system are 1) the number of waste cells, n; 2) the year waste cell i begins operation, t_i'; 3) the number of synthetic liners in each waste cell, m_i; and 4) parameters defining the probability-distribution functions for the lifetimes for each synthetic liner. The first three items are directly specified by the owner-operator. Item 4), however, requires some interpretation.

The performance of each synthetic liner can be characterized with the six parameters presented in Table 4.3. Choosing actual values for these parameters is necessarily subjective, especially considering that most liner materials used for hazardous waste containment have been commercially available for less than 15 years.

In a report prepared by the U.S. EPA [1983], an average service life for synthetic liners in hazardous waste applications was estimated to range from 5 to 45 years, depending on the type of liner used and the type of waste contained. A case study involving 27 failures in 384 years of operation at 39 sites [Burman et al, in press] indicated the average life of a landfill before a breach of containment occurred was 14 years. Overall, an average service life of 15 years appears to be a reasonable estimate.

For the base case described in Table 4.3, the values of the

Table 4.3 - Parameters Used to Characterize Liner Performance

Parameter	Units	Range	Interpretation	Base Case
a	none	0 - 1.0	Probability the liner has failed prior to operation as a result of manufacture or installation inadequacies.	0.05
b	none	0 - 1.0	Probability liner fails as a result of events that have an equal chance of occurring in any year.	0.65
c	none	0 - 1.0	Probability liner fails as a result of "old age" or wear.	0.3
	yr^{-1}	0.01 - 1	Annual probability that an event occurs which will cause the liner to fail prematurely.	0.2
	none	1 - 50	Shape parameter for curve representing failure due to wear. The spread of the curve will decrease as increases.	18
	yr^{-1}	.01 - .05	Measure of the expected life of liners that fail due to wear.	0.025

parameters are as follows: a=.05, b=.65, c=.30, λ =.20 (1/yr), β =18, and α =.025 (1/yr). For these values, the expected life is life given by:

$$\mu_{ji} = .65(5)+0.3(40) \quad (1.056)$$
$$= .65(5)+0.3(40).969$$
$$= 14.9 \text{ years} \qquad (4.26)$$

In Figure 4.8, the probabilities of breach for a single waste cell with one, two, and three synthetic liners are compared. The attributes of each liner are described by the base case in presented in Table 4.3. The effect of additional liners is to reduce the number of early breaches due to events that have equal annual probabilities of occurrence. An inescapable fact, however, is that reducing the probability of early breaches increases the probability of late breaches. The total probability of breach over an infinite time period must equal 1.0. This very fundamental principal seems to be often overlooked in many analyses.

Figure 4.9 compares breaching probabilities for one, two, and three waste cells each lined with a single synthetic membrane. The effect of additional cells is to increase the number of early breaches while decreasing the number of late breaches.

From an owner-operator's point of view, late breaches are much preferred to early breaches because of the effects of discounting described in Chapter 3. Future risks are less significant to the owner-operator than present risks. Figure 4.10 illustrates how

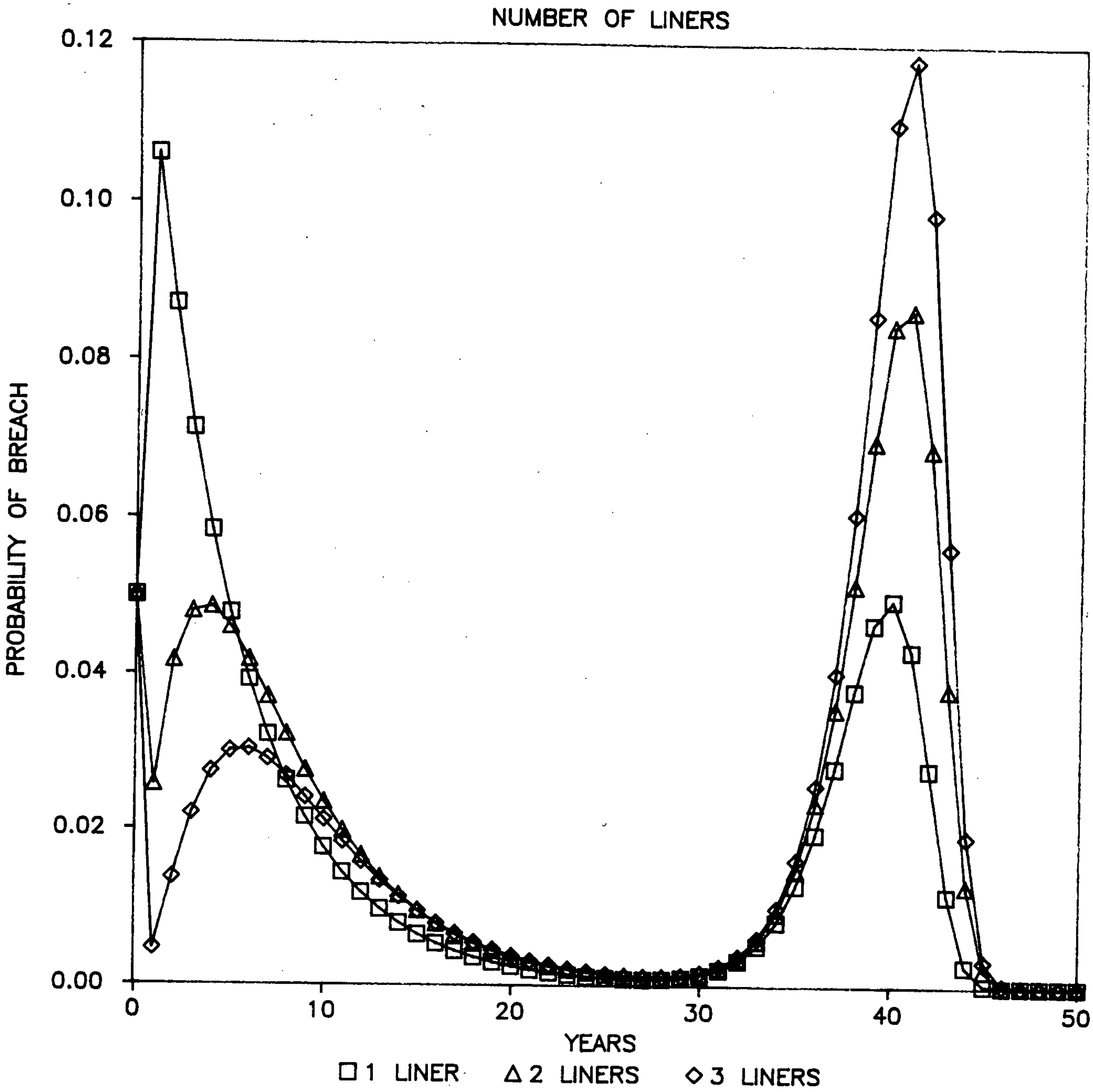


Figure 4.8 - Probability of Breach for Different Number of Liners

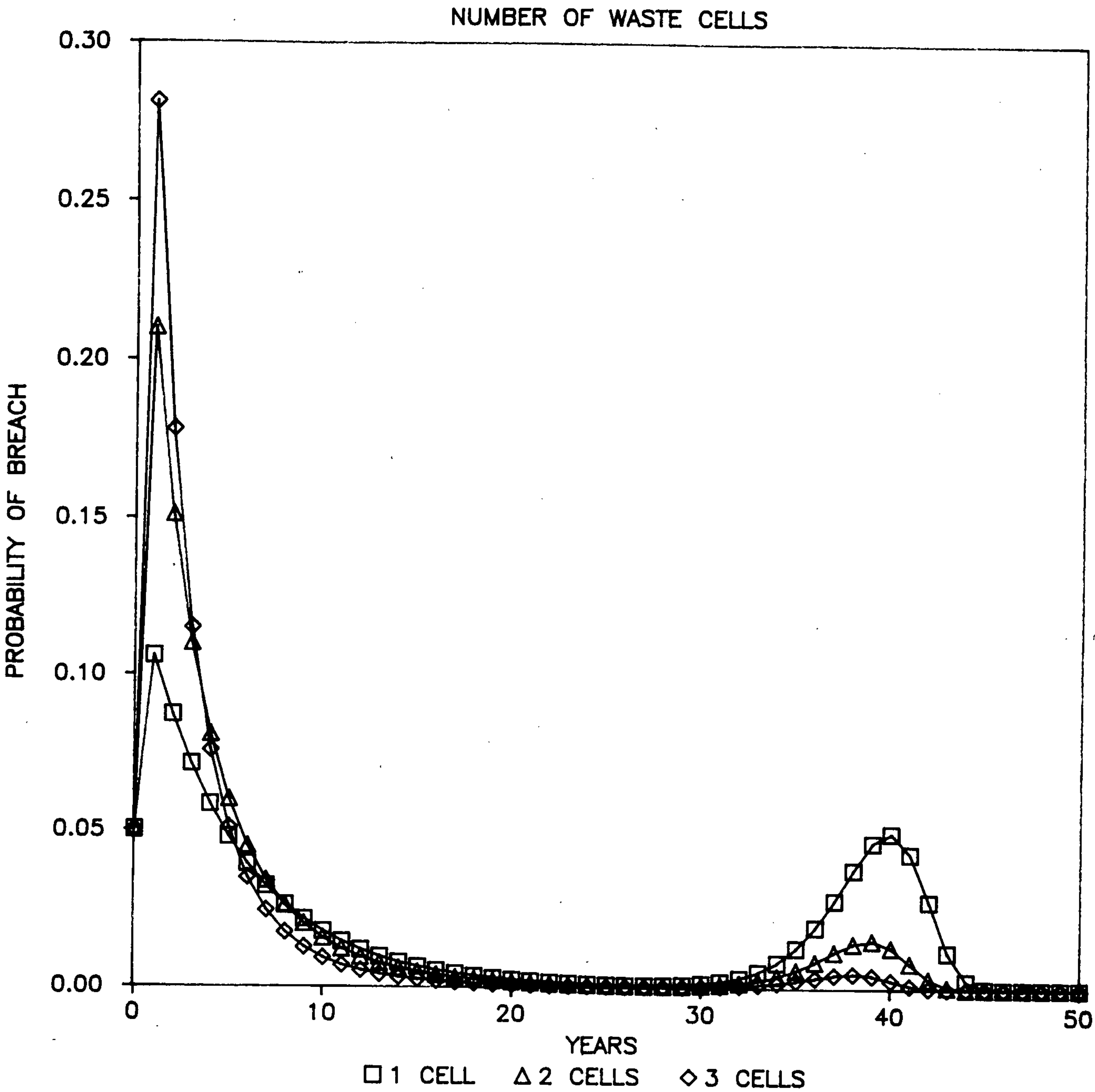


Figure 4.9 - Probability of Breach for Different Number of Cells

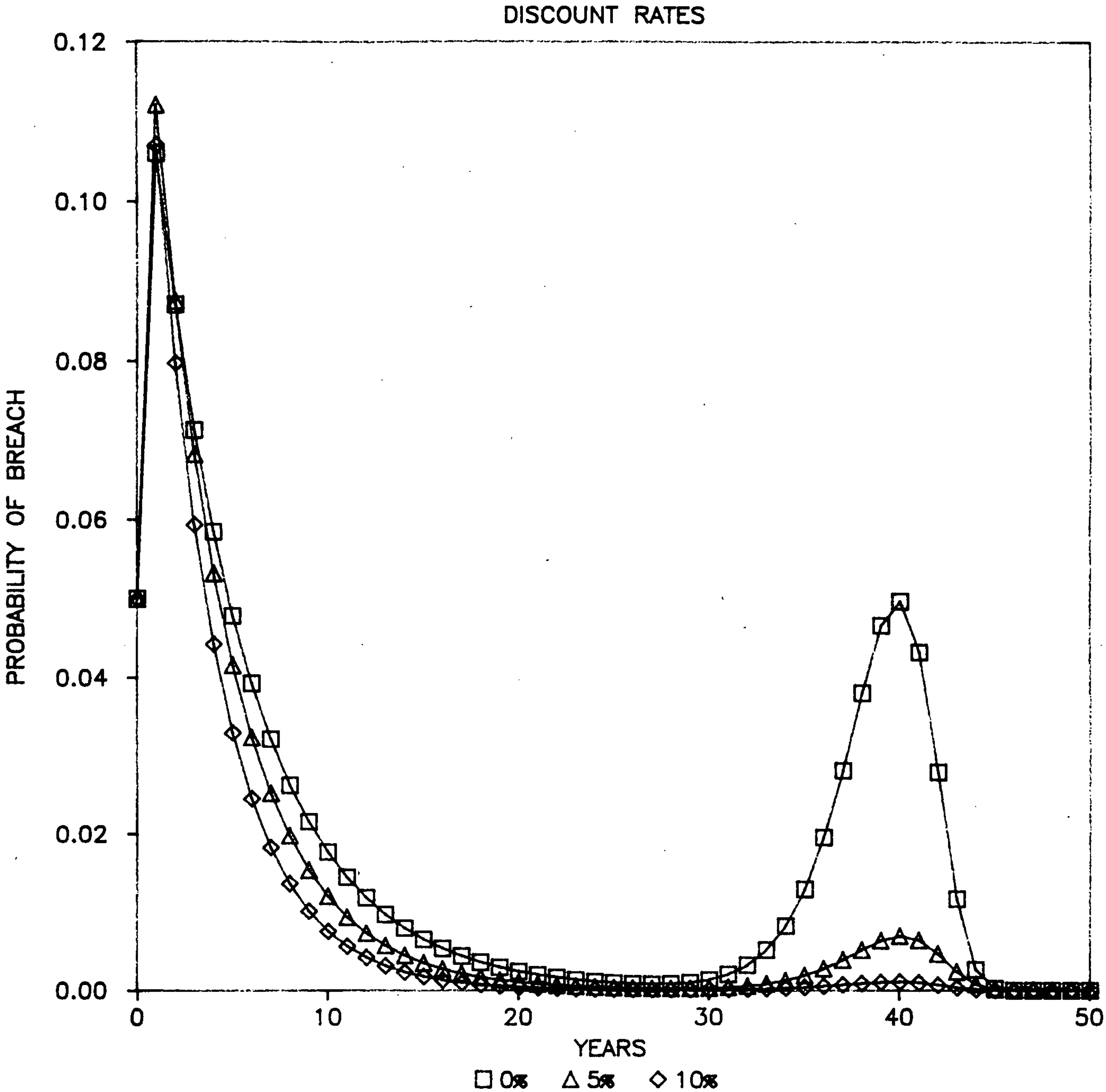


Figure 4.10 - Effects of Discount Rates on Probability of Breaching

the risks of future breaches are affected by various discount rates. With a discount rate of 10 percent, breaches due to degradation or wear are nearly insignificant.

From society's point-of-view, however, late failures may be as undesirable as early failures. In fact, early failures may be preferred since the responsible parties can be more easily identified.

Since breaches due to wear or degradation have limited effects upon an owner-operator's decision process, the remaining examples in this section investigate early breaches whose probabilities are governed by the exponential distribution. The performance of each liner can be described with a single parameter: the exponential rate constant. Recall that this constant can be considered as the annual probability that an event occurs which will cause the liner to fail prematurely.

Another change that will be made in the remaining examples is the way in which results are presented. The logarithm of the probability of breach will be plotted as a function of time in the remaining figures. With this format, the plots will be linear for all single-liner systems.

Figure 4.11 compares the effects of exponential rate constants. The slope of the function is directly proportional to the magnitude of the rate constant. As the probability of an event which causes breaching increases, the slope of the plot steepens.

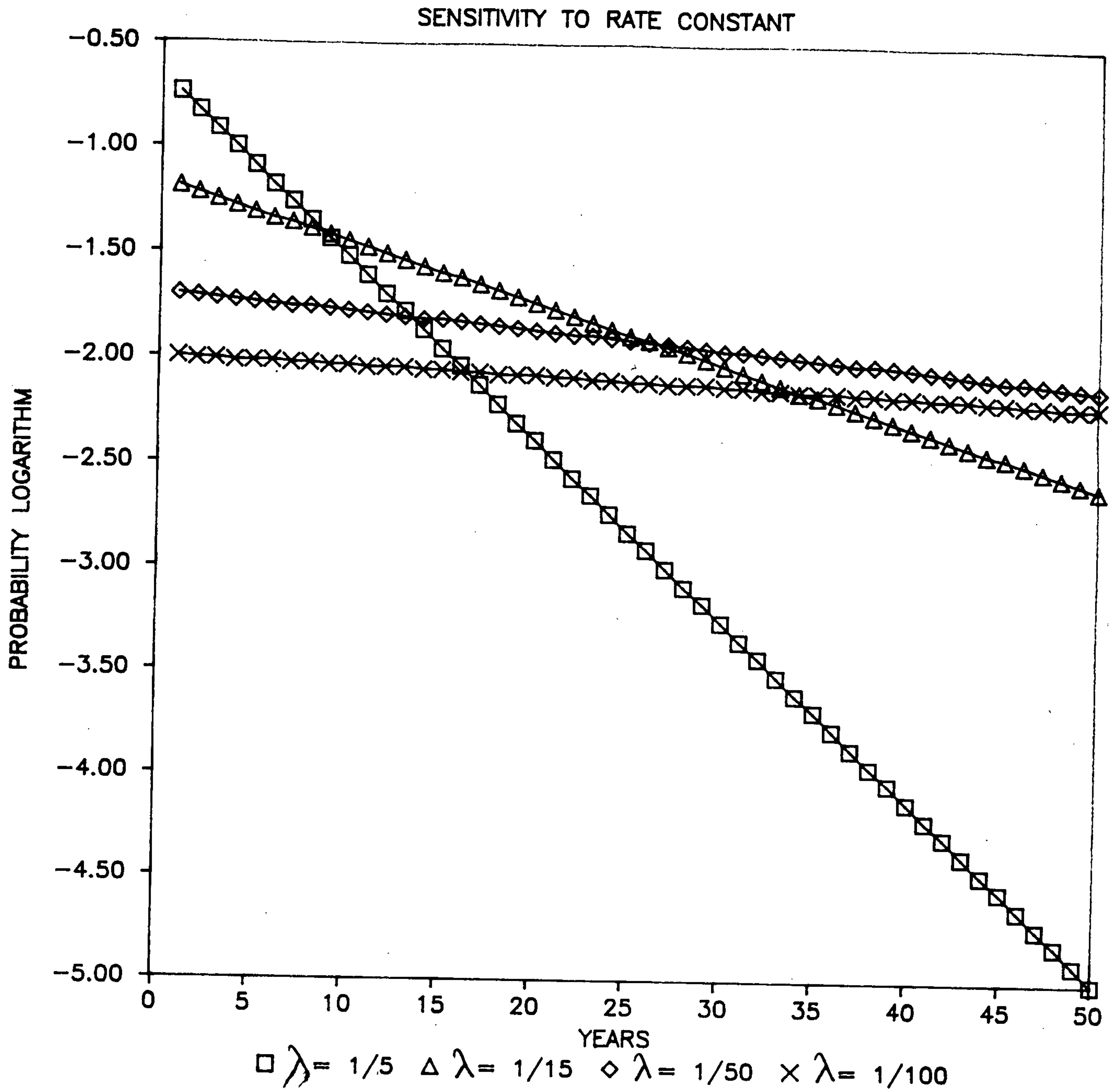


Figure 4.11 - Effect of Rate Constant on Probability of Breaching Assuming Exponential Distribution

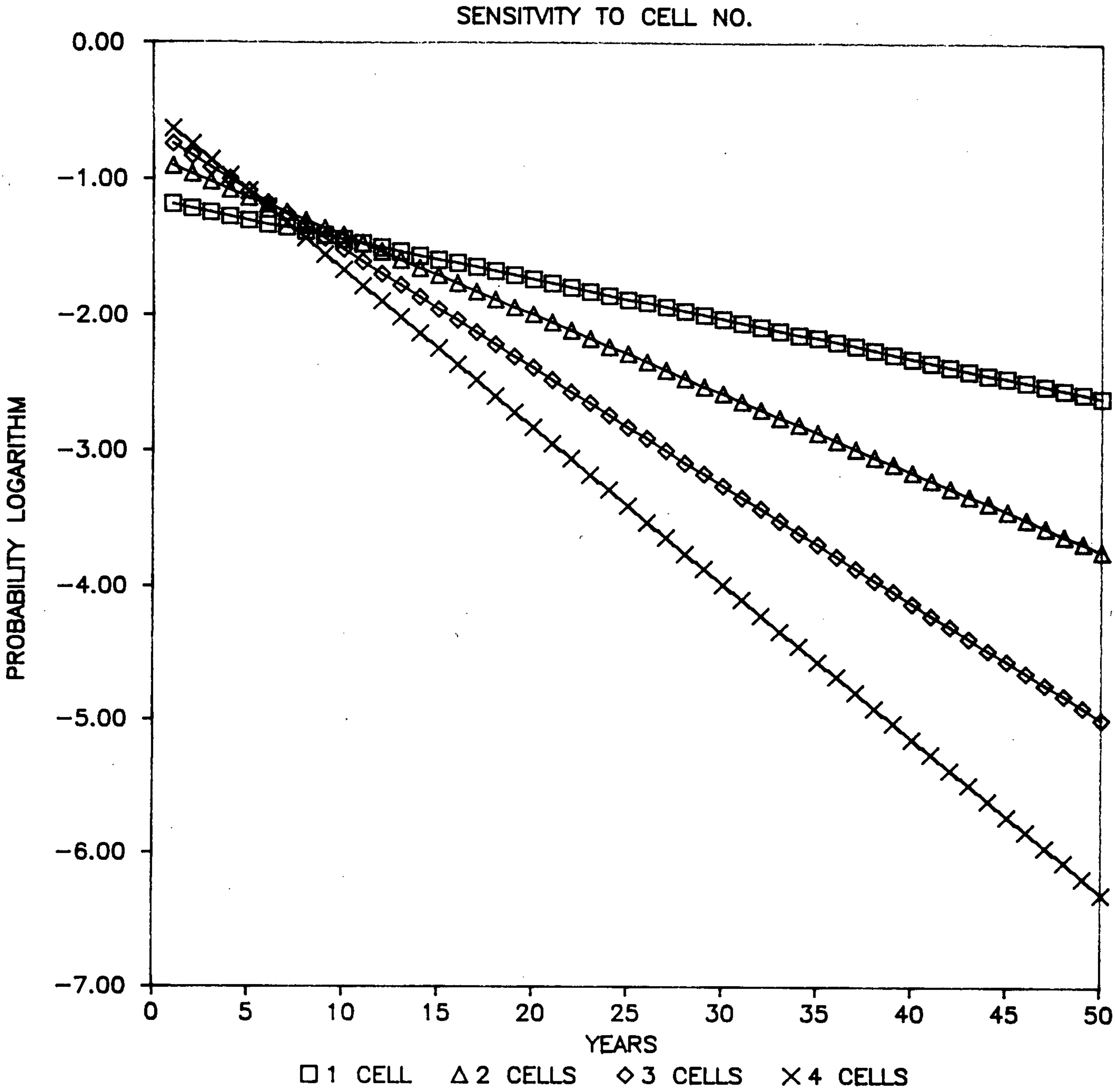


Figure 4.12 - Effect of Number of Cells on Probability of Breaching Assuming Expontial Distribution

In Figure 4.12, the effects of the number of waste cells are compared. Increasing the number of cells causes the slope of the plot to steepen in a manner similar to the effects of the rate constant. In fact, two cells, each with a rate constant of .03 (1/yr) will perform identically to a single cell with a rate constant of .06 (1/yr). If we assume the rate constant is proportional to the area of the waste cells, then we are implicitly assuming that events which cause early failures tend to be areally distributed on a local scale. This may be a valid assumption for breaches due to such things as flaws in liner manufacture or installation, differential settlements, or burrowing animals or roots. It is not valid for breaches due to events which do not depend upon area, such as earthquakes or administrative failures.

Figure 4.13 illustrates how the probability of breach is affected by the year in which a second waste cell begins operation. A second cell has two effects upon the breach curve: 1) a discontinuity in which the probability of breach jumps up in the year the second cell is installed, and 2) an increase in the slope. It is interesting to note that the size of the discontinuity is the same for each case. For the cell beginning operation in year 5, the logarithm of probability jumps from -1.28 to -1.02 while for the cell beginning in year 20 the jump is from -1.71 to -1.45. In terms of actual probability, however, the jump at five years represents an increase from 0.05 to 0.09 and the jump at 20 years represents an increase from 0.02 to

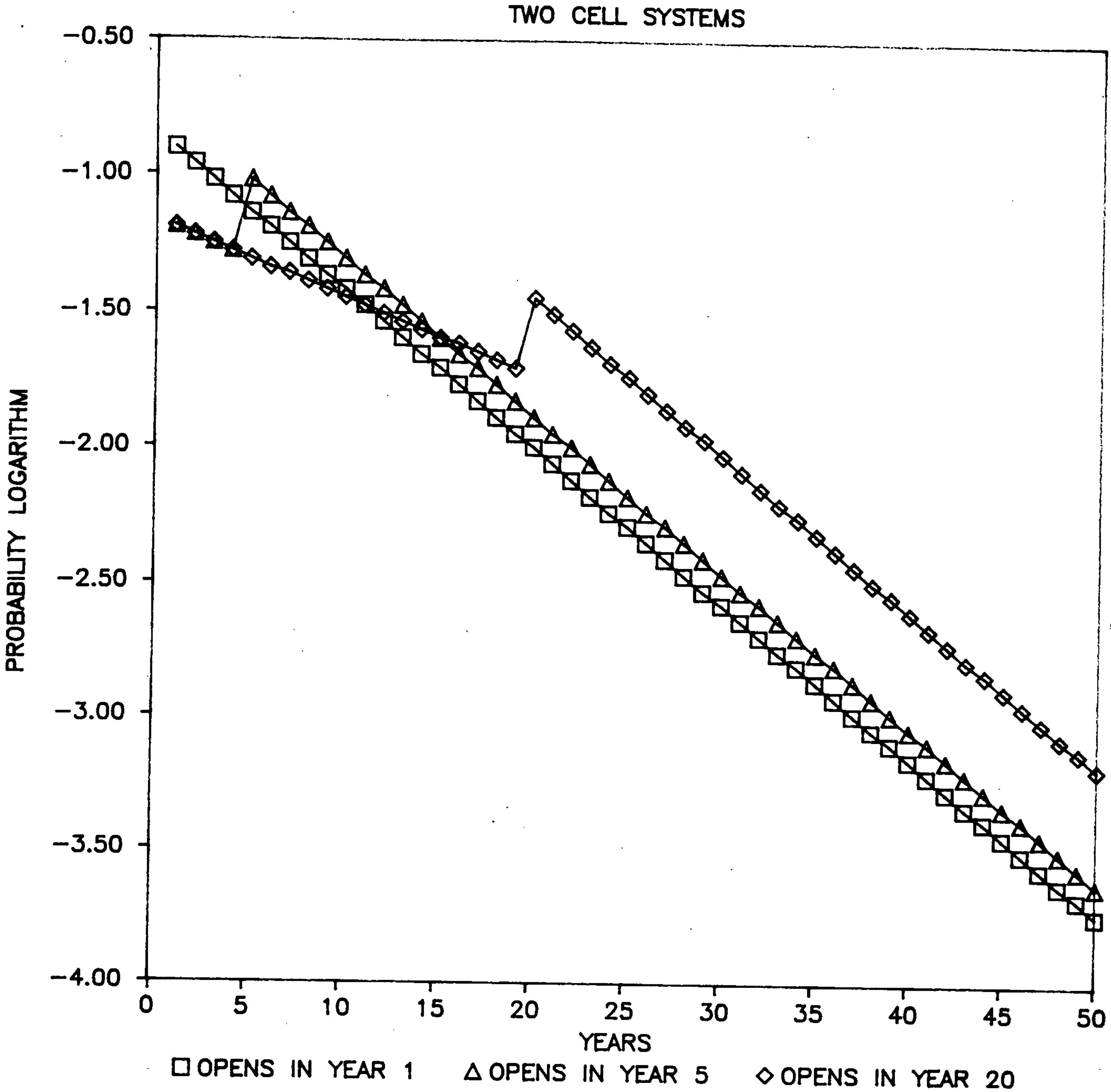


Figure 4.13 - Effects of the Year that Second Cell Begins Operation on Probability of Breaching for Cells with One Liner

0.04. Cells installed at later times have less effect on the overall system performance, even without considering the effects of discounting.

It is assumed in Figure 4.13 that the rate constant for the liner in the second cell is the same as that for the first cell. Figure 4.14 shows the effects of different rate constants for the liner in the second cell. This second cell is assumed to begin operation in year 10. As the rate constant increases, the size of the discontinuity increases and the slope steepens.

For multiple liner systems, the probability of breach is no longer a linear function of time, as shown in Figure 4.15. Additional liners reduce the probability of early breaches and increase the probability of late breaches. From an owner-operator's point of view they improve facility performance by shifting breaches into the future. Similar behavior was noted earlier when the complete mortality curve was considered.

Finally, Figure 4.16 illustrates the effect of the year in which a second waste cell begins operation. The second cell causes the curve to steepen and shifts the peak of the curve to the right.

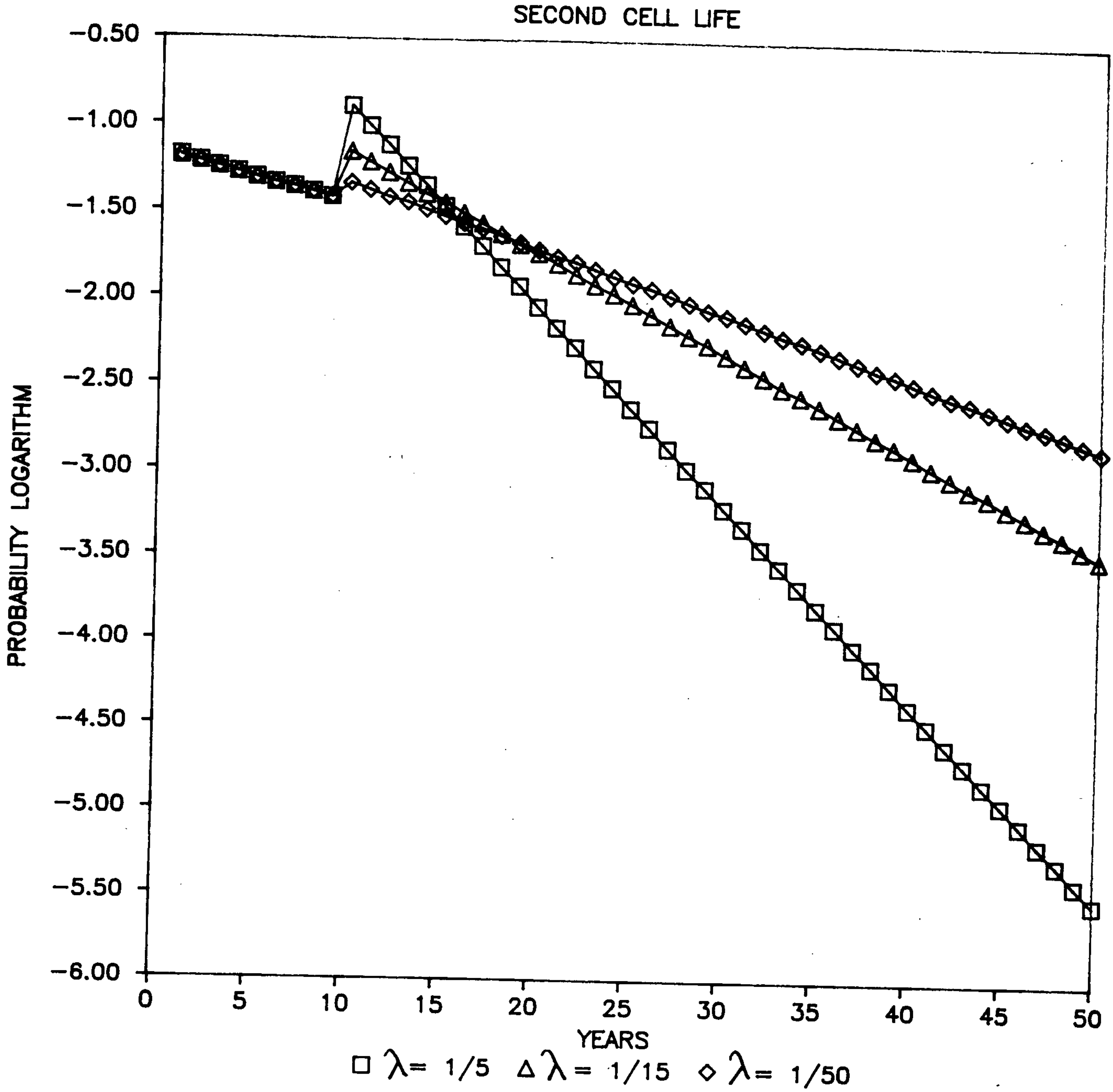


Figure 4.14 - Effects of Rate Constant for Liner in Second Waste Cell on Probability of Breaching

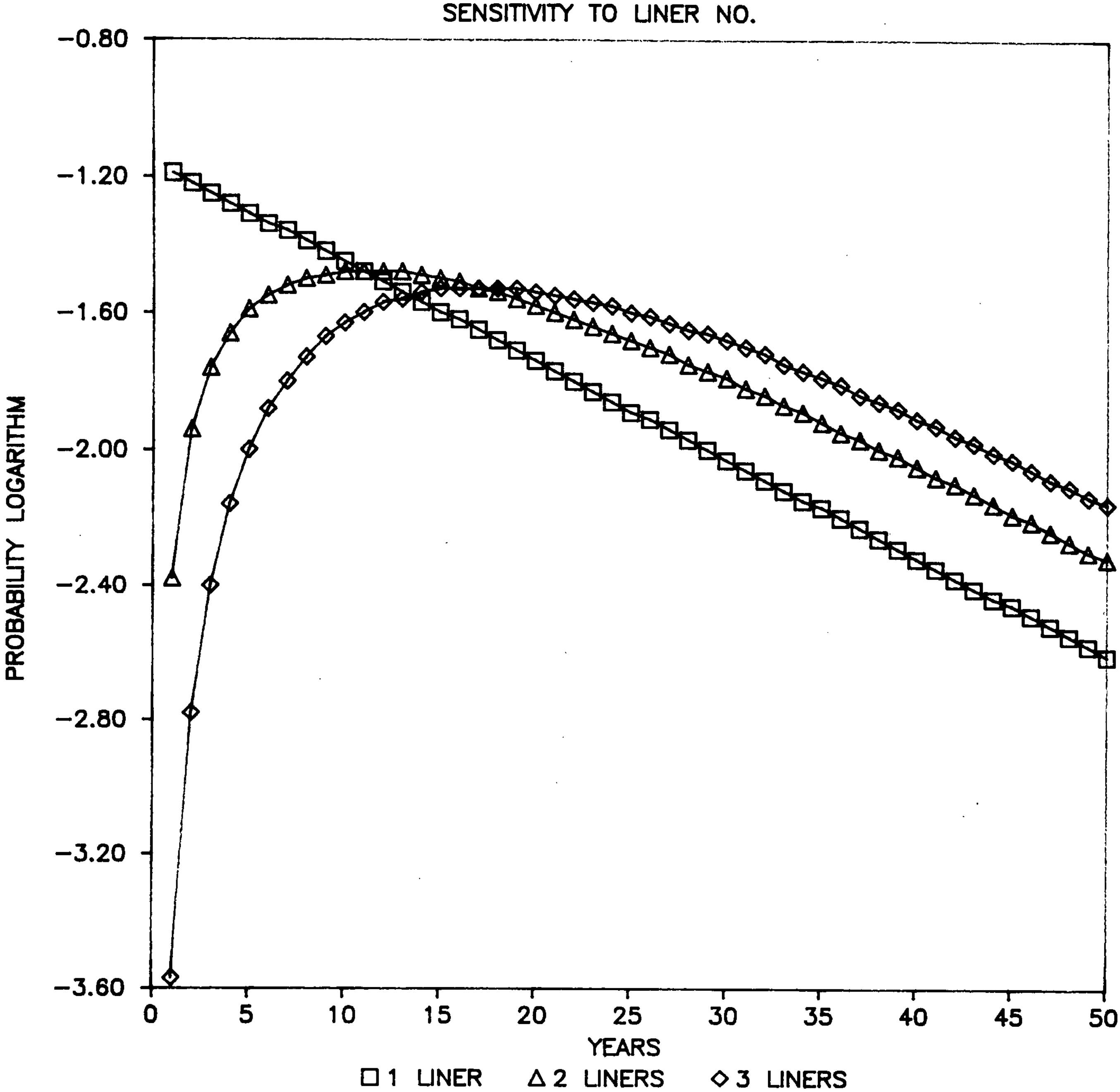


Figure 4.15 - Effects of Number of Liners on Probability of Breaching Assuming Exponential Distribution

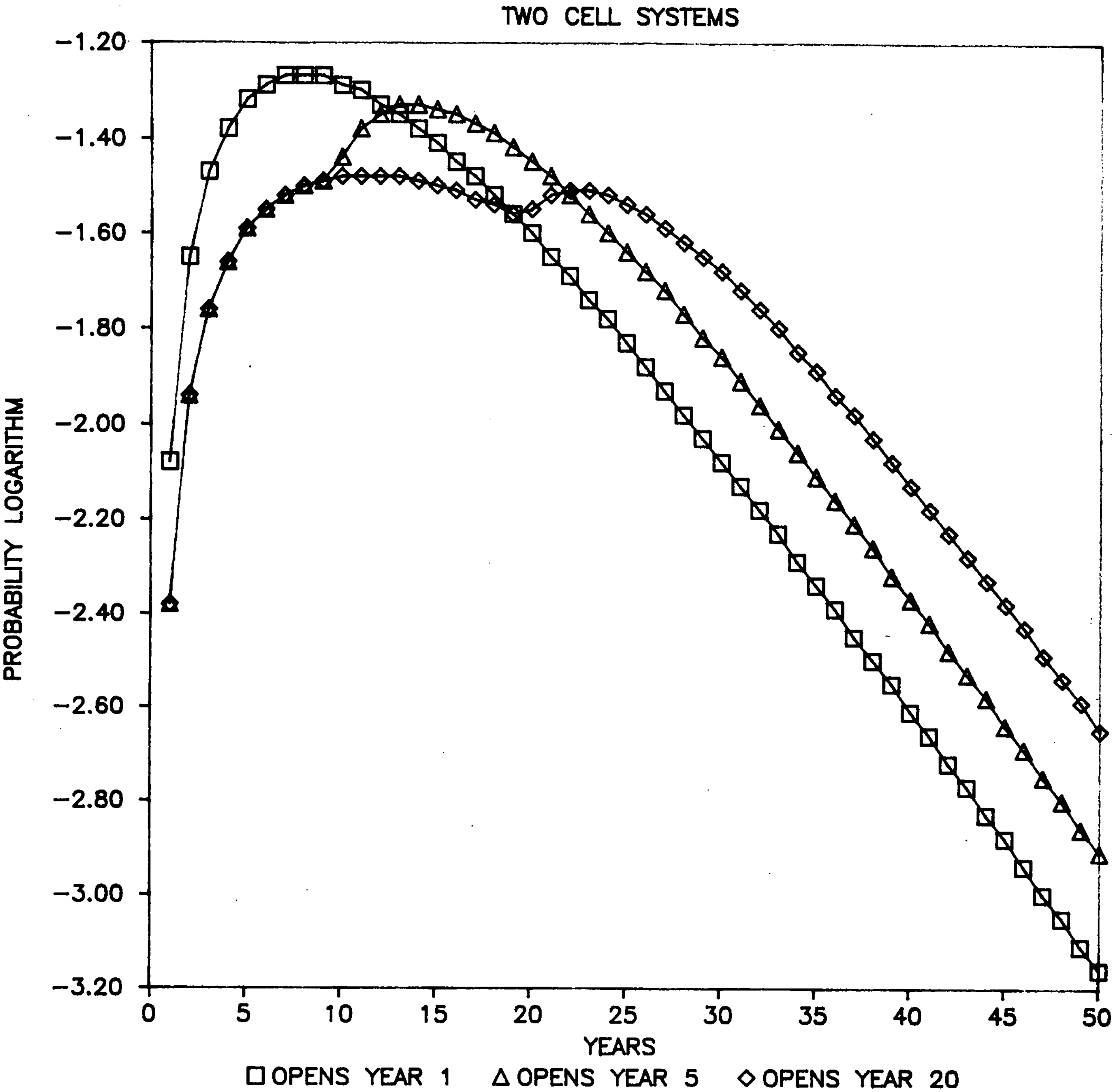


Figure 4.16 - Effects of the Year That Second Cell Begins Operation on Probability of Breaching for Cells with Two Liners

4.3 Summary of Assumptions and Conclusions

To model the waste management facility using reliability analysis, a number of assumptions must be made. Once these assumptions have been accepted, conclusions can be drawn pertaining to the design of containment structures. These assumptions and conclusions are summarized in this section.

The principal assumptions are:

1) Containment is achieved primarily by synthetic liners.
2) Mechanisms of breach are too many and too complex to model using physically-based approaches.
3) Individual liners and individual waste cells function independently.
4) The performance of individual liners can be modeled using the mortality curve shown in Figure 4.3.

Of these assumptions, the first and third are the most troublesome. Firstly, synthetic liners are not the sole containment mechanism at most waste management facilities. Additional containment features include low permeability covers, leachate collectors, and leak detection systems. Waste cells which incorporate these features could be modelled using reliability theories, but the configuration would no longer be a simple parallel structure and the structure function would be somewhat more complex. There is no theoretical or computational roadblock which prevents incorporating these additional features into the analysis; it would simply require more effort.

The third assumption, however, is of a more fundamental nature. Although individual waste cells may function independently, individual liners clearly do not. For example, if the upper liner of a two-liner system were to fail because of reactions with the leachate, the probability of breaching for the second liner, given that the first liner failed, would certainly be greater than the probability of breaching before the first liner failed. We can compensate for these effects to some extent by assigning higher rate constants to upper liners, but the model would still behave as an independent system. On the other hand, modeling the interactions among the different components would require a very complex set of conditional probabilities and would demand very difficult, if not impossible integrations. Considering the scarcity of data pertaining to the performance of containment components, such an analysis is probably not justified.

The conclusions that can be drawn from the analyses described in this chapter are as follows.

1) Waste management systems can be modelled as a system of waste cells configured in a series structure.

2) Waste cells can be modelled as a systems of synthetic liners configured in a parallel structure.

3) The probability of breaches due to a) manufacture and installation failures, b) external events which have equal

annual probabilities of occurrence, and c) liner degradation and wear can be separately identified if the mortality curve shown in Figure 4.3 is used to model liner performance.

4) Additional liners reduce the number of early breaches due to external events and increase the number of late breaches due to degradation or wear.

5) Additional cells increase the number of early breaches due to external events and decrease the number of late breaches due to degradation and wear.

6) Because of the effects of discounting future losses, breaches due to degradation or wear do not significantly affect owner-operators. From an owner-operator's point-of-view, then, the performance of liners can be effectively modeled using the exponential probability distribution, which models breaches due to external events with equal annual probabilities of occurrence.

7) If late breaches are neglected, the logarithm of the probability of breach for single-liner systems is linear. The slope steepens as the exponential rate constant increases and as the number of waste cells increases.

8) If the primary cause of early breaches is assumed to be due to events that occur at points locally distributed on a site scale, then the exponential rate constant should be

directly proportional to the area of the waste cells.

9) Waste cells which begin operation later in the system life play a relatively minor role in the overall performance of the waste management system, even without the effects of discounting.

10) Reliability analyses provide a valuable framework for studying waste management systems and for quantifying the effects of design parameters.

5. RANDOM FIELDS AND PROBABILISTIC CONTAMINANT TRAVEL TIMES

In addition to a breach of the containment structure, a second event must occur before the waste management system can be said to have failed: the contaminant plume must migrate through the hydrogeologic environment to the compliance surface. The way in which failure was defined requires this to occur during the facility's compliance period if the owner-operator is to be held responsible. The variable of interest in determining the probability of failure is the contaminant travel time.

The travel time is uncertain because of uncertainties associated with 1) the physical and chemical processes which govern contaminant transport, 2) physical and chemical parameters describing the geologic materials, and 3) physical and chemical parameters describing the groundwater. These uncertainties can be quantified by treating the travel time as a random variable with some specified probability distribution function.

This dissertation considers single, inorganic, non-radioactive contaminant species in a steady-state, saturated flow system in a high-permeability sand and gravel formation. With these restrictions, the plume migration can be modeled with the advective transport equation and the probability distribution for contaminant travel time can be estimated using computer models. The general development of these computer models will be presented in this chapter.

5.1 Solute Transport Processes

There are five general mechanisms involved in solute transport in saturated groundwater flow systems (Gillham and Cherry, 1982). These are advection, diffusion, dispersion, retardation, and decay. Advection, diffusion, and dispersion are principally physical mechanisms while retardation and decay are principally chemical and biological mechanisms. The relative importance of each mechanism depends upon geology, hydraulic gradients, groundwater chemistry, and the spatial and temporal scales of the particular problem that is being assessed.

5.1.1 Advection

Advective transport, illustrated in Figure 5.1, involves the movement of solute at the average linear velocity of the groundwater. In one-dimensional flow fields, advective transport is synonamous with plug flow. In two- and three-dimensional flow fields, advective transport can result in irregularly shaped plumes caused by fingering and bifurcations, but the concentration within the plume is the same at all points and is equal to the concentration at the source. The concentration as a function of distance from the source at some fixed time is a step function, as shown in Figure 5.1b. Similarily, the concentration as a function of time at some fixed location is also a step function, as shown in Figure 5.1c.

For advective flow, the mass flux of solute is dependent upon the hydraulic conductivity, the concentration, and the hydraulic

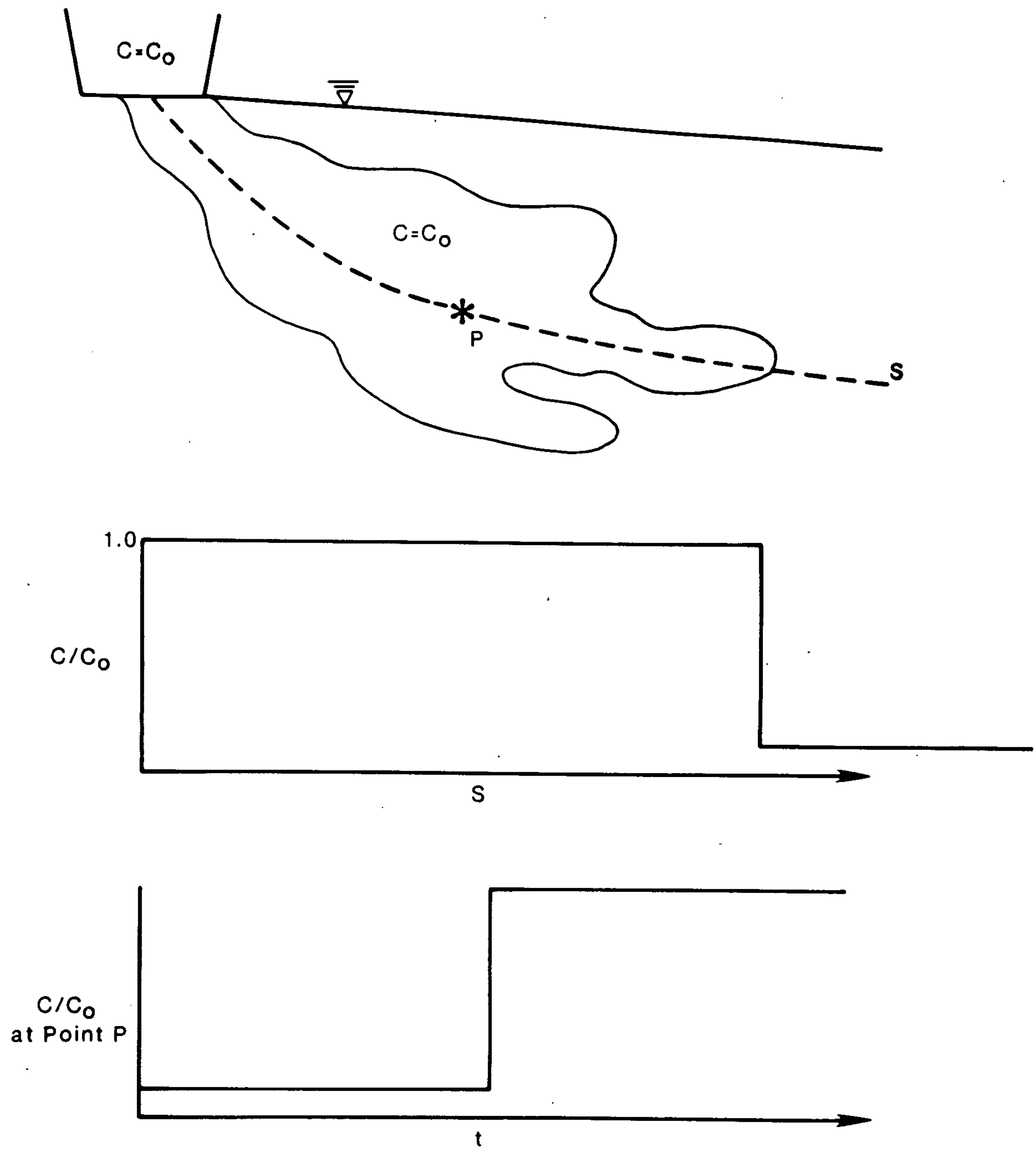


Figure 5.1 - Advective Transport

gradient:

$$J_a = -K(dh/dx)C \tag{5.1}$$

where

J_a = solute flux due to advection ($M/L^2/T$)

K = saturated hydraulic conductivity (L/T)

dh/dx = hydraulic head gradient

C = concentration of solute in groundwater (M/L^3)

5.1.2 Diffusion and Dispersion

Field and laboratory observations indicate that solute concentrations are not constant throughout plumes and that some degree of mixing takes place at the plume edges, as shown in Figure 5.2. The concentration as a function of distance from the source at some fixed time, illustrated in Figure 5.2b, and the concentration as a function of time at some fixed location, illustrated in Figure 5.2c, show mixing or spreading. This mixing is generally attributed to molecular diffusion and to mechanical dispersion. The first of these, molecular diffusion, is governed by Fick's Law:

$$J_m = -nD_m(dC/dx) \tag{5.2}$$

where

J_m = solute flux due to molecular diffusion ($M/L^2/T$)

n = porosity

D_m = molecular diffusion coefficient for the solute in the porous media (L^2/T)

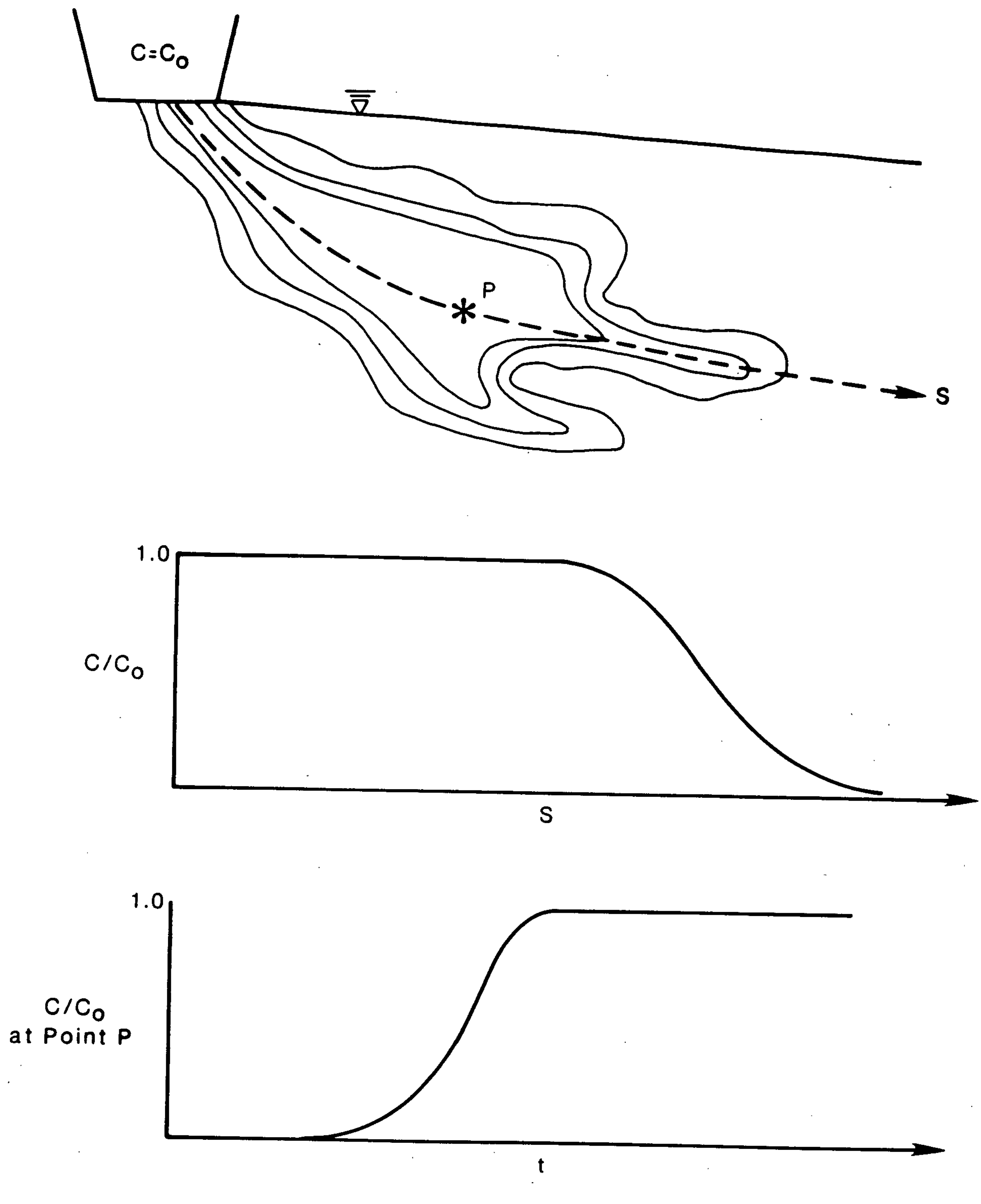


Figure 5.2 - Effects of Diffusion and Dispersion

dC/dx = concentration gradient (M/L^4)

The molecular diffusion coefficient for solute in porous media ranges from 10^{-5} to 10^{-6} cm^2/sec. The values are generally less than the diffusion coefficient in free solution, in part because the solute flow paths are more tortuous in porous media than in free solution.

The second mixing process, mechanical dispersion, is generally attributed to velocity variations within the porous medium. These variations occur within individual soil pores on a microscopic scale and within soil layers because of permeability variations on a larger scale. Mechanical dispersion is generally represented by an equation with the same form as Fickian diffusion:

$$J_d = -nD_d(dC/dx) \tag{5.3}$$

where

J_d = solute flux due to mechanical dispersion ($M/L^2/T$)

D_d = mechanical dispersion coefficient (L^2/T)

The dispersion coefficient is often calculated as a product of the average groundwater velocity and a parameter defined as the dispersivity:

$$D_d = \alpha v \tag{5.4}$$

where

α = dispersivity (L)

v = average linear groundwater velocity (L/T)

Dispersivities are assigned different values for directions perpendicular and parallel to the average direction of groundwater flow. Laboratory experiments indicate that longitudinal dispersivities typically exceed transverse dispersivities by a factor of 10 to 20 (Pickens, 1978).

The assumption that mechanical dispersion can be modeled with a diffusion type of equation is more a result of computational convenience than physical evidence. The assumption allows the effects of diffusion and dispersion to be combined in a single parameter defined as the hydrodynamic dispersion:

$$D = D_d + D_m \tag{5.5}$$

$$= \alpha v + D_m \tag{5.6}$$

Recent studies have attempted to address the effects of dispersion using conceptual models that are more physically realistic than the classical hydrodynamic dispersion model. Two of the more popular models are shown on Figure 5.3. Figure 5.3a illustrates that hydraulic conductivity stratifications can result in concentration profiles in monitoring wells similar to what is predicted using the hydrodynamic dispersion models [cf. Guven et al., 1984; Molz et al., 1986]. These types of hydraulic conductivity stratifications can be the result of very subtle textural changes in geologic materials that may not be visually apparent.

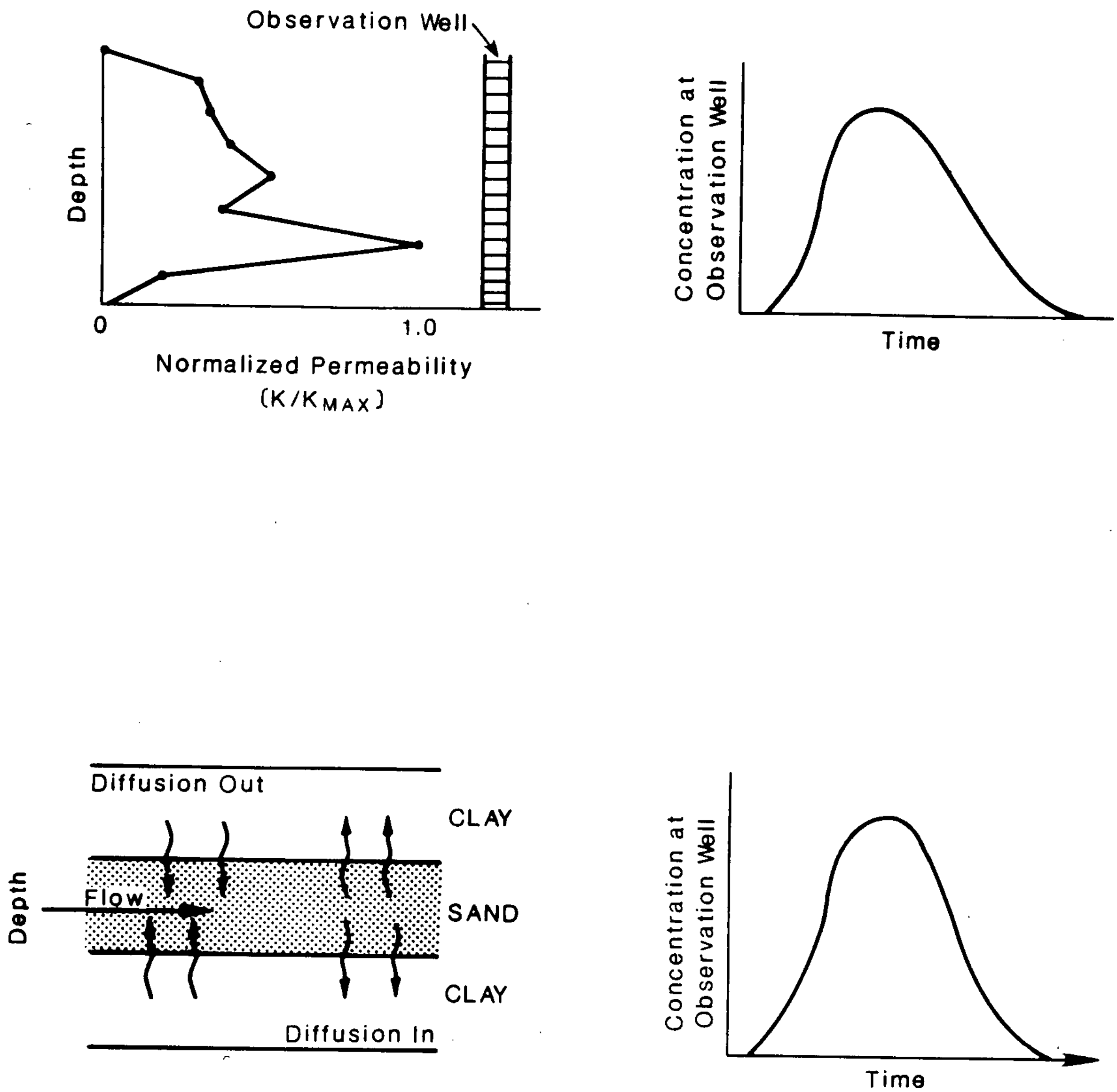


Figure 5.3 - Apparent Dispersion as a Result of a) Hydraulic Conductivity Variations Within a Geologic Unit, and b) Diffusion Into and Out of Low Permeability Layers

A second model that has recently been proposed is the advection-diffusion model, illustrated in Figure 5.3b. Solute which is transported advectively through higher conductivity layers may diffuse into and out of lower conductivity layers [Gillham et al., 1984]. The result of this reversible diffusion is again a concentration profile similar to what is predicted using hydrodynamic dispersion models.

5.1.3 Retardation and Decay

Advection, diffusion, and dispersion are the predominant transport processes for nonreactive or conservative solutes. For reactive or nonconservative solutes, concentrations also depend upon chemical and biochemical processes. These processes can be divided into two general catergories: retardation processes and decay processes. The primary difference between retardation and decay is that retardation processes are generally reversible while decay processes are generally irreversible.

Retardation is the process by which solute is transferred between the liquid and solid phases because of adsorption-desorption reactions, oxidation-reduction reactions, and precipitation-dissolution reactions. The overall effect of retardation is to partition chemical constituents between the solid and liquid phases. For most groundwater applications, both retardation and decay are assumed to be very rapid relative to natural groundwater flow so that local, steady-state equilibrium is maintained.

Retardation processes can be quantified using laboratory experimments in which the solid concentration is measured as a function of the dissolved concentration. The relationship between the solid and dissolved concentration is typically expressed by using a distribution coefficient, K_d, defined as:

$$K_d = S_m/C^b \tag{5.7}$$

where

K_d = distribution coefficient

S_m = mass of solute adsorbed per unit bulk dry mass of the porous medium (M/M)

C = concentration in liquid phase (M/L^3)

b = empirical coefficient

For most applications, b is assumed to be 1.0 so that the concentration in the solid phase is a linear function of the concentration in the liquid phase.

Retardation causes the average linear velocity of the solute front to be less than the average linear velocity of the groundwater. If distribution coefficients are used to model retardation, the relationship between the velocity of the solute front and the average linear groundwater velocity is given by:

$$v/v_s = 1. + K_d(\rho_b/n) \tag{5.8}$$

V_s = velocity of solute front (L/T)

ρ_b = mass density of solids (M/L^3)

n = porosity

The ratio of the average linear velocity of the groundwater to the velocity of the solute front is termed the retardation factor and is given by terms on the right side of Equation 5.8. Retardation factors may range from 1 to over 10,000, depending upon the solute and the geologic media [Miller and Benson, 1983].

As is the case with the equation for hydrodynamic dispersion, the main appeal of the retardation factor approach is mathematical convenience [Rubin, 1983]. The approach has limited or unproven predictive capabilities in field or even laboratory applications [Gillham and Cherry, 1982].

The second set of chemical and biological mechanisms that affect nonconservative solutes in groundwater are irreversible decay processes. Solute decay is a result of processes such as oxidation and reduction, microbial conversions, the formation of complex ions, and radioactive decay. A sink/source term that represents the rate at which the dissolved species is removed from solution is usually used to quantify the effects of decay. The decay term is given by:

$$SC = \text{Solute decay} \tag{5.9}$$

where

S = decay constant ($1/T$)

C = solute concentration (M/L^3)

5.1.4 The Importance of Advective Flow in Engineering Design

The relative importance of the five transport processes described in the previous section depends on the spatial and temporal scales of the problem that is being assessed. The importance of dispersion and diffusion can be illustrated by comparing the average linear groundwater velocity with the rate at which a solute front advances along a flow path. For advective transport, as illustrated in Figure 5.1, these two quantities are identical. When dispersion and diffusion are included, the relationship between the velocity of the groundwater and the velocity of the solute front depends upon the magnitude of the groundwater velocity and the way in which the solute front is defined, as shown in Figure 5.4.

Figure 5.4a compares groundwater velocity and solute front velocity when the solute front is defined by $C/C_o = 0.5$, where C_o is the source concentration. With this definition, advection is the dominant transport process if the average linear groundwater velocity is greater than about 0.5 cm/yr. When the solute front is defined by $C/C_o = 0.01$, advection dominates if the average groundwater velocity is greater than about 100 cm/yr, as shown on Figure 5.4b.

The relative importance of the various transport mechanisms can also be illustrated by studying case histories which present shapes and sizes of contaminant plumes. Table 5.1 summarizes the results of 10 case histories of contaminant plumes in unconsolidated geologic media [Freeze, personal communication, 1986]. These case histories generally confirm that the

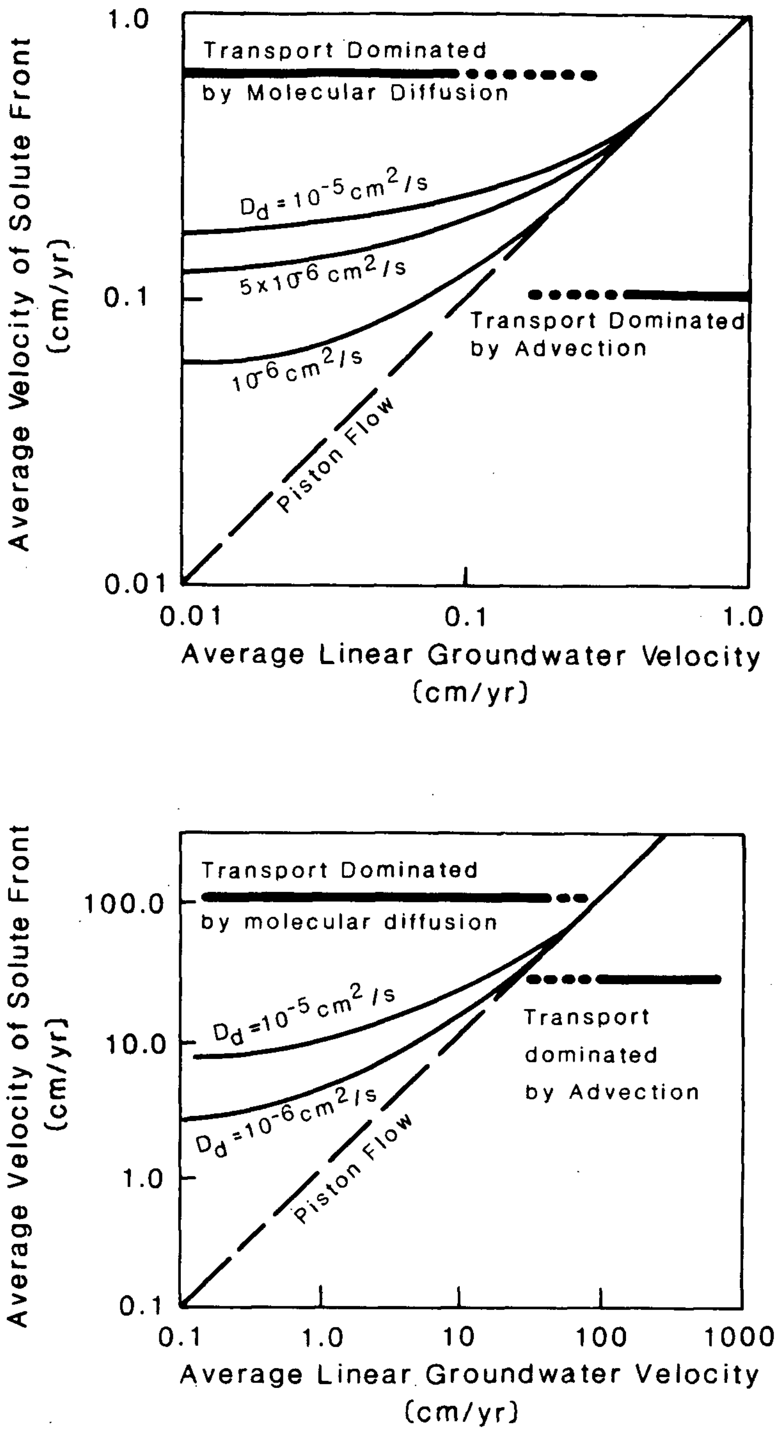


Figure 5.4 - Groundwater Velocity and Solute Front Velocity if Solute Front is Defined by a) C/Co = 0.5, and b) C/Co = 0.01

Table 5.1 - Groundwater Contamination Case Histories (from R. A. Freeze, 1986)

Location	Hydraulic Conductivity ($m/s \times 10^{+4}$)	Porosity	Hydraulic Gradient	Average Velocity ($m/s \times 10^{+6}$)
1	10.0	0.40	0.0025	6.2
2	0.4	0.38	0.0090	1.0
3	7.5	0.35	0.0025	5.4
4	8.0	0.35	0.0017	3.8
5	6.0	0.38	0.0050	7.8
6	10.0	0.25	0.0030	12.0
7	1.1	0.35	0.0077	2.4
8	0.4	0.35	0.0030	0.3
9	2.5	0.40	0.0019	1.0
10	12.0	0.30	0.0090	36.0

Location	Plume Length (meters)	Plume Width (meters)	Plume Thickness (meters)	Plume width/ Source width
1	820	300	10	1.0
2	300	30	5	1.0
3	1300	300	20	4.2
4	3200	700	25	1.2
5	700	90	25	1.1
6	3100	580	23	1.2
7	400	150	20	1.9
8	700	600	20	2.2
9	6700	1100	25	1.1
10	8800	1200	18	2.0

Location	Estimated Age of Plume (years)	Plume Velocity (m/yr)	Average Velocity/ Plume Velocity	Species
1	na	na	na	TCE
2	22	14	2.5	Strontium 90
3	12	118	1.4	Chromium +6
4	42	76	1.6	Boron
5	16	44	5.6	Strontium 90
6	27	113	3.3	Chloride
7	13	31	2.4	Chloride
8	39	18	0.6	Chloride
9	34	200	0.2	TDS
10	19	460	2.4	Chloride

1 = South Brunswick, NJ
2 = Chalk River, Ontario
3 = Nassau County, NY
4 = Cape Cod, MA
5 = Wood River Junctin, RI
6 = Babylon, NY
7 = Gloucester, Ontario
8 = Borden, Ontario
9 = Barstow, CA
10 = Denver, CO

predominant transport mechanism is advection if the average linear groundwater velocity is greater than several meters per year.

For groundwater contamination from waste management facilities, spatial scales are typically on the order of tens or hundreds of meters and temporal scales are on the order of tens of years. Solute front velocities on the order of meters or tens of meters per year are required for groundwater contamination to be important within these time and space scales. Contaminant plumes that develop at a slower rate will not have a significant impact on owner-operators' decisions because of the effects of discounting future risks to present value. Clean-up costs that will be incurred far in the future for slow-moving plumes will be negated by discounting.

In summary, the following conclusions can be drawn regarding contaminant tranport mechanisms:

1) Of the many solute species that would likely be present in landfill leachate, those that move most quickly are the most important with regard to the risks associated with groundwater contamination. The species that move most quickly are those that do not decay, react, or absorb.

2) With the definition of failure that has been adopted in this study, contaminant plumes must generally move meters per year for the risks associated with groundwater contamination to impact an owner-operator's objective

function. Advection is the dominant transport mechanism under these conditions.

3) Hydraulic conductivities on the order of 0.01 to 0.001 cm/s are needed to a have groundwater velocities on the order of meters per year. Sands and gravels generally represent the only types of unconsolidated deposits in which these magnitudes of hydraulic conductivities are observed. Plumes that move meters per year cannot generally occur in media of lower conductivity.

5.2 Modelling Solute Transport

5.2.1 General Transport Model

The combined effects of advection, dispersion, diffusion, retardation, and decay can be modeled with a differential equation that is generally termed the advection-dispersion equation. The equation can be developed by considering the flux into and out of a fixed elemental volume. For a nonreactive solute, the flux will be a summation of the advective flux, given by Equation 5.1, diffusive flux, given by Equation 5.2, and dispersive flux, given by Equation 5.3. The effects of retardation are generally incorporated using a retardation factor and decay is incorporated with a sink/source term. The three-dimensional form of the advection-dispersion equation is [Javandel et al, 1984]

$$R \frac{\partial C}{\partial t} = \frac{\partial}{\partial x_i} D_{ij} \frac{\partial C}{\partial x_j} - \frac{\partial}{\partial x_i}(CV_i) - SCR \qquad (5.10)$$

where

C	=	solute concentration $[M/L^3]$
D_{ij}	=	dispersion coefficient tensor $[L^2/T]$
n	=	porosity
v_i	=	seepage velocity in direction x_i $[L/T]$
R	=	retardation factor
S	=	decay constant $[1/T]$
x_i	=	Cartesian coordinate $[L]$

The seepage velocities, v_i, are determined from Darcy's Law:

$$V_i = -K_{ij}(\partial h/\partial x_j) \tag{5.11}$$

where

K_{ij} = hydraulic conductivity tensor [L/T]

n = effective porosity

h = hydraulic head [L]

The distribution of hydraulic head values is determined by solving the groundwater flow equation:

$$\frac{\partial}{\partial x_i}\left[K_{ij}\,\frac{\partial h}{\partial x_j}\right] = S_s\,\frac{\partial h}{\partial t} \tag{5.12}$$

where S_s is the specific storage [1/L].

The advection dispersion equation given by Equation (5.10) can, in theory, be solved for a specified set of boundary and initial conditions to obtain solute concentration as a function of time and location in the flow field. In practice, however, the modeling process is rife with difficulties and complications.

The problems generally fall into the following categories:

1) The equations used to describe dispersion, retardation, and decay represent gross simplifications of actual physical processes.

2) The input parameters for the equation, which are typically

heterogeneous and anisotropic, are difficult to measure or even estimate.

3) Boundary and initial conditions are often unknown and difficult to determine.

4) In its most general form, the advection-dispersion equation cannot be solved using closed-form analytical techniques. Numerical techniques, which may require large computational efforts, can be inaccurate.

Although these difficulties apply to all five of the transport mechanisms described in the previous section, some mechanisms are better understood and more easily modeled than others. In general terms, advection and diffusion processes can be modeled with some degree of confidence while dispersion, retardation, and decay processes often cannot.

5.2.2 Advective Transport Model

For the reasons discussed in Section 5.1.4, the contaminant front must move on the order of meters or tens of meters per year for groundwater contamination to be an important factor in the owner-opertor's risk-cost-benefit equation. Dispersion and diffusion are dominated by advective transport when the groundwater velocities are on the order of meters per year. Dispersion and diffusion will therefore be neglected in this study.

The effects of retardation and decay will also be neglected for the analyses presented in this dissertation. Most waste management facilities will receive a variety of different wastes and will therefore generate a variety of contaminants. Some of these contaminants will likely be retarded or will decay and some will likely not. Neglecting the effects of retardation and decay has the same effect as considering only the fastest-moving contaminant.

By considering only advective solute transport, the solute front will move at the average linear groundwater velocity. The concentration at any point will be zero if the plume has not arrived and will be C_o if it has, where C_o is the source concentration. The time required for the plume to travel from point s_1 to point s_2 is given by:

$$t_2 - t_1 = \int_{t_1}^{t_2} dt = \int_{s_1}^{s_2} ds/V \tag{5.13}$$

5.2.3 Stream Functions

The traditional approach for determining groundwater travel times is to 1) determine the spatial distribution of hydraulic heads (or fluid potentials) by solving Equation (5.12), 2) determine the spatial distribution of velocities by solving Equation (5.11), and 3) determine travel times between two points by solving (5.13). However, for steady state flow systems, a more efficient and often times more accurate approach is to reformulate the groundwater flow equation in terms of stream functions rather than fluid potentials or hydraulic heads [Frind and Matanga, 1985; Bear, 1979].

By definition, a streamline, s, is a curve that is everywhere tangent to the velocity vector, as illustrated in Figure 5.5 for a two dimensional flow field. The slope of the streamline equals the slope of the velocity vector at all points in the flow field:

$$dx_2 \,/\, dx_1 = q_2 \,/\, q_1 \tag{5.14}$$

where

$dx_2 \,/\, dx_1$ = slope of streamline

$q_2 \,/\, q_1$ = slope of velocity vector

A stream function , $\Psi(x_1,x_2)$, is defined as a function that is constant along a streamline, s. The derivative of the stream function with respect to x_1 and x_2 is therefore zero along s. Using the total derivative:

$$\partial\Psi(x_1,x_2) = \frac{\partial\Psi}{\partial x_1}dx_1 + \frac{\partial\Psi}{\partial x_2}dx_2 = 0 \tag{5.15a}$$

$$dx_2/dx_1 = \frac{-\partial\Psi/\partial x_1}{\partial\Psi/\partial x_2} \tag{5.15b}$$

Combining Equations (5.14) and (5.15) gives:

$$q_2/q_1 = \frac{-\partial\Psi/\partial x_1}{\partial\Psi/\partial x_2} \tag{5.16}$$

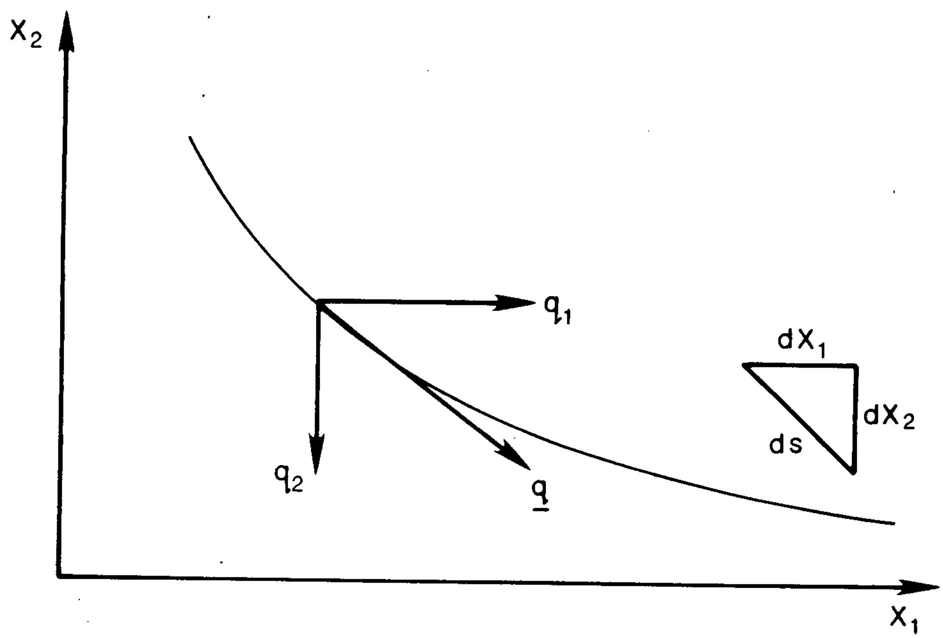


Figure 5.5 - Definition of a Streamline

From Equation (5.16):

$$q_2 = -\partial\Psi / \partial x_1 \tag{5.17a}$$

$$q_1 = \partial\Psi / \partial x_2 \tag{5.17b}$$

Combining Equations (5.17) with the two-dimensional form of Darcy's Law gives:

$$\begin{bmatrix} q_1 \\ q_2 \end{bmatrix} = - \begin{bmatrix} K_{11} & K_{12} \\ K_{21} & K_{22} \end{bmatrix} \begin{bmatrix} \partial\phi / \partial x_1 \\ \partial\phi / \partial x_2 \end{bmatrix} = \begin{bmatrix} \partial\Psi / \partial x_2 \\ -\partial\Psi / \partial x_1 \end{bmatrix} \tag{5.18}$$

The mathematical relationships between fluid potentials and stream functions that is given by Equation (5.18) are termed the Cauchy-Riemann conditions. They are illustrated on Figure 5.6.

Equation (5.18) can be solved for fluid potentials by inverting the hydraulic conductivity tensor:

$$\begin{bmatrix} \partial\phi / \partial x_1 \\ \partial\phi / \partial x_2 \end{bmatrix} = -\underline{K}^{-1} \begin{bmatrix} \partial\Psi / \partial x_2 \\ -\partial\Psi / \partial x_1 \end{bmatrix} = -\underline{K}^{-1} \begin{bmatrix} q_1 \\ q_2 \end{bmatrix} \tag{5.19}$$

where

$\underline{K}^{-1}$ = inverse of hydraulic conductivity tensor

It can be shown [cf. Frind and Matanga, 1985; Bear, 1979] that in a steady state flow field the conservation of forces requires:

$$\nabla \times (\underline{K}^{-1} \underline{q}) = 0 \tag{5.20}$$

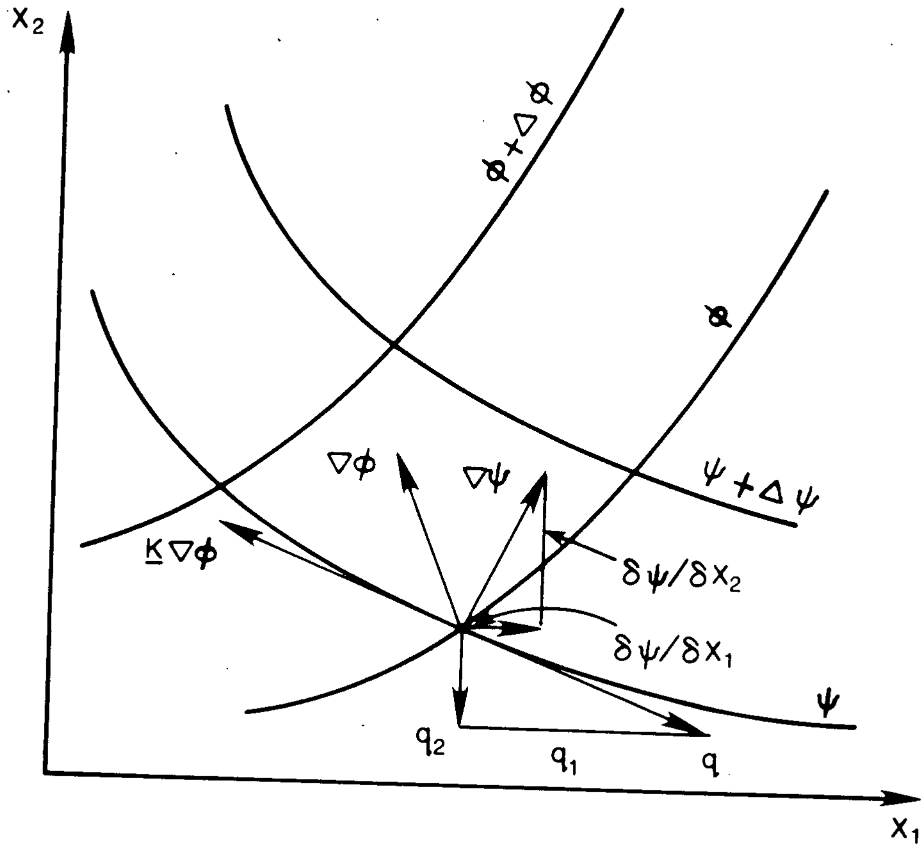


Figure 5.6 - Relationships Between Fluid Potentials and Stream Functions

where X denotes a vector cross product.

Combining Equations (5.19) and (5.20) gives the steady-state groundwater flow equation in terms of stream values:

$$\nabla \frac{1}{|K|} \underline{K} \nabla \Psi = 0 \tag{5.21}$$

where

$|K|$ = determinant of hydraulic conductivity tensor

The discharge between two streamlines, shown in Figure 5.7, is given by:

$$dQ = \int_{p_1}^{p_2} q \; p \; dp = \int_{p_1}^{p_2} q_1 dx_2 - q_2 dx_1 \tag{5.22}$$

Combining Equations (5.18) and (5.22)

$$dQ = \int_{p_1}^{p_2} \frac{\partial \Psi}{\partial x_2} dx_2 + \frac{\partial \Psi}{\partial x_1} dx_1 = \int_{p_1}^{p_2} d\,\Psi(x_1, x_2) \tag{5.23}$$

$$dQ = \Psi(p_2) - \Psi(p_1) \tag{5.24}$$

Equation (5.24) shows that the value of a stream function at point p_2 equals the value of a reference stream function at point p_1 plus the discharge between p_1 and p_2. The fluid velocity in the stream tube of width dp is:

$$v = dQ/dpn = d\Psi/dpn \tag{5.25}$$

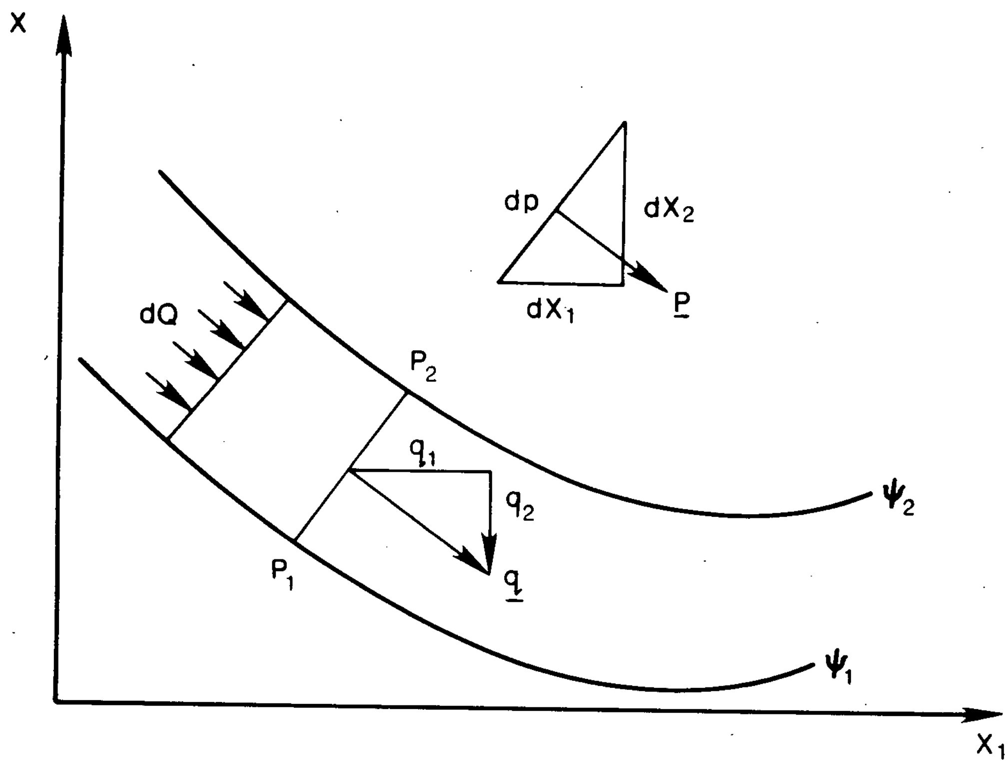


Figure 5.7 - Discharge Between Two Streamlines

where

n = effective porosity

The distance traveled along a streamline during time interval dt is:

$$ds = vdt = \frac{d\Psi dt}{dp(s)n} \qquad (5.26)$$

where

$dp(s)$ = width of stream tube at position s

The travel time between points s_1 and s_2 can be determined by integrating Equation (5.26):

$$t_2 - t_1 = \int_{t_1}^{t_2} dt = \frac{n}{d\Psi} \int_{s_1}^{s_2} dp(s)ds \qquad (5.27)$$

Equation (5.27) is the stream function alternative to Equation (5.13). One of the advantages of the stream function formulation is that computer models that solve for stream values are more accurate than computer models that solve for fluid potentials, especially for unconfined flow in relatively long and shallow aquifers. This will be discussed in more detail in Section 5.2.4.

A second advantage of the stream function formulation is that the travel times can be determined in fewer steps. With the hydraulic head formulation, the travel times are determined in a three step procedure: 1) the groundwater flow equation (Equation 5.12) is solved to determine hydraulic heads, 2) Darcy's law (Equation 5.11) is solved to determine velocities, and 3) the velocities are used in Equation (5.13) to determine travel times. With the stream function formulation, the travel times are determined in a two step procedure: 1) the groundwater flow equation (Equation 5.21) is solved to determine stream values, and 2) the stream values are used in Equation (5.27) to determine travel times.

Before the stream function formulation of the groundwater flow equation can be solved, it is necessary to specify boundary conditions in terms of stream values. There are two general types of boundary conditions for steady-state flow fields. The First Type or Dirichlet boundary condition is a boundary along which the stream function is known:

$$\Psi = \Psi(s) \tag{5.28}$$

The Dirichlet boundary condition can be rewritten using Equations (5.23) and (5.24):

$$\Psi(s) = \Psi(s_o) = \int \underline{q} \cdot \underline{n} \, ds \tag{5.29}$$

where

$\Psi(s_o)$ = reference stream value

q = specified flux at boundary

n = normal vector to boundary

The Dirichlet boundary is used to incorporate boundaries with prescribed flows. For impermeable boundaries, the stream function is a constant.

The Second Type or Neuman boundary condition corresponds to boundaries with a known gradient for the stream function, as shown on Figure 5.8. The stream function gradient can be related to fluid potential gradients by rewriting Equation (5.19):

$$\begin{bmatrix} g_1 \\ g_2 \end{bmatrix} = \frac{1}{|K|} \underline{K} \begin{bmatrix} \partial\Psi / \partial x_1 \\ \partial\Psi / \partial x_2 \end{bmatrix} = \begin{bmatrix} \partial\phi / \partial x_2 \\ -\partial\phi / \partial x_1 \end{bmatrix} \tag{5.30}$$

where

g_1 = stream function gradient in the x_1-direction

g_2 = stream function gradient in the x_2-direction

The stream function gradients normal to the boundary are given by the dot product of the gradients and the normal vector for the boundary:

$$\underline{g} \cdot \underline{n} = \frac{\partial\phi}{\partial x_2} n_1 - \frac{\partial\phi}{\partial x_1} n_2 \tag{5.31}$$

where n_1 and n_2 are the direction cosines for the unit normal vector.

Equation (5.31) can be rewritten in terms of the vector tangent to the boundary, as shown on Figure 5.8:

$$\underline{g} \cdot \underline{n} = \frac{-\partial\phi}{\partial x_1} r_1 - \frac{\partial\phi}{\partial x_2} r_2 \tag{5.32}$$

where

$r_2 = -n_1$

$r_1 = n_2$

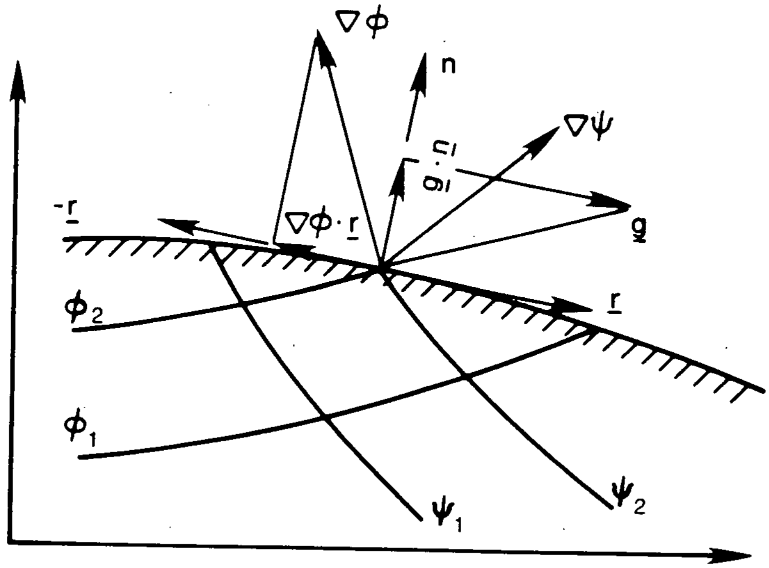


Figure 5.8 - Second Type or Neuman Boundary Conditions for Stream Functions

Equation (5.32) shows that the Neuman boundary condition on the stream function equation corresponds to a boundary with a specified fluid potential.

Figure (5.9) illustrates stream function boundary conditions for a typical cross-sectional, steady-state flow system. The left boundary is a prescribed flux boundary, the lower and right boundaries are impermeable boundaries, and the upper boundary is a prescribed head boundary.

5.2.4 Finite Element Solutions

The groundwater flow equation written in terms of stream functions, Equation (5.22), can be solved analytically for flow fields with relatively simple geometries, boundary conditions, and material properties. For more complex flow fields, numerical solutions in the form of finite-difference or finite-element computer models are required. The finite element method is used in the present study.

In general terms, the finite element procedure involves discretizing the flow field into a large number of elements. Each element can have its own set of boundary conditions and physical parameters. The differential equation describing groundwater flow is approximated with a (large) set of algebraic equations. There is usually one algebraic equation for each element. The complete set of algebraic equations can be solved by computer programs to obtain stream values at the nodes of the

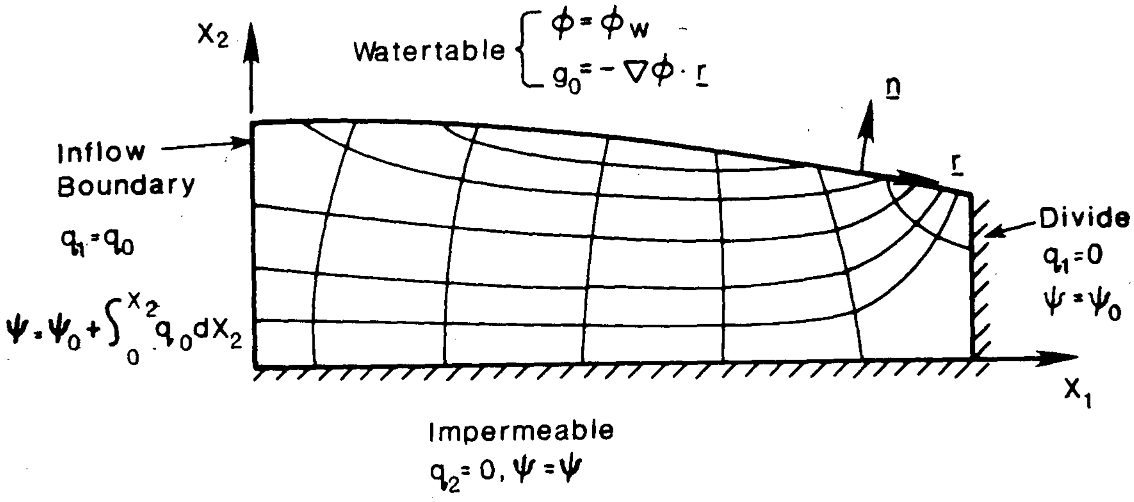


Figure 5.9 - Stream Function Boundary Conditions for a Cross-Sectional, Steady-State Flow System

mesh used to discretize the flow field.

Solving the groundwater flow equation with the finite element method has become a relatively standard procedure in hydrogeology during the last decade. A large number of texts present the details of the approach [cf. Pinder and Gray, 1977; Smith, 1982].

The Galerkin formulation with triangular elements and linear interpolation is used in this dissertation. Table 5.2 summarizes the element stiffness matrix and the right hand side vector used to solve the stream function formulation of the groundwater flow equation. Each element in the flow field has a matrix and vector of this form. The matrices and vectors are combined to obtain a set of algebraic equations that are solved to determine stream values at each node in the flow field.

The nodal stream values can be used to determine groundwater velocities within individual blocks of the flow field. Each block is composed of two triangular finite elements, as shown in Figure 5.10. The groundwater velocities within the block are given by [Frind and Matanga, 1985]:

$$v_1 = \frac{1}{2n(x_{23}-x_{22})}(-\Psi_1 - \Psi_2 + \Psi_3 + \Psi_4) \tag{5.33a}$$

$$v_2 = \frac{1}{2n(x_{12}-x_{11})}(\Psi_1 - \Psi_2 - \Psi_3 + \Psi_4) \tag{5.33b}$$

where x_{ij} is the x_i coordinate at point j. These velocities can be incorporated into Equation (5.27) to determine advective contaminant travel times between two points.

Table 5.2 - Element Stiffness Matrix and Right-Hand-Side Vector for Stream Value Finite Element Formulation

$$
\frac{1}{4A}\begin{bmatrix}
\dfrac{a_i a_i}{K_2} + \dfrac{b_i b_i}{K_1} & \dfrac{a_i a_j}{K_2} + \dfrac{b_i b_j}{K_1} & \dfrac{a_i a_k}{K_2} + \dfrac{b_i b_k}{K_1} \\[2ex]
\dfrac{a_j b_i}{K_2} + \dfrac{b_j b_i}{K_1} & \dfrac{a_j a_j}{K_2} + \dfrac{b_j b_j}{K_1} & \dfrac{a_j a_k}{K_2} + \dfrac{b_j b_k}{K_1} \\[2ex]
\dfrac{a_k a_i}{K_2} + \dfrac{b_k b_i}{K_1} & \dfrac{a_k a_j}{K_2} + \dfrac{b_k b_j}{K_1} & \dfrac{a_k a_k}{K_2} + \dfrac{b_k b_k}{K_1}
\end{bmatrix}
$$

$a_i = x_{2j} - x_{2k} \qquad a_j = x_{2k} - x_{2i} \qquad a_k = x_{2i} - x_{2j}$

$b_i = x_{1k} - x_{1j} \qquad b_j = x_{1i} - x_{1k} \qquad b_k = x_{1j} - x_{1i}$

A = area of element = $(b_k a_j - b_j a_k)/2.0$

$$
\text{RHS} = \frac{-1}{2K_2}\left[\frac{\partial \psi}{\partial x_1}\bigg|_{ij} a_k + \frac{\partial \psi}{\partial x_2}\bigg|_{ki} a_j \right] \frac{-1}{2K_1}\left[\frac{\partial \psi}{\partial x_2}\bigg|_{ij} b_k + \frac{\partial \psi}{\partial x_1}\bigg|_{ki} b_j \right]
$$

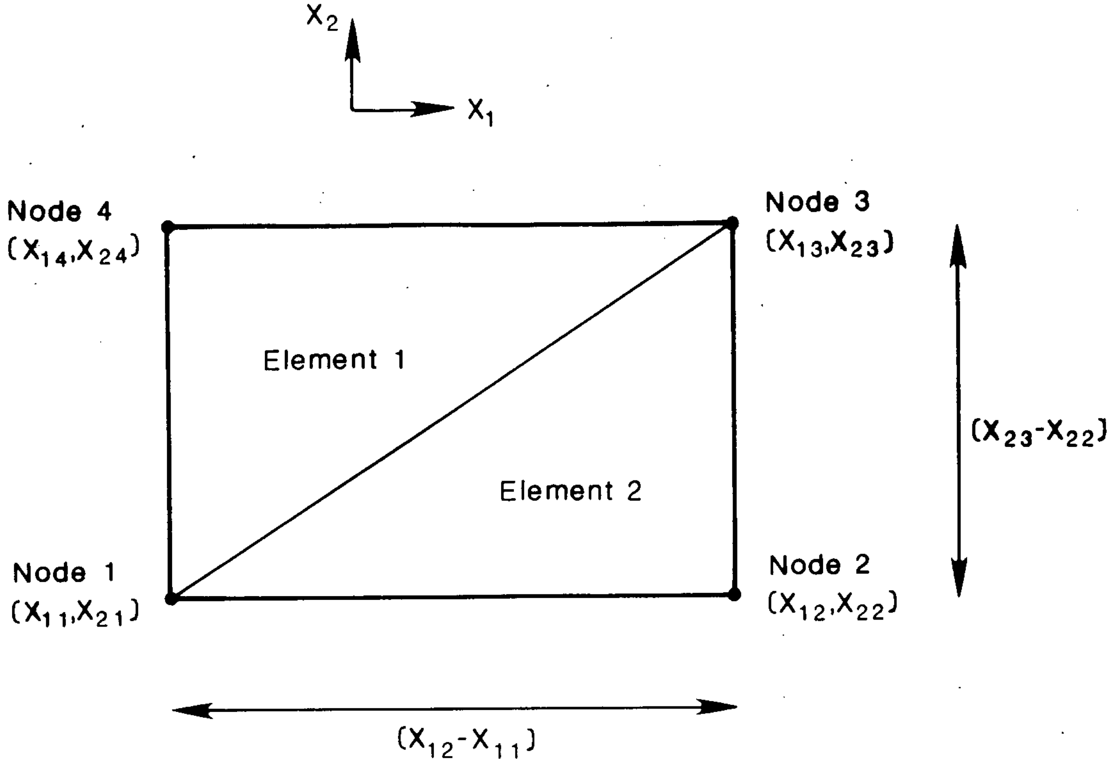


Figure 5.10 - Variables Used to Define Hydraulic Conductivity Blocks and Fluid Velocities

5.3 Quantifying Parameter Uncertainty

The steady-state groundwater flow equation written in terms of either stream values (Equation 5.22) or fluid potentials (Equation 5.12) can be used to predict solute travel times if 1) solute transport is known to be predominately due to advection, 2) the region of flow and boundary conditions are known, and 3) the spatial distributions of hydraulic conductivity and porosity are known. In most applications, however, some amount of uncertainty will be associated with each of these three sets of information. These uncertainties result in three general types of prediction error: model error, input error, and parameter error.

Model error is due to differences between actual physical and chemical processes and the physical and chemical processes that are modeled with differential equations. For example, the assumption that advection is the predominant transport mechanism will introduce model error if fluid velocities are low enough to cause dispersion and diffusion to become important. Using distribution coefficients to model the effects of retardation will also introduce errors when applied to groundwater flow systems in which local equilibrium is not achieved. Modeling three dimensional flow fields with one- or two-dimensional models adds additional model error.

Input error is due to uncertainty in flow field geometry and boundary conditions. The boundary conditions can be uncertain in terms of both the type of boundary and the magnitude of prescribed flows or fluid potentials. Both the size and shape of the flow field may also be uncertain.

The most difficult prediction error to eliminate is parameter error. Parameter error is caused by our inability to accurately incorporate the actual spatial distribution of material properties into computer models. The two material properties required to model advective transport of conservative solutes are porosity and hydraulic conductivity. The hydraulic conductivity values for naturally-occurring, unconsolidated geologic deposits can range from as low as 10^{-13} meters per second for very tight clay deposits to as high as 10 meters per second for gravels [Freeze and Cherry, 1979]. Even relatively uniform deposits may have hydraulic conductivities which range over several orders of magnitude as a function of location. Porosity generally varies over a much smaller interval. Values typically range from approximately 0.1 to 0.6.

Our inability to incorporate the spatial distribution of material properties into computer models is due to both uncertainty and variability. Uncertainty and variability have been deliberately distinquished. Uncertainty is a subjective value that depends upon the person performing the analysis. Variability is an objective, though likely unknown, value that depends upon geology. As discussed below, uncertainty and variability can be combined by treating hydraulic conductivity as a random field in a Bayesian framework [Hachich and Vanmarke, 1983].

5.3.1 Hydraulic Conductivity as a Stochastic Process

A random variable is any variable whose value cannot be predicted with certainty. A stochastic or random process is any process that generates sets of random variables. These sets of random variables can be distributed as functions of time (time series) or as functions of location (space series or random fields). Stochastic processes can be further characterized as processess which generate sets of discrete random variables and processes which generate continuous random variables. In the present study, in which finite element programs are used to predict contaminant travel times, the hydraulic conductivity of each element is viewed as a spatially discrete random variable and the complete set of hydraulic conductivities is treated as a discrete random field.

Hydraulic conductivity is defined by Darcy's Law:

$$K = -Qdl/Adh \tag{5.34}$$

where

A = area perpendicular to flow (L^2)

Q = flux through area A ($M/L^3/T$)

dh/dl = hydraulic head gradient.

By definition, hydraulic conductivity is an average value for the volume defined by the product of the area perpendicular to the direction of flow, A, and the length of the flow field, dl. Because of this averaging, hydraulic conductivity is a scale-dependent parameter. For most applications, hydraulic conductivity can be defined over three general scales [Dagan, 1986]: laboratory scale, local scale, and regional scale. Typical dimensions for these three scales are presented in Table 5.3. Often, hydraulic conductivity is measured on one scale and used to make predictions on a larger scale; laboratory measurements are used to make predictions at local scale and local measurements are used to make predictions at regional scale.

When treated as a discrete random field, hydraulic conductivity is dependent upon volume, V, a location dimension, X, and a non-physical, sample dimension, W. The dependence upon the location dimension is clear: a particular flow field consists of a large number of discrete hydraulic conductivity values. The dependence upon the non-physical sample dimension is more subtle. The actual hydraulic conductivity of any particular element is a single random variable. This single random variable has a sample space associated with it. The range of possible values that we assign to the sample space for any particular element is a function of our uncertainty. For example, the sample space might range from 10^{-13} to 10 meters per second if we know very little or might range from 10^{-6} to 10^{-1} meters per second if we know the element is composed of clean sand. It is important to note that while the hydraulic conductivity is treated as a discrete random variable in the location dimension, it is a continuous random variable in the sample dimension.

Table 5.3 - Scales used to Define Hydraulic Conductivity (after Dagan, 1986).

Scale	Extent of Flow Domain (meters)	Dimension of Averaging Volume to Define Point Variables (meters)
Laboratory	10^{-1} - 10^{0}	10^{-3} - 10^{-2}
Local	10^{1} - 10^{3}	10^{-1} - 10^{0}
Regional	10^{4} - 10^{5}	10^{1} - 10^{2}

In general terms, then, the hydraulic conductivity of an element in a flow-field is a spatially-dependent, scale-dependent, uncertain variable: K(V,X,W). To simplify notation, we will assume for the present time that the volume of each element is the same and will define the random variable K_i as the hydraulic conductivity of element i. In general terms, each K_i will have a unique cummulative probability distribution function defined as

$$F_i(w_i) = \mathrm{Prob}[K_i < w_i] \tag{5.35}$$

where

$F_i(w_i)$= cummulative probability distribution function for the hydraulic conductivity of element i

The corresponding probability density function is given by

$$f_i(w_i) = dF_i(w_i)/dw_i \tag{5.36}$$

For each hydraulic conductivity element, an expected value and a variance is defined as

$$E[K_i] = \int_{-\infty}^{\infty} K_i f_i(w_i) dw_i \tag{5.37}$$

$$\mathrm{Var}[K_i] = \int_{-\infty}^{\infty} (E[K_i] - K_i)^2 f_i(w_i) dw_i \tag{5.38}$$

where

$E[K_i]$ = expected value of hydraulic conductivity of element i

$\mathrm{Var}[K_i]$ = variance of hydraulic conductivity of element i

The integrations in Equations (5.37) and (5.38) are performed over the sample dimension and not over the space dimension. The expected hydraulic conductivity is the best estimate of K_i and the variance reflects the amount of uncertainty that is associated with K_i.

To fully describe the complete set of conductivities that make up the flow field, it is necessary to specify a multivariate cummulative distribution function:

$$F(w_1, w_2, \ldots\ w_n) = \mathrm{Prob}(K_1 < w_1, K_2 < w_2 \ldots K_N < w_N) \tag{5.39}$$

where N is the number of elements in the flow field.

The multivariate cummulative distribution function is related to the multivariate probability density function in the following manner:

$$f(w_1, w_2, \ldots w_N) = \frac{\partial^N F(w_1, w_2, \ldots w_N)}{\partial w_1\, \partial w_2 \ldots \partial w_N} \tag{5.40}$$

If the hydraulic conductivities of each element are unrelated or independent, then the multivariate cumulative distribution function is simply the product of the individual distribution functions:

$$F[w_1, w_2, \ldots w_N] = \prod_{i=1}^{N} F_i[w_i] \tag{5.41}$$

5.3.2 Correlation and Covariance

Hydraulic conductivities do not vary in space in a purely random, unstructured manner. Neighboring values in a discretized flow field are correlated. For example, in a direction parallel to formation bedding, above-average values will be grouped together and below-average values will be grouped together, as shown in Figure 5.11a. Conversely, in a direction perpendicular to the bedding, low values might follow high values in regular manner, as shown in Figure 5.11b.

Covariances can be used to quantify the spatial relationships among random variables such as hydraulic conductivity. The covariance between two random variables K_i and K_j is defined as the expected value of the product of the deviations from their respective means [Vanmarke, 1983]:

$$Cov(K_i,K_j) = C_{ij} = E[(K_i-E[K_i])(K_j-E[K_j])] \tag{5.42}$$

where:

$E[Ki]$ = expected value of random variable Ki

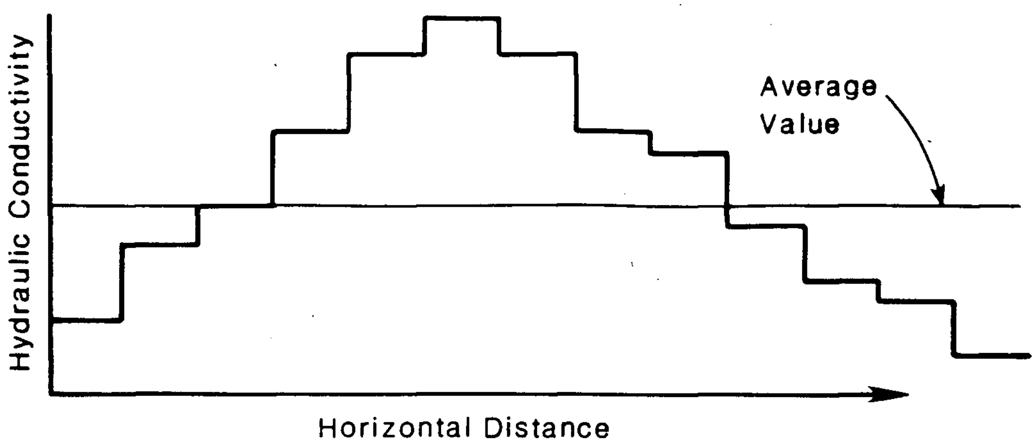
The expectation given in Equation (5.42) is given by:

$$C_{ij} = \int_{-\infty}^{\infty}\int_{-\infty}^{\infty}(K_i - E[K_i])(K_j - E[K_j])f_{ij}(w_i,w_j) \tag{5.43}$$

A parameter directly related to covariance is the correlation coefficient:

$$\rho_{ij} = C_{ij}/ [\sqrt{Var(K_i)}\sqrt{Var(K_j)}] \tag{5.44}$$

Correlation coefficients range from -1 to +1. For the hydraulic conductivity of finite elements, correlation coefficients will



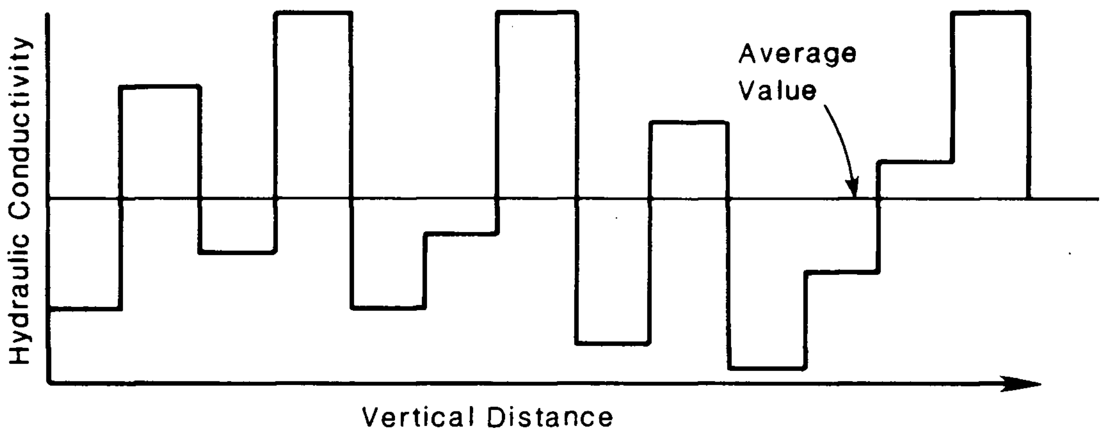


Figure 5.11 - Examples of Horizontally- and Vertically-Correlated Hydraulic Conductivity Values

generally be functions of the distance between the elements. A correlation function, $\rho(l)$, can be defined where l is the lag or the distance between element centers. Example correlations for the geologic formation shown on Figure 5.11 are presented on Figure 5.12.

A variety of different functions can be used to model correlation. Some of the more frequently used functions are illustrated on Figure 5.13. The linear function, shown on 5.13a, is the most simple. The spherical and exponential functions, shown on 5.13b and 5.13c, have very similar shapes and differ only in how quickly they approach zero. The Gaussian, shown on Figure 5.13d, is flat near the vertical axis, indicating strong correlation over small distances.

In many instances it is desirable to specify a single parameter to characterize the correlation structure of a set of random variables. A parameter commonly used is the integral scale or fluctuation scale, defined as:

$$\varepsilon = \int_0^{\infty} \rho(l)dl \qquad (5.45)$$

A second parameter that is used to charactarize the correlation structure is the correlation length, which is defined as the distance over which the correlation is positive.

It should be noted that neither the correlation length nor the integral scale uniquely define the correlation structure. For stationary random fields, which will be defined in the following

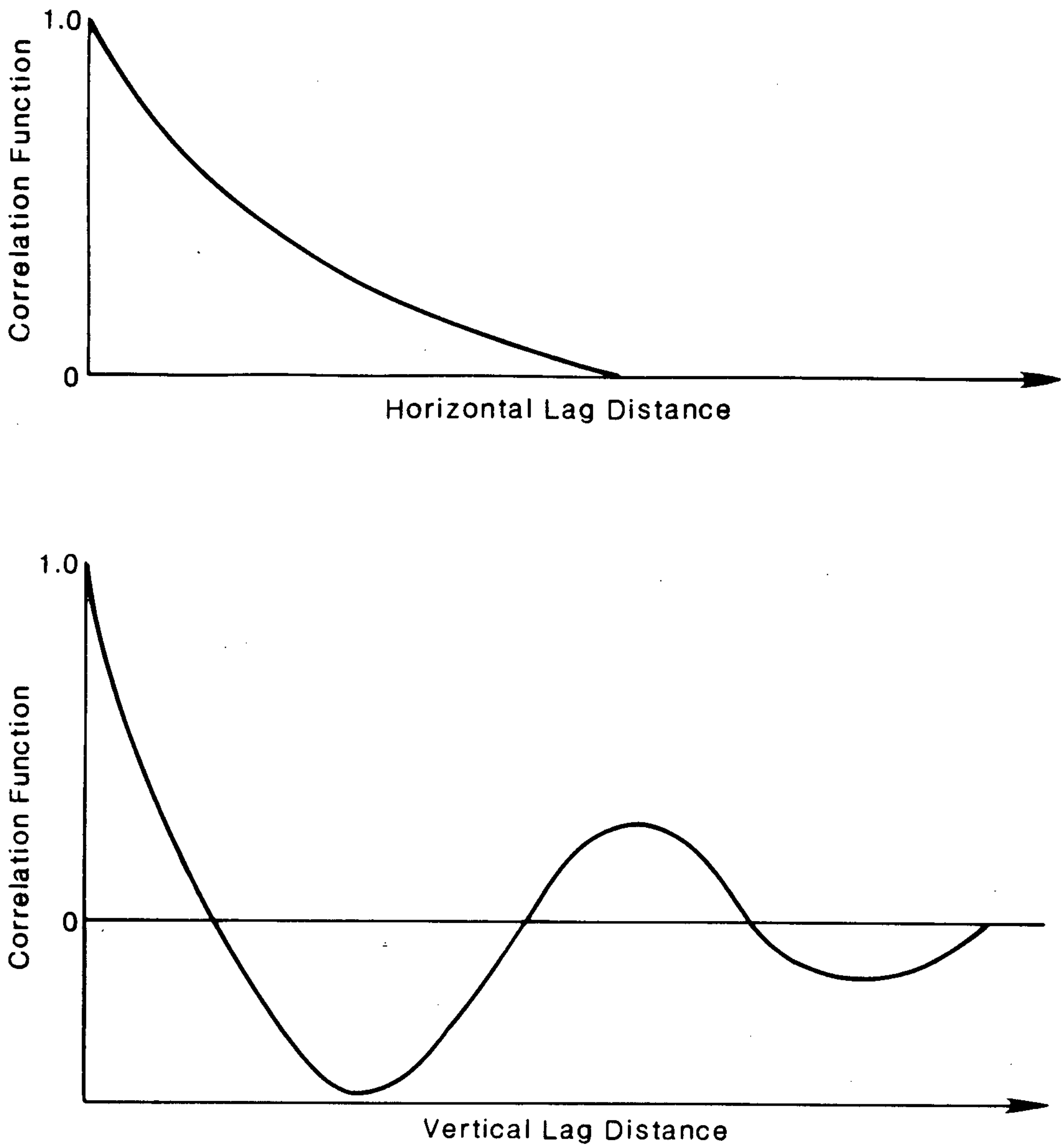


Figure 5.12 - Examples of Horizontal and Vertical Correlation Functions for the Hydraulic Conductivity Values Presented in Figure 5.11

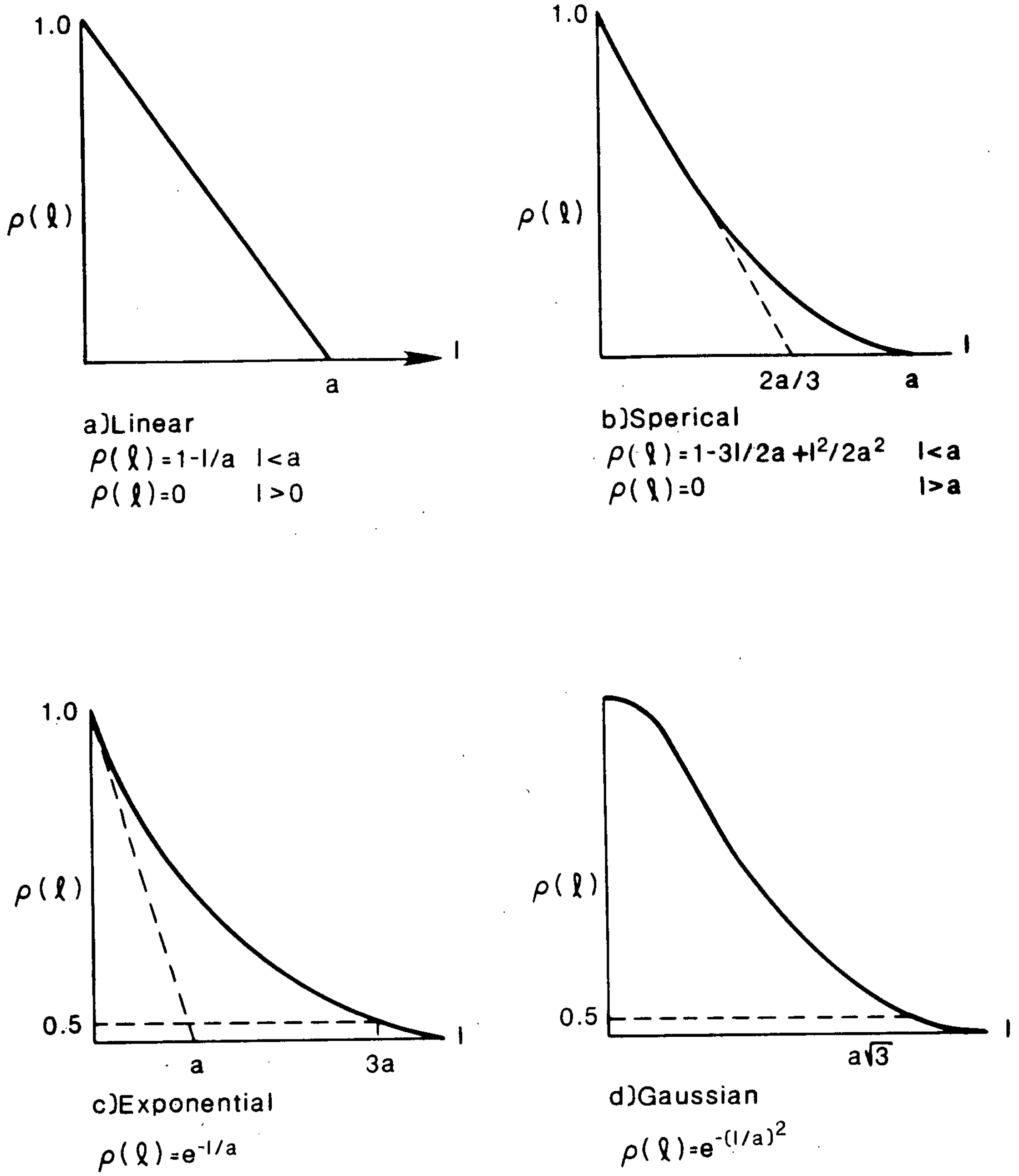


Figure 5.13 - Example Correlation Functions

section, the form of the correlation function must also be specified to uniquely define the structure. In the more general case of non-stationary fields, a complete N by N matrix of covariances is required to fully define the correlation among a set of N random variables.

5.3.3 Stationarity and Ergodicity

To fully characterize the set of hydraulic conductivities that comprise the flow field, the complete multivariate probability distribution function must be specified. The data requirements for this type of characterization are overwhelming if probabilities are interpreted from a frequency or classical viewpoint. As discussed in Chapter 2, this viewpoint dictates that probabilities have meaning only when assigned to events or experiments that can be repeated a large number of times. Enough data must be collected from these experiments so that the form of the probability distribution function (eg. normal, lognormal, or exponential) can be estimated for each hydraulic conductivity block, the parameters of these distributions (eg. expected values and variances) can be estimated, and the complete covariance matrix can be estimated.

Several simplifying assumptions are often made to reduce these data requirements. The two most common assumptions are stationarity and ergodicity. First-order or strict stationarity assumes that the form and parameters of the probability distribution functions are the same for all the hydraulic conductivity blocks. With this assumption, the correlation coefficients depend only upon the distance between elements and the correlation matrix can be replaced with a single correlation function. This assumption also allows the complete random field to be modeled with a single, univariate probability distribution function.

An assumption somewhat less restrictive than first-order stationarity is stationary increments, in which the values $K(x) - K(x+h)$ do not depend upon locations but only upon the distance between hydraulic conductivity blocks. The advantages of assuming stationary increments rather than first-order stationarity are 1) the random hydraulic conductivity field can exhibit linear trends and still have stationary increments, and 2) the random variables need not have finite variances or well-defined correlation structures.

The second general assumption that is often made to reduce data requirements is ergodicity. A random field is ergodic if the form and parameters of the probability distribution functions for the hydraulic conductivity blocks are the same in both the location dimension, X, and the sample dimension, W. With this assumption, samples taken at various locations within the random field can be used to infer probability distributions for each hydraulic conductivity block. Without this assumption, the frequentist approach would have no applicability in treating hydraulic conductivities as stochastic or random fields.

The assumptions of ergodicity and stationarity are most often made to appease the concerns of those who adopt the frequentist or classical viewpoint of probability. Neither assumption is necessary when a personalistic or subjective approach is used. This later approach is adopted in the present study.

5.3.4 Effects of Mesh Size and Geometry

As discussed in section 5.3.1, hydraulic conductivity is an average value defined for a particular volume. The hydraulic conductivity at point X is more precisely the hydraulic conductivity of some volume that is centered at point X. The values that should be assigned to the expected values and variances depend upon the size and shape of the volumes used to discretize the flow field. The more rudimentary ideas of variances and expected values as functions of volume will be presented in this section. A more detailed introductory discussion is presented by Clark [1979] and a more advanced analysis is presented by Vanmarcke [1983].

In a strict sense there is no such thing as a point value for hydraulic conductivity. However, in many instances the size of the sample used to estimate the conductivity is so much smaller than the scale of the flow field that the sampled value is essentially a point value. A "continuous" series of such measurements along a line may give a sample function, K(X), similar to that shown in Figure 5.14a. If the random field is stationary, a mean value, a variance, and a correlation length

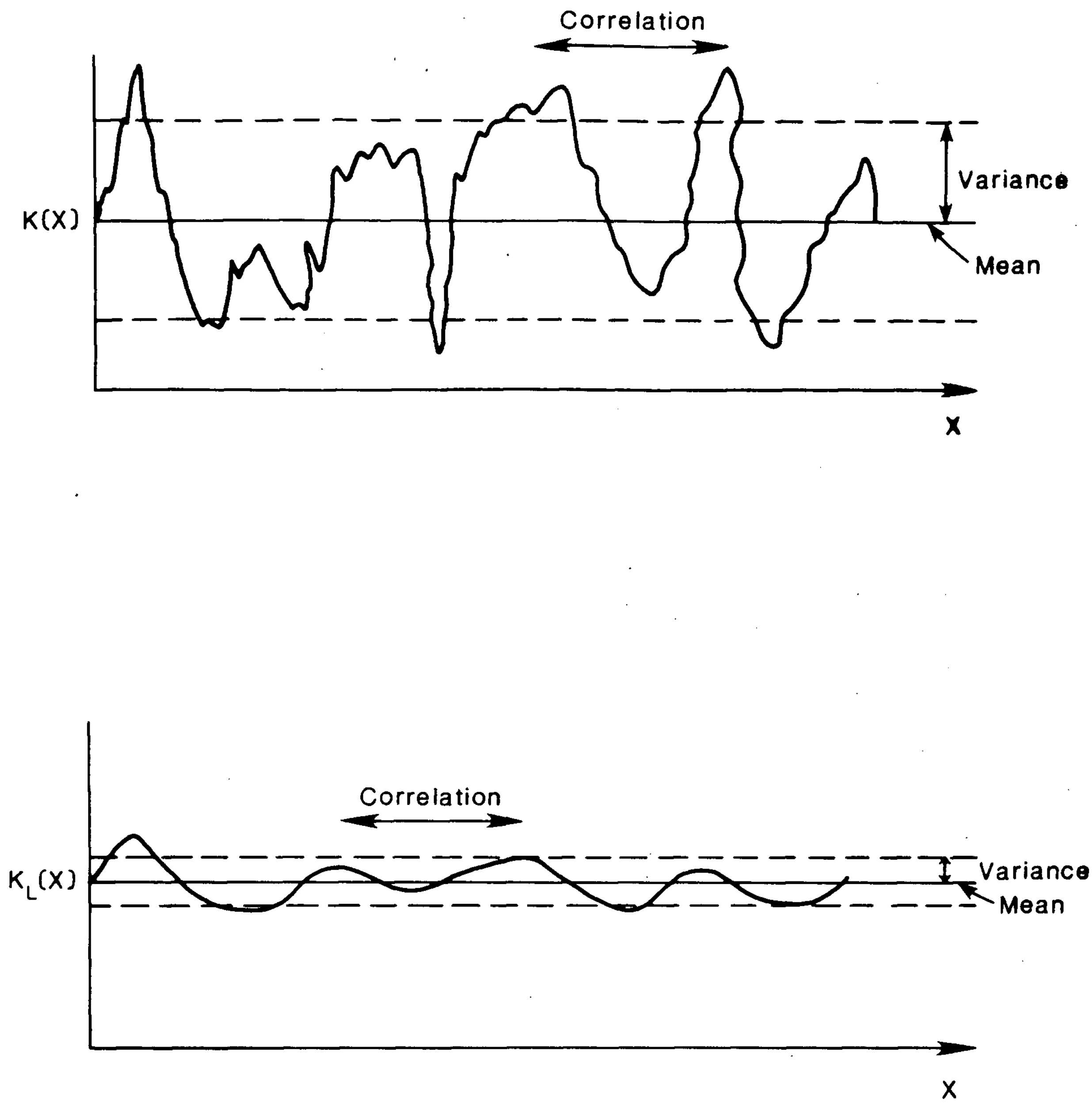


Figure 5.14 - Sample Function for a) Point Values of Hydraulic Conductivity and b) Locally-Averaged Values of Hydraulic Conductivity

can be defined over the location dimension, X.

From the continuous function of point values, a family of moving averages can be obtained from:

$$K_L(x) \quad = \quad 1/L \int_{x-L/2}^{x+L/2} K(x)dx \qquad (5.46)$$

where L denotes the averaging length. The relationship between the point values and the locally averaged values is shown in Figure 5.14b. The local averages are smoother than the point values. Increasing the averaging length, L, causes even more smoothing. This smoothing is equivalent to a reduction in variance with an increase in averaging length.

A relationship between the variance of the point values and the variance of the locally averaged values can be defined:

$$Var(K_L) \quad = \quad \gamma(L)Var(K) \qquad (5.47)$$

where $\gamma(L)$ is defined as the variance function of K(X). It measures the reduction of the point variance due to local averaging. The relationship between the variance function and the correlation function is [Vanmarcke, 1983]:

$$\gamma(L) \quad = 2/L \int_0^L (1.-1/L)\, \rho(1)d1 \qquad (5.48)$$

For the triangular correlation function shown on Figure 5.15a, the variance function is given by

$$\gamma(L) \quad = \quad 1 - L/(3a) \qquad L < a \qquad (5.49)$$

$$\gamma(L) \quad = a/L(1-a/(3L)) \qquad L > a$$

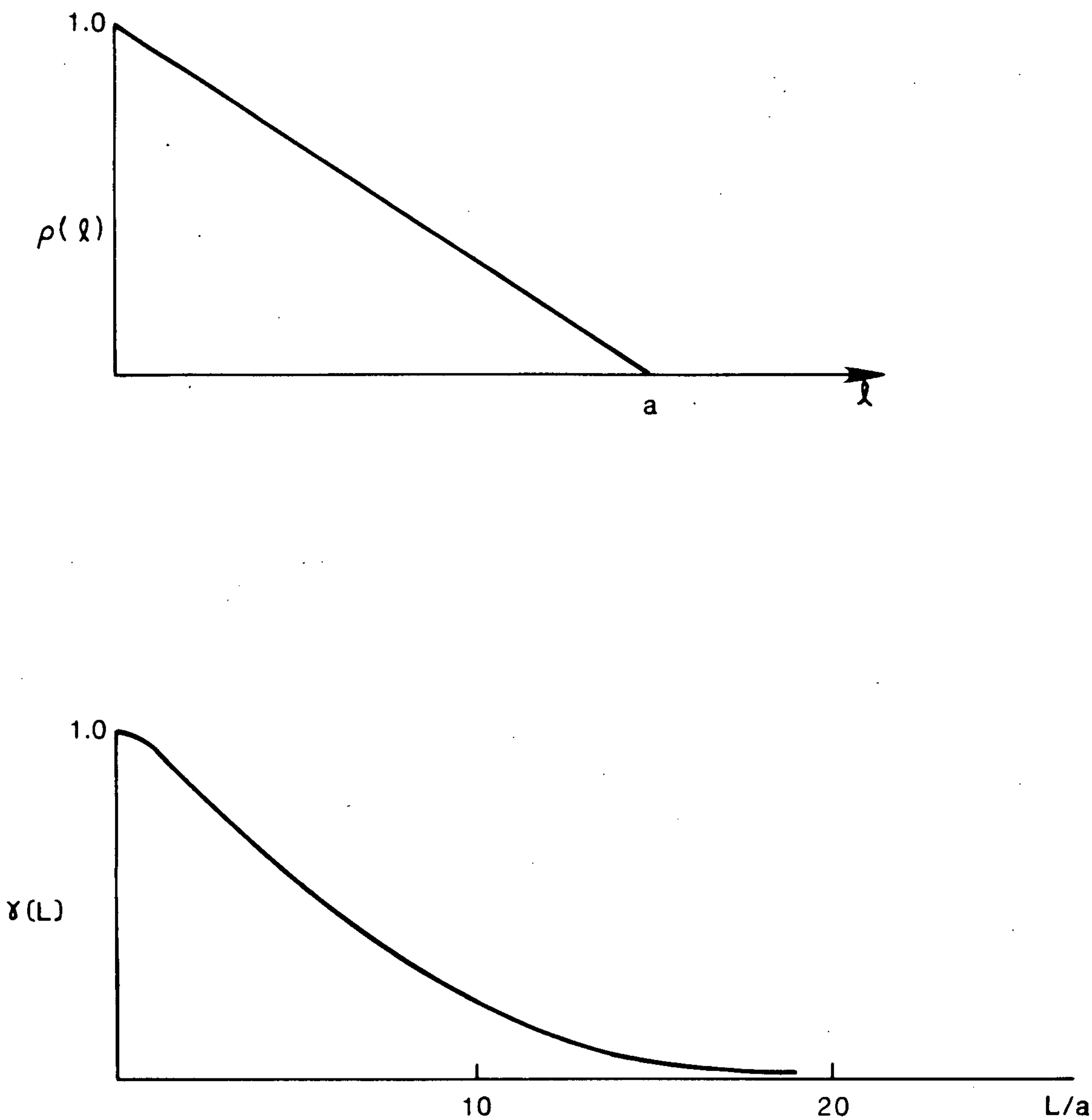


Figure 5.15 - Triangular a) Correlation Function and b) Variance Function

The variance function given by Equation (5.49) is shown on Figure 5.15b.

The variance function can be used to determine the correlation of local averages. Consider the two segments L and L' shown in Figure 5.16. The local averages are defined as

$$K_L = K_L(x) = 1/L \int_{x-L/2}^{x+L/2} K(x)dx \tag{5.50a}$$

$$K_{L'} = K_{L'}(x) = 1/L' \int_{x-L'/2}^{x+L'/2} K(x')dx' \tag{5.50b}$$

The correlation of these local averages is given by [Vanmarcke, 1983]:

$$\rho(K_L, K_{L'}) = \sum_{k=0}^{3} \frac{(-1)^k \quad (L_k)}{2[\Delta(L)\ \Delta(L')]^{1/2}} \tag{5.51}$$

where L_0, L_1, L_2, and L_3 are defined in Figure 5.16 and $\Delta(L)$ is given by

$$\Delta(L) = L^2 \gamma(L) \tag{5.52}$$

A set of equations similar to Equations (5.46) through (5.52) can also be developed for the two-dimensional random field shown on Figure 5.17. The two-dimensional correlation function is assumed

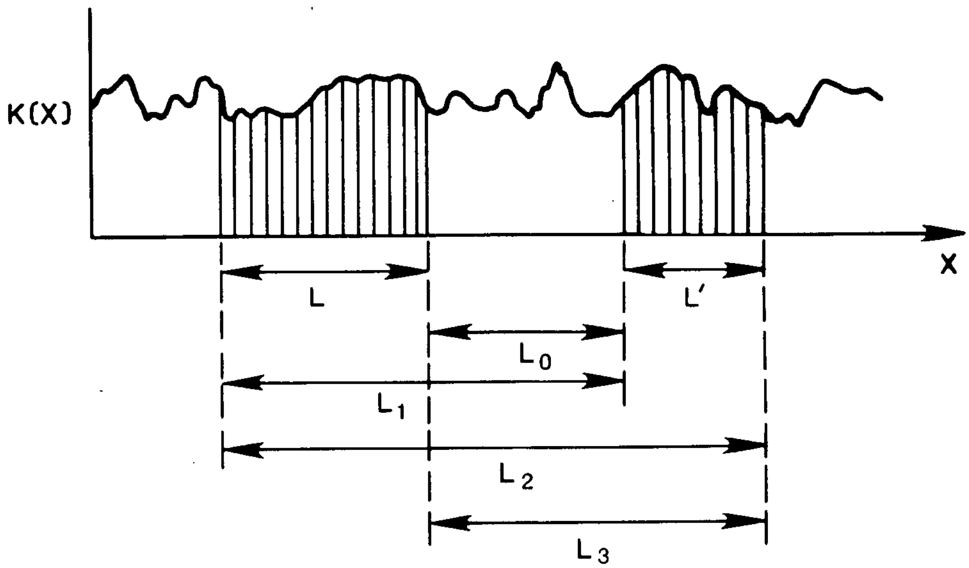


Figure 5.16 - Intervals Used to Determine Correlations of Local Averages

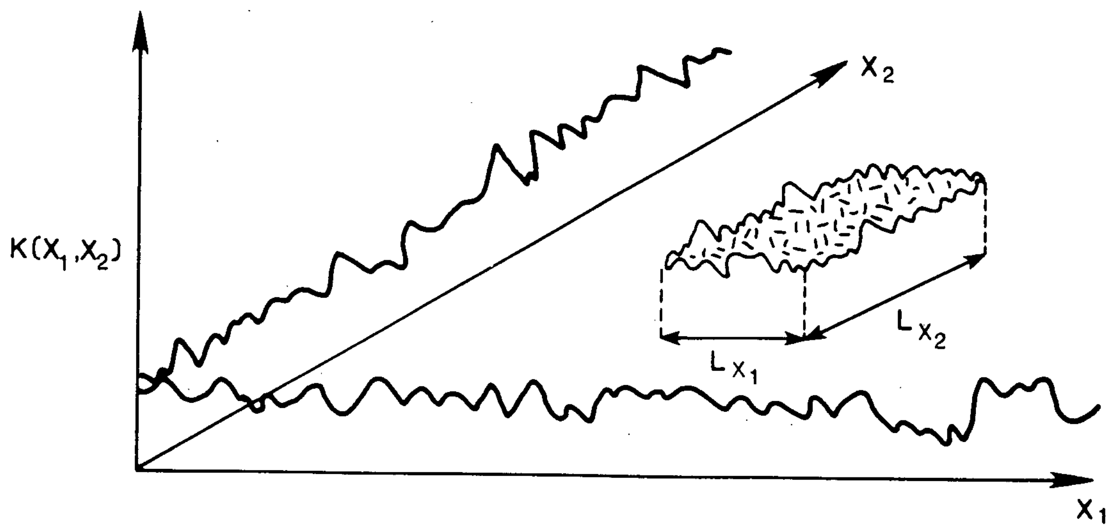


Figure 5.17 - Local Averaging Over a Rectangular Area

to be given by

$$\rho(h_1,h_2) = (1 - |h_1|/\lambda_1)(1 - |h_2|/\lambda_2) \qquad (5.53)$$
$$\text{for } |h_1| < \lambda_1 \text{ and } |h_2| < \lambda_2$$

This function, which is shown on Figure 5.18, is termed separable since it can be expressed as a product of one-dimensional functions [Vanmarcke, 1983]. The advantage of using a separable correlation function is that the corresponding variance function is also separable. For the correlation function given by Equation (5.53), the variance function is defined as

$$\gamma(L_1,L_2) = \gamma_1(L_1)\gamma_2(L_2) \qquad (5.54)$$

The one-dimensional variance functions $\gamma_1(L_1)$ and $\gamma_2(L_2)$ are given by Equation (5.49).

We are now in a position to define the covariance of two-dimensional local averages. The local averages for the two rectangular areas A and A' shown on Figure 5.19 are

$$K_A(x_1,x_2) = 1/A \int_{x_1-L_1/2}^{x_1+L_1/2} \int_{x_2-L_2/2}^{x_2+L_2/2} K(x_1,x_2)dx_1dx_2 \qquad (5.55a)$$

$$K_{A'}(x_1',x_2') = 1/A' \int_{x_1'-L_1'/2}^{x_1'+L_1'/2} \int_{x_2'-L_2'/2}^{x_2'+L_2'/2} K(x_1',x_2')dx_1'dx_2' \qquad (5.55b)$$

The covariance of these local averages is given by [Vanmarcke, 1983]

$$\mathrm{Cov}(K_A,K_{A'}) = \frac{\mathrm{Var}(K)}{4(AA')^2} \sum_{k=0}^{3} \sum_{l=0}^{3} (-1)^k(-1)^l \Delta(L_{1k},L_{2l}) \qquad (5.56)$$

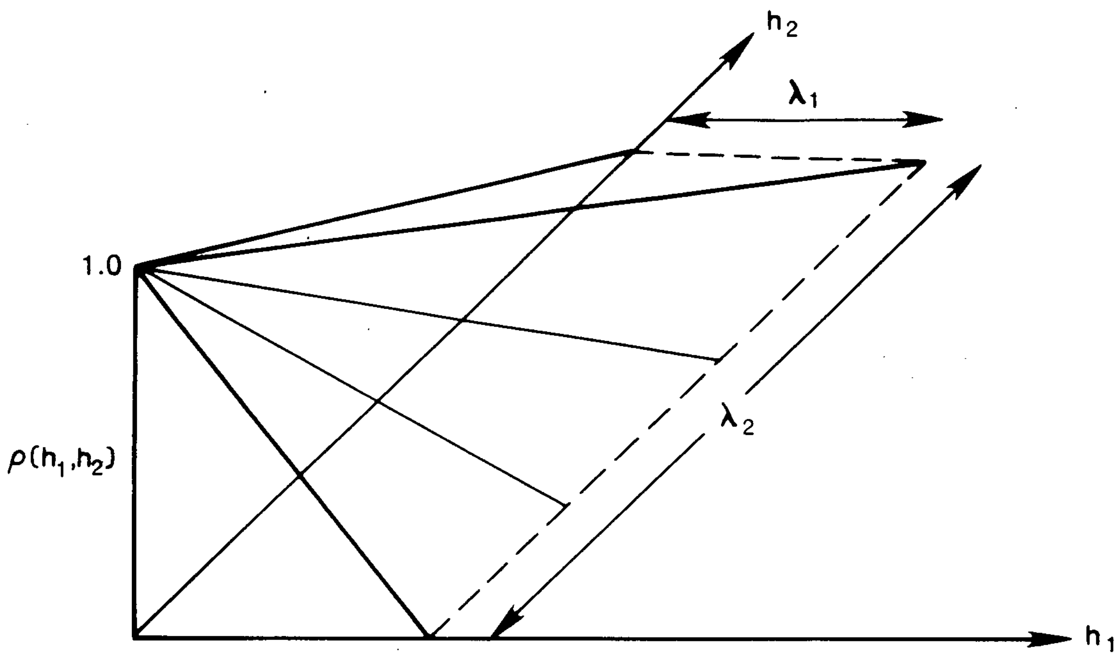


Figure 5.18 - Two-Dimensional Triangular Correlation Function

where L_{1k} and L_{21} are defined on Figure 5.19 and $\Delta(L_{1k},L_{21})$ is given by

$$\Delta(L_{1k},L_{21}) = (L_{1k}L_{21})^2 \gamma(L_{1k},L_{21}) \quad (5.57)$$

Equations (5.56) and (5.57), combined with (5.54) and (5.49), are used to generate covariance matrices for spatially averaged values of hydraulic conductivity.

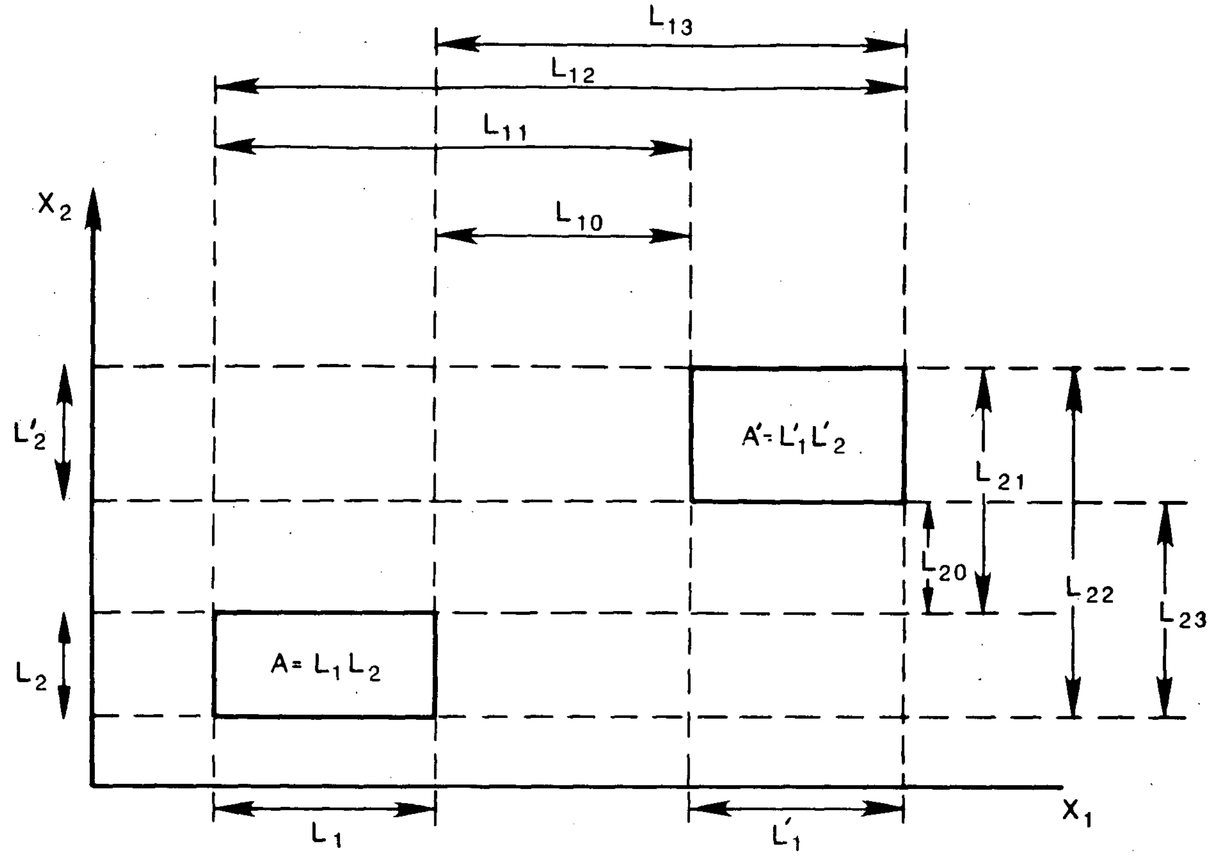


Figure 5.19 - Intervals Used to Determine Correlations of Two-Dimensional Local Averages

5.4 Summary

The procedures used to estimate contaminant travel times through the hydrogeological environment are presented in this chapter. For reasons presented at the conclusion of Section 5.1, advection is the most important transport mechanism with regard to risks associated with groundwater contamination from waste management facilities. Advective travel times can be estimated by solving Equations 5.21 and 5.27 using finite element methods. Equation 5.21 is the steady-state groundwater flow equation written in terms of stream values and Equation 5.27 relates stream values and travel times. Boundary conditions are implemented using Equations (5.29) and (5.30).

The travel times predicted using finite element representations of Equations (5.21) and (5.27) are uncertain, primarily due to uncertainty and variability in hydraulic conductivity. This uncertainty is quantified by treating the hydraulic conductivity of an element in the flow field as a spatially-dependent, scale-dependent, uncertain variable. The multivariate cummulative distribution function given by Equation (5.39) is used to fully describe the complete set of conductivities that make up the flow field.

In its most general form, the multivariate distribution function given by Equation (5.39) includes an NxN covariance matrix, where N is the number of elements in the flow field. Because of the effects of spatial averaging, the magnitude of the variances and covariances that are incorporated into this matrix are scale-dependent. Variance functions, given by Equations (5.48) and (5.54), can be used to estimate the variances and covariances of spatial averages from the variances and covariances estimated from point measurements of hydraulic conductivity. For the two dimensional flow fields used in this study, Equations (5.56) and (5.57) can then be used to estimate the elements of the covariance matrix for the hydraulic conductivities elements.

6. INCORPORATING HYDRAULIC CONDUCTIVITY MEASUREMENTS AND MONITORING WELLS

The previous chapter presents the premise that at waste management facilities in which the risks of groundwater contamination are likely important, advection is often the predominant transport mechanism. The acceptance of this premise allows contaminant travel times to be predicted with some confidence using computer models. The primary sources of uncertainty in travel time predictions for advective flow systems are uncertainty and variability in hydraulic conductivity. The hydraulic conductivity uncertainty and variability can be quantified by treating hydraulic conductivity as a random field in a Bayesian framework. This approach was introduced in the previous chapter.

Techniques for incorporating the effects of hydraulic conductivity measurements are described in this chapter. These measurements reduce uncertainty associated with hydraulic conductivity estimates and with travel time predictions. Section 6.1 discusses impacts of measurements on hydraulic conductivity estimates and Section 6.2 discusses impacts of measurements on travel time predictions. Section 6.3 describes techniques to incorporate the effects of groundwater monitoring efforts in reducing the probability of failure for the waste management facility.

6.1 Effects of Measurements on Parameter Uncertainty

At most waste-management sites, some type of site investigation will be performed. These investigations are used to determine stratigraphy and to determine material properties. In the present study, we assume that the basic stratigraphy is known and that we are concerned primarily with investigations aimed at determining material properties; specifically, spatially-averaged values of hydraulic conductivity. These hydraulic conductivity measurements can be used to reduce uncertainty by applying a number of different approaches and techniques. Several of these will be described in this section.

6.1.1 Unconditional and Conditional Operations

Finite-element computer models are used in the present study to predict advective contaminant travel times. The flow field is divided into a large number of discrete elements. As discussed in the previous chapter, the hydraulic conductivity of each element in the flow field is a spatially-dependent, scale-dependent, uncertain variable. To fully describe the complete set of hydraulic conductivities that make up the flow field, it is necessary to specify a multivariate cumulative distribution function:

$$F(w_1, w_2, \ldots w_N) = \text{Prob}(K_1 < w_1, K_2 < w_2, \ldots K_N < w_N) \tag{6.1}$$

where

K_i = hydraulic conductivity of element i

N = number of elements in the flow field.

The derivative of the cumulative distribution function is defined as the multivariate probability density function (PDF):

$$f(w_1,w_2, \ldots w_N) = \frac{\partial^N F(w_1,w_2, \ldots w_N)}{\partial w_1 \partial w_2 \ldots \partial w_N} \tag{6.2}$$

The notation used in Equations (6.1) and (6.2) can be simplified:

$$\begin{aligned} f(\overline{w}_N) &= f(w_1,w_2, \ldots w_N) \\ F(\overline{w}_N) &= F(w_1,w_2, \ldots w_N) \end{aligned} \tag{6.3}$$

where $\overline{w}_N$ denotes a vector with N elements.

If the form and parameters of the cumulative distribution function are assumed to be the same for all the hydraulic conductivity blocks, the random field is stationary and can be modeled with a single, univariate cumulative distribution function, $F(w) = \text{Prob}[K < w]$. A univariate probability density function can also be defined:

$$f(w) = dF(w)/dw \tag{6.4}$$

The information obtained from the hydraulic conductivity measurements can be used in two operations:

1) To modify the parameters of the probability density functions, and
2) To modify our best estimate of hydraulic conductivity at the unmeasured locations.

If the locations of the measurements are included, the operation is termed conditional. If the locations are not included, it is unconditional. Unconditional operations preserve stationarity while conditional operations generally do not.

The first step in incorporating hydraulic conductivity measurements is to determine the unconditional, posterior PDF from Bayes Theorem. If we assume that hydraulic conductivity measurements are made in J of the N blocks , where J = N-M, then:

$$f_u'(w_N) = GL(K_{M+1}, K_{M+2}, \ldots K_N) f_u(\overline{w_N}) \tag{6.5}$$

where

$L(K_{M+1}, \ldots\ldots K_N)$	=	likelihood of obtaining the sample $(K_{M+1}, \ldots\ldots K_N)$
G	=	normalizing constant

The likelihood function is not dependent upon the absolute locations of the (N-M) measurements. The subscripts, u, in Equation (6.5) are used to denote unconditional operations.

The best estimate of K_i without including the locations of the measurements is given by the unconditional, posterior expected value:

$$E[K_i]_u' = \int_{-\infty}^{\infty} K_i f_u'(w_i) dw_i \tag{6.6}$$

The unconditional, posterior covariance is given by:

$$[C_{ij}]_u' = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} (K_i - E[K_i]_u')(K_j - E[K_j]_u') f_u'(w_i, w_j) dw_i dw_j \tag{6.7}$$

The unconditional, posterior variance, $[C_{ii}]_u'$, may be larger or smaller than the prior variance, $[C_{ii}]_u$, depending on whether $f_u'(w)$ is more or less diffuse than $f_u(w)$.

The unconditional posterior expected value given by Equation (6.6) is the best estimate if measurement locations are not included. However, a better estimate is given by the conditional expected value, which includes locations:

$$E[K_i]_c' = \int_{-\infty}^{\infty} K_i f_c'(w_i) dw_i \tag{6.8}$$

where $f_c'(w)$ = conditional posterior probability density function. The subscripts, c, are used to denote conditional operations. Even though the unconditional posterior PDF, $f_u'(w)$, was constant for all blocks, the conditional posterior PDF, $f_c'(w)$, is generally not.

The conditional posterior covariance is given by:

$$[C_{ij}]_c' = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} (K_i - E[K_i]_c')(K_j - E[K_j]_c') f_c'(w_i, w_j) dw_i dw_j \tag{6.9}$$

The conditional, posterior covariance given by Equation (6.9) is always less than or equal to the unconditional, posterior covariance given by Equation (6.7). However, it may be larger or smaller than the prior covariance.

The conditional posterior PDF is related to the unconditional PDF by the following formula (Dagan, 1982):

$$f_c'(w_1, w_2, \ldots w_M \,/\, K_{M+1}, K_{M+2}, \ldots K_N) = f_u'(\overline{w}_N)/f_M(\overline{w}_M) \quad (6.10)$$

where

$$f_M(\overline{w}_M) \quad = \quad \text{marginal probability density function}$$

The marginal PDF is obtained by integrating the measured values out of the unconditional posterior PDF:

$$f_M(w_M) = \int_{-\infty}^{\infty} \int_{-\infty}^{\infty} \cdot \cdot \cdot \cdot \int_{-\infty}^{\infty} f_u'(\overline{w}_N) dw_{M+1} \ldots dw_N \quad (6.11)$$

These integrations are usually cumbersome at best, and often are analytically impossible.

Two assumptions are generally used to avoid some of the integration problems inherent in determining the best possible estimates at the unmeasured points. The first assumption is that the variables, or some transformation of the variables, have a multivariate, normal PDF. The second assumption is that a linear function of the measured points is used to estimate the values at the unmeasured points.

6.1.2 Multivariate Normal Distribution

There is a fairly extensive literature, both theoretical and empirical, that suggests that hydraulic conductivities are often lognormally distributed [e.g. Freeze, 1975; Hoeksema and Kitandis, 1985; Jury, 1987]. If Y_i is defined as the logarithm of K_i, then the set of Y_i values constitutes a set of random variables with a normal PDF function. The multivariate normal

PDF for a set of N elements is given by:

$$f(w_N) = \frac{|U|^{1/2}}{(2\pi)^{N/2}} \text{EXP}[-1/2 \sum_{i=1}^{N} \sum_{j=1}^{N} U_{ij}(w_i-E[Y_i])(w_j-E[Y_j])] \quad (6.12)$$

where

U = inverse of the covariance matrix

$|U|$ = determinant of U

The assumption that the logarithms of hydraulic conductivities are normally distributed greatly simplifies the integrations that are needed to determine the marginal and conditional posterior PDF's presented in Equations (6.10) and (6.11). In fact, the marginal distribution has the same form as Equation (6.12). However, the i,j element in the matrix U is replaced by:

$$U_{ij} - \sum_{l=M+1}^{N} \sum_{m=M+1}^{N} U_{il}U_{jm}[C_{ij}]_{u}' \quad (6.13)$$

where

$[C_{ij}]_{u}'$ = unconditional, posterior covariances

For two-dimensional analyses, the stochastic structure incorporated in the multivariate normal distribution is fully represented by four sets of parameters: 1) a vector of mean values, E[Y], 2) a standard deviation, σ_Y, 3) a correlation length in the x-direction, λ_x, and 4) a correlation length in the z-direction, λ_z. In some ways it is easier to grasp values given in terms of K rather than Y. We therefore present some of our results in terms of the mean hydraulic conductivity $\bar{K}$, the standard deviation σ_K, and the correlation ρ_K rather than in terms of $\bar{Y}$, σ_Y, and ρ_Y. These variables are related by the following expressions [Vanmarcke, 1983]:

$$E[K] = \bar{K} = EXP(E[Y] + \sigma_Y^2/2) \tag{6.14}$$

$$\sigma_K^2 = [EXP(\sigma_Y^2) - 1]EXP(2E[Y] + \sigma_Y^2) \tag{6.15}$$

$$\rho_K(x_1,x_2) = \frac{[EXP(\rho_Y(x_1,x_2)\sigma_Y^2) - 1]}{EXP(\sigma_Y^2) - 1} \tag{6.16}$$

6.1.3 The Observational Model

The observational model is used to estimate hydraulic conductivities at unmeasured locations from measured locations. Again, we assume that there are N elements in the flow system and that observations are made in J of these elements and no measurements are made in M elements (M+J=N). From the set of observations, a Jx1 vector of log conductivities is determined by linear regression:

$$[R] = [P][Y] + [e] \tag{6.17}$$

where

R = Jxl vector of observed values,

Y = Nxl vector of actual values,

P = JxN matrix of regression coefficients, and

e = Jxl vector of zero-mean measurement errors.

The matrix of regression coefficients, [P], can be used to incorporate measurement biases or to allow for the fact that hydraulic conductivities may be inferred from some alternative parameter, such as a grain size value. In the simplest case with no biases, the matrix [P] would contain only zeros and ones.

The observational model given by Equation (6.17) can be used to directly determine the conditional, posterior probability density function. If the log conductivity values, [Y], are normally distributed and if the measurement errors are normally distributed with mean zero, then the conditional posterior PDF is normally distributed with means and covariances given by:

$$E[Y]_c' = E[Y]_u' + [C_Y]_u'[P]^T([P][C_Y]_u'[P]^T + [E])^{-1}([R]-[P]E[Y]) \tag{6.18}$$

$$[C_Y]_c' = [C_Y]_u' - [C_Y]_u'[P]^T([P][C_Y]_u'[P]^T + [E])^{-1}[P][C_Y]_u' \tag{6.19}$$

where the primed terms are after measurements have been made and where

$[E]$ = covariance matrix for measurement errors

$E[Y]_u'$ = unconditional, posterior expected value,

$[C_Y]_u'$ = unconditional, posterior covariances,

$E[Y]_c'$ = conditional, posterior expected value, and

$[C_Y]_c'$ = conditional, posterior covariances.

A detailed development of these equations can be found in the text by Bryson and Ho [1969]. They are used in a seepage analysis by Hachich and Vanmarcke [1983] and in an aquifer parameter estimation study by Clifton and Neumann [1982].

6.1.4 Sensitivity Studies

The covariances given by Equation (6.19) can be used to evaluate the sensitivity of the observational model to various input parameters and to qualitatively evaluate the effectiveness of various sets of observations in reducing uncertainty in hydraulic conductivity at unmeasured locations. A simple, one-dimensional flow field can be used to illustrate the effects that different parameters in Equation (6.19) have on hydraulic conductivity uncertainty. The flow field is divided into ten equally-sized elements numbered from left to right. The hydraulic conductivity of the flow field is assumed to have a linear correlation function with a fluctuation scale equal to the length of four elements. The unconditional variance of the hydraulic conductivity is assumed equal to 1.0, the measurement error variance is assumed equal to 0.25, and the measurements are assumed to be unbiased so the regression coefficients are assigned values of 1.0. Measurements are assumed to be made in element number 3 and in element number 7.

Figure 6.1a illustrates the effects of measurement errors on the conditional hydraulic conductivity variance. The upper line shows a case with a measurement error of 4.0. The hydraulic conductivity uncertainty decreases as the measurement errors decrease to 1.0 and then to 0.25. However, no matter how large the measurement error becomes, measurements always decrease the unconditional variance. Figure 6.1b illustrates the effects of regression coefficients. As the coefficients increase, the conditional variance decreases. Large regression coefficients imply more sensitive measurement techniques. Finally, Figure 6.1c illustrates the effects of correlation lengths. Correlation lengths enter the analysis through the unconditional covariance matrix. As the correlation length increases, the "zone-of-influence" of the measurements also increases. If the fluctuation scale is only one element long, measurements reduce uncertainty only in the measured elements. Conversely, if the correlation length is more than four elements long, the measurements reduce uncertainty in all the elements.

To illustrate the impacts of correlation lengths in more detail, measurements were also simulated in the two-dimensional flow

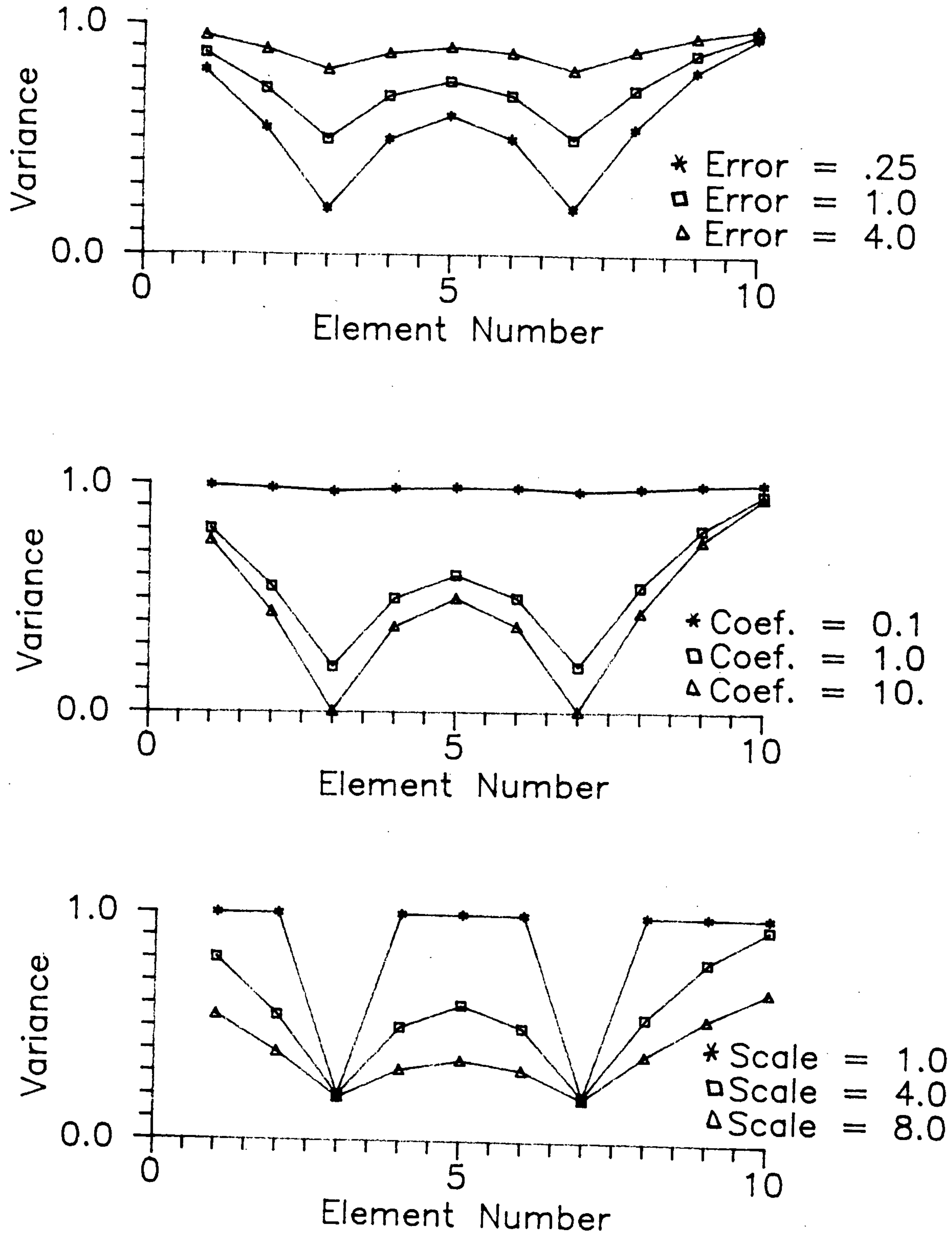


Figure 6.1- Example Sensitivities for Hydraulic Conductivity Measurements in a One-Dimensional Flow Field

field shown on Figure 6.2. In this figure, which is in plan view, the dots indicate hydraulic-conductivity measurement points and the cross-hatching illustrates areas in which the conditional standard deviation is less than one-half the unconditional standard deviation. These areas can be thought of as areas in which the measurements reduce uncertainty by one-half.

The hydraulic conductivities were assumed to have a triangular correlation structure similar to that shown on Figure 5.18. On the upper diagram, Figure 6.2a, the fluctuation scale in the x-direction is L/6 and the fluctuation scale in the z-direction is L/18. This represents a geology with relatively small correlation scales, such as a glacial till or an alluvial deposit. On the lower diagram, Figure 6.2b, the fluctuation scale in the x-direction is L/2 and the fluctuation scale in the z-direction is again L/18. This represents a geology with larger correlation scales, such as a lacustrine deposit. Figure 6.2 clearly shows that measurements are much more effective in geologies which exhibit more extensive correlation structures.

6.1.5 Multivariate Normal Analyses and Kriging

In recent years, considerable attention has been focused on using Kriging to contour hydraulic conductivities and to study the impacts of hydraulic conductivity measurements on hydraulic conductivity uncertainty. Kriging is an interpolation technique that incorporates the correlation structure of the hydraulic conductivity field in determining weighting coefficients.

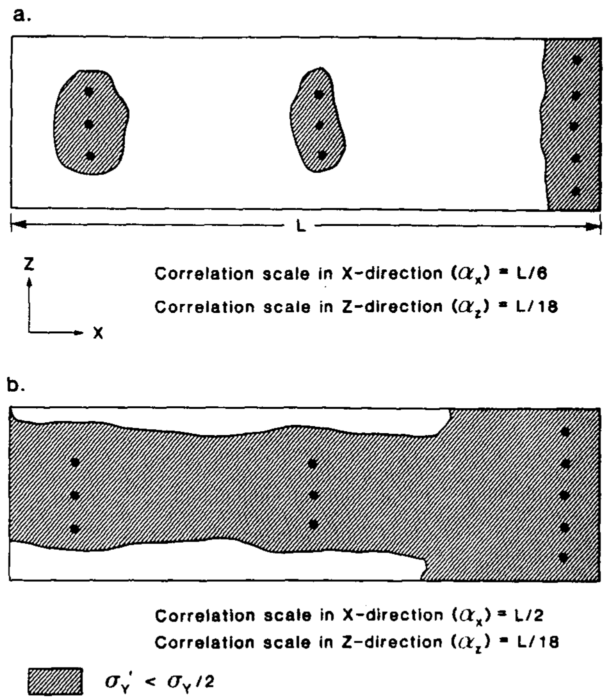


Figure 6.2 - Impact of Correlation Scales on Measurement Effectiveness

Several of the attractive features of Kriging are 1) it preserves the measured values, 2) it gives minimum variance estimates at unmeasured points, and 3) it provides a measure of uncertainty at the unmeasured points.

Dagan [1982] provides a detailed development that shows the Kriging equations and Equations 6.18 and 6.19 should give equivalent results if stationary increments are assumed. As a check on Equations 6.18 and 6.19, a one-dimensional flow field consisting of 50 hydraulic conductivity values was generated. This flow field was then "sampled" at selected locations and Kriging equations and multivariate normal equations (Equations 6.18 and 6.19) were used to estimate the hydraulic conductivities at the unmeasured locations.

Figure 6.3 presents log hydraulic conductivities for the flow field. These values, which represent "reality," were generated numerically. The population from which they were selected had a mean value of $Y = -12$, a variance equal to 0.65, and a linear correlation structure with a fluctuation scale equal to 5. The actual 50 values shown on Figure 6.3 have a mean and variance quite close to -12 and 0.65, respectively. However, the fluctuation scale for the 50 values is approximately 7, rather than 5. This will have an impact on the Kriging estimates, as discussed below.

Hydraulic conductivity measurements were made at 16 equally-spaced locations in the flow field. These 16 measurements were

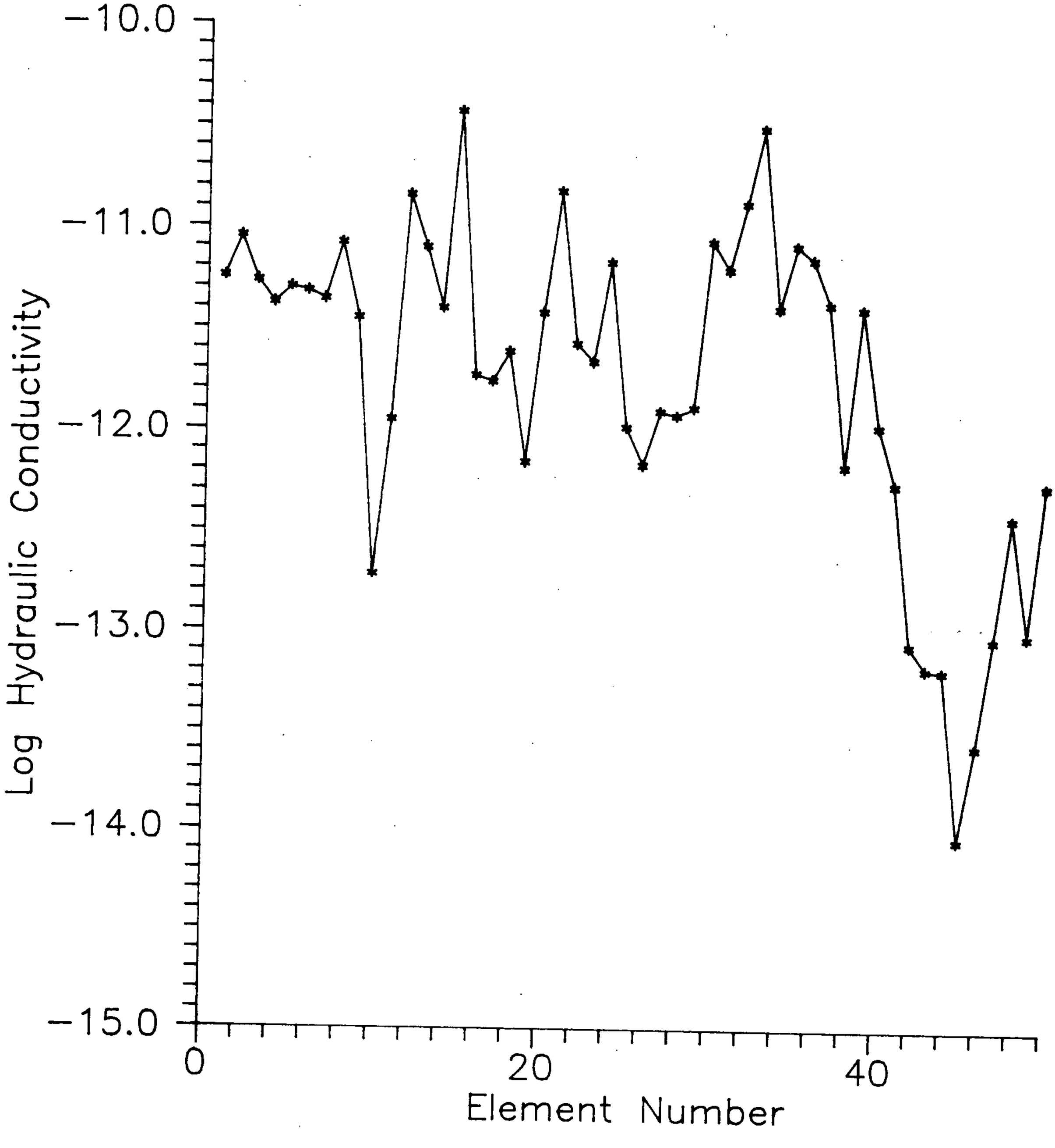


Figure 6.3 - Log Hydraulic Conductivities for Hypothetical Flow Field

then used to estimate the values at the unmeasured points using Equations 6.18 and 6.19 and using the Kriging equations. The software developed by Devary and Hughes [1984] was used to obtain the Kriged values. The results, which are presented on Figure 6.4, illustrate that the two procedures give very similar results. The slight difference between the two may be caused by the correlation structure. The Kriging software used in this study determines the correlation length directly from the data while Equations 6.18 and 6.19 use a prior fluctuation scale that is not dependent upon the data. The Kriged values are based upon a fluctuation scale of approximately 7 while the multivariate normal values are based upon a fluctuation scale of 5.

Figure 6.5 presents the root mean square error between actual values and predicted values as a function of the number of measurement locations. The solid line is for values predicted using the multivariate equations and the dashed line is for values predicted using the Kriging equations. Again, the two methods yield very similar results. The upper line included on Figure 6.5 is for unconditional estimates in which the measurement locations are not included in the analysis. With this approach, the mean value of the measurements is assigned to all locations. With the conditional approaches, the root mean square error approaches zero as all locations are measured. With the unconditional approach, the root mean square error approaches the sample standard deviation as all locations are measured.

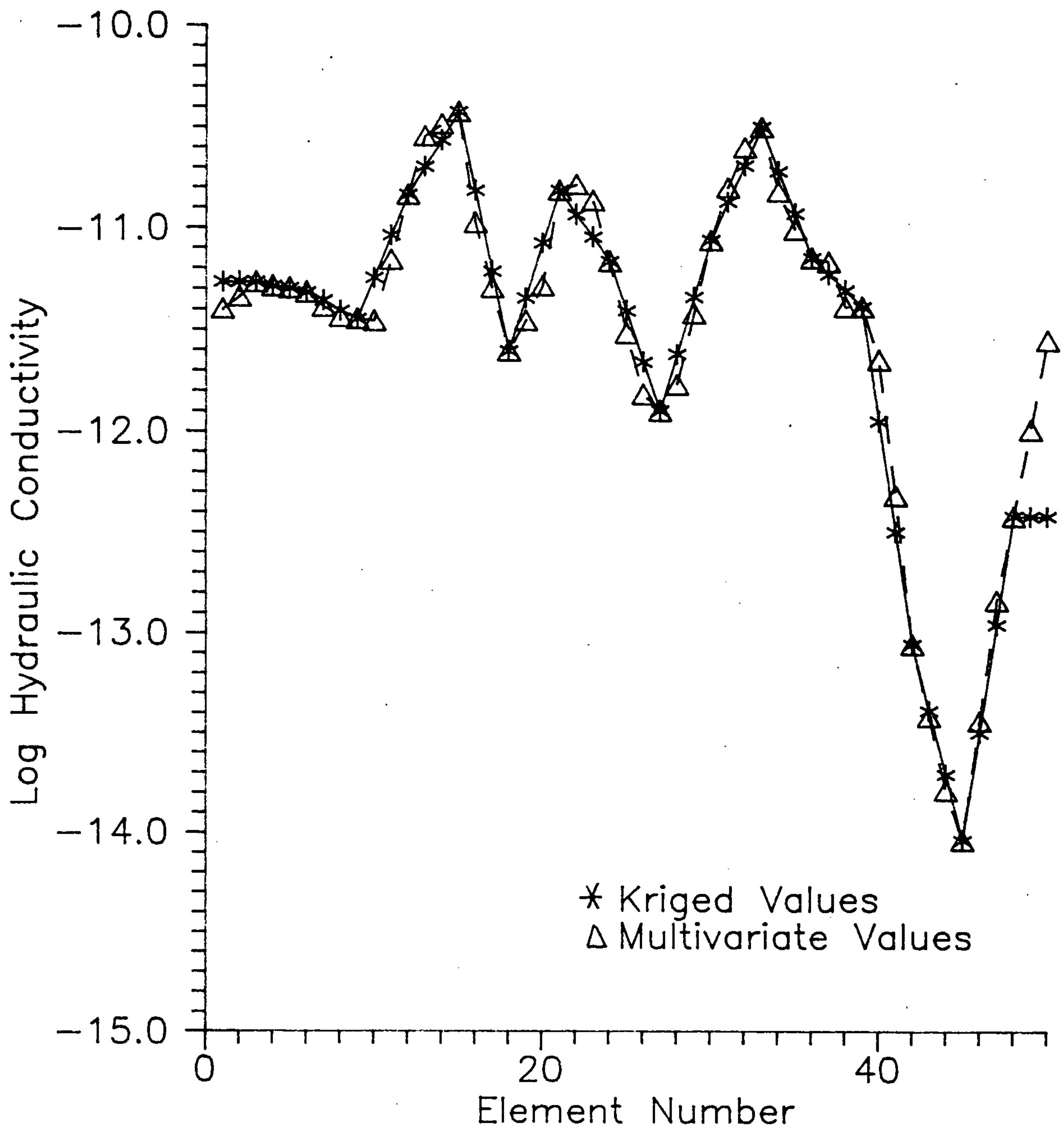


Figure 6.4 - Estimated Hydraulic Conductivities Using Kriging and Multivariate Analyses

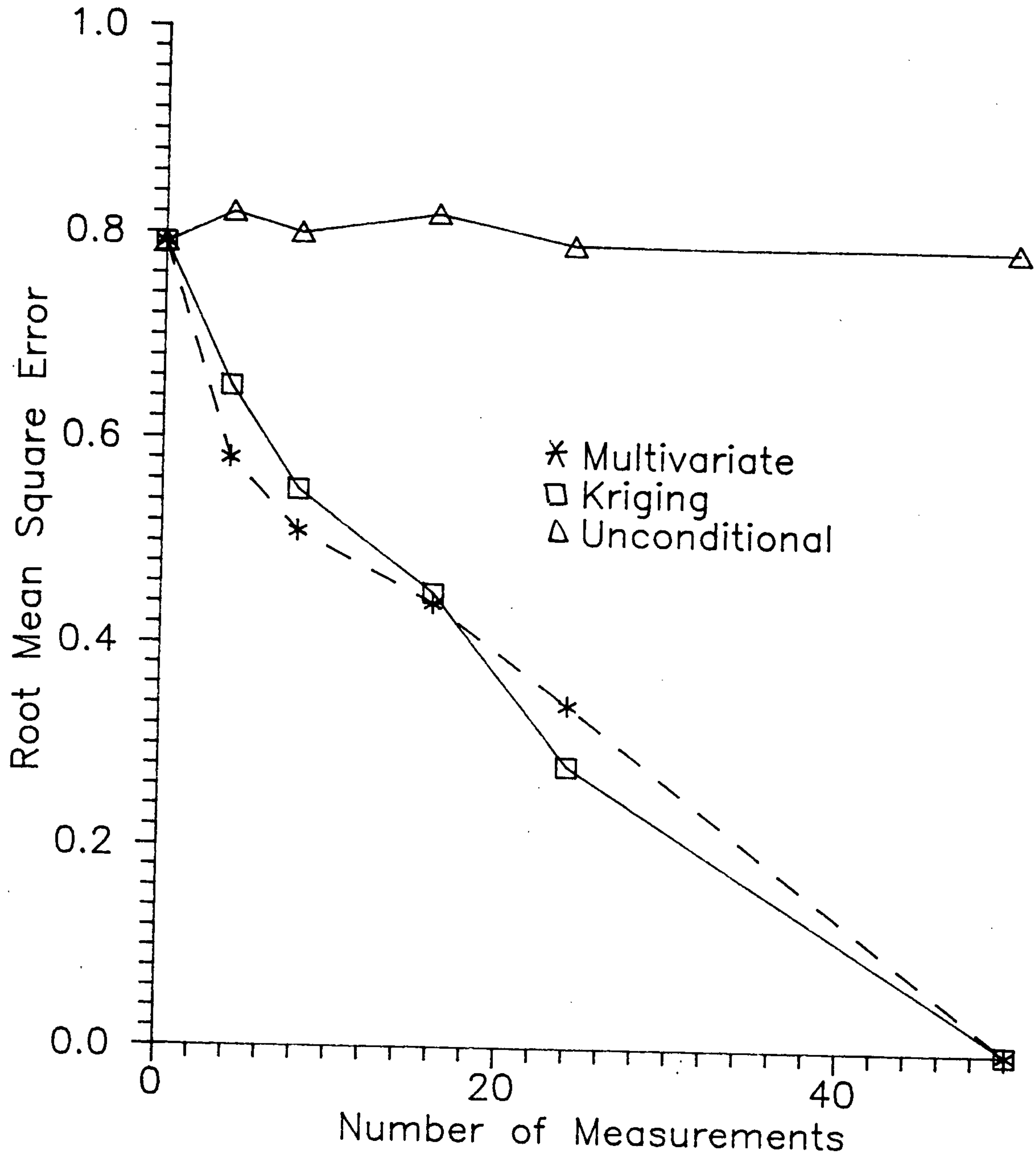


Figure 6.5 - Root Mean Square Error for Predicted and Actual Log Hydraulic Conductivities

6.2 Incorporating Parameter Uncertainties in Transport Models

The discussions in the previous section have been concerned with techniques for estimating how hydraulic conductivity measurements impact hydraulic conductivity uncertainties. In this section, we will go one step further and look at techniques for estimating how uncertainties in hydraulic conductivities translate into uncertainties in travel time predictions. Depending upon the complexity of the problem, there are three general approaches for incorporating hydraulic conductivity uncertainties into advective transport models. For relatively simple flow fields, analytical methods can be used. For more complex fields, analytical expressions are replaced with Taylor series expansions. Finally, for complex flow fields and for flow fields that have a high degree of uncertainty associated with the hydraulic conductivity, Taylor series methods do not give reliable results and Monte Carlo methods are required.

6.2.1 Analytical Methods

The simplest cases for estimating how uncertainties in hydraulic conductivity translate into uncertainties in travel times are those in which the travel time can be represented as a closed-form, analytical function of hydraulic conductivity. In these cases, the PDF for the travel times can be directly determined from the PDF for the hydraulic conductivity. In most instances in which analytical methods are used, hydraulic conductivities are viewed as a continuum, rather than as discrete values.

As an example, for a one-dimensional flow field with length, L, divided into N elements, with porosity, n, and with hydraulic gradient $(H_1 - H_0)/L$, the travel time is given by:

$$T = \left[\frac{L^2 n}{N(H_1 - H_0)} \right] R \tag{6.20}$$

where $R = \sum_{i=1}^{N} 1/K_i$

The PDF for the travel time can be directly determined from the PDF for R, if it is known. The travel time mean and variance can also be determined analytically as:

$$E[T] = \left[\frac{L^2 n}{N(H_1 - H_0)} \right] E[R] \tag{6.21a}$$

$$\sigma_T^2 = \left[\frac{L^2 n}{N(H_1 - H_0)} \right]^2 \sigma_R^2 \tag{6.21b}$$

Analytical solutions can also be obtained for more complex flow fields, as shown by Gelhar et al [1979], Bakr et al [1978], and Mizell et al [1982]. The approach that has been used on these more complex flow fields has been to transform the differential equation describing groundwater flow into a stochastic differential equation that is then solved using Fourier-Stieltjes integrals. In practice, the stochastic differential equations must be linearized before they can be solved. This linearization generally limits the validity of the approach to situations in which the variance of Y is less than 1.0. The approach is also limited in terms of the types of geometries and boundary conditions that can be modeled.

6.2.2 Taylor Series Expansions

A second approach for translating hydraulic conductivity uncertainties into travel time uncertainties is to approximate the functional relationship between travel time and hydraulic conductivities using a Taylor series approximation. This approach is also termed the method of moments. If the travel time, T, is a general function of hydraulic conductivity, g(K), then the Taylor series approximation for T is given by:

$$T \approx g(E[K]) + (K-E[K])\frac{dg}{dK} + \frac{(K-E[K])^2}{2}\frac{d^2g}{dK^2} + \ldots \qquad (6.22)$$

where the derivatives are evaluated at the mean value of K.

The mean and variance of T can be determined directly from Equation (6.22):

$$E[T] \approx g(E[K]) + \frac{\sigma_K^2}{2}\,\frac{d^2g(E[K])}{dK^2} \qquad (6.23a)$$

$$\sigma_T^2 \approx \sigma_K^2 \left[\frac{d^2g(E[K]\ ^2}{dK^2}\right] \qquad (6.23b)$$

For the one-dimensional example described in the previous section, the travel time mean and variance given by the Taylor series approximation are:

$$E[T] = \frac{L^2 n}{N(H_1-H_0)} \sum_{i=1}^{N} 1/E[K_i] + \sum_{i=1}^{N} K_i^2/K_i^3 \qquad (6.24a)$$

$$\sigma_T^2 \approx \left[\frac{L^2 n}{N(H_1-H_0)}\right]^2 \left[\sum_{i=1}^{N} \sigma_{Ki}^2/K_i^4 + \sum_{j=1}^{N}\sum_{i=1}^{N} \mathrm{Cov}(K_i,K_j)/K_i^2K_j^2\right] \qquad (6.24b)$$

Taylor series approximations can also be used for fairly complex flow fields that are modeled numerically using finite element or finite difference computer models [Tang and Pinder, 1977, Dettinger and Wilson, 1981] Sagar [1978] describes a fairly efficient approach for converting finite element groundwater flow models into stochastic models using Taylor series approximations. The primary limitation in the approach is that it is strictly valid only in the vicinity of the mean value. Large variations about the mean value will tend to give erroneous results.

To estimate the range of hydraulic conductivity variabilities over which Taylor series approximations would give reasonable results, travel time means and variances were calculated for a one-dimensional flow field using Equations (6.24) and using the Monte Carlo method that is described below. These calculations indicate that the Taylor series method underestimates travel time variances. The amount of underestimation becomes quite significant when the coefficient of variation for hydraulic conductivity exceeds approximately 0.3. Because hydraulic conductivities often exhibit coefficients of variation greater than 1.0, the Taylor series method was not used.

6.2.3 The Monte Carlo Method

The third general approach for estimating travel time uncertainty as a function of hydraulic conductivity uncertainty is the Monte Carlo approach. This approach, though somewhat demanding from a computational viewpoint, is conceptually quite simple. In its most basic form, the mechanics of the Monte Carlo method are as follows:

1) Generate sets of input values that have the desired probability distribution.
2) For each set of input values, use the physical model (computer model) to calculate output values.
3) From the output values, estimate probability distribution functions and distribution function parameters.

The number of sets of input values that are required to get good results depends upon the shape of the probability distribution for the input and upon the functional relationship between the input and the output. The minimum is several hundred and several thousand are not atypical.

Although computationally inefficient, the advantages of the Monte

Carlo approach are 1) flow fields with complex geometries and boundary conditions can be modeled, 2) hydraulic conductivity fields with large uncertainties and variabilities can be modeled, and 3) deterministic computer models can be easily adapted and used in the analysis. The approach, which has been used to study the effects of hydraulic conductivity uncertainties in groundwater flow and mass transport predictions by Freeze [1975], Smith and Freeze [1979], and Smith and Schwartz [1981], is used in the present study.

The first step in the approach, generating sets of input values that have the desired probability distribution function and correlation structure, generally requires the most effort. A number of different approaches have been used in the past. Mejia and Rodriquez-Iturbe [1974] generate realizations using spectral analysis techniques. Smith and Freeze [1979] use an autoregressive, nearest-neighbor approach. Delhomme [1979] and Montaglou and Wilson [1981] use a turning bands method in which two- and three-dimensional random fields are generated by summing up one-dimensional fields. Clifton and Neumann [1982] use an approach based upon a Cholesky decomposition of the covariance matrix.

The turning bands and Cholesky decomposition methods were considered in the present study. Both methods were used in numerical experiments and the Cholesky decomposition method was chosen because it was numerically more efficient for two-dimensional flow fields and it was simpler to incorporate into existing computer programs.

To generate realizations of Y values with the desired statistical properties, we first generate a vector, S, of normally-distributed, independent values with mean zero and variance equal to one. This can be done with any standard univariate normal random number generator. The desired realizations are then obtained by the following operation [Clifton and Neumann, 1982]:

$$[Y] = E[Y]_c' + [D][S] \tag{6.25}$$

where [D] is the lower triangular matrix defined as

$$[D][D]^T = [C]_c' \tag{6.26}$$

and where

$[Y]$ = vector of random values with desired mean, variance, and correlation structure,

$E[Y]_c'$ = vector of posterior, conditional mean values,

$[S]$ = vector of uncorrelated random values with mean zero and variance one, and

$[C]_c'$ = posterior, conditional covariance matrix.

The matrix [D] is most easily determined by a Cholesky decomposition of the posterior, conditional covariance matrix. This decomposition needs to be performed once for each suite of realizations that is generated. After the decomposition has been completed, the individual realizations require three operations: 1) generation of a vector of uncorrelated, normal random values, 2) multiplication of this vector with the matrix [D], and 3) adding the product to the vector of posterior conditional mean values, $E[Y]_c'$.

After a vector of Y values with the desired statistical properties has been generated, the second step in the Monte Carlo procedure is to convert these Y values into hydraulic conductivities using the expression:

$$K = e^Y \tag{6.27}$$

The hydraulic conductivities are then used to estimate travel times using the advective transport model described in Section 5.2. One travel time is calculated for each vector of hydraulic conductivities that is generated. The flow path with the minimum travel time from the source to the compliance surface is chosen to calculate the travel time for each hydraulic conductivity realization.

The final step in the Monte Carlo approach is to statistically evaluate the travel times. This evaluation includes calculating means and variances and also estimating the form of the probability distribution function for the travel times. The Chi-squared test [Benjamin and Cornell, 1970] was used to check if travel times were either normally or lognormally distributed. In the majority of instances, the lognormal distribution gave a better fit for the travel times. Once the probability distribution function and parameters have been estimated for travel times, the second term in the expression for the probability of failure for the waste management facility

(Equation (3.7)) can be calculated.

6.2.4 Travel Time Sensitivities

In this section, sensitivity studies are presented which show how travel times statistics are affected by mean hydraulic conductivity values, by hydraulic conductivity uncertainties, by fluctuation scales, and by hydraulic conductivity measurements. The flow field that is used in the sensitivity studies is illustrated on Figure 6.6. In this figure, which is in plan view, the cross-hatched area represents the landfill and the right boundary represents the compliance surface. The boundary conditions that are assumed are 1) the left boundary of the flow field is a prescribed flux boundary, 2) the right boundary is a prescribed head boundary, and 3) the upper and lower boundaries are impermeable. The flow field is assumed to be 1000 meters long and 700 meters wide and the landfill is assumed to be 100 meters wide. For the base case, the mean hydraulic conductivity is assumed to be 1500 meters per year, the hydraulic conductivity standard deviation is assumed to be 1500 meters per year, and the fluctuation scales in the x- and z-directions were both assumed to be 300 meters. The porosity was assigned a value of 0.20 and the head drop across the flow field is 8.2 meters. After experimenting with a number of different values, two hundred and fifty Monte Carlo realizations was found to give reliable results.

Table 6.1 illustrates the effects of the mean hydraulic

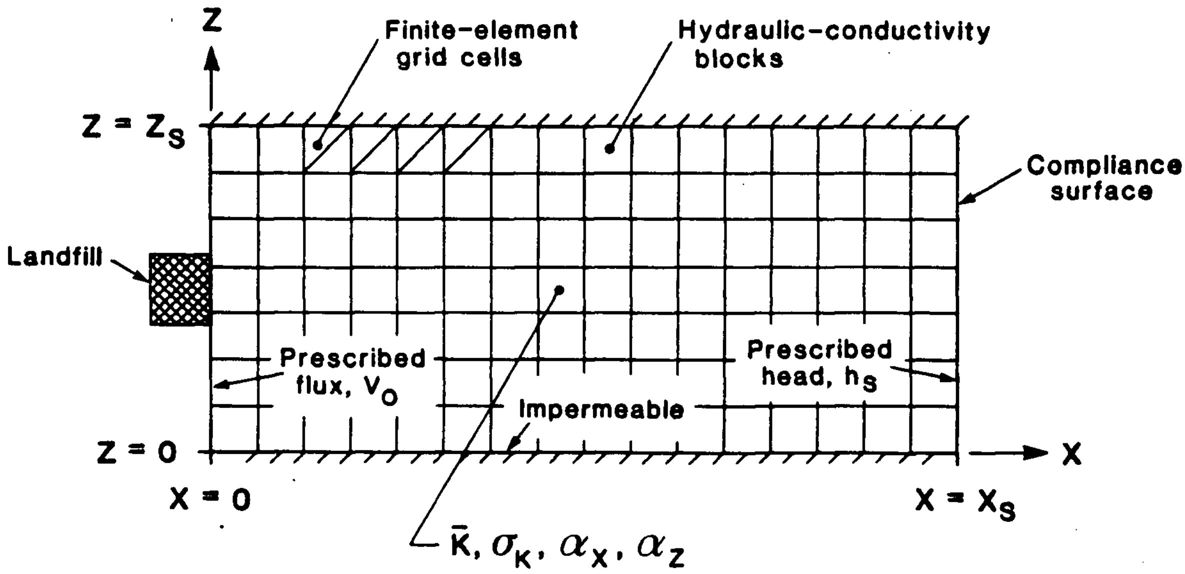


Figure 6.6 - Flow Field Used in Sensitivity Studies

conductivity. As the mean conductivity increases, the expected value of the travel time decreases, as does the travel time standard deviation. As might be expected from a simple application of Darcy's law, there is a direct proportional relationship between the mean conductivity and the mean travel time.

The effect of hydraulic conductivity variability on travel times is illustrated in Table 6.2. As the conductivity variability increases, the mean travel time decreases and travel time standard deviation increases. With an increase in hydraulic conductivity variability, there will be flow paths with hydraulic conductivities considerably higher and considerably lower than average. The contaminant plume will always travel the high-permeability path and therefore the average travel time will decrease with increasing conductivity variability. The increase in travel time variability with increasing conductivity variability is expected. It is interesting to note that the standard deviation for the travel times increases at a slower rate than the standard deviation for hydraulic conductivity. For example, increasing the conductivity standard deviation by a factor of 30 (from 150 to 4500 m/yr) results in an increase in travel time standard deviation by a factor of approximately 12 (from 0.7 to 8.0 years).

In Table 6.3, the hydraulic conductivity mean value and variance where both changed so that the coefficient of variation remained constant and equal to 1.0. With a constant coefficient of

Table 6.1 - Sensitivity of Travel Time Statistics to Mean Hydraulic Conductivity

Hydraulic Conductivity			Travel Time		
Mean Value (m/yr)	Standard Deviation (m/yr)	Coeff. of Var.	Mean Value	Standard Deviation	Coeff. of Var.
150.	1500.	10.	137.	108.	0.79
1500.	1500.	1.0	16.3	5.8	0.36
3000.	1500.	0.5	8.3	1.7	0.20

Table 6.2 - Sensitivity of Travel Time Statistics to Hydraulic Conductivity Standard Deviation

Hydraulic Conductivity			Travel Time		
Mean Value (m/yr)	Standard Deviation (m/yr)	Coeff. of Var.	Mean Value	Standard Deviation	Coeff. of Var.
1500.	150.	0.1	16.7	0.7	0.04
1500.	1500.	1.0	16.3	5.8	0.36
1500.	4500.	3.0	14.8	8.0	0.54

Table 6.3 - Sensitivity of Travel Time Statistics to Hydraulic Conductivity Statistics with Coefficient of Variation Equal to 1.0

Hydraulic Conductivity			Travel Time		
Mean Value (m/yr)	Standard Deviation (m/yr)	Coeff. of Var.	Mean Value	Standard Deviation	Coeff. of Var.
150.	150.	1.0	163.	58.	0.36
1500.	1500.	1.0	16.3	5.8	0.36
3000.	3000.	1.0	8.1	2.9	0.36

variation, the mean travel time was linearly proportional to the mean conductivity and the travel time standard deviation was linearly proportional to the conductivity standard deviation. Thus, the travel time coefficient of variation also remained constant. For example, increasing the conductivity mean and standard deviation by a factor of ten (from 150 to 1500 m/yr) caused a decrease in mean travel time by a factor of ten (from 163 to 16.3 years) and an increase in travel time standard deviation by a factor of ten (from 5.8 to 58 years). This linear variation is not observed if the coefficient of variation is allowed to fluctuate.

The effects of correlation lengths is illustrated in Table 6.4. Table 6.4a illustrates that as the correlation length in a direction parallel to flow increases, the mean travel time decreases and the travel time standard deviation increases. The cause of these relationships is similar to the relationships observed with increasing conductivity variability. With larger correlations, there are more flow paths with conductivities considerably higher and considerably lower than average. The contaminant plume will again always travel the high-permeability path. Table 6.4b illustrates the effects of increasing the correlation length in a direction perpendicular to flow. The impact on mean travel time is negligible, as might be expected. The reason for the decrease in travel time standard deviation is not clear.

Table 6.4 - Sensitivity of Travel Time Statistics to Hydraulic Conductivity Fluctuation Scales

a. Sensitivity of fluctuation scale parallel to flow direction

Fluctuation Scales xdirect. (m)	Fluctuation Scales zdirect. (m)	Expected Travel Time (yr)	Travel Time Standard Deviation (yr)
100	300	16.2	3.4
300	300	16.3	5.8
900	300	15.6	7.2

b. Sensitivity of fluctuation scale perpendicular to flow direction

Fluctuation Scales xdirect. (m)	Fluctuation Scales zdirect. (m)	Expected Travel Time (yr)	Travel Time Standard Deviation (yr)
300	300	16.3	5.8
300	900	16.4	2.7

Table 6.5 - Sensitivity of Travel Time Statistics to Hydraulic Conductivity Measurements

a. Sensitivity of measurements with fluctuation scale parallel to flow direction equal to 170 meters.

	Expected Travel Time (yr)	Travel Time Standard Deviation (yr)
Without measurements	16.9	4.4
With measurements	17.0	4.0

b. Sensitivity of measurements with fluctuation scale parallel to flow direction equal to 500 meters.

	Expected Travel Time (yr)	Travel Time Standard Deviation (yr)
Without measurements	15.4	5.1
With measurements	17.3	2.9

Figure 6.2 illustrates how hydraulic conductivity measurements reduce hydraulic conductivity uncertainty in different types of geologies. Table 6.5 shows how these same measurements affect the travel time statistics. Table 6.5a is for a geology with relatively small correlation scales. Hydraulic conductivity measurements reduce travel time uncertainty, but not by a great deal. The negligible impact on the mean travel time is due to our assumption in this particular case that the mean value of the observed hydraulic conductivities is equal to the prior mean conductivity. Table 6.5b shows the effects of measurements in a geological deposit with larger correlation scales. The measurements are considerably more effective in reducing travel time uncertainty in this type of geological environment.

6.3 Monitoring Systems and the Probability of Plume Detection

The expression for the probability of failure for the waste management facility, Equation 3.8, is comprised of three terms. The first term was associated with the probability of containment breaching, the second term was associated with the probability of contaminant plume migration through the hydrogeologic environment, and the third term was associated with the probability of plume detection. Techniques for estimating the probability of breaching are discussed in Chapter 4 and techniques for estimating the probability of plume travel times are discussed in Section 6.2. In this section, the method used to estimate plume detection is described.

6.3.1 Objectives and Effects of Groundwater Monitoring

Monitoring is an option that is available to both the owner/operator and the regulatory agency, but the objectives are different for each. The owner/operator uses monitoring as a warning against potential failure. His monitoring network must lie between the source and the compliance surface. The regulatory agency uses monitoring for enforcement of performance standards. Its monitoring must be carried out at the point(s) of compliance.

In general, there is no economically feasible monitoring network that can be expected to detect all possible plumes arising from a particular waste management facility. There is a probability of detection associated with any specific monitoring network and it can be expected to increase with increased density of the network and/or increased frequency of sampling.

Nevertheless, we have assumed for the purposes of this study that the probability of detection of the regulatory agency is unity; that is, we assume that any failure caused by the facility will be detected by society. This seems to be the only ethically defensible viewpoint for the owner/operator to take in his risk-cost-benefit analysis. (Any other viewpoint opens the door to consideration of even less ethical probabilities such as the probability of enforcement by the regulatory agency should a failure occur or the probability of conviction if the enforcement is fought in a court of law.)

For the owner/operator's monitoring network, on the other hand, the probability of detection will, in general, be less than unity. For a monitored system, we can define

$$P_f'(t) = P_f(t)(1 - P_d) \tag{6.28}$$

where

$P_f'(t)$ = probability of failure in year t for the unmonitored facility,

$P_f(t)$ = probability of failure in year t for the facility with a monitoring network in place,

P_d = probability of detection by the owner/operator's monitoring network.

Concepts similar to those incorporated in (6.28) appear in the arguments of Raucher [1983] and Schecter [1985].

By reducing the probability of failure, the monitoring network in turn reduces the risks defined in Equation 3.2 as the product of the cost of failure and the probability of failure. It should be clear that there is a possible trade-off for the owner operator between the costs of a denser or more frequently sampled monitoring network and the risk reduction that can be gained thereby.

As discussed in Section 3.4, it should also be recognized that there is a cost to the owner/operator associated with the detection of contaminants at the monitoring network. By definition, this probabilistic cost constitutes a risk. It is a lesser risk than that associated with detection at the compliance surface, but it is a risk nevertheless. It is the potential cost of the remedial work that would have to be carried out to avoid the continued migration of the plume from the monitoring network to the compliance surface. Using the techniques for estimating travel times that are described in Section 6.2, we can define $P_p(t)$ as the probability of plume arrival at any monitoring point in year t. The travel time to the monitoring network is defined using the quickest flow path from the source to any monitoring point.

In a monitored facility, then, there will be two risk terms in the owner/operator's risk-cost-benefit analysis. One risk will be a cost associated with detection at the compliance surface (the cost of failure) and the second risk will be a cost associated with detection at the monitoring network.

An implicit assumption in Equation (6.28) is that an owner/operator will completely avert a failure at the compliance surface if he detects the plume at his monitoring network. Given the low remedial success rates noted in Chapter 1, and the documented presence of contaminants in the hydrogeological environment, this is likely an optimistic presumption. If defensible data were to become available, it might be possible to add a remedial success factor to Equation (6.28).

6.3.2 Estimating Detection Probabilities

The probability of detection used in Equation (6.28) can be estimated from the plume migration analysis that is described in Section 6.2. Because we have neglected the effects of diffusion and dispersion, a contaminant plume emanating from a breach in the containment structure will be detected by a monitoring system only if the groundwater flow lines which pass through the contaminant source also pass through one or more of the monitoring wells. If the breach occurs over a segment with width L_s and if there are n_s such segments along the length of the landfill, then the probability of detection by monitoring system j is given by

$$P_d(j) = \sum_{i=1}^{n_s} P_d(j/S_i)P_b(S_i) \tag{6.29}$$

where

$P_d(j)$ = probability of detection by monitoring system j,

$P_d(j/S_i)$ = probability of detection by monitoring system j given that breaching occurs in segment i,

$P_b(S_i)$ = probability that breach occurs in segment i given that it occurs at all.

The probability of detection by monitoring system j given that breaching occurs in segment i, $P_d(j/S_i)$, can be determined by the same Monte Carlo simulations used to predict travel times. For each hydraulic conductivity field that is generated, the advective transport finite element program described in Chapter 5 is used to calculate flow lines. As illustrated in Figure 6.7a, if the flow tubes which pass through segment i in the landfill also pass through one of the monitoring points, the plume is detected. The probability of detection is given by

$$P_d(j/S_i) = \frac{\text{number of plumes detected}}{\text{total number of fields generated}} \qquad (6.30)$$

It should be noted that this rather idealized treatment of monitoring ignores many of the practical difficulties faced in the real world: laboratory errors, instrument errors, sampling errors, detection limits, type I and II errors, and so on. Many of these difficulties would tend to reduce the probability of detection. The framework that is proposed would allow consideration of these issues if data were available.

6.3.3 Detection Sensitivities

The number of contaminant plumes that are detected by the owner/operator's monitoring network depends upon the number and location of the monitoring points, the hydrogeological environment, and the size and location of the breach that emanates from the source area to create the contaminant plume. To illustrate some example sensitivities, probabilities of detection have been calculated for monitoring wells in the hypothetical horizontal flow field used for the base case in Section 6.2. This base case flow field is illustrated in Figure 6.6. It is assumed that the monitoring wells fully penetrate the aquifer and that they are monitored often enough to delineate travel times for the contaminants should breaching occur.

Figure 6.7b shows the location of nine monitoring wells and the probabilities that each well would individually detect a contaminant plume, given the base case assumptions of a mean hydraulic conductivity of 1500 m/yr and a breach length of 20 meters as the contaminant source. As expected, wells near the source and in the middle of the flow field are most apt to detect plumes. For the relatively small breach size of 20 meters, the probabilities of detection are quite small. The overall probability of detection for a monitoring system composed of the three wells on the left is 0.19, which is the value used as a base case.

Examples of the effects of geology and source size on the

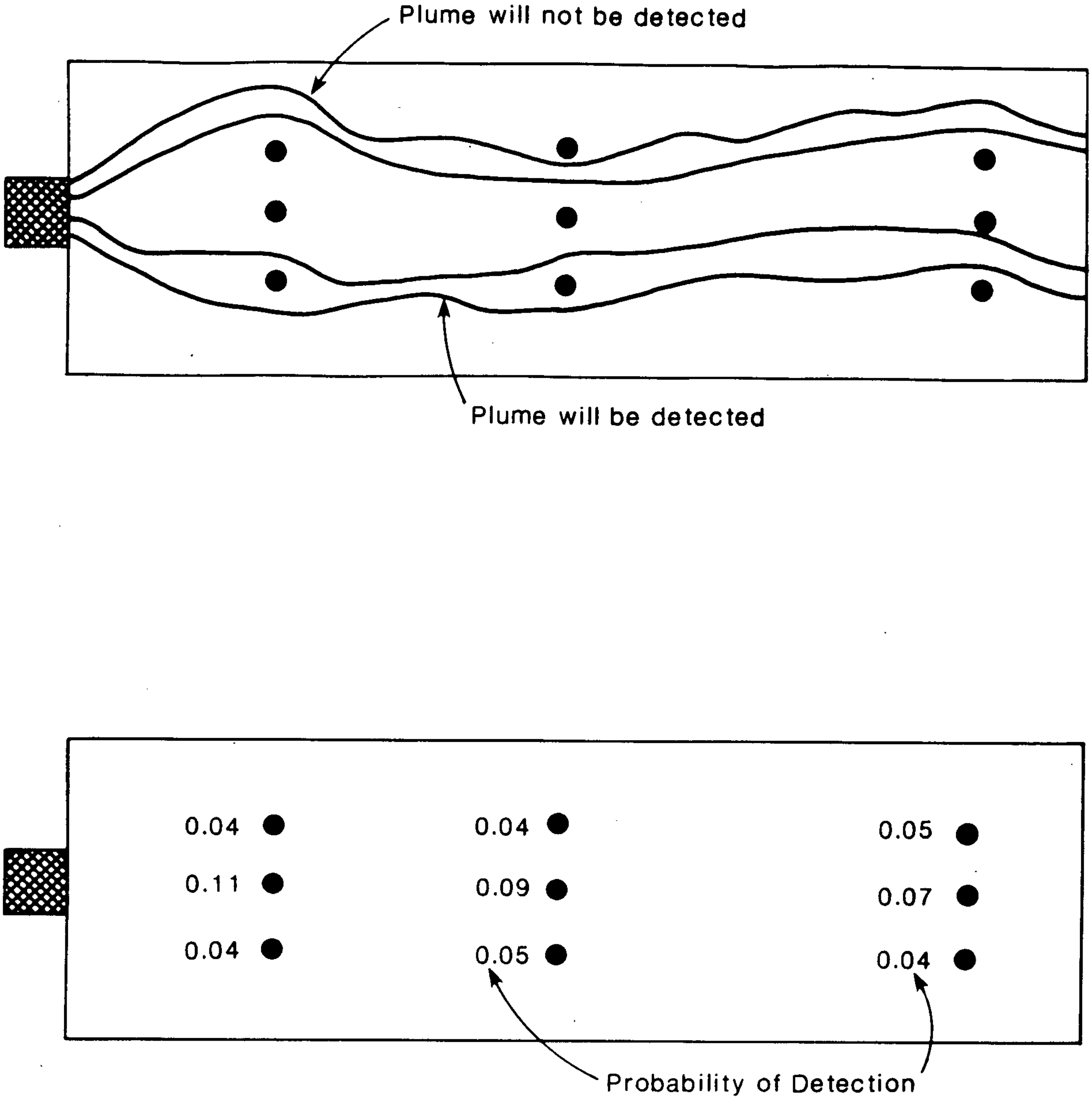


Figure 6.7 - Example Probabilities of Detection for Base Case

probability of detection are presented in Table 6.6. The monitoring system that is modeled consists of the three left-most wells shown on Figure 6.7. Table 6.6a illustrates how detection probabilities are affected by mean hydraulic conductivity. As the mean conductivity increases, the probability of detection also increases. Table 6.6b shows how detection probabilities depend upon hydraulic conductivity variability. Monitoring wells are more effective in less variable deposits. The effects of fluctuation scales, both parallel and perpendicular to the direction of flow, are illustrated in Table 6.6c. Although the detection probabilities increase with increasing correlation lengths, the effect is negligible. The more critical parameter with regard to monitoring system effectiveness is the length of the source or breach, as shown in Table 6.6d. Large breaches are much more detectable than small breaches.

The probabilities of detection presented in Table 6.6 and shown on Figure 6.7 are likely much smaller than most practitioners would expect from past experience. If the effects of diffusion and dispersion were to be included in the analysis, the plume would be wider and the detection probabilities would increase. A second consideration is that most existing plumes emanate from landfills and other sources that were not been lined or otherwise controlled. These unlined landfills produce large sources that would result in wider plumes that would have detection probabilities in the higher range noted on Table 6.6d. On the other hand, we have also made some assumptions that would tend to overestimate probabilities of detection. Included in these are the assumption of fully penetrating monitoring wells and neglecting laboratory errors, instrument errors, and sampling errors.

Table 6.6 - Example Sensitivities for the Probability of Contaminant Plume Detection

a. Sensitivity to mean hydraulic conductivity

Mean Hydraulic Conductivity (m/yr)	Probability of Detection (decimal fraction)
150	.142
1500	.188
3000	.202

b. Sensitivity to hydraulic conductivity standard deviation

Hydraulic Conductivity Standard Deviation (m/yr)	Probability of Detection (decimal fraction)
150	.200
1500	.188
4500	.167

c. Sensitivity to hydraulic conductivity fluctuation scale

Hydraulic Conductivity Fluctuation Scale		Probability of Detection (decimal fraction)
xdirect. (m)	zdirect. (m)	
100	300	.191
300	300	.188
900	300	.193
300	800	.196

d. Sensitivity to length of contaminant source

Length of Source (m)	Probability of Detection (decimal fraction)
10	.094
20	.188
100	.836

7. RISK-COST-BENEFIT SENSITIVITY STUDIES

In Chapter 3, a risk-cost-benefit analysis was developed from the perspective of an owner-operator of a landfill in which the primary design feature is one or more synthetic liners in a multiple-barrier configuration. To review, the objective function is given by:

$$\Phi_o = \sum_{t=0}^{T_o} [B(t) - C(t) - R(t)]/(1+i)^t \tag{7.1}$$

where the benefits, B(t), costs, C(t), and risks, R(t), include engineering, hydrogeologic, economic, regulatory, and political components. In particular, the risks are given by:

$$R(t) = P_f'(t)CF(t)u(CF) \tag{7.2}$$

where $P_f'(t)$ is the probability of failure of a monitored facility, CF(t) is the expected cost associated with a failure, and u(CF) is a utility function. Failure is defined as the occurrence of a groundwater contamination event that leads to the exceedance of performance standards at a compliance surface established by a regulatory agency. Failure requires three conditions. First the containment structure must be breached. Next, the contaminant plume resulting from the breach must migrate to the compliance surface. Finally, the plume must escape detection by any monitoring network the owner-operator has installed. These three conditions are represented by the three terms on the right hand side of the following expression:

$$P_{f'}(t) = Pr(t^* = t')Pr(t^{**} = t - t_{op} - t')(1-P_d) \qquad (7.3)$$

where t^* is the time until breaching, t^{**} is the migration time, and P_d is the probability of detection of the monitoring network. In Chapter 4, it is shown that the probabilities associated with the time until breaching can be estimated using reliability theory. The probabilities associated with migration times and detection are based on Monte Carlo analyses using finite-element simulations of two-dimensional, horizontal, steady-state, advective contaminant transport through a hydrogeological environment in which the hydraulic conductivity values are defined stochastically. The techniques used to estimate travel time and detection probabilities are described in Chapters 5 and 6.

This chapter has two parts. In the first, it is shown how the risk-cost-benefit analysis can be used by the owner-operator in a decision framework to assess the merits of alternative design strategies. In the second, it is shown how the analysis can be used by the regulatory agency to assess alternative regulatory policy, but only in an indirect manner, by examining the response of an owner-operator to the stimuli of various policies.

Throughout the chapter, the sensitivity analysis used to assess alternatives is carried out with respect to a hypothetical base-case. The base-case, which is denoted as Case B in the tables that follow, is a mid-sized landfill with an area, A, of 30,000

m^2, a capacity, Z, of 450,000 tons, and an annual throughput, V, of 11,250 tons/yr. It is sited on a shallow unconsolidated sand-and-gravel aquifer with a mean hydraulic conductivity, $\overline{K}$, of 1,500 m/yr and a standard deviation, σ_K, that is also 1,500 m/yr. The distance, X_s, from the landfill to the compliance point is 1,000 m, and over this distance the hydraulic head drop, H, is 8.2 m. The design features one synthetic liner (m=1) with a mean breach time, $\bar{t}^*$, of 15 years. The exploration drilling in the shallow aquifer, Y_x, totals 90 m. The total installed depth of monitoring wells, Y_m, is also 90 m. The unit charge for waste handled is set at \$90/ton. Remedial costs in the event of failure, C_R, are assumed to be \$5.75 million. A regulatory fine, C_P, of \$5.0 million will be levied in the event of failure. In that event, litigation costs (including the costs of damages assessed) are expected to reach \$5.0 million. The time horizon, T_O, for the owner-operator is 46 years, with the design and construction period, t_{op}, that precedes operation covering 6 years. The discount rate, i_m, is taken as 0.10. The complete set of parameter values used for the full suite of cost, benefit, and risk terms is listed in Table 3.1.

A plot of the time stream of benefits, costs, and risks for the base-case is shown on Figure 7.1a. Benefits and costs are relatively constant through the operation period; the risks peak at t = 26 years. For this case, the value of the objective function is \$1.1 million. This is the net present value in constant value 1980 U.S. dollars of the rather complex stream of

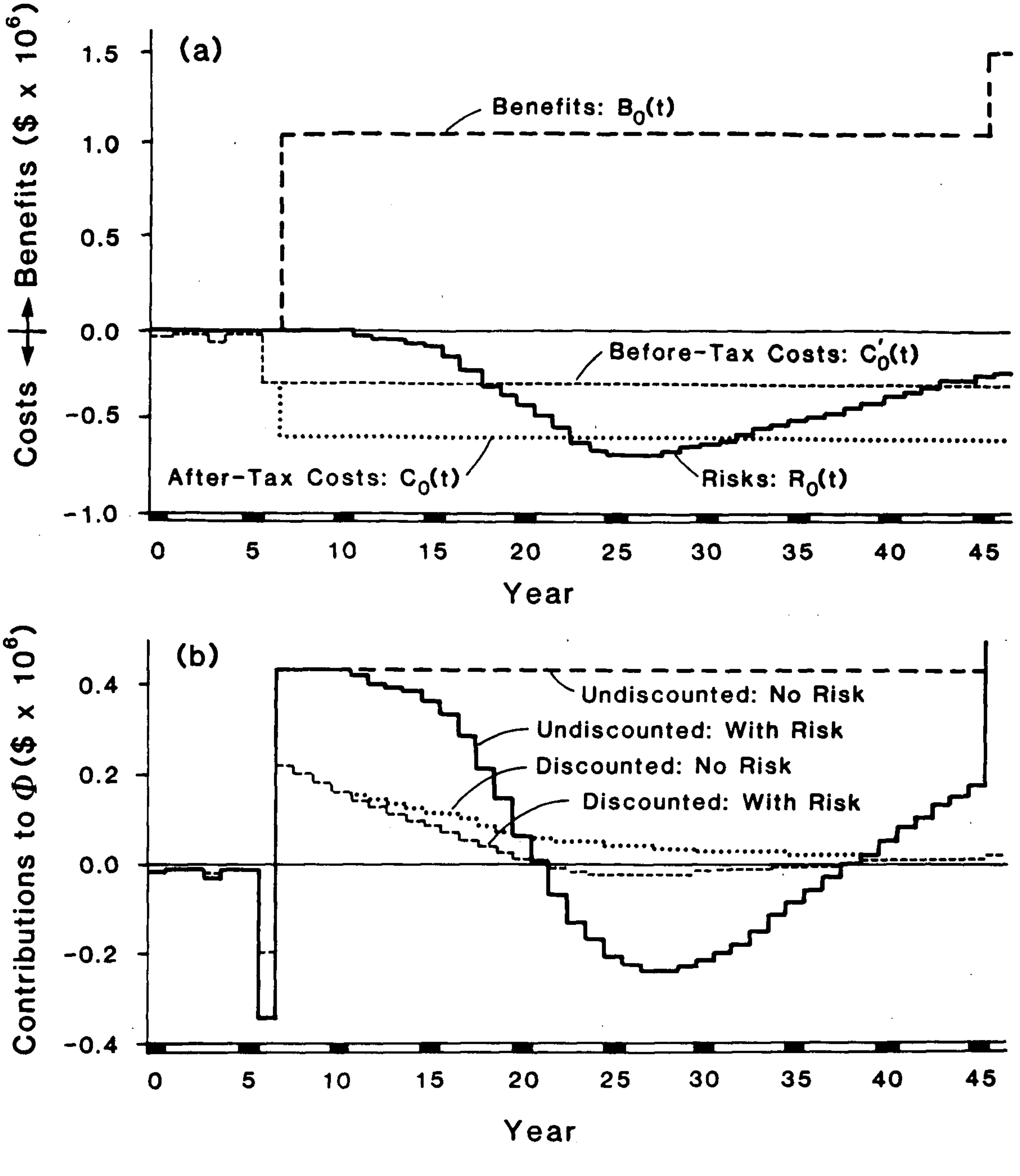


Figure 7.1 - Example Benefits, Costs, and Risks for Base Case

benefits, costs, and risks shown on Figure 7.1a.

Figure 7.1b shows the makeup of the annual undiscounted and discounted contributions to the objective function, both with and without the risk term included. Note that the discounted contributions to the objective function that include the influence of risk are negligible after about 20 years. The curves showing the contributions without risks are included to emphasize that the curves with risk are expected values rather than "actual" values that would be experienced by the owner-operator. In practice, there will either be a failure or there will not. If there is not, the owner-operator will experience contributions to his objective function that have the current values shown on the undiscounted curve (or the present values shown on the discounted curve). If a failure occurs, he will experience these values until the time of failure, after which the curve will plunge into the negative range under the influence of regulatory penalties and remedial costs.

There are several criteria that could be used to compare streams of benefits, costs, and risks similar to those shown on Figure 7.1. Following Dieter [1983] and Mishan [1976], this dissertation does not use a benefit-cost ratio or a discounted, cash-flow, internal rate of return, since both give ambiguous and contradictory results. Instead, the net present value of the integrated stream of benefits, costs and risks afforded by a direct comparison of the alternative values of the objective function, Φ_o. A comparison of the break-even unit charge, B_R, which is the value of the unit charge, B_R, that is just sufficient to produce $\Phi_O = 0$ at the specified rate, i_m, is also provided.

In the tables that follow, in addition to Φ_O and B_R, the probability of detection of the monitoring network, P_d, will be report for each case. Two parameters that will enter the discussion only in connection with the assessment of alternative regulatory policies will also be presented. These latter two measures are the total probability of failure, $\sum P_f'$, over the time horizon, T_O, and the risk reduction effectiveness, r, introduced by Vanmarcke and Bohenblust [1982]. They are defined as follows:

$$\sum P_f = \sum_{t=0}^{T_O} P_f'(t) \tag{7.4}$$

$$r = 1 - [(\sum P_f')_2 \;/\; (\sum P_f')_1] \tag{7.5}$$

where alternative 2 associated with $(\sum P_f')_2$ in Equation (7.5) is a more conservative alternative than alternative 1 associated with $(\sum P_f')_1$. The parameter, r, is a measure of risk reduction that is valuable in the qualitative assessment of alternatives from a social perspective.

7.1 Assessment of Alternative Design Strategies

The alternative design strategies available to the owner-operator revolve around the allocation of resources among: (1) site-exploration activities, (2) containment-construction activities, and (3) monitoring activities. These three groups of design activities affect the three terms on the right hand side of Equation (7.3); containment-construction activities affect the probability associated with breaching, $Pr(t^* = t')$, site-exploration activities affect the probability associated with plume migration, $Pr(t^{**} = t-t_{op}-t')$, and monitoring activities affect the probability of plume detection, P_d. Each of these activities are treated in turn.

7.1.1 Site Exploration

The primary purpose of site-exploration is to obtain estimates of the mean value, $\bar{K}$, the standard deviation σ_K, and the correlation scales, λ_x and λ_z, for the hydraulic conductivity field in the region between the landfill and the compliance surface. With the Bayesian approach to site-exploration, a prior set of values is assumed on a subjective basis before measurements are taken and the field measurements of K are then used to update these values in the manner described in Chapters 5 and 6. Additional measurements will confirm or change the estimates of the spatial variability of K, and they will reduce the uncertainty as to the actual values in the vicinity of the measurement points.

The values of $\bar{K}$, σ_K, λ_x and λ_z influence the estimated travel time statistics, $\bar{t}^{**}$ and σ_t, which in turn influence the probability of failure, $P_f(t)$, the estimated risks, $R(t)$, and ultimately the objective function, Φ_O. The sensitivity of travel times to the various hydrogeological parameters is presented in Section 6.2 using the hypothetical flow field shown in Figure 6.6. The next step in the analysis is to use the risk-cost-benefit analysis described in Chapter 3 to quantify how travel time statistics, which depend upon the site exploration activities, impact overall risks and the value of the objective function.

Before examining the impact of measurement programs, let us first look at how the site properties will influence the owner-operator's objective function. Table 7.1 indicates the influence of the mean hydraulic conductivity, $\bar{K}$, on the owner-operator's decision criteria. For a particular design (i.e., for a particular exploration, containment, and monitoring allocation), site Q with $\bar{K}$ = 3,000 m/yr would allow the net present value of all future expected rewards (as embodied in the value of Φ_O) to reach only \$0.1 million, whereas site P with $\bar{K}$ = 150 m/yr will produce an expected \$2.1 million. Case B is the base-case, with K = 1,500 m/yr and $_O$ = \$1.1 million.

The same information is conveyed by the B_R data on Table 7.1. At a site with $\bar{K}$ = 150 m/yr, the owner-operator would have to charge \$25/ton for waste handled in order for benefits to equal costs plus risks; at a site with $\bar{K}$ = 3,000 m/yr he would have to charge

Table 7.1 - Comparison of Three Sites with Different Mean Hydraulic Conductivity

Parameter	Alternative P	Alternative B	Alternative Q
Mean hydraulic conductivity	150	1500	3000
Objective function	$\$2.1x10^6$	$\$1.1x10^6$	$\$0.1x10^6$
Break-even unit charge	\$25	\$53	\$85
Probability of detection	0.19	0.19	0.19
Total probability of failure	$0.5x10^{-5}$	0.64	0.74
Risk-reduction effectiveness	¶0.99	0.14	0.00

\$85/ton. The probability of detection is the same in all three cases because the monitoring network is identical in each case.

The total probability of falure, $\sum P_f'$, over the owner-operator's time horizon of T_O = 46 years, is 0.74 for site Q, but is reduced by over five orders of magnitude at site P. The implications of this risk reduction are discussed later in the chapter with respect to regulatory policy options.

With the value of low-permeability sites firmly established, let us now apply the risk-cost-benefit approach to investigate the worth of alternative measurement programs. The effectiveness of measurements in reducing uncertainty depends upon geology, as discussed in Section 6.1 and shown on Figure 6.2. Measurements reduce not only uncertainty in hydraulic conductivity, but also uncertainty in travel times, as discussed in Section 6.3. The next logical step would be to investigate how hydraulic conductivity measurments impact risks and the objective function. Unfortunately this is not easily done. The calculations performed in Chapter 6 were carried out under the assumption that the observed mean hydraulic conductivity after measurements were taken was unchanged from the prior mean value assumed before measurements were taken. In general, however, a measurement program can be expected to lead to a new mean as well as a reduced variance. The impact of this fact on the comparison of alternative exploration strategies is explored in Table 7.2.

The question addressed in Table 7.2 is whether an increase in

Table 7.2 - Comparison of Alternative Exploration Strategies

Parameter	Alternative B	Alternative S1	Alternative S2
Depth of exploratory drilling	90	330	330
Prior mean hydraulic conductivity	1500	1500	3000
Observed hydraulic conductivity	1500	1500	300
Objective function	$1.1x10^6	$0.45x10^6	$1.9x10^6
Break-even unit charge	$53	$77	$31
Probability of detection	0.19	0.19	0.19
Total probability of failure	0.64	0.64	0.15
Risk-reduction effectiveness	0.00	0.00	0.77

exploratory drilling, Y_x, from 90 m (Case B) to 330 m (Case S) is beneficial, assuming that hydraulic conductivity measurements are obtained every 10 m of exploratory drilling in both cases, and that site properties are identical in all other respects. In Case S_1, the observed mean conductivity is unchanged from the prior; in Case S_2, it is reduced. For S_1, the additional exploration actually decreases the owner-operator's objective function. The benefits of the decreased travel time uncertainty are outweighed by the costs of the additional exploration. For Case S_2, however, where the conservativeness of the prior mean conductivity has been exposed by the measurement program, the owner-operator's objective function value is increased. The results of S_1 and S_2 confirm the intuitive fact that the better our prior understanding of a site, the less valuable will be the results of an exploration program.

Unfortunately, the worth of prior estimates is not known until they have been tested by exploration. To properly analyse the value of data not yet collected, it is necessary to introduce the concept of economic regret [Maddock, 1973]. An owner-operator suffers regret if he makes a decision based upon the assumption that the site has a particular set of hydraulic conductivity values when in fact it has a different set. As a simple example, assume the owner-operator's site has a mean hydraulic conductivity of either 3,000 m/yr or 1,000 m/yr. If the owner-operator uses a one-liner system, assume his objective function is $\$1x10^6$ if the expected hydraulic conductivity is 3,000 m/yr and $\$5x10^6$ if it is 1,000 m/yr. If he uses a two-liner system, assume his objective function is $\$3x10^6$ if the hydraulic conductivity is 3,000 m/yr and $\$4x10^6$ if it is 1,000 m/yr. If he decides the mean value is 3,000 m/yr he will choose the two-liner system and if he decides the mean value is 1,000 m/yr, he will choose the one-liner system. His regret in assuming 3,000 m/yr if in fact it is 1,000 m/yr is $\$1x10^6$ ($\$5x10^6$ - $\$4x10^6$). His regret in assuming 1,000 m/yr if in fact it is 3,000 m/yr is $\$2x10^6$ ($\$3x10^6$ - $\$1x10^6$).

Because the actual hydraulic conductivity values represent a random field, the owner-operator's regret is a random variable. An expected regret can be estimated based upon probabilities associated with each possible hydraulic conductivity field. In the example presented above, by assigning probabilities to the 1,000 m/yr and 3,000 m/yr cases, the owner-operator's expected regret can be calculated. The best exploration strategy is the one that minimizes the owner-operator's expected regret.

To determine if additional exploration will reduce the owner-operator's expected regret, the objective function must be calculated for a much larger number of possible outcomes than the two shown on Table 7.2, and probabilities must be assigned to each outcome. This type of analysis, although conceptually straightforward, can involve a considerable amount of computational effort. The design of optimal exploration programs for waste-management facilities, based on the framework presented in this dissertation and using the concept of expected regret summarized here, could form the topic of later research, but it is outside the scope of the present work.

7.1.2 Containment Design

The landfill system modeled in this study consists of one or more cells, with the wastes in each cell contained by one or more liners. Each cell will function so long as at least one liner is functioning and the complete landfill system will function so long as all cells are functioning. The probability of a breach of containment for such a system, based on time-dependent reliability theory and the assumption that times to failure for liners are exponentially distributed is given by Equations (4.23) and (4.25). Example sensitivities are presented in Section 4.2.

Additional liners clearly increase costs and reduce risks. The question is whether the additional costs are compensated by the risk reduction. Table 7.3 provides a summary of the risk-cost-benefit output from which a comparison of three alternatives (no liner, one liner, two liners) can be made by the owner-operator. Case B is the base case. The superiortiy of Case C, the two-liner case, is indicated by the values for both Φ_O and B_R. The value of Φ_O is calculated by assuming a charge of \$90/ton for waste handled. At this rate, Case A is not expected to be profitable. It would take a charge of \$100/ton to produce $\Phi_O = 0$ at $i_m = 0.10$ for Case A. For Cases B and C, on the other hand, the same result could be achieved at \$53/ton and \$32/ton,

Table 7.3 - Comparison of Three Design Alternatives

Parameter	Alternative A	Alternative B	Alternative C
Number of liners	0	1	2
Objective function	-$\$0.26\times10^6$	$\$1.1\times10^6$	$\$1.9\times10^6$
Break-even unit charge	$100	$53	$32
Probability of detection	0.19	0.19	0.19
Total probability of failure	0.81	0.64	0.06
Risk-reduction effectiveness	0.00	0.21	0.92

respectively. For this comparison, then, the two-liner design, which would probably be preferred by the regulatory agency on the basis of its risk-reduction effectiveness, is also preferred by the owner-operator because it will maximize his expected profitability. As shown later, this identity of interests is not always present.

7.1.3 Monitoring Network Design

The probability of failure of a waste-management facility will be reduced if the owner-operator can install a monitoring network on his property between the source and the regulatory compliance point. The level of reduction in the probability of failure is dependent on the probability of detection of the monitoring network, which is in turn dependent on the number and location of the monitoring wells, the hydrogeological environment, and the size of the breach that emanates from the source area to create the contaminant plume. Techniques for estimating detection probabilities and example sensitivities for these probabilities are presented in Section 6.3.

As with containment, it is clear that additional monitoring increases costs and reduces risks. Once again, the risk-cost-benefit framework can be used to assess whether the additional costs are compensated by the risk reduction. Cases G, B, and H in Table 7.4 show the impact that three levels of monitoring would have on the owner-operator's risk-cost-benefit analysis. The various monitoring scenarios are illustrated on Figure 6.2.

Table 7.4 - Comparison of Three Monitoring Alternatives

Parameter	Alternative G	Alternative B	Alternative H
Total depth of monitoring wells	0	90	330
Objective function	$1.2x10^6$	$1.1x10^6$	$1.1x10^6$
Break-even unit charge	$50	$53	$56
Probability of detection	0.00	0.19	0.46
Total probability of failure	0.79	0.64	0.33
Risk-reduction effectiveness	0.00	0.19	0.59

For Case B, the three left-most monitoring wells are used and for Case H, all 11 monitoring wells are included.

All three involve a single-liner design. The design variable in this case is the total depth, Y_m, of the installed monitoring wells. It is assumed in all three cases that there is one monitoring point for every 10 m of well depth and that the network is sampled every three months. Drilling costs, installation costs, and chemical analysis costs are constant. The results on Table 7.4 suggest that from the owner-operator's point of view, the three monitoring alternatives are of roughly equal value. Apparently the benefits accorded by having monitoring wells in place are almost exactly offset by their costs. Given this result, the rational owner-operator may well choose Case G with no monitoring and a probability of detection of $P_d = 0$. Even if he chooses cases B or H, P_d only goes up to 19% and 46% respectively. For the base-case conditions we have analysed, it appears that the installation of a monitoring network is of less value to the owner-opertor than a more conservative containment design as a component in the maximization of his objective function. To determine whether this is true in general would require a much larger suite of simulations, over a much greater range of paramters, than have been carried out for this study.

7.2 Assessment of Alternative Regulatory Policies

The assessment of alternative regulatory policies by the regulatory agency is now considered. The risk-cost-benefit analysis set up from the owner-operator's perspective will still be used, but the emphasis will be placed on the owner-operator's response to various regulatory stimuli. By carrying out such an exercise an indirect comparison of the worth of various policy options can be provided. It should be emphasized that these policy options are not based on those embedded in the U.S. Resource Conservation and Recovery Act (RCRA). The present study is not intended as an assessment of any particular current legislation.

The reader will recall that Chapter 3 investigated the feasibility of setting up a risk-cost-benefit analysis from the perspective of the regulatory agency, which would have allowed a direct comparison of regulatory options, but that this attempt floundered on the difficulties associated with placing a dollar value on the worth of human life. Nevertheless, it is clear that the primary burden placed on a regulatory agency by society is the protection of human health and life, so a comparison of the merits of alternative policies must be based on some measure that reflects their relative success in this area. In the present study, the total probability of failure, ΣP_f, is used as that measure. As discussed in Section 3.2, it is a surrogate for acceptable risk.

7.2.1 Regulatory Options

In general, the objectives of social policy should be embedded in regulations that are (1) comprehensive, (2) logical, (3) practical, (4) equitable, (5) politically acceptable, (6) cost efficient, and (7) simple to administer [Eheart, 1980, Fischoff et al, 1981]. There is a much larger literature on the development of regulatory policies for the protection of surface-water quality [cf. Kneese and Bower, 1968; Freeman, 1980; Brill, 1979] than there is for groundwater quality; however, it appears that many of the ideas are transferable.

In both cases, a regulatory philosophy can take one of two forms: (1) economic incentives or (2) direct regulation; and in each case there are several alternatives. In the environmental economics literature there is widespread support for the use of economic incentives, but in practice almost all legislation, both for surface water and groundwater, is based on direct regulation.

Direct regulation involves setting standards. Such standards may be one of two types; (1) design standards or (2) performance standards [Dieter, 1983; Freeman, 1980]. Design standards require that facilities be constructed with specified methods and to a certain standard. In groundwater-pollution legislation they take the form of containment requirements and/or requirements on the monitoring network.

Performance standards require that facilities achieve a certain level of performance without reference to how that performance is achieved. For potential groundwater contamination at waste-management facilities, possible performance standards for a particular contaminant [Cartwright et al, 1981; Domenico and Palciauskas, 1982; LeGrand, 1982] are those relating to:

1. Maximum concentration levels at the compliance surface.

2. Maximum flux across the compliance surface.

3. Pre-emplacement advective travel time from the source to the compliance surface.

4. Contaminant travel times from the source to the compliance surface.

Design standards almost never stand alone; there are usually performance standards associated with regulatory monitoring activities even when facilities must be built to design standards.

The licencing role of a regulatory agency requires that successful applicants be issued a permit in which the applicable standards are clearly outlined. When regulatory monitoring programs uncover a failure to meet performance standards, the enforcement role of the agency comes into play. In this study, enforcement may involve (1) imposing fines, (2) withholding the return of a performance bond, and (3) closing the plant.

In the following sections, sensitivity analyses are invoked to look at the following regulatory issues:

1. Relative merits of design standards vis-a-vis performance standards.

2. Relative merits of design standards on the monitoring network vis-a-vis design standards on the containment structure.

3. Relative merits of fines vis-a-vis performance bonds for the enforcement of violation of standards.

4. Impact of closure.

5. Importance of siting.

7.2.2 Design Standards and Performance Standards

Table 7.3 presented a comparison of three design alternatives. In the associated discussion it was explained how the owner-operator would choose the two-liner design over the no-liner or one-liner design on the basis of a comparison of the values of his objective function, Φ_O, and the break-even unit charge, B_r. Now let us view the situation through the regulator's eyes. His interest centers on the total probability of failure, $\sum P_f$, and the risk-reduction effectiveness, r. With the one-liner design (and a monitoring network with a probability of detection, P_d = 0.19), the total probability of failure over the life of the facility, $\sum P_f$, is still 64% and the risk-reduction effectiveness, r, over the no-liner design is only 21%. With the two-liner design, on the other hand, P_f = 6% and r = 92%. Although Case B would likely be unacceptable to the regulatory agency, Case C may possibly satisfy the constraint recognized by the regulatory agency for a politically acceptable probability of failure.

For the comparison shown on Table 7.3, the interests of the owner-operator and those of the regulatory agency are both met by the most conservative design. If such were always the case, one could argue that there would be no need for the regulatory agency to impose design standards on the owner-operator to force the use a safe design.

However, it is not difficult to construct comparisons where this identity of interests is not present. Table 7.5 shows a summary of three cases that differ from those in Table 7.3 only in terms of the estimated costs associated with the probability of failure. In Table 7.5 these costs ($C_J + C_P + C_R$) total \$1.26 million; in Table 7.3 they total \$15.75 million. In Table 7.5, the one-liner and two-liner cases have nearly identical Φ_O, and the one-liner case will actually be slightly favoured on the basis of the B_R values. The equality of the Φ_O values for cases E and F imply that the net present value of the added costs of the second liner are just balanced by the net present value of the reduced risks afford by its presence. From the owner-operator's point of view, the costs come early and the risk reduction comes late, so discounting tends to reduce the influence of the latter on his design decisions. However, as in Table 7.3, the risk reduction afforded by the two-liner design is

Table 7.5 - Comparison of Three Design Alternatives Under Conditions Where There Are Lower Costs Associated with Failure than Those Used in Table 7.3.

Parameter	Alternative G	Alternative B	Alternative H
Number of liners	0	1	2
Objective function	$0.16x10^6$	$0.19x10^6$	$0.19x10^6$
Break-even unit charge	$30	$29	$30
Probability of detection	0.19	0.19	0.19
Total probability of failure	0.81	0.64	0.06
Risk-reduction effectiveness	0.00	0.21	0.92

significant and from a societal perspective it may be necessary. In the absence of a two-liner design standard, it would be very unlikely that the owner-operator would select this societally-preferable option. For this case, it may be necessary to have design standards in place to meet societal objectives. It appears that design standards, despite their unpopularity in the economics literature, and to some degree in the engineering community, too, are effective in reducing risks to politically acceptable levels.

7.2.3 Design Standards and Monitoring

There is another type of design standard that can be invoked. Regulatory agencies can require that an owner-operator install a specified level of monitoring network. Cases G, B, and H on Table 7.4 showed the impact that three levels of monitoring would have on the owner-operator's risk-cost-benefit analysis. Recall that from his point of view there was little reason to choose between them. From a societal point of view, however, we must look at the $\sum P_f$ values in Table 7.4. They vary from 0.79 to 0.33; Case H is clearly preferred. It is possible, however, that none of these values would be politically acceptable. If a one-liner design is allowed, more severe design standards on the monitoring network than those of Case H may be required to reduce risk to acceptable levels. More generally, some combination of design standard for the liner and the monitoring network might be in order. For example, if the expanded monitoring network with Y_M=330 m is applied to the two-liner design of Case C in Table

7.3, the probability of failure is halved from 6% to 3%. In addition, calculations presented in Section 6.3 suggests that if large source lengths are anticipated, higher probabilities of detection can be attained with economically feasible, monitoring networks. In such cases design standards on the monitoring network might be quite effective. In general, it appears that design standards on the monitoring network are effective in reducing risk, but not as effective as design standards on the containment.

7.2.4 Fines and Performance Bonds

Let us now turn to performance standards. If one assumes that an owner-operator will make his design decisions on the basis of economic analyses, the effectiveness of performance standards will be controlled by the impact that they have on the owner-operator's risk-cost-benefit analysis through the penalties asssessed for failure to meet the performance standards. Table 7.6 shows such an impact for three values of $C_P + C_J$, where C_P is the regulatory fine imposed on the owner-operator in the event of failure and C_J is the estimated cost of litigation. There is no question that such penalities affect the expected profitability of the facility. The question is how the owner-operator will respond to the stimulus. It has been shown that increased design standards force a risk reduction on the owner-operator, but the response to increased penalties for violation of a performance standard does not necessarily lead to risk reduction. The owner-

Table 7.6 - Comparison of Three Levels of Regulatory Penalty

Parameter	Alternative G	Alternative B	Alternative H
Regulatory penalty plus cost of litigation and damage	$\$5.0 \times 10^6$	$\$10.0 \times 10^6$	$\$20.0 \times 10^6$
Objective function	$\$1.3 \times 10^6$	$\$1.1 \times 10^6$	$\$0.7 \times 10^6$
Break-even unit charge	$46	$53	$67
Probability of detection	0.19	0.19	0.19
Total probability of failure	0.64	0.64	0.64
Risk-reduction effectiveness	0.00	0.00	0.00

operator has two routes he can follow: (1) he can improve his design and try to improve Φ_0 by reducing the risk, R(t), or (2) he can increase his charges, B_R and improve Φ_0 by increasing his benefits, B(t). If there is an active competitive market in the provision of waste-management services, he may be forced to follow the first route, but given the political difficulties in siting facilities, such active markets are less likely to develop, and without them, owner-operators may tend to follow the easier second route. In Table 7.6, if the penalties are increased from $10 million (Case B) to $20 million (Case K) the owner need only boost his charges from $53/ton to $67/ton to maintain his economic position. If he does so, societal goals will not be met with respect to acceptable risk.

An alternative regulatory scheme to the use of penalties at the time of failure is the posting of a bond at $t = 0$, which is returned with interest at $t = T_0$ if no violation of a performance standard occurs. Table 7.7 summarizes the impact of the two approaches on the financial position of an owner-operator. Comparison of Case M with Case B shows that substitution of a performance bond for a prospective penalty of greater value can turn a venture that is expected to be profitable into one that is expected to be unprofitable. In this case it is not likely that the owner-operator would respond by increasing his charges, as an increase from $53/ton to $113/ton would be required and it is unlikely that he could remain competitive at the higher rate. If the venture could be made profitable by improved design and

Table 7.7 - Comparison of the Impact of Performance Bond Posted Before Construction Relative to Prospective Penalties Imposed at the Time of Failure

Parameter	Alternative L	Alternative B	Alternative M	Alternative N
Regulatory penalty plus cost of litigation and damage	$\$10 \times 10^6$	$\$10 \times 10^6$	0	0
Performance bond posted	$\$3 \times 10^6$	0	$\$3 \times 10^6$	$\$4 \times 10^5$
Objective function	$-\$1.1 \times 10^6$	$\$1.1 \times 10^6$	$-\$0.7 \times 10^6$	$\$1.2 \times 10^6$
Break-even unit charge	\$127	\$53	\$113	\$49
Probability of detection	0.19	0.19	0.19	0.19
Total probability of failure	0.64	0.64	0.64	0.64
Risk-reduction effectiveness	0.00	0.00	0.00	0.00

reduced risk, he would probably take that route. Comparison of Cases B and N shows that performance bonds can be set at a much lower level than penalties to achieve comparable impact on the owner-operator's financial position. Comparison of Cases L and M suggest that, if a large performance bond is required, the presence or absence of a regulatory penalty in the event of failure is not very important to the owner-operator.

7.2.5 Facility Closure

The results of Tables 7.6 and 7.7 are another indication of the importance of the discount rate in removing expected future impacts from consideration in the making of current decisions. It can also be shown, for similar reasons, that the question of whether the facility is closed at the time of failure is not of particular consequence to the owner-operator in the decisions he makes at time zero. Actual, immediate costs always have much greater impact than prospective future loss of revenues.

7.2.6 Siting

The influence of the mean hydraulic conductivity of the hydrogeological environment on the owner-opertor's decision criteria was discussed earlier in connection with Table 7.1. This table can also be viewed from a regulatory perspective in the context of siting. It documents the total probability of failure, P_f, for the base-case-design for three different values of K. For Case B, with K = 1,500 m/yr, $P_f = 0.64$; for case P, with K = 150 m/yr, $P_f = 0.5 \times 10^{-5}$. A reduction in mean hydraulic conductivity of one order of magnitude produces a decrease in risk of five orders of magnitude. It has long been recognized in the hydrogeological community that siting can be of greater importance than either design or regulation in reducing the risk borne by society from waste-management facilities. This study provides quantitative confirmation of this fact. It is unfortunate that siting remains largely in the political arena; careful siting is in most cases the easiest way to meet societal constraints with respect to acceptable risk.

8. CASE STUDIES

Two case studies are presented in this chapter. The first is the Cape May County Landfill located in Woodbine, New Jersey and the second is the Carlson Landfill located near Vancouver, Washington. Although these facilities are solid waste landfills which receive mostly municipal refuse, relatively stringent state regulations require that they be designed and operated in a manner very similar to hazardous waste facilities. These particular landfills were chosen primarily because of the willingness and cooperation of the owners in providing information describing their facility.

The principal motive for including the case studies is to illustrate that the relatively large amount of data required for the analysis presented in this dissertation can be obtained for fairly typical applications. No attempt is made to draw conclusions pertaining to the design or operation of the Cape May County or Carlson facilities.

8.1 Cape May County Landfill

The sources for much of the information presented in this section include consultants' reports [Geraghty and Miller, 1982; Geraghty and Miller, 1983; PQA Engineering, 1982; Emcon and Associates, 1983; Soil Testing Services of Wisconsin, 1982], the operating plan for the facility [Cape May County Municipal Utilities Authority, 1983], environmental regulations [New Jersey Department of Environmental Protection, 1981], and discussions with Cape May County officials. Although the objective function components presented in Chapter 3 are in SI units, the variables used in the consulting reports and operating plans are given in English units. English units will be used in this chapter.

8.1.1 General Site Description

Cape May County is located in the southern portion of New Jersey. The area, which serves as a summer resort for inhabitants of many east coast cities, contains beaches, state parks, pine forests, and wetlands. Because of the area's environmental sensitivity, all landfill sites in the county were closed and a relatively sophisticated and secure sanitary landfill was constructed during 1983 and placed in operation in May, 1984.

This new landfill is located near the village of Woodbine, approximately 60 miles south of Philadelphia and 25 miles southwest of Atlantic City. The landfill is owned by the Cape May County Municipal Utilities Authority (CMCMUA) but is operated by a privately owned company. The design and operation of the facility is regulated by the New Jersey Department of Environmental Protection [NJDEP, 1981]. The landfill receives municipal and non-hazardous industrial wastes.

The facility will be developed in 6 stages. In each stage, which will last approximately 3 years, a 15 to 20 acre waste cell will be filled. These cells will be constructed with two synthetic liners and a leachate collection system.

The property boundaries for the facility, shown in Figure 8.1, encompass an area of 304 acres. Of this area, only 96 acres will actually be used for solid waste disposal. The property is relatively flat and is covered with pine and hardwood lowlands and forests.

The site is underlain by Coastal Plain sediments consisting of alternating strata of sand, silt, and clay. These sediments were deposited in fluvial and marine environments. The two formations of primary significance to the landfill study are the Bridgeton Formation and the Cohansey Sand. The relative vertical positions of these two formations under the landfill site are illustrated in Figure 8.2.

Groundwater at the site occurs in a very shallow unconfined aquifer in the Bridgeton Formation and in a confined aquifer in the Cohansey Sand. The groundwater in the upper aquifer, which is recharged by precipitation, flows to the east and southeast toward the water-filled gravel pits shown in Figure 8.1. The

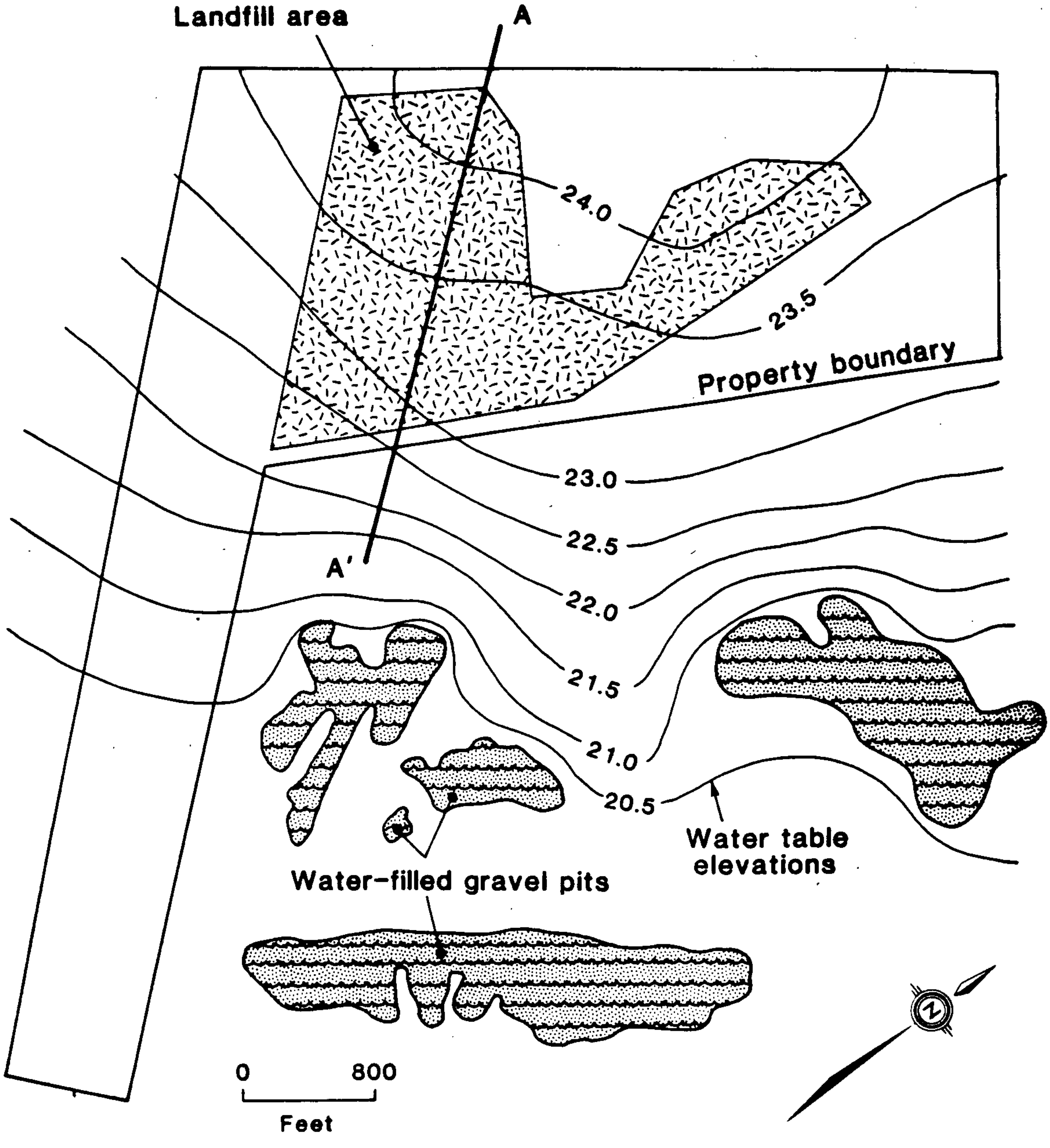


Figure 8.1 - Plan View of Cape May County Landfill Property

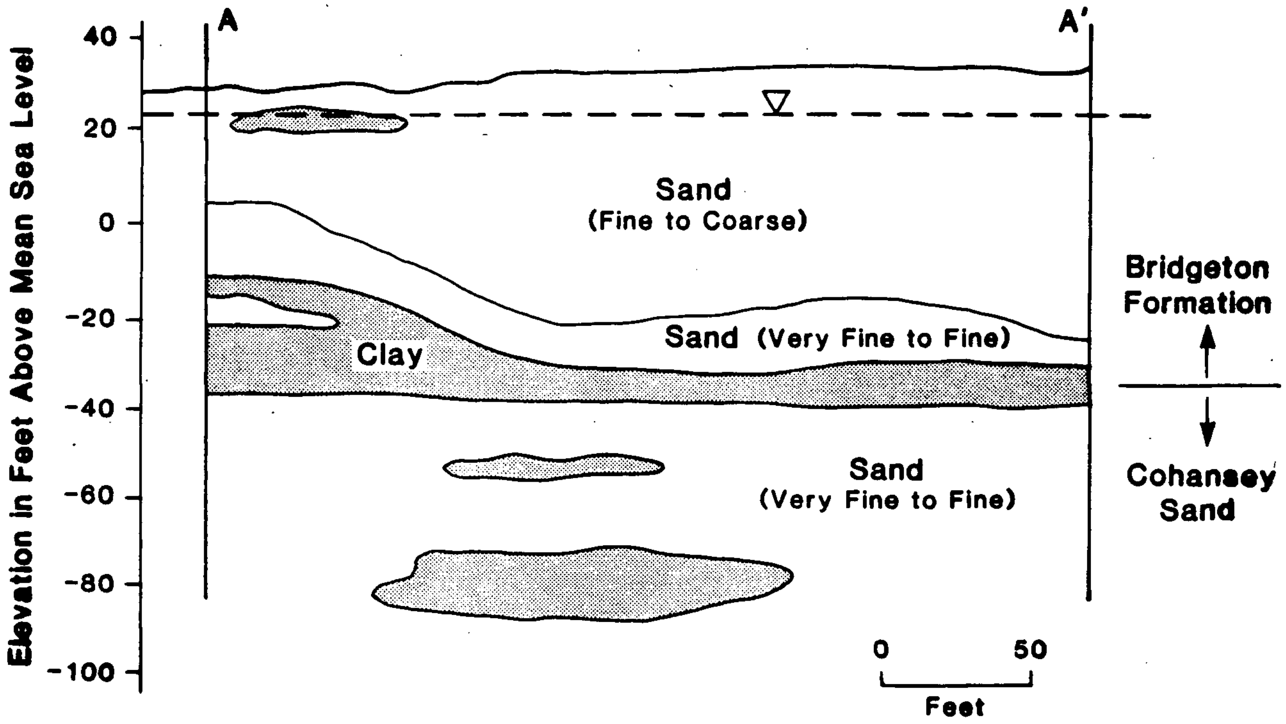


Figure 8.2 - Geologic Cross-Section for Cape May County Facility

gravel pits apparently act as groundwater sinks due to atmospheric evaporation at the water surface. Because precipitation in the area is evenly distributed throughout the year, the elevation of the water table is relatively constant. Depth to the water table at the site ranges from 4 to 14 feet, depending upon the elevation of the ground surface.

Groundwater in the Cohansey Sand, which is used for domestic, municipal, and industrial supplies, also flows in a southeasterly direction. Several wells used for public water supply are located northwest of the landfill site. A slight downward flow gradient exists between the water-table aquifer and the lower confined aquifer.

8.1.2 Hydrogeologic Explorations

Site exploration activities at the landfill site were completed in two phases. In the first phase, 24 small-diameter groundwater observation wells and three test borings were completed in the upper 50 feet of the Bridgeton Formation. Six additional test borings and five additional observation wells were installed in the second phase. These additional borings and wells were as deep as 150 feet and penetrated the Cohansey Sand.

Split-spoon soil samples were obtained at five-foot depth intervals during drilling for the test borings and observation wells. The geologic logs from these samples identified the subsurface stratigraphy that is shown in Figure 8.2. The samples indicate that the Bridgeton Formation beneath the site consists of fine to coarse sand, with some gravel, silt, and clay. The sands are generally poorly sorted and are composed primarily of quartz. The contact between the Bridgeton Formation and the underlying Cohansey Sand is characterized by layers of clay, silt, and fine sands apparently deposited in transitional deltaic environments. The Cohansey Sand formation consists of fine to very fine sand with occasional lenses of clay.

The hydraulic conductivity of the sand comprising the Bridgeton Formation was estimated from grain-size analyses performed on soil samples obtained during installation of the monitoring wells and soil borings. The results of these analyses are summarized in Table 8.1. The mean hydraulic conductivity of the samples is 0.011 cm/s and the coefficient of variation is 1.5.

The hydraulic conductivities calculated from the soil samples are in good agreement with values reported in the literature for the Bridgeton Formation. A New Jersey Department of Conservation publication (Gill, 1962) lists an estimated transmissivity for the Bridgeton Formation in Cape May County of 15,000 gallons per day per foot. Dividing this value by 50 feet, which is the approximate thickness of the saturated zone beneath the site, gives an overall hydraulic conductivity of 0.014 cm/s.

Water levels from the observation wells were used to construct potentiometric surfaces for the upper water-table aquifer and for the lower confined aquifer. The map for the confined aquifer, indicates that flow is to the east. The map for the water-table

Table 8.1 Hydraulic conductivities estimated from grain size-curves for the Cape May County Landfill [Geraghty and Miller; 1982, 1983].

Boring	Depth (ft)	Hydraulic conductivity (cm/s)
B1	15-16	8.1x10-3
B2	45-46	1.0x10-2
B3	2-4	9.0x10-4
B3	25-26	4.0x10-2
B3	45-46	8.1x10-3
B16	15-16	1.0x10-2
B17	30-31	8.1x10-3
B28	0-2	4.0x10-4
B28	4-6	6.2x10-2
B28	40-42	1.6x10-3
W4D	5-6	1.0x10-4
W13D	45-46	1.7x10-2
W23	0-2	2.2x10-2
W23	50-52	9.0x10-3
W24	0-2	9.0x10-4
W24	2-4	2.5x10-3
W24	25-27	3.2x10-2
W25	2-4	2.5x10-3
W25	4-6	1.7x10-2
W26	2-4	1.6x10-3
W26	4-6	6.2x10-4
W26	15-17	2.2x10-4

aquifer, presented in Figure 8.1, shows that the shallow groundwater flows to the southeast toward the gravel pits. A comparison of the two maps indicates a slight downward gradient between the two aquifers. Water level measurements made at several different times during 1982 and 1983 suggest that a steady-state flow system exists.

Groundwater samples collected from Wells W3, W7, W13, and W13D were chemically analyzed during the first phase of the hydrogeologic exploration. The results show that the groundwater is low in dissolved materials, has very little hardness, and is slightly acidic. None of the priority pollutant organics or metals were present and there were no radioactive compounds or coliform bacteria. The water quality was what would be expected from an undeveloped, natural site.

8.1.3 Facility Design and Operation

The landfill at Woodbine, which serves all of Cape May County, receives approximately 116,500 tons of refuse each year. The types of materials that are accepted and prohibited are listed in Table 8.2. The site will be developed in 6 stages. In each stage, a 15 to 20 acre waste cell will be filled. The volume of each cell is estimated to be 908,000 cubic yards. Depending upon how densely the refuse is compacted, the active life of an individual waste cell will be between 30 and 41 months, as shown in Table 8.3. The design density is 1,000 pounds per cubic yard which will result in an active cell life of roughly 3 years.

Table 8.2 - Types of wastes that are accepted and prohibited at the Cape May County Landfill [CMCMUA, 1983].

Accepted Wastes	Prohibited Wastes
Municipal waste	Dry hazardous waste
Dry sewage sludge	Oil-spill clean-up waste
Bulky waste	Infectious waste
Dry non-hazardous chemicals	Waste oil and sludges
Vegatation	Bulk liquids
Animal processing wastes	Liquid sewage sludge
Food processing wastes	Septic tank wastes
Non-chemical industrial	Liquid hazardous waste
	Liquid chemical wastes

Table 8.3 - Active life for waste cells as a function of compaction densities for the Cape May County Landfill [CMCMUA, 1983].

Compaction Rate (lb/cubic yd)	Loading Rate (cubic yd/yr)	Cell Life (yr)
1100.	265,000.	3.4
1000.	291,000.	3.1
900.	324,000.	2.8
800.	3640000.	2.5

Based on landfill volume of 908,000 cubic yards, 3:1 side slopes, 116,500 tons per year of refuse, and 29,000 tons per year of cover materials.

Because of the shallow water table at the site and the overall sensitivity of the area's envrionment, the New Jersey Department of Environmental Protection (NJDEP) has required a dual synthetic liner and leachate collection system for each waste cell. The liner system is constructed with an upper 36 mil Hypalon liner and a lower 30 mil PVC liner separated by 18 inches of sand. Above each liner is a leachate collection system consisting of 6-inch diameter perforated PVC pipes placed on 125 feet centers and connected to an 8-inch diameter header pipe. The header pipes are connected to independently operated sumps. The leachate is disposed of at a nearby wastewater treatment plant. The amount of leachate that is anticipated for each cell while it is in operation ranges from 18 million gallons per year for an empty cell to 3 million gallons per year after the cell has been filled. A final cover consisting of a 20 mil PVC liner covered with 24 inches of soil is expected to prevent any further infiltration into the landfill after it has been closed. However, some additional leachate will be generated after closure as the waste materials consolidate.

A system of 15 wells is used to monitor groundwater quality at the landfill. The parameters to be measured and the sampling frequency are specified by the NJDEP. As required by the regulations, samples must be taken quarterly and analyzed for the seven indicator parameters listed in Table 8.4a. Once a year, the samples must be analyzed for the expanded list of parameters

Table 8.4 - Chemical parameters for groundwater quality monitoring at Cape May County landfill [Geraghty & Miller, 1983]

8.4a - Quarterly parameters	8.4b - Annual parameters
Chloride (Cl)	Coliform bacteria
Hardness	Turbidity
Iron	Color
Phenolic compounds	Taste
Total dissolved solids	Odor
COD	Arsenic
BOD	Barium
	Cadmium
	Chromium
	Cyanide
	Flouride
	Lead
	Selenium
	Silver
	Chloride
	Copper
	Hardness
	Iron
	Mangenese
	Nitrate
	Phenolic compounds
	Sodium
	Sulfate
	Dissolved solids
	Zinc
	COD
	BOD
	ABS/LAS (substances contained in synthetic detergents)

Table 8.5 - Landfill development, expansion, and operating costs for the Cape May County Landfill [CMCMUA, 1983].

A.	Initial Development	Cost (1983 Dollars)
	Land	3000./acre
	Utility	13000.
	Scale house	95000.
	Scale and equipment	44000.
	Equipment shed	63000.
	Drainage	63000.
	Fencing	69000.
	Access road	63000.
	Landscaping	19000.
	Yard lighting	19000.
	Monitoring wells	25000.
	Liner (20 acre site)	567000.
	Leachate collection (20 acre site)	65000.
	Gas venting	63000.
	Equipment	863000.
	Engineering, legal, and administrative	35% of capital costs
B.	Periodic site expansion (20 acres)	
	Monitoring wells	25000.
	Liner	567000.
	Leachate collection	25000.
	Gas venting	63000.
	Drainage	13000.
	Internal roads	6000.
	Final clay cap	90300.
C.	Annual operating and maintenance	
	Equipment maintenance	100000.
	Labor	240000.
	Utilities and miscellaneous	63000.

given in Table 8.4b.

The costs associated with constructing and operating the Woodbine landfill, which are summarized in Table 8.5 [CMCMUA, 1983], can be grouped into four categories: 1) initial development costs, 2) periodic site expansion costs, 3) annual operating and maintenance costs, and 4) post-closure care costs. The initial development costs include the costs of land, access roads, installation of utilities and other services, monitoring wells, landfilling equipment, fencing, scale and equipment housing, and engineering, legal, and administrative sevices. Periodic expansion costs are incurred each time a new waste cell is constructed. These costs include those for liner and leachate collection systems, gas venting systems, and additional access roads. Included in annual operating and maintenance costs are equipment costs, labor costs, costs of intermediate and daily cover materials, cost of utilities, and costs of groundwater monitoring. The post-closure care costs are primarily for groundwater monitoring and site maintenance.

8.1.4 Results of Analysis

The information provided by the various sources that were consulted for this study allowed many of the input variables required for the analysis to be either directly determined or relatively easily inferred. However, some data gaps do exist. The parameters that were not directly available can be separated into two different categories: those that can be estimated with some confidence based upon similar activities at other sites and those that must be arbitrarily assumed. Included in the first group are costs associated with monitoring, site explorations, and remedial actions, the lengths of the fluctuation scales for the hydraulic conductivities, and the discount rate. Included in the second group are the costs of regulatory penalties and litigation, the expected length of the breaches, and the expected liner life. The approach used in the analysis was to assume a range of values for these parameters. The analysis was most sensitive to the expected life of the liners, the discount rate, and the costs of failure.

Table 8.6 presents a summary of the values used to model the Cape May County landfill. A brief description of the rationale used in selecting these values is also included in the table. A more detailed description of the variables is presented in Table 3.1 For those variables that were unknown, a range of values is given. The stream of benefits, costs, and risks for the base case is presented on Figure 8.3. Even before discounting, the magnitude of the risks are small in relation to the benefit and cost terms. The risks, which do not become significant until after approximately 30 years, are signficantly reduced by discounting.

The results of the analysis for the Cape May County Facility are summarized in Tables 8.7 through 8.8. Table 8.7 compares the effect of different liner lives, Table 8.8 shows the influence of the discount rate, and Table 8.9 compares the costs of failure

Table 8.6 - Summary of input variables for the Cape May County Landfill

Parameter	Value	Rationale
NTOO	49 yrs.	One year of construction. Three years of operation per waste cell. Six waste cells. Thirty years of post-closure care.
NTDEV	1 yr.	All construction and preparation completed in first year.
NU	6	Six waste cells.
CL	$288,000.	Ninety-six acres times $3,000/acre.
CS	$930,700.	Scale house: $139,000. Utilities: 13,000. Equipment shed: 63,000. Drainage: 50,000. Fencing: 69,000. Access road: 63,000. Landscaping: 19,000. Lighting: 19,000. Leachate collection: 40,000. Consulting services: 455,700.
CY	$100./m	Assumed value for hollow-stem augers with supervision by engineer or geologist.
CN	$100.	Assumed cost for grain size analyses to determine hydraulic conductivity.
CA	$8.32	A double liner system for a 16 acre site costs $539,000. Dividing this by 64,780 square meters per acre gives $8.32 per square meter for a double liner system.
CQ	$860,000.	Specified in CMCMUA operating plan.
CU	$1.40/t	Equipment maintenance, utilities, and other miscellaneous costs are estimated to be $163,000/year. Dividing this by the expected annual throughput of 116,500 tons/year gives $1.40 per ton.
CT	$0.00	Assume no pre-emplacement treatment costs.

Table 8.6 - continued.

Parameter	Value	Rationale
CB	\$19.80/hr	Calculated from annual labor costs equal to \$240,000.
CW	\$0.00	Cost of residual disposal included in annual maintenance costs.
CD	\$1.12	Cost of restoration per square meter as specified in CMCUA operating plan.
CP	\$5 million	Arbitrarily assumed.
CJ	\$5 million	Arbitrarily assumed.
CE	\$0.00	Energy included in maintenance costs.
CV	\$500.	Calculated from costs for similar monitoring well installations.
CC	\$450.	Assumes indicator tests cost \$100 per sample and full scans cost \$1000 each. Cost of collecting sample assumed to be \$100.
ANUM	0.2	Assume average depth of exploration holes is 10 meters and assume two hydraulic conductivity analyses per hole.
ALUM	0.1	Assume one monitoring point per monitoring hole.
AKUM	4	Groundwater monitoring is required four times per year.
EE	0	Energy costs included in maintenance.
EL	.05 hr/t	Labor requirements specified in CMCMUA operating plan.
WHY	530 m	Amount of exploration drilling specified in Geraghty and Miller reports.
DISC	.10	Discount rate arbitrarily set to 10%.

B1	$28.75	Charge per ton of waste specified in conversations with Cape May County officials.

Table 8.6 - continued.

Parameter	Value	Rationale
R1	0	Assume no recycle benefits.
R2	0	Costs of disposal of residual wastes included in annual maintenance costs.
THETA	0	Assume no benefits after failure.
DELTA	0	Assume scale effects negligible.
DIST	610 m	Assume compliance surface corresponds to edge of gravel pits.
PD	.250	Calcualated for monitoring system with 15 wells as specified in CMCMUA operating plan. Assumes expected hydraulic conductivity equals 3480 m/yr, hydraulic conductivity coefficient of variation equals 1.5, fluctuation scales equal to 60 m, and porosity equal to 0.2
ETM	17.7 yr	Expected time to monitoring system calculated for hydrogeologic parameters presented above.
VTM	45.1 yr2	Variance in time to monitoring system calculated for hydrogeologic parameters presented above.
ETF	62.7 yr	Expected time to compliance surface calculated for parameters presented above.
VTF	326. yr2	Variance in time to compliance surface calcuated for parameters presented above.
BETA	0.0	Assume expected value approach.
WHYM	150 m	Assumes fifteen monitoring wells, each well 15 m deep.
CBP	0	Assume no bond posted.

Table 8.6 - continued.

Parameter	Value	Rationale
DB	20 m	Depth of slurry walls needed to contain contaminant plume assumed to be 20 m.
SC	$8.00	Cost coefficient for surface seal for contaminant plume. Based upon a total containment cost of $2,100,000.
WC	$150.	Cost coefficient for slurry wall for contaminant plume. Based upon a total containment cost of $2,100,000.
S1	200 m	Assumed maximum width of area to be enclosed with slurry wall.
S2	610 m	Assumed maximum length of area to be enclosed with slurry wall.
AREA	64,800 m2	Area of each waste cell specified in CMCMUA operating plan.
CAP	350,000 t	Capacity of each waste cell specified in CMCMUA operating plan.
LOS	20 m	Length of contaminant source resulting from breach arbitrarily assumed to equal 20 meters.
TSTAR1	50 yrs	Expected life of each synthetic liner arbitrarily assumed to equal 50 years.

Table 8.7 - Results of Cape May Landfill Analyses: Effect of Expected Liner Life

	Expected Liner Life in Years		
	10	50	100
Present value of benefits	$.25x10^8$	$.25x10^8$	$.25x10^8$
Present value of costs	$.17x10^8$	$.17x10^8$	$.17x10^8$
Present value of risks	$.51x10^6$	$.14x10^6$	$.50x10^5$
Objective function	$.77x10^7$	$.81x10^7$	$.82x10^7$
Total probability of failure	.47	.19	.07

Table 8.8 - Results of Cape May Landfill Analyses: Effect of Discount Rate

	Discount Rate in Percent		
	5	10	15
Present value of benefits	$.37x10^8$	$.25x10^8$	$.18x10^8$
Present value of costs	$.24x10^8$	$.17x10^8$	$.13x10^8$
Present value of risks	$.47x10^6$	$.14x10^6$	$.78x10^5$
Objective function	$.13x10^8$	$.81x10^7$	$.53x10^7$
Total probability of failure	.19	.19	.19

Table 8.9 - Results of Cape May Landfill Analyses: Effect of Cost of Failure

	Failure Cost in Millions of Dollars		
	2	12	20
Present value of benefits	$.25x10^8$	$.25x10^8$	$.25x10^8$
Present value of costs	$.17x10^8$	$.17x10^8$	$.17x10^8$
Present value of risks	$.79x10^6$	$.14x10^7$	$.19x10^7$
Objective function	$.82x10^8$	$.81x10^8$	$.80x10^8$
Total probability of failure	.19	.19	.19

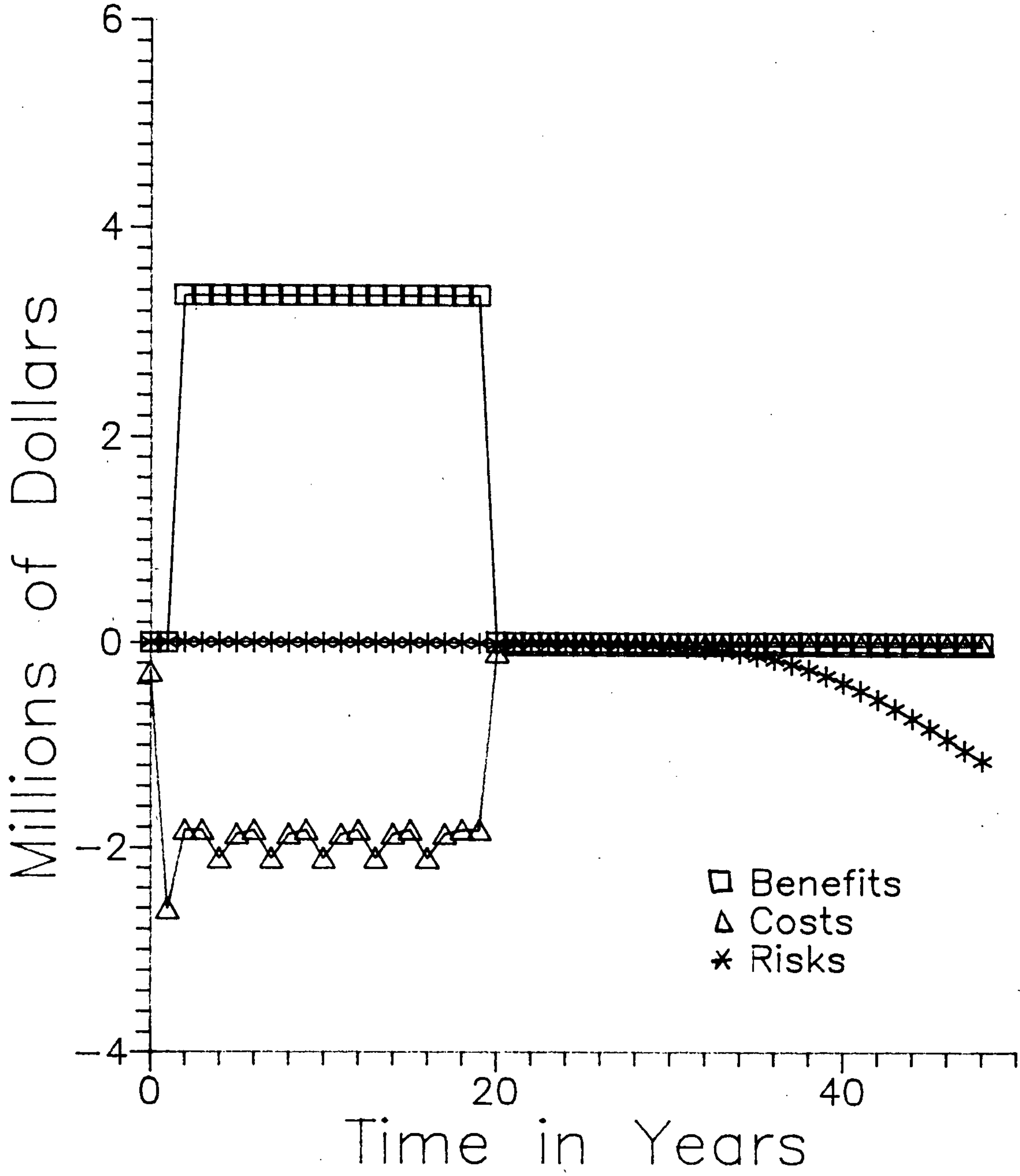


Figure 8.3 - Benefits, Costs, and Risks for Cape May County Landfill

(litigation costs plus penalties). In all cases, the owner/operator's objective function is over $\$5 \times 10^6$. The expected present value of the risks are two orders of magnitude less than the expected present values of both benefits and costs. Because of the relatively large benefits and costs, the analysis is insensitive to parameters that relate to the probability and risk of failure. The parameters that were arbitrarily assumed therefore had relatively minor effects on the overall analysis.

The value report for the total probability of failure in Tables 8.7 through 8.9 should be viewed in a relative, rather than an absolute, sense. It must be recognized that this analysis did not consider the contributions to risk reduction of the leachate collection system or any other components of the landfill design other than the twin liners. The framework that has been described in this dissertation is capable of treating more complex designs, but the necessary additional program modules have not been developed.

8.2 Carlson Landfill

The Carlson Landfill site is located in Clark County, in the southern part of Washington. The site, which is approximately 15 miles north of Vancouver, Washington, is presently operated as a small, privately-owned demolition landfill that accepts dry, non-hazardous waste (primarily construction debris). The Clark County Board of Commissioners has made plans to purchase the property and to upgrade the facility so that it can become the municipal landfill for all of Clark County. As part of the negotiations for determining a fair market price for the landfill property, the Board of Commissioners and the current owner of the property sponsored a study to estimate the present value of future benefits, costs, and risks associated with the proposed municipal landfill [Hart Crowser, 1986]. The general approach that has been described in this dissertation was incorporated into the study. This section summarizes some of the results of that study.

As compared to the Cape May County facility, the Carlson Landfill is much earlier in the planning and design process. The level of detail that has been incorporated into investigations at the Carlson site are more characteristic of siting studies than of design-level studies. The sources for much of the information presented in this section include consultants' reports [Hart Crowser, 1985; CH_2M Hill, 1986; Hart Crowser, 1986], environmental regulations [Washington Department of Ecology, 198?], and discussions with the owner and operator of the Carlson

Landfill.

8.2.1 General Site Description

The landfill property, shown on Figure 8.4, is located in a relatively flat area that is surrounded by alder and fir forests, pastures, and farmland. A few rural residential properties are located in the general vicinity of the site. McCormick Creek, which flows just east of the property, is reported to provide a spawning and rearing habitat for steelhead salmon and cutthroat trout [CH_2M Hill, 1986].

The property boundaries for the facility encompass an area of approximately 110 acres. Of this area, 75 acres will actually be used for solid waste disposal. The design and operation of the facility is regulated by the Washington Department of Ecology [WAC, 1981]. The landfill will receive municipal and non-hazardous industrial wastes.

The site is underlain by three major unconsolidated geologic units: 1) the Older Columbia River Alluvium, 2) the Upper Troutdale Formation, and 3) the Lower Troutdale Formation. The relative vertical positions of these three formations under the landfill site are illustrated in Figure 8.5. The three units are comprised of clays, silts, sands, and gravels and were deposited by streams and rivers.

The Older Columbian River Alluvium, which forms the surficial geologic unit, consists of clayey silt. Only slight variation in

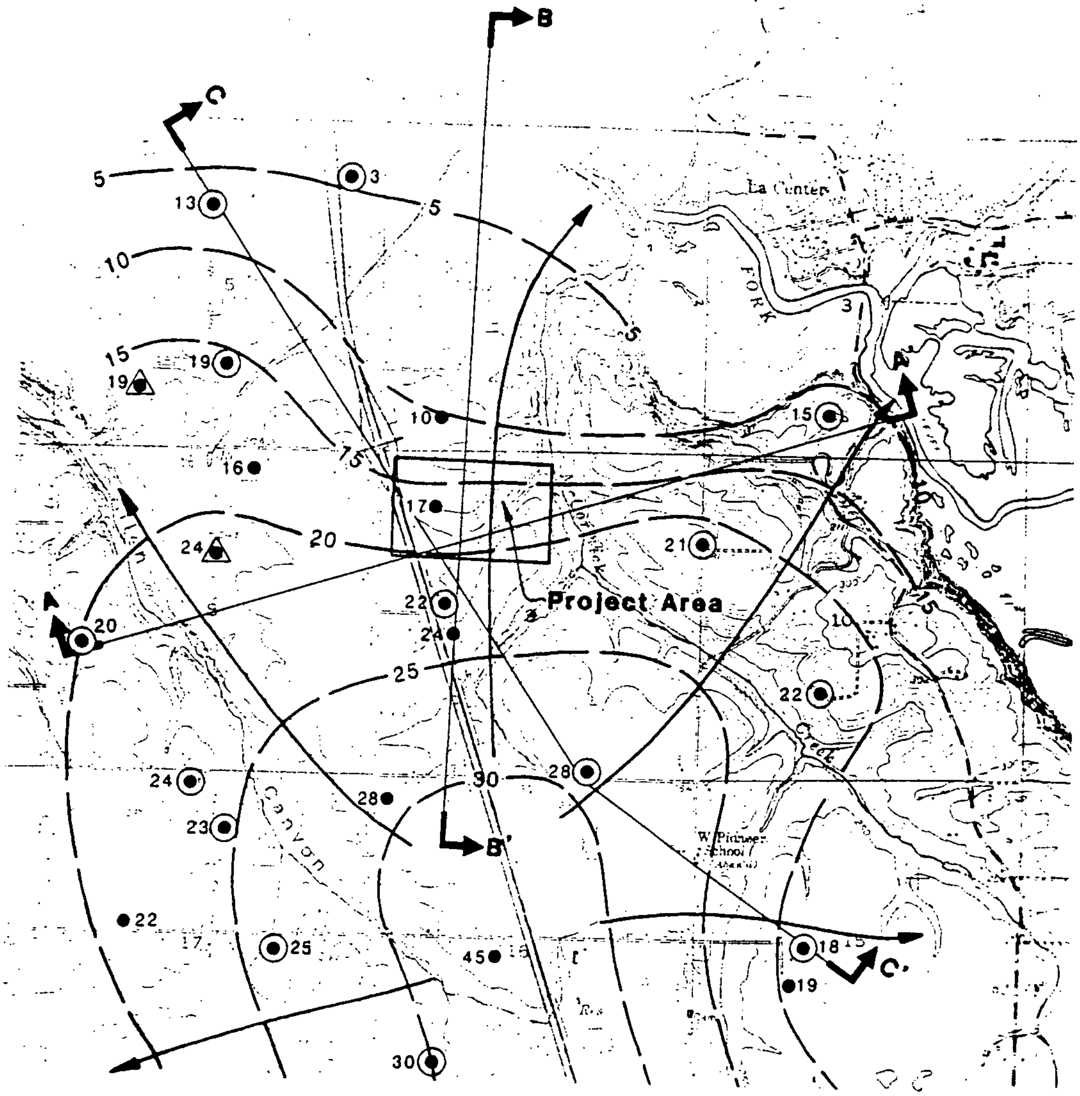


Figure 8.4 - Plan View of Carlson Landfill Property

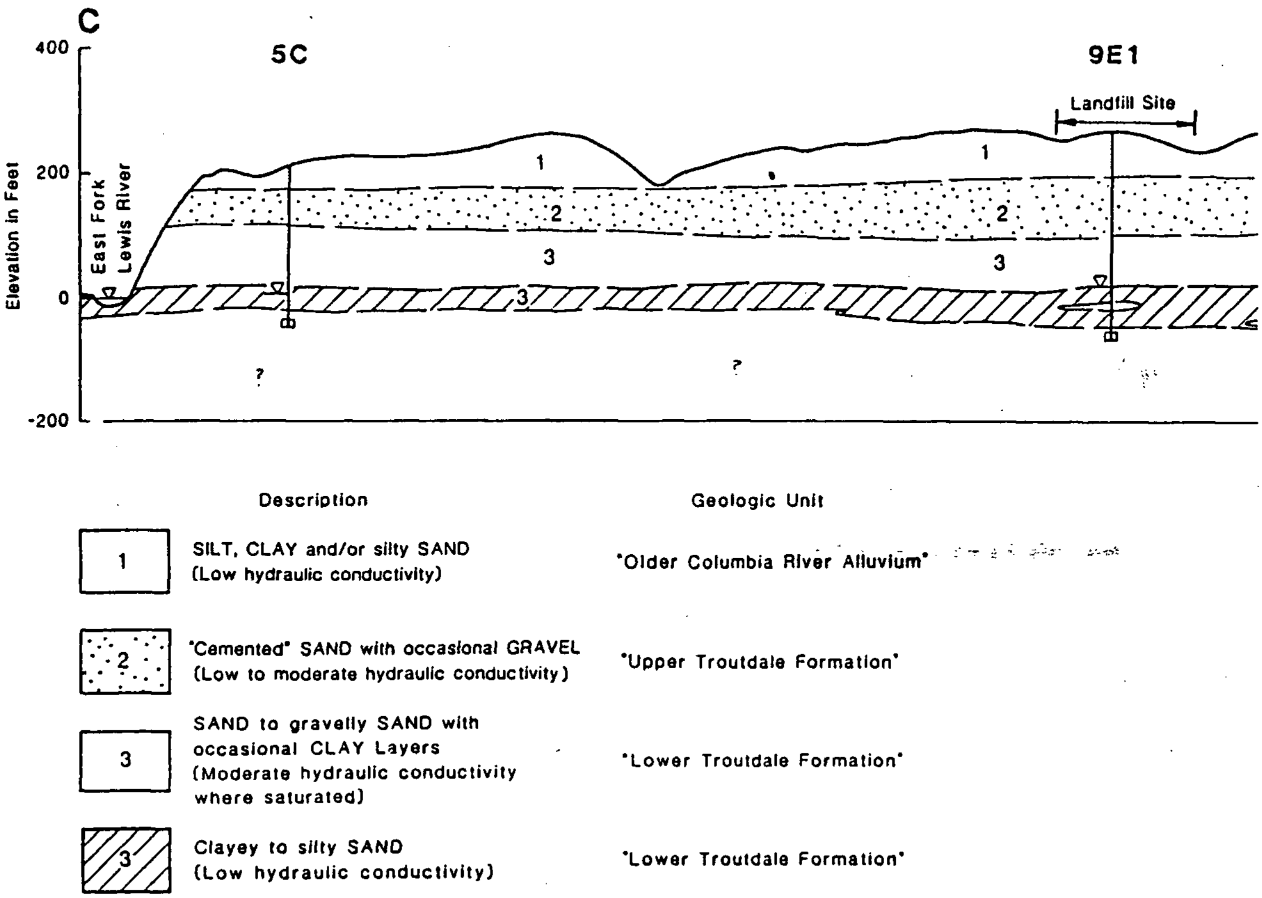


Figure 8.5 - Geologic Cross-Section for Carlson Facility

grain size distribution over the site has been observed, based on nine test pits and three shallow borings made in the alluvium. The alluvium is typically 50 to 70 feet thick over the site. The Upper Troutdale Formation is comprised of slightly cemented sand and gravel that is easily separated into loose material. Field investigations indicate considerable variability in grain-size distribution within this geologic unit. The Lower Troutdale formation is comprised of more fine-grained materials and is described as "fine sand" and "sandy clays" [Hart Crowser, 1985].

Sand units within the Lower Troutdale Formation form the first groundwater aquifer beneath the site. This Lower Troutdale aquifer is the only significant aquifer in the area, as indicated by logs for 88 wells located within 2 kilometers of the landfill site. The aquifer, which lies at or below sea level, is encountered at depths of 250 to 300 feet below the ground surface.

A water table elevation contour map based on a field survey of 29 wells and on information contained in logs for 59 other wells, indicates that groundwater is recharged in the uplands area south of the landfill site and flows toward the three surrounding rivers (the Lake River, the Lewis River, and the East Fork Lewis River). As shown in Figure 8.4, the regional groundwater flow direction beneath the Carlson Landfill site is to the north. Smaller surface-water bodies in the immediate vicinity of the landfill site such as ponds, streams, and creeks are not hydraulically connected to the Lower Troutdale aquifer. No seeps or springs from perched zones have been observed in the vicinity of the landfill site.

8.2.2 Hydrogeologic Explorations and Evaluations

The hydrogeologic exploration activities that have been completed at the Carlson Landfill site, which are quite limited, are fairly typical of what might be accomplished during the siting phase of landfill development. Because of the lack of detailed information regarding the hydrogeology at the site, the approach used to evaluate contaminant travel times through the hydrogeologic environment is to assume realistic, though worst-case conditions. If the estimated risks associated with groundwater contamination are negligible under these worst-case assumptions, as was the case for the Cape May County facility, then a more refined analysis is not warranted. If the estimated risks are important, these assumptions will be re-evaluated.

As indicated earlier, geologic logs from 88 wells have been identified within the general vicinity of the landfill. Only one of these wells is actually located on the Carlson Landfill property, as shown on Figure 8.4. Water level measurements were made at 29 locations to estimate the regional groundwater flow direction shown on Figure 8.4.

Exploration activities that have been completed on the actual landfill property include 1) general geologic reconnaissance including field mapping of surface topography, 2) excavating 27 test pits, 3) performing four in-situ infiltrometer tests to assess hydraulic conductivity, 4) collecting soil samples for grain size analyses (five from the Older Columbia River Alluvium and three from the Upper Troutdale formation), and 5) performing laboratory permeability tests on three samples of the Older Columbia River Alluvium compacted to different densities. The hydraulic conductivity tests indicate that the Older Columbia Alluvium has a saturated hydraulic conductivity in the range of $1x10^{-6}$ to $1x10^{-7}$ cm/s and the Upper Troutdale Formation has a saturated hydraulic conductivity in the range of $1x10^{-4}$ to $1x10^{-5}$ cm/s.

The only available water quality information are electrical conductivity, temperature, and pH measurements made on groundwater samples from 12 wells in the vicinity of the site. These data do not indicate the presence of any groundwater contamination.

Groundwater recharge in the vicinity of the Carlson site is derived from precipitation percolating downward to the Lower Troutdale formation. A mass balance analysis for existing, natural conditions indicates that approximately 8 to 12 inches of precipitation per yer recharges the groundwater system [Hart Crowser, 1985]. Field reconnaissance work and discussions with local residents do not indicate any seeps or springs in the vicinity of the landfill site. Based upon these observations, leachate generated from the landfill will flow downward to the aquifer in the Lower Troutdale formation and then north away from the site.

The compliance point, as specified in the Washington State Regulations, is the uppermost aquifer at a point directly beneath the landfill boundary. To obtain a conservative estimate of the contaminant travel time from the landfill to the aquifer in the Lower Troutdale Formation, the following assumptions are made 1) the groundwater flow direction is vertical and is caused by a unit hydraulic gradient, 2) enough leachate is generated in the landfill to cause continual ponding along the bottom liner, 3) the unsaturated hydraulic conductivity is equal to the saturated hydraulic conductivity, and 4) the storage and retardation capabilities of the geologic materials are neglected.

Two populations of hydraulic conductivity were assumed, one for the Older Columbia Alluvium and one for the Upper Troutdale Formation. The expected value for the hydraulic conductivity was assumed to be 0.3 meters per year ($1x10^{-6}$ cm/s) for the Older Columbia Alluvium and 30 meters per year ($1x10^{-4}$) for the Upper Troutdale Formation. For both materials, the coefficient of variation was assumed to be 1.0 and the vertical fluctuation scale was assumed to be 1 meter. These values were for a mesh size of 1 meter. The Older Columbia Alluvium was assumed to be 20 meters thick and the Upper Troutdale Formation was assumed to be 40 meters thick.

Based upon these conservative assumptions, the expected travel time from the landfill to the aquifer in the Lower Troutdale Formation is estimated to be 20.0 years. The travel time standard deviation is estimated to be 4.47 years.

8.2.3 Facility Design and Operation

The Carlson Landfill has a design capacity of 7.3 million cubic yards which will be recieved during a 20-year operations period. Approximately 170,300 tons of refuse will be disposed in the landfill each year. The site will be developed in 5 stages. In each stage, a 15 acre waste cell will be filled. The volume of each cell is estimated to be approximately 1.5 million cubic yards.

Two relatively large costs are associated with the development of the Carlson site. A large part of the proposed landfill volume is below the existing ground surface. A significant cost will be incurred in excavating these materials. The second large cost is due to some existing refuse material that must be excavated, stockpiled, and then recompacted in the proposed landfill.

Regulations for the State of Washington will require a liner and a leachate collection system for each waste cell in the Carlson landfill. The proposed liner system will consist of a 50 mil Hypalon liner placed over 2 feet of compacted silt material with an hydraulic conductivity less than $1x10^{-6}$ cm/s. Above the synthetic liner will be a leachate collection system. The leachate will be collected and treated at a nearby wastewater treatment plant. The amount of leachate that is anticipated for each cell while it is in operation ranges from 1.6 million gallons per year for an empty cell to 110,000 gallons per year after the cell has been filled and covered. The final cover will consist of 2 feet of compacted silt material with an hydraulic conductivity less than 1×10^{-6} cm/s. All daily, intermediate, and final covers will be constructed using existing materials present onsite.

A system of 4 wells will be used to monitor groundwater quality at the landfill. The parameters to be measured and the sampling frequency are specified by the State of Washington. As required by the regulations, samples must be taken quarterly and analyzed for the indicator parameters. Once a year, the samples must be analyzed for an expanded list of parameters.

The costs associated with constructing and operating the Carlson landfill, which are summarized in Table 8.10, can be grouped into four categories: 1) initial development costs, 2) periodic site expansion costs, 3) annual operating and maintenance costs, and 4) post-closure care costs. The initial development costs include costs for excavation, disposal of existing refuse, monitoring wells, fencing, scale and equipment housing, landscaping, berm construction, and engineering, legal, and administrative sevices. Periodic expansion costs are incurred each time a new waste cell is constructed. These costs include those for liner and leachate collection systems, gas venting systems, and surface drainage systems. Included in annual

Table 8.10 - Landfill development, expansion, and operating costs for the Carlson Landfill site [CH_2M Hill, 1986; Hart Crowser, 1986; Management Advisory Services, 1986].

	Cost (1986 Dollars)
A. Initial Development	
Clearing and grubbing	45,000.
Landscaping and access roads	304,920.
Screening berms	300,000.
Scale	55,000.
Fencing	58,000.
Surface water control	17,000.
Excavation of existing demolition material	2,900,000.
Excavation of first waste cell (15 acres)	1,452,000.
Liner construction (15 acres)	369,500.
Leachate collection and drainage (15 acres)	274,500.
Gas venting wells (15 acres)	45,000.
Construction inspection (15 acres)	112,500.
Monitoring wells	66,000.
Engineering, legal, and administative	1,500,000.
Subtotal:	$7,499,275.
B. Periodic site expansion (15 acres)	
Excavation	1,452,000.
Liner construction	369,500.
Leachate collection and drainage	274,500.
Gas venting wells	45,000.
Construction inspection	112,500.
Clay cap	150,000.
Subsurface drainage layer	190,500.
Topsoil and seeding	39,000.
Subtotal:	$2,633,000.
C. Annual operating and maintenance	
Labor and benefits	272,000.
Equipment	298,000.
Daily and intermediate cover	170,300.
Utilities	115,800.
Leachate treatment	366,000.
Engineering and administrative	60,000.
Subtotal:	$1,282,100.

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Table 8.11 - Summary of input variables for Carlson Landfill.

Parameter	Value	Rationale
NTOO	51 yrs.	One year of construction. Four years of operation per waste cell. Five waste cells. Thirty years of post-closure care.
NTDEV	1 yr.	All construction and preparation completed in first year.
NU	5	Five waste cells.
CL	0	Land will not be sold to operator
CS	\$5,180,000.	Scale house: \$55,000. Landscaping/access roads 304,920. Clearing and grubbing 45,000. Screening berms 300,000. Fencing 58,000. Surface water control 17,000. Excavate existing waste 2,900,000. Consulting services 1,500,000.
CY	\$135./m	Hollow-stem augers with supervision by engineer or geologist.
CN	\$1000.	Assumed cost for in-situ hydraulic conductivity testing and reporting.
CA	\$37.10	Periodic site expansion including excavation and a single liner system for a 15 acre site costs \$2,253,500. Dividing this by 4049 square meters per acre gives \$37.10 per square meter for each waste cell. The actual liner cost is \$6.08 per square meter.
CQ	\$0.	Equipment cost are incuded as an annual expense.
CU	\$3.78	Equipment purchase and maintenance, utilities, leachate treatment and other miscellaneous costs are estimated to be \$644,100/year. Dividing this by the expected annual throughput of 170,300 tons/year gives \$3.78 per ton.

CT	\$0.00	Assume no pre-emplacement treatment costs.

Table 8.11 - continued.

Parameter	Value	Rationale
CB	\$31.94/hr	Calculated from annual labor costs equal to \$272,000.
CW	\$0.00	Cost of residual disposal included in annual maintenance costs.
CD	\$6.25/m^2	Cost of clay cap, subsurface drainage layer, topsoil, and seeding.
CP	\$10x10^6	Arbitrarily assumed, includes cost of estimated cost of groundwater treatment if plume reaches aquifer.
CJ	\$5x10^6	Arbitrarily assumed.
CE	\$0.00	Energy included in maintenance costs.
CV	\$1000.	Calculated from costs for similar monitoring well installations.
CC	\$23,275.	Includes \$12,400 dollars per year for groundwater monitoring and \$360,000 per year for leachate treatment costs.
ANUM	0.022	Assume average depth of exploration holes is 90 meters and assume two hydraulic conductivity analyses per hole.
ALUM	0.011	Assume one monitoring point per monitoring hole.
AKUM	4	Groundwater monitoring is required four times per year.
EE	0	Energy costs included in maintenance.
EL	.05 hr/t	Labor requirements specified in CMCMUA operating plan.
WHY	360 m	Amount of drilling to install monitoring wells.
DISC	.10	Discount rate arbitrarily set to 10%.

Table 8.11 - continued.

Parameter	Value	Rationale
B1	$35.00	Charge per ton of waste specified in MAS operating plan. Based on rates charged at nearby landfills.
R1	0	Assume no recycle benefits.
R2	0	Costs of disposal of residual wastes included in annual maintenance costs.
THETA	0	Assume no benefits after failure.
DELTA	0	Assume scale effects negligible.
DIST	60 m	Assume compliance surface corresponds to the top of the aquifer located in the Lower Troutdale formation.
PD	.00	Monitoring wells are not able to detect contamination.
ETM	NA	Contaminant plume will reach compliance surface before it reaches monitoring points.
VTM	NA	Contaminant plume will reach compliance surface before it reaches monitoring points.
ETF	13.8 yr	Expected time to compliance surface calculated assuming the following hydrogeologic parameters: unit gradient, porosity equal to 0.3, expected value of hydraulic conductivity equal to 0.3 meters per year for the Older Columbia Alluvium and 30 meters per year for the Upper Troutdale formation, coefficient of variation equal to 1.0 and fluctuation scale equal to 1 meter for both the Older Columbia Alluvium and the Upper Troutdale.

VTF	20.0 yr2	Variance in time to compliance surface calcuated for parameters presented above.

Table 8.11 - continued.

Parameter	Value	Rationale
BETA	0.0	Assume expected value approach.
WHYM	360 m	Four monitoring wells, each well 90 m deep.
CBP	0	Assume no bond posted.
DB	NA	Slurry walls not applicable.
SC	NA	Slurry walls not applicable.
WC	NA	Slurry walls not applicable.
S1	200 m	Slurry walls not applicable.
S2	610 m	Slurry walls not applicable.
AREA	60,730 m2	Area of each waste cell specified in MAS operating plan.
CAP	680,000 t	Capacity of each waste cell specified in MAS operating plan.
TSTAR1	50 yrs	Expected life of the synthetic liner arbitrarily assumed to equal 50 years.

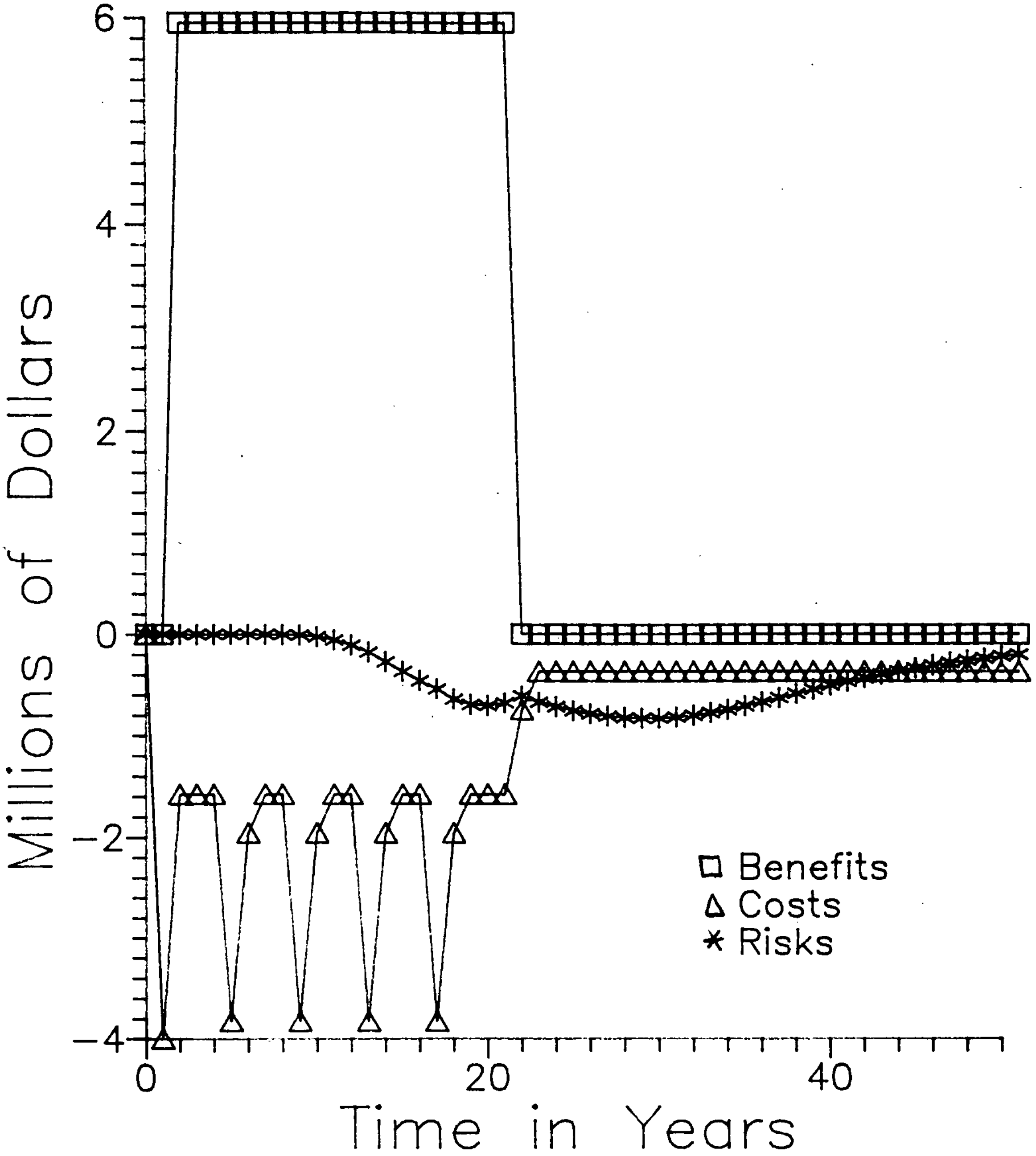

Figure 8.6 - Benefits, Costs, and Risks for Carlson Landfill

Table 8.12 - Results of Carlson Landfill Analyses: Effect of Expected Liner Life

	Expected Liner Life in Years 10	50	100
Present value of benefits	$.46\mathrm{x}10^8$	$.46\mathrm{x}10^8$	$.46\mathrm{x}10^8$
Present value of costs	$.24\mathrm{x}10^8$	$.24\mathrm{x}10^8$	$.24\mathrm{x}10^8$
Present value of risks	$.42\mathrm{x}10^7$	$.18\mathrm{x}10^7$	$.21\mathrm{x}10^6$
Objective function	$.18\mathrm{x}10^8$	$.20\mathrm{x}10^8$	$.21\mathrm{x}10^8$
Total probability of failure	.999	.92	.73

Table 8.13 - Results of Carlson Landfill Analyses: Effect of Discount Rate

	Discount Rate in Percent 5	10	15
Present value of benefits	$.71\mathrm{x}10^8$	$.46\mathrm{x}10^8$	$.33\mathrm{x}10^8$
Present value of costs	$.35\mathrm{x}10^8$	$.24\mathrm{x}10^8$	$.18\mathrm{x}10^8$
Present value of risks	$.53\mathrm{x}10^7$	$.18\mathrm{x}10^7$	$.69\mathrm{x}10^6$
Objective function	$.31\mathrm{x}10^8$	$.20\mathrm{x}10^8$	$.13\mathrm{x}10^8$
Total probability of failure	.92	.92	.92

Table 8.14 - Results of Carlson Landfill Analyses: Effect of Cost of Failure

	Failure Cost in Millions of Dollars 25	50	100
Present value of benefits	$.46\mathrm{x}10^8$	$.46\mathrm{x}10^8$	$.46\mathrm{x}10^8$
Present value of costs	$.24\mathrm{x}10^8$	$.24\mathrm{x}10^8$	$.24\mathrm{x}10^8$
Present value of risks	$.18\mathrm{x}10^7$	$.34\mathrm{x}10^7$	$.67\mathrm{x}10^7$
Objective function	$.20\mathrm{x}10^8$	$.19\mathrm{x}10^8$	$.15\mathrm{x}10^8$
Total probability of failure	.92	.92	.92

operating and maintenance costs are equipment costs, labor costs, costs of intermediate and daily cover materials, cost of utilities, leachate treatment costs, and costs of groundwater monitoring. The post-closure care costs are primarily for leachate treatment, groundwater monitoring, and site maintenance.

8.2.4 Results of Analysis

As discussed earlier, the design of the Carlson Landfill is in its preliminary phases. The hydrogeologic exploration activities that have been completed at the Carlson Landfill site are quite limited and the actual facility design is still conceptual. The approach used to evaluate the relative importance of risks associated with groundwater contamination was to assume realistic, though worst-case conditions. If the estimated risks associated with groundwater contamination are negligible under these worst-case assumptions, as was the case for the Cape May County facility, then a more refined analysis is not warranted. If the estimated risks are important, these assumptions will be re-evaluated.

Table 8.11 presents a summary of the values used to model the Carlson landfill. A brief description of the rationale used in selecting these values is also included in the table. For those variables that were unknown, a range of values is given.

The results of the analysis for the Carlson Facility are summarized in Tables 8.12 through 8.14. Table 8.12 compares the effect of different liner lives, Table 8.13 shows the influence of the discount rate, and Table 8.14 compares the costs of failure (litigation costs plus penalties). In all cases, the owner/operator's objective function is over $\$15x10^6$. The expected present value of the risks are an order of magnitude less than the expected present values of both benefits and costs. Because of the relatively large benefits and costs, the analysis is insensitive to parameters that relate to the probability and risk of failure. The parameters that were arbitrarily assumed had relatively minor effects on the overall analysis and a more refined analysis of the contaminant travel times is therefore not warranted.

Relatively low risks for the owner-operator, as compared to benefits and costs, may impact the types of regulatory policies that would be effective in preventing groundwater contamination. Policies based upon performance standards that rely upon fines or penalties would be less effective than policies based upon design standards.

Once again, the value report for the total probability of failure in Tables 8.12 through 8.14 should be viewed in a relative, rather than an absolute, sense. It must be recognized that this analysis did not consider the contributions to risk reduction of the leachate collection system or any other components of the landfill design other than the synthetic liners. The framework that has been described in this dissertation is capable of treating more complex designs, but the necessary additional program modules have not been developed.

9. REVIEW, SUMMARY, AND CONCLUSIONS

9.1 Review and Summary of Dissertation

The preceeding eight chapters have presented and discussed, to varying levels of detail, a relatively broad and lengthy set of topics associated with groundwater contamination from waste-management sites. In this section, the materials that have been presented are reviewed and an attempt is made to put the various topics in perspective. Each chapter is treated by first summarizing the purpose and content of the chapter and then highlighting the more important material.

9.1.1 Review and Summary of Chapter 1 - Introduction

Chapter 1 introduces the topic of the dissertation, including an overview of limitations and assumptions. The general framework used to evaluate the problem is introduced and the relationship to previous studies is described. A comparison is made between the approach that is proposed and the approach that has been often used in the past by design engineers working on geotechnical and hydrogeological projects.

The two general approaches that are generally adopted in developing regulations dealing with groundwater contamination are 1) basing regulations upon available technologies, and 2) basing regulations based upon the management of risks. The approach based upon technologies, although more easily administered, often results in inflexible regulations prone to obsolescence. The approach based upon risk management, although often resulting in regulations that are difficult to administer, is more flexible and forms the basis for the methodology described in this dissertation.

Regulations related to groundwater contamination are one part of a complex system of physical, economic, and social processes that includes engineering designs, hydrogeological environments, free-market economies, and ethical and political decisions. The inter-relationships among these various processes can be evaluated using risk-cost-benefit analyses. The foundation for these analyses is the contention that waste-management facilities are operated within an adversarial environment in which the objective of an owner-operator to maintain profitability may conflict with the objectives of a regulatory agency established to address the safety and environmental concerns of society.

Although the approach that is developed is applicable to a variety of waste-management scenarios, the dissertation is concerned with the design, operation, and regulation of new landfills in which the primary mechanism of failure involves a breach of containment across engineered barriers and migration through a hydrogeological environment. It is assumed that the siting process has been completed prior to the analysis and that the facilities will be placed in unconsolidated, permeable deposits. The predominant transport mechanism for contaminants is assumed to be advective flow in a saturated, two-dimensional, horizontal aquifer.

To calculate the risk terms in the objective function, it is necessary to estimate probabilities of failure. Geotechnical and hydrogeological engineers have not generally used this design approach. In the past, design engineers working on geotechnical problems have tended to view themselves as "protectors-of-the-public." Safety factors, generally defined as a ratio of capacity to demand, are used in the traditional design approach. The "protector-of-the-public" approach may result in intra-project inefficiencies.

Intra-project inefficiencies are due to trade-offs between levels of effort expended in various activities within a single project. These activities can generally be classified as site investigation, design/construction, monitoring, and remedial actions. Trade-offs exist between the levels of effort expended in each of these categories. If an improvement in performance can be made by shifting resources from one of these activities to another, then intra-project inefficiencies exist.

Intra-project inefficiencies can be avoided by approaching design as a process of balancing the risks of failure against the costs of reducing these risks. The risk balancing role requires 1) an explicit acceptance of the possibility of failure, 2) the adoption of probabilistic analyses in lieu of the more traditional deterministic approaches, 3) the explicit incorporation of engineering economics into the design process, 4) an attempt to quantify the uncertainties inherent in engineering analysis, and 5) an attempt to quantify the consequences of failure in terms of economic and life losses.

9.1.2 Review and Summary of Chapter 2 - Techniques for Selecting Design and Regulatory Strategies

Chapter 2 presents the decision analysis framework that is used 1) to compare alternative design strategies available to owner-operators of waste management facilities and 2) to compare alternative regulatory strategies available to agencies established to address the safety and environmental concerns of society. The overall decision structure is developed and values used to measure uncertainties and consequences are presented. Various decision-making criteria are discussed and techniques for predicting the value of information are described.

Decisions can be described with four components: decision variables, state variables, consequences, and constraints. The consequences that result when a decision variable is selected depends upon state variables, which are often uncertain. To assess alternative decisions, it is necessary to quantify degrees of uncertainty by assigning probabilities.

A number of different interpretations of probability have been proposed. These interpretations range from the very objective to the very subjective. For decisions in geotechnical and hydrogeological projects, the subjective or degree-of-belief interpretation is generally more applicable.

With the degree-of-belief interpretation, discrepancies or biases often develop between a person's assessment of probabilities and an accurate description of his actual underlying judgment. Probability encoding techniques have been developed to help minimize the effects of these biases. Additional data can be incorporated into subjective probabilities in an unbiased manner using Bayes Theorem.

Consequences can be measured in terms of both monetary and utility units. The two are related by a utility curve, which quantifies risk-averseness. In simplified terms, risk averseness is incorporated by assigning disproportionately high utility values to large anticipated economic losses.

Various criteria can be used to compare alternative decisions. These included the maxi-min criterion, the mini-max criterion, the maximum likelihood criterion, and the maximum expected utility criterion. The maximum expected utility criterion has been shown to provide "best" decisions.

The expected value of additional information can be evaluated using the concept of regret. Additional information has value if its cost is less than the reduction in regret that it is expected to provide.

9.1.3 Review and Summary of Chapter 3 - The Risk-Cost-Benefit Equation

In Chapter 3, an objective function comparing benefits, costs, and risks is developed for both owner-operators and regulatory agencies. A description of the specific terms included in the objective function and the effects of the time value of money are presented. The method used to quantify risk is discussed in some detail. The approaches used to estimate the value of life and health and the value of clean water are also reviewed.

The objective function used in the risk-cost-benefit analysis treats the stream of future benefits, costs, and risks in a net present value calculation. For assessing alternatives by the owner-operator, the costs are the capital costs and operational costs of constructing and operating a waste-management facility. The benefits are primarily in the form of revenues for services provided. The risks are defined as the expected costs associated with the probability of failure. The costs associated with the probability of failure are those that affect his profitability: fines, taxes, or charges levied by the regulatory agency; costs of litigation; costs of remedial action; and the value of any revenues forgone if operations must be curtailed or stopped.

For assessing alternatives by the regulatory agency, the costs are the administrative costs of maintaining the regulatory agency. The benefits to society are primarily those associated with the preservation of clean water. The costs associated with the risk of failure are the costs of remedial action where these are not borne by the owner-operator, the value of the benefits undone by the contamination incident (in the form of reduced groundwater quality), and the societal costs associated with the impairment of human health or the loss of human lives.

In this dissertation, it is assumed that the regulatory agency is responsible for protecting public safety and that design engineers will not concern themselves with this issue if an adequate regulatory system is in place. The cost of human life is not included in the risk term used in the risk-cost-benefit analysis of public policy alternatives. For any given alternative, the economic costs and benefits are kept in one account and the lives saved or lives exposed in a separate account. This approach is put in place by maximizing the objective function for the regulatory agency subject to a constraint on the total probability of failure for the facility.

The evaluation of economic decisions in terms of net present value requires that a discount rate be specified. For the owner-operator, it is generally proper to set the discount rate equal to the current market rate on private borrowing. The selection of a discount rate for decisions in the public sector is more controversial. As a lower bound, the social discount rate should be at least as large as the risk-free, constant-dollar interest rates paid on long-term government bonds. The current market rate represents an upper bound.

The time horizon for the regulatory agency is likely much longer than the horizon used by owner-operators. Owner-operators typically make decisions based on 10 to 50 year time horizons while the concerns of regulatory agencies demand a time horizon on the order of at least 100 to 200 years. This incompatibility of time horizons is one of the stumbling blocks preventing the development of effective regulatory policies.

Of the two objective functions, the framework developed for the owner-operator is by far the most valuable. The formulation for owner-operators is much more specific and reasonable estimates are available for the input parameters.

9.1.4 Review and Summary of Chapter 4 - Reliability Theory and the Probability of Containment Breaches

Chapter 4 describes the techniques used to estimate the probabilities associated with the breaching of landfill liners. The causes of breaching are summarized and the reliability equations used to describe the containment structure are developed. The sensitivities of these equations to various input parameters are studied and a summary of assumptions and conclusions is presented.

The complexity of breaching mechanisms precludes using physically- based approaches for calculating the probabilities of individual breaching modes. Because of this, an empirical approach using time-dependent reliability theory is used to estimate breaching probabilities. Physical attributes of the containment structure are included in the analysis, but the actual physical mechanisms of breaching are not considered.

The waste management facility modeled in the present study consists of one or more units or cells. The wastes in each cell are contained by one or more synthetic liners. Each cell will function so long as at least one liner is functioning and the complete system will function so long as all cells are functioning.

Reliability theory allows the probability distribution function for the time until breaching for the complete landfill to be estimated from the probability distribution functions for the individual liners. A large number of distribution forms have been proposed for liner- type components. One of the more general forms is the "human mortality" curve. This curve can be used to represent early failures which may result from construction and installation inadequacies, failures due to external events that have an equal chance of occurring in any given year, and failure as a result of "old-age" or wear.

The parameters used to describe the waste management system are 1) the number of waste cells, 2) the year that each waste cell begins operation, 3) the number of synthetic liners in each waste cell, and 4) parameters defining the probability-distribution functions for the lifetimes for each synthetic liner. The first three items are directly specified by the owner-operator. Item 4), however, requires some interpretation. Based on case histories and empirical data, an average service life of 10 to 50 years appears to be a reasonable estimate.

Sensitivity studies were performed to evaluate the effects of various input parameters, including the number of liners and the number of waste cells. The effect of additional liners is to reduce the number of early breaches due to events that have equal annual probabilities of occurrence. An inescapable fact, however, is that reducing the probability of early breaches increases the probability of late breaches. This very fundamental principal seems to be often overlooked in many analyses.

The effect of additional cells is to increase the number of early breaches while decreasing the number of late breaches. From an owner-operator's point of view, late breaches are much preferred to early breaches because of the effects of discounting. From society's point-of-view, however, late failures may be as undesirable as early failures. In fact, early failures may be preferred since the responsible parties can be more easily identified.

9.1.5 Review and Summary of Chapter 5 - Random Fields and Probabilistic Contaminant Travel Times

The procedures used to estimate probability distribution functions for solute travel times are presented in Chapter 5. The general solute transport mechanisms are reviewed and arguments are made for limiting the analysis to advective transport of stable species. The equations governing solute transport in groundwater are presented and the computer models used to solve these equations are briefly described.

Discussions are presented on treating hydraulic conductivities as random fields in a Bayesian framework. The effects of differences in measurement scales and modeling scales are described.

There are five general mechanisms involved in solute transport in saturated groundwater flow systems. These are advection, diffusion, dispersion, retardation, and decay. The relative importance of the five transport processes depends on the groundwater velocity. For groundwater contamination from waste management facilities, velocities on the order of meters or tens of meters per year are required for groundwater contamination to be important. Case histories and theoretical arguments indicate that advection is the predominant transport mechanism with these velocity magnitudes. The discussions included in this dissertation are limited to single inorganic, non-radioactive contaminant species in a steady-state, saturated flow system in a high-permeability sand and gravel formation.

By considering only advective solute transport, the solute front will move at the same velocity as the average linear groundwater. Groundwater velocities and travel times are estimated by using a finite element computer model which solves the groundwater flow equation in terms of stream functions.

Three general types of error are associated with predictions of groundwater velocities: model error, input error, and parameter error. The most difficult prediction error to eliminate is generally parameter error. Parameter error is due to our inability to accurately incorporate the spatial distribution of hydraulic conductivity into computer models. This inability is due to both uncertainty and variability. Uncertainty and variability can be combined by treating hydraulic conductivity as a random field in a Bayesian framework.

In the present study, in which finite element programs are used to predict contaminant travel times, the hydraulic conductivity of each element is viewed as a spatially-dependent, scale-dependent, uncertain variable and the complete set of hydraulic conductivities is treated as a discrete random field. To fully describe the complete set of conductivities that make up the flow field, it is necessary to specify a multivariate cumulative distribution function that includes covariances to recognize the inter-dependence among hydraulic conductivity values.

The values that should be assigned to the expected values and variances for hydraulic conductivity depend upon the size and shape of the volumes used to discretize the flow field. The relationship between the variance of the point values and the variance of the locally-averaged values can be defined using a variance function which measures the reduction in the variance due to local averaging.

9.1.6 Review and Summary of Chapter 6 - Incorporating Hydraulic Conductivity Measurements and Monitoring Wells

Chapter 6 presents techniques for incorporating the effects of hydraulic conductivity measurements and groundwater monitoring wells. The impacts of hydraulic conductivity measurements on hydraulic conductivity uncertainties are discussed. Methods for estimating how uncertainties in hydraulic conductivities translate into uncertainties in travel-time predictions are presented. Techniques to incorporate the effects of groundwater monitoring efforts in reducing the probability of failure for the waste management facility are also presented. Sensitivity studies are presented to quantify these effects and impacts.

The information obtained from the hydraulic conductivity measurements can be used in two operations: 1) to modify the parameters of the probability density functions, and 2) to modify our best estimate of hydraulic conductivity at the unmeasured locations. For the first operation, a probability distribution must be assumed. There is a fairly extensive literature that suggests that hydraulic conductivities are often lognormally distributed. For two-dimensional analyses, the multivariate lognormal distribution is fully represented by four sets of parameters: 1) a vector of mean values, 2) a standard deviation, 3) a fluctuation scale in the x-direction, and 4) a fluctuation scale in the z-direction.

For the second operation, modifying hydraulic conductivity estimates at unmeasured locations, an observational model is assumed. The model used in this study is based upon linear regression.

Sensitivity studies are presented to qualitatively evaluate the effectiveness of various sets of observations in reducing uncertainty in hydraulic conductivity at unmeasured locations. As expected, the hydraulic conductivity uncertainty decreases as the measurement errors decrease. As the fluctuation scale increases, the "zone-of-influence" of the measurements also increases. This indicates that measurements are much more effective in geologies which exhibit correlation structures.

A comparison is made between hydraulic conductivity predictions that are made using multivariate normal equations and Kriging equations. The results illustrate that the two procedures give essentially the same results.

Depending upon the complexity of the problem, there are three general approaches for incorporating hydraulic conductivity uncertainties into advective transport models: 1) analytical methods, 2) Taylor series methods, and 3) Monte Carlo methods. For relatively simple flow fields, analytical methods can be used. For more complex fields, analytical expressions are replaced with Taylor series expansions. Finally, for complex flow fields and for flow fields that have a high degree of uncertainty associated with the hydraulic conductivity, Taylor series methods do not give reliable results and Monte Carlo methods are required. The Monte Carlo approach is used in the present study.

Sensitivity studies for travel times are performed using a hypothetical flow field. Included in the sensitivity studies are analyses which show how travel times statistics are affected by mean hydraulic conductivity values, by hydraulic conductivity uncertainties, by fluctuation scales, and by hydraulic conductivity measurements. As the mean conductivity increases, the expected value of the travel time decreases, as does the travel time standard deviation. As the conductivity variability increases, the mean travel time decreases and travel time standard deviation increases. Finally, as the fluctuation scale in a direction parallel to flow increases, the mean travel time decreases and the travel time standard deviation increases.

For a geology with relatively small correlation scales, hydraulic conductivity measurements reduce travel time uncertainty, but not by a great deal. The measurements are considerably more effective in reducing travel time uncertainty in a geological environment that exhibits spatial correlation.

The objectives and impacts of groundwater monitoring are presented. The owner/operator uses monitoring as a warning against potential failure. The regulatory agency uses monitoring for enforcement of performance standards. It is assumed for the purposes of this study that the probability of detection of the regulatory agency is unity. This seems to be the only ethically defensible viewpoint for the owner/operator to take in his risk-cost-benefit analysis. For the owner/operator's monitoring network, on the other hand, the probability of detection will generally be less than unity.

The owner-operator's monitoring network reduces risks by reducing the probability of failure. However, there is a cost to the owner/operator associated with the detection of contaminants at the monitoring network. By definition, this probabilistic cost constitutes a risk. In a monitored facility, then, there will be two risk terms in the owner/operator's risk-cost-benefit analysis. One risk will be a cost associated with detection at the compliance surface (the cost of failure) and the second risk will be a cost associated with detection at the monitoring network.

The probability of detection can be estimated using the same Monte Carlo simulations used to predict travel times. The number of contaminant plumes that are detected by the owner/operator's monitoring network depends upon the number and location of the monitoring points, the hydrogeological environment, and the size and location of the breach that emanates from the source area to create the contaminant plume. To illustrate some example sensitivities, probabilities of detection were calculated for monitoring wells in a hypothetical horizontal flow field. As expected, wells near the source and in the middle of the flow field are most apt to detect plumes. The probability of detection increases as the mean conductivity increases and as the conductivity standard deviation decreases. The effects of fluctuation scales are negligible. Finally, the most critical parameter with regard to monitoring system effectiveness is the length of the source or breach.

The way that monitoring is treated in the dissertation ignores many of the practical difficulties faced in the real world: laboratory errors, instrument errors, sampling errors, detection limits, type I and II errors, and so on. Many of these difficulties would tend to reduce the probability of detection. The framework that is proposed would allow consideration of these issues if data were available.

9.1.7 Review and Summary of Chapter 7 - Risk-Cost-Benefit Sensitivity Studies

Chapter 7 has two parts. In the first, it is shown how the risk-cost-benefit analysis can be used by the owner-operator in a decision framework to assess the merits of alternative design strategies. In the second, it is shown how the analysis can be used by the regulatory agency to assess alternative regulatory policy, but only in an indirect manner, by examining the response of an owner-operator to the stimuli of various policies. Throughout the chapter, the sensitivity analyses used to assess alternatives are carried out with respect to a hypothetical base-case.

The net present value of the integrated stream of benefits, costs and risks is used to compare the alternatives. The break-even unit charge, which is the value of the charge per ton of waste that is just sufficient to make benefits equal to costs plus risks, is also used. The total probability of failure over the compliance period is used to compare alternatives from the regulatory agency's point of view.

For the owner-operator, the alternative design strategies which are investigated are 1) site investigation activities, 2) containment construction activities, and 3) monitoring activities. To fully analyse monitoring activities, it is necessary to introduce the concept of regret. The best exploration strategy is the one that minimizes the owner-operator's expected regret. To determine if additional exploration will reduce the owner-operator's expected regret, the objective function must be calculated for a very large number of possible outcomes. This type of analysis, although conceptually straightforward, can involve a considerable amount of computational effort and is not incorporated into the present study.

For regulatory agencies, a comparison of the merits of alternative policies must be based on some measure that reflects their relative success in protecting human health and the environment. The total probability of failure over the compliance period is used as that measure. As discussed in Chapter 3, it is a surrogate for acceptable risk.

A regulatory philosophy can take one of two forms: (1) economic incentives or (2) direct regulation; and in each case there are several alternatives. In the environmental economics literature there is widespread support for the use of economic incentives, but in practice almost all legislation, both for surface water and groundwater, is based on direct regulation. Direct regulation involves setting standards. Such standards may be one of two types; (1) design standards or (2) performance standards. However, design standards almost never stand alone; there are usually performance standards associated with regulatory monitoring activities even when facilities must be built to design standards.

Sensitivity studies are used to investigate the following regulatory issues: 1) the relative merits of design standards vis-a-vis performance standards, 2) the relative merits of design standards on the monitoring network vis-a-vis design standards on the containment structure, 3) relative merits of fines vis-a-vis performance bonds to enforce the violation of standards, 4) the impact of closure, and 5) the importance of siting. The results and implications of the sensitivity studies for both the owner operator and the regulatory agency are summarized in Section 9.3, Conclusions.

9.1.8 Review and Summary of Chapter 8 - Case Studies

Two case studies are presented in Chapter 8. The first is the Cape May County Landfill located in Woodbine, New Jersey and the second is the Carlson Landfill located near Vancouver, Washington. The sources of data used to complete the analyses are described and the results of limited sensitivity studies are presented.

The principal motive for including the case studies is to illustrate that the relatively large amount of data required for the analysis presented in this dissertation can be obtained for fairly typical applications. These data include general site descriptions, hydrogeologic explorations and evaluations, landfill designs, and facility operating plans. These particular landfills were chosen primarily because of the willingness and cooperation of the owners in providing information describing their facility.

For the Cape May County facility, the information provided by the various sources that were consulted for the study allowed many of the input variables required for the analysis to be either directly determined or relatively easily inferred. However, some data gaps do exist. The approach used in the analysis was to assume a range of values for these parameters. The analyses were most sensitive to the expected life of the liners, the discount rate, and the costs of failure.

As compared to the Cape May County facility, the Carlson Landfill is much earlier in the planning and design process. The level of detail that has been incorporated into investigations at the Carlson site are fairly typical of what might be accomplished during the siting phase of landfill development. Because of the lack of detailed information regarding the hydrogeology at the site, the approach used to evaluate contaminant travel times through the hydrogeologic environment is to assume realistic, though worse-case conditions. If the estimated risks associated with groundwater contamination are negligible under these worse-case assumptions, then a more refined analysis is not warranted.

The stream of benefits, costs, and risks for both the Cape May County and Carlson landfills indicate that, even before discounting, the magnitude of the risks are small in relation to the benefit and cost terms. For the Cape May County facility, the owner-operator's objective function is over $\$5x10^6$. The expected present-value of the risks are two orders of magnitude less than the expected present values of both benefits and costs. For the Carlson Landfill, the owner/operator's objective function is over $\$15x10^6$. The expected present value of the risks are an order of magnitude less than the expected present values of both benefits and costs.

Because of the relatively large benefits and costs, the analyses are insensitive to parameters that relate to the probability and risk of failure. The parameters that were arbitrarily assumed therefore had relatively minor effects on the overall analysis. The conclusions and implications that can be inferred from the case studies are included in Section 9.3.

9.2 A Summary of Principal Assumptions

Although the general procedures and techniques that are presented in this dissertation are believed to be applicable to a variety of conditions, the specific conclusions and results that are reported are based upon a number of assumptions. The more important assumptions are summarized below.

* The waste management facility is a landfill, for which the primary design feature is one or more synthetic liners.

* Individual liners and individual waste cells function independently, and the performance of individual liners can be modeled using the mortality curve.

* The analysis is intended to aid in the design of new waste management facilities rather than in the design of clean-up operations at failed facilities.

* The site process has been completed prior to the analysis and the facility will be placed in unconsolidated, permeable deposits.

* The contaminant released is a single, inorganic, non-radioactive, conservative species.

* It is released into a steady-state, saturated, groundwater flow system that can be analyzed with a two-dimensional plan view analysis.

* The flow system is developed in a clean, unconsolidated formation of sand and gravel of high hydraulic conductivity in which the advective component of contaminant migration outweighs the influences of dispersion, diffusion, and retardation.

* The principal source of uncertainty in contaminant travel times is due to uncertainty and variability in hydraulic conductivity.

* The variation in hydraulic conductivity is lognormally distributed and exhibits linear spatial autocorrelation.

* Regulatory compliance is achieved at a compliance point with continuous monitoring, so that the probability of detection of a failure by the regulatory agency is unity.

* The owner-operator will completely avert a failure at the compliance surface if he detects a plume at his monitoring network. There is no risk to society associated with a landfill leak that is detected and contained before contamination reaches the compliance surface.

* If the owner-operator is a municipality or other government agency, it will act much like a free-market, owner-operator.

The assumptions listed above influence the results reported in this study and may impact some of the conclusions that are reached. Two assumptions have been identified for more detailed discussion.

The first is the assumption that the advective component of contaminant migration outweighs the influences of dispersion and retardation. If lateral dispersivity is significant, the probabilities of detection reported for monitoring networks will be too low, and the value of such networks in reducing risk will be greater than the analysis suggests. It should be noted that published case histories for contamination events in alluvial sands generally reveal plume widths that are less than two times their source widths, so that the effect on detection probabilities of lateral dispersion would be similar to the effect on Table 6.6b of an increase in contaminant source width from 10 m to 20 m.

If longitudinal dispersion is significant, and if performance standards are set at concentrations that are a small percentage of the maximum concentrations existing in the plume, then the actual travel times for a particular case will be less than the travel times predicted using advective transport models. This would lead to higher probabilities of failure in any given year and lower values for the objective function. These effects are counterbalanced by the effects of retardation which would tend to increase actual travel times over those reported. It is believed that the results for a complete analysis that integrates advection, dispersion and retardation would be rather similar to the results based on advection alone for contamination events in high-permeability unconsolidated deposits.

The second assumption for discussion is the one that assumes that the owner-operator will completely avert a failure at the compliance surface if he detects a plume at his monitoring network. In reality, it is unlikely that cleanup can be accomplished with 100% effectiveness. The net result would be an increase in the owner-operator's risk over that reported in the results included in this dissertation.

In summary then, some of the assumptions lead to underestimates of risk for the owner-operator, and some lead to overestimates. From the point of view of the regulatory agency some are conservative and some are not. It should be emphasized once again that all the assumptions listed above can be removed or improved within the framework and methodology introduced in this study.

9.3 Summary of Conclusions

Throughout the dissertation, a number of general conclusions have been drawn concerning various topics. The more important conclusions are summarized below.

* Waste management systems can be modeled as a system of waste cells configured in a series structure and waste cells can be modeled as a system of synthetic liners configured in a parallel structure.

* Additional liners reduce the number of early breaches due to external events and increase the number of late breaches due to degradation or wear while additional cells increase the number of early breaches due to external events and decrease the number of late breaches due to degradation and wear.

* Because of the effects of discounting future losses, breaches due to degradation or wear do not significantly affect owner-operators. From an owner-operator's point-of-view, then, the performance of liners can be effectively modeled using the exponential probability distribution, which models breaches due to external events with equal annual probabilities of occurrence.

* Measurements of hydraulic conductivity are most valuable in reducing uncertainties with respect to migration times in hydrogeologic environments with large spatial correlation properties.

* The effectiveness of monitoring networks is greater in hydrogeologic environments that have little variability in hydraulic conductivity, and for contamination events in which the size of the breach is large relative to the spacing of the monitoring wells.

* For the specific base case chosen for detailed analysis 1) the owner-operator's objective function is maximized by a two-liner design relative to a design with either one liner or no liner, 2) the installation of a dense monitoring network is of less value to the owner-operator than a more conservative containment design, and 3) the travel time statistics and the owner-operator's objective function are sensitive to the outcome of site exploration activities. To fully quantify the value of site exploration activities requires an expected-regret analysis that is beyond the scope of this study.

* It is also possible to view the sensitivity analyses from the perspective of the regulatory agency. Alternative regulatory policies can be assessed in an indirect manner, by examining the response of an owner-operator to the stimuli of various policies. Among the conclusions that arise from such an analysis for the base case are the following: 1) design standards are more effective than performance standards in reducing risks, 2) design specifications on the containment structure are more

effective in reducing risk than those on the monitoring network, 3) performance standards are required to identify sites that fail, 4) the nature of the regulatory penalty for violation of a performance standard may not have a particularly large impact on the design decisions of the owner-operator, 5) performance bonds posted before construction have a greater potential to influence design than prospective penalties to be imposed at the time of failure, and 6) siting on low-conductivity deposits is a more effective method of risk reduction than any form of regulatory influence (for the specific case that was analyzed, a reduction of one order of magnitude in mean hydraulic conductivity led to a reduction in risk of five orders of magnitude.

* Although a sensitivity analysis on the length of the time horizon was not performed, it is clear that the incompatibility of the time horizons of the owner-operator of a waste management facility and the regulatory agency assigned to protect the societal interest is a major stumbling block to the development of effective regulatory policy.

* Application of the methodology to two case histories reveals that the relatively large amount of data required for this type of analysis can be obtained for a typical site. The results of the analyses indicate that an owner-operator's risk may be quite small relative to the overall benefits and

costs of operating a waste managment facility. Under such circumstances, regulatory agencies must use design standards and siting criteria rather than performance standards and penalties to ensure groundwater quality.

* The results reported in this dissertation are influenced by the assumptions that underlie the study, as summarized in Section 9.2. Some of the assumptions lead to underestimates of risk for the owner-operator and some lead to overestimates. From the point of view of the regulatory agency, some are conservative and some are not. The policy conclusions reached in this dissertation should not be extrapolated to cases that lie outside the assumed conditions.

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