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Mitigative Techniques for Ground-Water Contamination Associated With Severe Nuclear Accidents

Case Study Analysis of Hydrologic Characterization and Mitigative Schemes

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Pacific Northwest Laboratory
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PREFACE

ORGANIZATION OF THE REPORT

This report is divided into two volumes. Volume 1, "Mitigative Techniques for Ground-Water Contamination Associated with Severe Nuclear Accidents: Analysis of Generic Site Conditions," examines generalized aspects of a nuclear core melt accident. The contents of Volume 1 are:

Executive Summary of Volumes 1 and 2

Section 1 - presents a overview of the study including: purpose of study, study objectives, and the scope of the study with associated limitations.

Section 2 - discusses the major types of severe commercial nuclear power reactor accidents considered for this study. Section 2 includes a discussion of radionuclide release mechanisms and rates expected following a reactor core melt accident.

Section 3 - describes the generic hydrogeologic classification scheme and presents the definition of each generic classification. Ground-water flow parameters (e.g., hydraulic conductivity, effective prorosity, etc.) and contaminant transport parameters (e.g., longitudinal dispersion, retardation, etc.) are discussed.

Section 4 - identifies the various ground-water contaminant mitigation techniques and strategies that may be applicable to ground-water contamination resulting from a severe accident.

Section 5 - presents the results of the evaluation of the radionuclide flux for each generic hydrogeologic classification with an assessment of appropriate mitigation measures.

Volume 2, "Mitigative Techniques for Ground-Water Contamination Associated with Severe Nuclear Accidents: Case Study Analysis of Hydrologic Characteristics and Interdictive Schemes," considers the site-specific aspects of selected hydrogeologic environments and individual mitigative techniques. Three case studies are presented to examine mitigative techniques in greater detail than is possible in the generic analysis. Volume 2 contains:

Section 6 - discusses the geologic and hydrologic conditions at the South Texas Plant. Included are simulations of premitigative contaminant migration, mitigative benefits of a cut-off wall and injection wells.

Section 7 - continues the discussion of the South Texas Plant in greater detail. Emphasis is placed on near-field simulations, design considerations and performance assessment.

Section 8 - presents an analysis of the special features of plant configuration and hydrologic characterization of a fractured anisotropic unit.

Section 9 - discusses the primary findings of the study and includes suggestions for further research.

Section 10 - presents the summary of conclusions.

Appendix A - presents a glossary of geotechnical terms used in this report.

Appendix B - provides a generalized guide to site characterization and code selecti n.

Appendix C - provides a description of the TRANS ground-water flow code.

Appendix D - gives a list supplemental references on contaminant mitigation.

PURPOSE

The case studies highlight the hydrogeologic methodologies required to characterize a site for analysis and the selection of a preferred mitigative scheme. The determination of an appropriate method to interdict ground-water contamination and design engineering structures can only be made at the case study level of analysis. General information is included on topical subjects for the convenience of the reader and to serve as a reference guide to further information. The case studies also demonstrate the conceptual model development that supports the accident scenario(s) and the use of selected mathematical models.

Furthermore, the case studies serve as a validation of the conclusions reached in the generic hydrogeological analysis. The case study results (e.g., maximum contaminant discharge flux and feasible mitigative techniques) are compared to generic based conclusions reached in Volume 1.

The components of each case study are designed to start with the information gained from the generic anlaysis and follow an iterative process of collecting more information and developing more sophisticated conceptual and numerical models. This process is outlined in Volume 1, Figure 1.5-2. In the event of a severe accident the process would be continued until either the analysis indicated that no contaminant interdiction was necessary or that the mitigative scheme in place would be an effective safeguard of environmental concerns.

Case Study Objectives

The case studies are conducted with two concurrent objectives. First, to consider to the greatest possible extent the effects of various contrasting geologic environments on selection of mitigative techniques. And secondly, to emphasize within each case study a separate component of the site characterization process such as, hydrologic description, model selection, code development, and cost effectiveness in contaminant interdiction. The core elements of the three case studies are given in Table I.

TABLE I. Overview of Case Study Content

Case Study No.	Name	Topics of Concentration
One	South Texas Plant	Unconsolidated hydrologic unit, hydrogeologic characterization, evaluation of mitigative methods.
Two	South Texas Plant	Performance assessment, cost effectiveness, mitigative scheme selection.
Three	Marble Hill Indiana	Consolidated fractured hydrologic unit, anisotropic flow field, plant structures

Selection of Sites For Case Study Analysis

The two locations chosen for case study analysis were selected from many sites that fulfilled the hydrogeologic requirements. The availablility and content of the geotechnical data bases were the overriding considerations in site selection. The South Texas Plant and Marble Hill Nuclear Generating Station have no known site or plant characteristics that would make these reactors prone to a severe nuclear accident.

English units of measure are used in Volume 2 so that this document is compatible with existing site documents (e.g., Final Saftey Analysis Reports and U.S. Geological Survey Reports, etc).

CONTENTS

6.0 CASE STUDY NO. 1

6.1 INTRODUCTION

Chapter 5.0 presents the results of pre-mitigative severe accident radionuclide release and transport analyses for each of the six generic site classifications. In addition, a set of matrices is provided which relate the feasibility of implementing selected mitigative techniques for each of the sites to composite hydrogeologic characteristics and technology constraints. The overall purpose of Chapter 5.0 is to provide a screening tool to determine the relative likelihood of significant radionuclide discharges at a given site by generic classification and to identify a preliminary set of feasible mitigative alternatives for further consideration. Clearly, determination of the "best" mitigative action(s) (in terms of technical feasibility, performance, maintenance requirements, service life and costs) requires a detailed evaluation of pre- and post-mitigative radionuclide transport through the ground-water system to potentially accessible environments. The South Texas Plant[a] (STP) case study described in this chapter is representative of the general methodology for performing such an evaluation. Emphasis is focused on the characterization and evaluation of ground-water flow and contaminant transport phenomena important to the South Texas Plant. The intent of this initial case study is to discuss some of the methods, procedures, and analyses necessary to determine the impact of various mitigative strategies on the ground-water flow regime of a specific site. Subsequent case studies will be more heavily involved with issues related to power plant configuration and more comprehensive mitigation performance tradeoff analysis.

6.1.1 Case Study Objectives

The primary objective of STP Case Study No. 1 is to demonstrate a general approach for hydrogeologic characterization and ground-water flow analysis for leading to site-specific evaluation of mitigative techniques. In addition, the case study is designed to:

- quantitatively assess achievable mitigation as a function of site hydrogeology, accident scenario and basic mitigative technique design characteristics proposed for the STP site, and
- numerically and graphically illustrate the spatial effects of selected mitigative techniques on ground-water potentials, flow velocities and travel times.

6.1.2 Relationship of Case Study No. 1 to Generic Classification - Mitigation Matrix

The hydrogeologic conditions underlying the STP site are representative of the porous unconsolidated silicate classification which is described in

(a) The South Texas Plant was selected solely because of adequate data availability. The case study is strictly hypothetical and is intended only to demonstrate certain analytical procedures.

Chapter 3.0 and discussed further in Section 5.6. Relative to the other generic hydrogeologic classifications, the porous unconsolidated silicate sites have high hydraulic conductivity, high effective porosity, low hydraulic gradient and slightly greater than average distances (compared to other power plant sites) to surface water. In general these characteristics apply to the STP site. Given this correlation between the STP site and the porous unconsolidated silicate classification, the matrix presented in Table 5.6.2-1 serves as a useful guide for preliminary selection of mitigative techniques for more indepth evaluation. In turn, the results of the case study will provide quantitative verification of the matrix.

6.1.3 General Methodology for Evaluation of Mitigative Alternatives

The recommended methodology for the evaluation of selected techniques for mitigation of possible ground-water contamination due to severe accidents at nuclear power plants consists of four main steps:

Step 1. Survey of regional ground-water hydrogeologic characteristics and regional flow analysis to determine local boundary conditions.

Step 2. Pre-mitigative local ground-water flow and transport analysis.

Step 3. Performance evaluation of feasible mitigative techniques based on ground-water and contaminant transport simulation.

Step 4. Sensitivity analyses of contaminant transport to hydrogeologic parameters.

This overall approach is intended to be universally applicable to most nuclear power plant sites. The appropriate means would be selected for analyzing ground-water flow and contaminant transport phenomena, dependent on site-specific conditions such as the geologic medium (e.g., porous sandstone, porous unconsolidated silicate, etc.), proximity to water users, and accident scenario. The computational requirements could range from simplified analytical representations to more sophisticated finite-difference or finite-element modeling depending on site conditions, data availability, and compatibility of computational approach with the objective of assessing the feasibility of mitigative alternatives.

A survey of regional ground-water flow characteristics is conducted in order to establish the general hydrogeologic conditions relevant to the study site. The regional ground-water flow analysis, using appropriate ground-water analytical and/or numerical modeling techniques provides the necessary data for determination of appropriate boundary conditions for the local analysis. The local flow and transport analyses are also performed by employing appropriate modeling techniques. As noted above, the techniques employed will be largely a function of the geologic medium hydraulic properties, data availability and the ability of the technique to determine the performance of particular mitigative

method. Generally, the primary measure of relative performance for the mitigative strategies will be contaminant flux at the location of the nearest downgradient surface water body or other accessible environment. Consideration should also be given to the contaminant flux at accessible off-site hydrologic units which may be used for water supply. Without some level of ground-water modeling of site-specific characteristics it would be virtually impossible to recommend, with confidence, an appropriate mitigative strategy.

The local flow and transport analyses are first applied to a pre-mitigative accident scenario to determine the baseline contaminant migration. These results provide the basis for subsequent trade-off analyses of the effectiveness of various mitigation approaches. Parametric studies are typically performed to determine possible limits of the effectiveness (i.e., performance) of a mitigative strategy in relation to both uncertainties in hydrogeologic parameters (e.g., hydraulic conductivity) and changes in major design characteristics of individual mitigative strategies (e.g., slurry walls).

The engineering interpretation of case study results will lead to consideration of performance related factors pertinent to the choice of mitigative alternatives. These factors include construction considerations, cost, durability, and the impact on water table elevations in the immediate vicinity of the plant. The durability issue is particularly important in light of the extended period of acceptable performance that may be required of the selected mitigative strategy. Durability considerations, which are dependent on the configuration of the mitigative strategy, may range from long-term effects of grout exposure to the hydrologic environment to mechanical equipment deterioration (e.g., pumps for well injection).

6.1.4 Case Study No. 1 Approach and Limitations

The approach taken for the STP case study is consistent with the general methodology described in Section 6.1.3. Specifically, a regional hydrogeologic analysis to determine local boundary conditions and a local flow and transport analysis are conducted using the TRANS ground-water flow and transport code developed by the Illinois Water Survey Division (Prickett et al. 1981). The criteria followed in the evaluation and selection of TRANS are discussed in Section 6.4.

For the remainder of this section, the term "model" is used to define a numerical computer code (e.g., TRANS) in conjunction with the data set or the site being studied (STP). In applying a code such as TRANS it is important to realize that a "model", as defined, is a simplification of the real world. However, when properly developed and validated a site specific model does approximate the attributes of the real ground-water system that are important to the objectives of the study. While not a perfect indicator of observed contaminant movement, a hydrologic flow and transport model can provide reconnaissance level (or better) understanding of the transport phenomena for the purpose of evaluating the effectiveness of various mitigative alternatives.

The primary limitation of the STP case study is that, due to the demonstrative nature and scope of this study, only previously published data are

used. If required data are sparse or unavailable, hypothesized data are generated based on the best information available and engineering judgment. In reality a field program would be conducted to provide a sufficient level of data to properly characterize the hydrogeologic properties of the site.

Other limitations of the case study relate to representing the STP aquifer system as a two-dimensional (horizontal) flow system and assuming steady-state simulation of water movement. Use of a two-dimensional model assumes vertically averaged flow and transport over the total aquifer thickness. It also assumes instantaneous mixing in the vertical. In terms of evaluating mitigative alternatives, injection and withdrawal wells and low permeability barriers are assumed to fully penetrate the aquifer. Further, it is assumed that no contaminant leakage occurs between the bottom of impermeable barriers and the aquifer bottom. Though these assumptions represent simplifications of the actual STP flow system, they do not detract significantly from the ability to realistically evaluate mitigative alternatives at the STP.

6.2 DEFINITION OF CASE STUDY NO. 1

6.2.1 Geographical Location and Physical Setting

The STP is situated in south-central Matagorda County, Texas approximately 4.9 km due west of the Colorado River. The STP is located on the Texas Gulf Coastal Plain approximately 17 km inland from Matagorda Bay and 24 km inland from the Gulf of Mexico. Figure 6.2.1-1 shows the site location within the State of Texas. The STP site is influenced by the coastal hydrometeorologic regime and tidal effects of the Gulf of Mexico. In summary, these influences result in (Houston Power and Light 1978):

1. high gross natural evapotranspiration rates,
2. high annual rainfall volumes and hourly intensities,
3. high winds,
4. tropical cyclones,
5. high ambient air temperatures,
6. high natural river water temperatures,
7. moisture-laden warm air masses, and
8. brackish surface water.

The geomorphology of Matagorda County (and the STP site) is typical of a slightly eroded coastal plain. The area is characterized by low relief, abandoned river valleys, marshes, and offshore barrier bars. The surface of the STP site is a depositional plain of the last ice age (Hammond 1969).

The STP site is within the humid subtropical region of Texas, and receives average annual precipitation on the order of 100 cm. Rainfall is normally well distributed on an annual basis with maximum rainfall usually occurring in

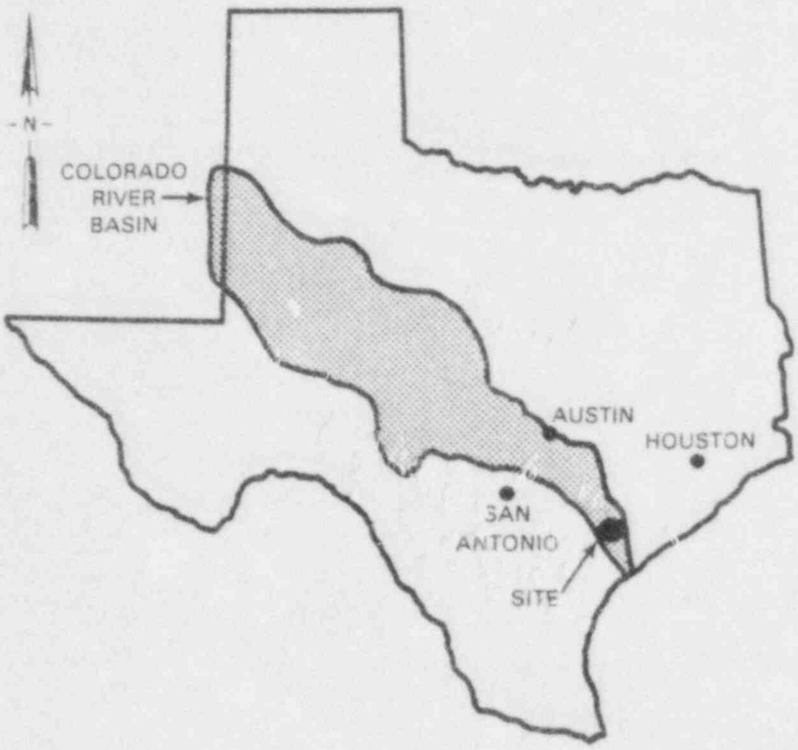


FIGURE 6.2.1-1. STP Site Location Map

September and minimum rainfall occurring in March (Hammond 1969). The area experiences long, hot summers with temperatures exceeding 32°C (90°F) for about 100 days each year. During the winter, cold fronts occasionally move down from the north which mix with the warm air lying over the Gulf of Mexico and produce cloudy, mild, but drizzly weather. Spring experiences mild days, brisk winds and frequent showers. Strong southeast winds begin in March but diminish in April and May and give way to pleasant sea breezes by mid-June. During late June, July and early August, the sea breeze greatly subsides and occasionally fails completely. The area is subject to tropical disturbances during summer and fall with potentially destructive winds. Thunderstorms are frequent but hail is infrequent and tornadoes are rare (NOAA 1980).

6.2.2 Reactor Design and Plant Configuration

The STP is composed of two units each having identical pressurized water reactors (PWR). The two units are roughly 180 m apart and use certain shared facilities including the cooling reservoir, spillway and blowdown facilities, and essential cooling pond. The reactor core-rated thermal power is 3,800 Mwt. High pressure light water serves as the coolant, neutron moderator, reflector, and solvent for the neutron absorber (Houston Power and Light 1978). The reactor containment building has a diameter of approximately 45 m

with a concrete basemat roughly 5.5 m thick. The containment is designed to withstand the internal pressure buildup following a loss of coolant accident.

Figure 6.2.2-1 shows the plant area (i.e., containment buildings, etc.) in relation to other station features. The plant grade is at 8.5 m MSL. The cooling reservoir is located south of the plant area and covers approximately 2800 ha or a little over half of the site property. The impoundment is supplied by water diverted from the Colorado River. The essential cooling pond, located east of the station, is intended to provide cooling water for safe shutdown of the plant. The essential cooling pond is an offstream impoundment which, under normal conditions is supplied with water from the cooling reservoir but has a backup well with 2000 ℓ/min pumping capacity (Houston Power and Light 1978). The essential cooling pond covers nearly 19 ha.

Offsite utility service could be important in considering the types of mitigative techniques that may be implementable. There are eight 345 kV transmission circuits from the STP 345 kV switchyard to the interconnecting grids of the STP owners. The transmission system provides reliable offsite power services any time power is unavailable from the station.

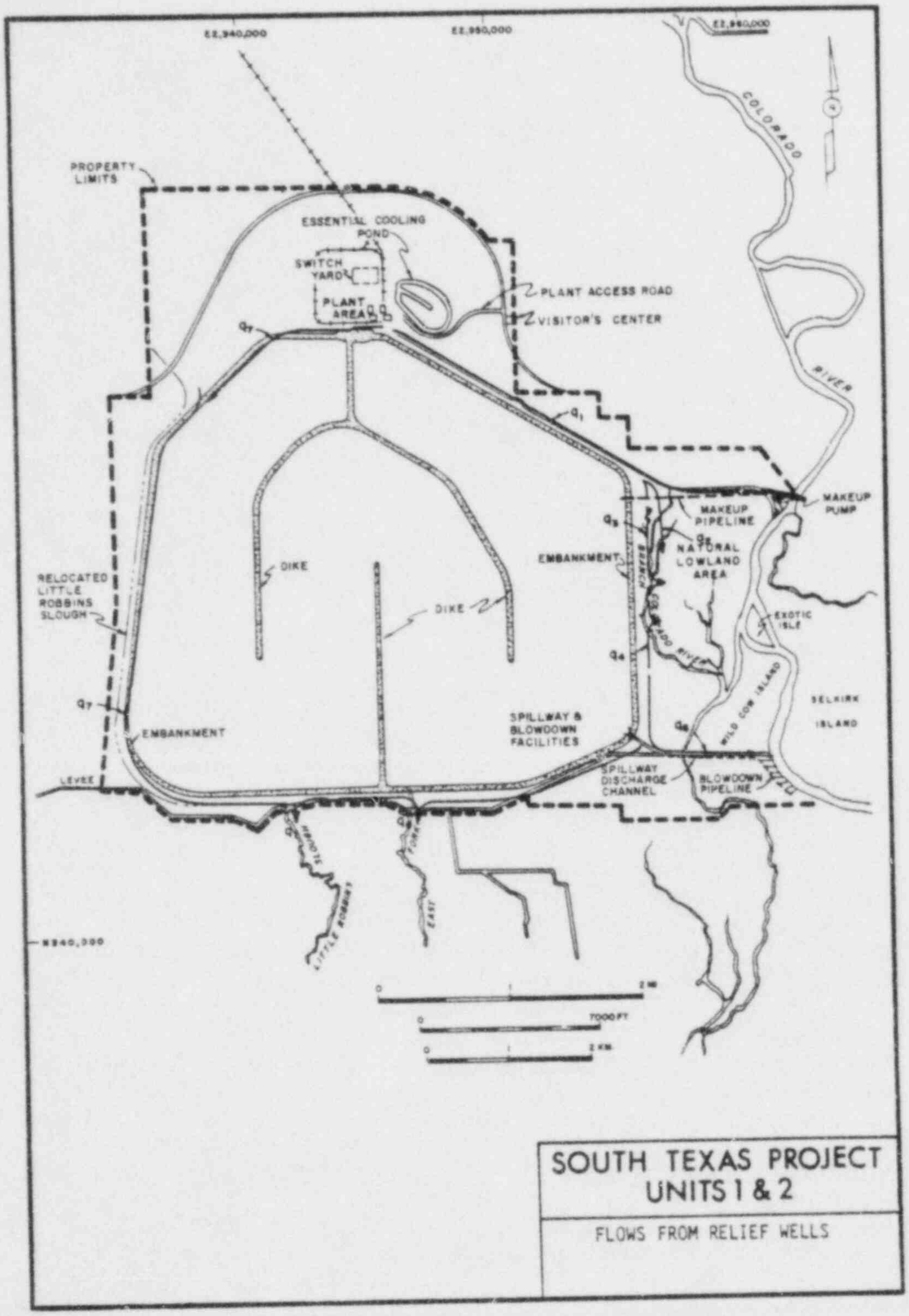
6.2.3 Definition of Accident Scenario

6.2.3.1 Severe Power Plant Accident

The South Texas Plant is a PWR incorporating a double loop for removal of heat from the reactor core. In a postulated severe accident, insufficient heat is removed from the reactor and the core materials overheat to the point of melting. The molten nuclear fuel and supporting materials could contain sufficient heat to melt through the reactor vessel and drop onto the floor of the containment building. The hot core materials would then thermally decompose and melt the concrete containment basemat (USNRC 1975).

The basemat structure could be penetrated (i.e., melted through) by the core melt mass or severely fractured allowing radioactive debris to enter the geologic materials below the power plant. Once the core debris containing nuclear fuel, steel, and liquified geomaterials entered the substratum, cooling would initiate solidification. The decay heat of the radionuclides in the debris would contain enough energy to prevent ground-water contact for about one year (Niemczyk et al. 1981). Ground water flowing through and around the core debris would leach radionuclides from the core melt mass and begin transporting contaminant away from the site. A more complete description of accident sequences and contaminant release is contained in Chapter 2.

In addition to core debris, the cooling water and water used in emergency spray systems could collect in the containment building sump. This water, referred to as "sump water", would become contaminated in the accident process and could be released into the geologic units beneath the plant due to basemat failure. The STP is capable of producing both types of contaminant releases in a severe accident.



<u>FIGURE 6.2.2-1.</u> STP Plant Area (Source: Houston Power and Light 1978)

In a severe nuclear accident, radionuclides of various half-lives, initial quantities, toxicities, and ground-water transport parameters would be released. It is not necessary to determine the ultimate position of all classes of radionuclides. This study focuses on radionuclides that would be released into the ground-water system (as opposed to those that would constitute an atmospheric release). The radionuclides having long half-lives are of concern because they would not decay to low levels very soon after an accident. Radionuclides in large quantities that are not strongly sorbed are also of concern because they have the potential to migrate away from the site more quickly and in high concentrations.

The experience gained in the generic characterization of all nuclear power sites is used to select radionuclides that can serve as indicators of contamination. In unconsolidated silicates (i.e., sand, silt and clay), the radionuclides which best characterize contamination are strontium-90 and cesium-137.

There are several accident scenarios that could result in a nuclear power plant core melt. This study has conservatively assumed that the accident sequence that would release the largest portion of the nuclear inventory has occurred at this site. The amount of radionuclides contained in the core is based on the thermal output of the STP in relation to a theoretical reference reactor described by USNRC (1975). The thermal output and a partial core inventory of the two reactors are listed in Table 6.2.3-1.

TABLE 6.2.3-1. Initial Amounts of Indicator Radionuclides

Radionuclide	Half-Life (days)	Reference Reactor pCi (USNRC 1975)	South Texas Plant pCi (Single Unit)
Strontium-90	10519	3.71×10^{18}	4.53×10^{18}
Cesium-137	11042	4.67×10^{18}	5.70×10^{18}

The radionuclides contained in the core would be partitioned into the core debris, the sump water and the containment atmosphere. The accident sequence preceeding the core melt determines the percentage of the initial inventory that would reside in each of the above partitions under the assumption of the most likely accident sequence. The radionuclide partitioning for this study assumes that the most severe accident sequence has occurred. The resultant initial amounts of key radionuclides available for release are listed in Table 6.2.3-2. A more complete description of the core melt source term is given in Section 2.2.

6.2.3.2 Release of Radionuclides into the Ground-Water System

The penetration of the core melt into the earth below the containment structure is a function of the accident sequence, size of the reactor, and the chemical composition of the geologic materials. Clay and sand at the STP would primarily be chemically composed of silicic minerals. The shape of core melt penetration into a silicate material has been calculated by Niemczyk et al.

TABLE 6.2.3-2. Release Fractions for the Indicator Radionuclides (Source: Niemczyk et al. 1981)

Radionuclide	Sump Water Release, %	Core Melt Debris Leach Release, %
Strontium-90	11	89
Cesium-137	100	0

(1981). The geometric configuration of the core debris would be approximately cylindrical with a radius of roughly 9 m (29 ft) and a depth of approximately 11 m (35 ft) below the basemat or about 25 m (80 ft below MSL). At this depth, the core debris would reside in the lower unit of the shallow-zone aquifer (see Section 6.3.2 for a detailed characterization of the STP site hydrogeology). The deep aquifer, which is used as a source of fresh water, would be isolated from the core melt by over 45 m (150 ft) of clay. The shallow aquifer sands are therefore the hydrostratigraphic unit that would transport the majority of radionuclides away from the site.

The heat contained in the core debris would temporarily vaporize the ground water adjacent to the melt and prevent transport under saturated conditions. It is estimated the top of the core melt in contact with sump water would cool below the boiling point of water in about six months. Similarly, the central portion of the core melt would cool in approximately one year (Niemczyk et al. 1981). The resaturation of the desiccation-alteration zone around the core debris would also delay the transport of contaminant. This study accounts for the temperature of the debris preventing saturation and conservatively does not consider the additional time required for resaturation.

The leach release of silicic materials occurs over a long period of time. Indeed, glass is often used as an isolation medium for radioactive wastes because of its isolation properties and low rate of decomposition. The dominant mechanism for the removal of radionuclides from core debris is matrix corrosion. The silicic leach processes are described more fully in Section 2.4.2. The majority of the radioactivity undergoes decay while still contained in the melt debris and does not enter the ground-water system. However, the leach release does continue for millennia at an exponentially decreasing rate. The leach rate is determined under the same assumptions as used in the generic examination of silicic core melts. The absolute rate is appropriately scaled to represent the thermal size of a single reactor at the STP. Figure 6.2.3-1 presents the release flux of strontium-90 from the core melt debris over time. Because almost 90% of the total strontium-90 present is contained in the core melt debris, sumpwater release, for simplicity, is not considered in this study.

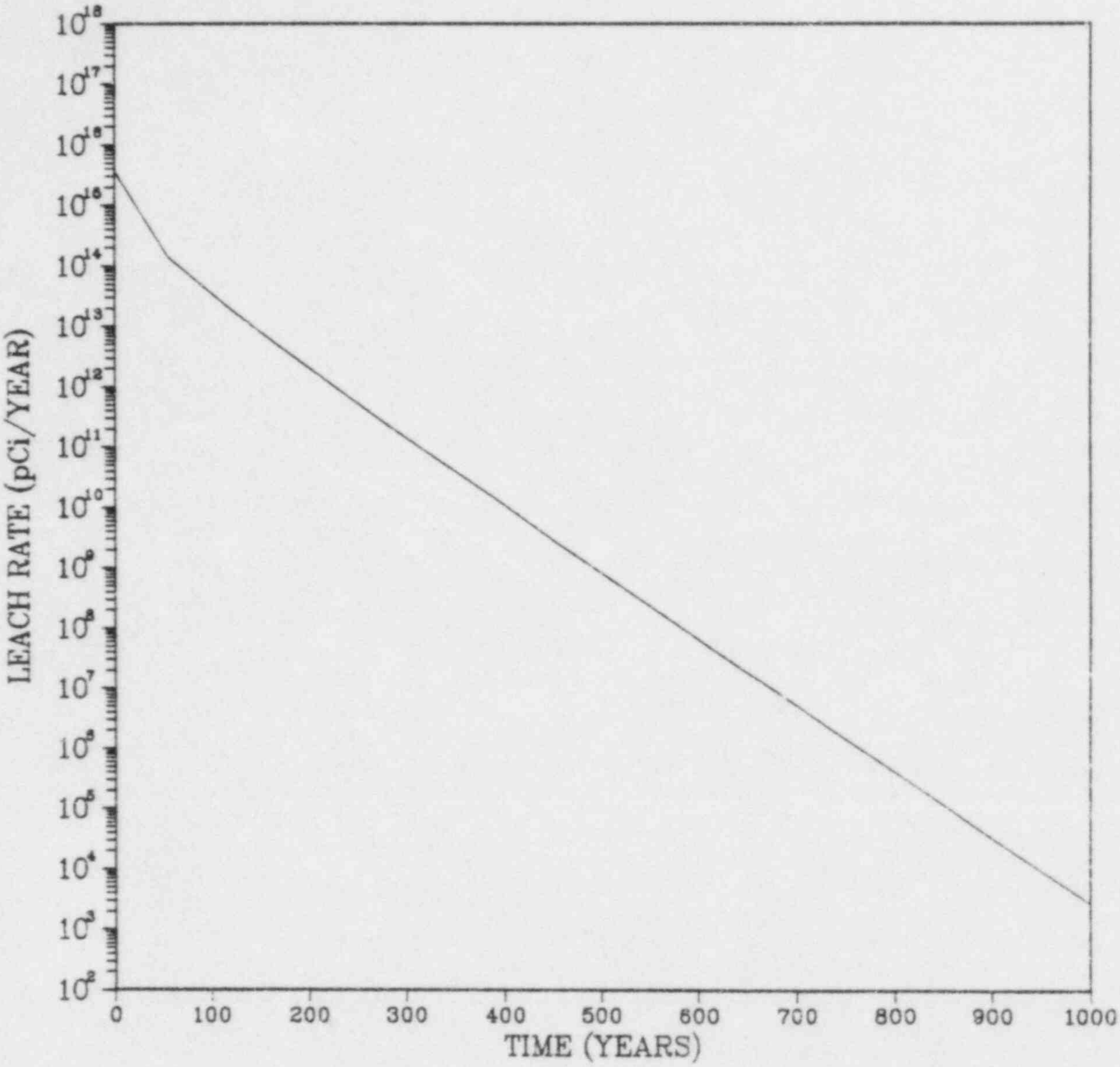


FIGURE 6.2.3-1. Hypothesized STP Leach Release of Strontium-90

6.3 REGIONAL ANALYSIS

6.3.1 Approach[a]

The regional hydrologic system is important in analyzing the hydrology and contaminant transport of more localized systems. This importance has been demonstrated in a ground-water modeling study of remedial action effectiveness

(a) English units of measure are used throughout the analysis because published data pertaining to the hydrogeologic properties of sites in the U.S. are typically in English units.

for the La Bounty Landfill in Charles City, Iowa presented by Cole et al. (1983). Their study also shows that boundary conditions for the local system must be determined from the regional system for pre-mitigative and post-mitigative flow conditions if reasonable estimates of travel times and ground-water flow rates are to be obtained. On this basis, a two stage modeling approach is developed for the STP site. The first stage consists of development of a coarse grid regional ground-water flow model, while the second stage involves development of ground-water flow and contaminant transport model for the immediate vicinity of the STP site.

The application of models to investigate ground-water flow and transport involves several areas of effort: data collection, data preparation for the model, history matching and predictive simulation (Mercer and Faust 1980). The interrelationships of these tasks is illustrated in Figure 6.3.1-1. The first phase of a ground-water model study consists of gathering the available geologic and hydrologic data on the ground-water system of interest. Typically this would include information on: surface and subsurface geology, precipitation, evapotranspiration, pumping, surface streamflows, soils, vegetation, irrigation, hydraulic potential, aquifer properties and boundary conditions. If available data are not adequate, a field data collection program may be required. All of the data are then used to develop a conceptual model of the basin.

As discussed by Boonstra and de Ridder (1981) a conceptual model is constructed based on preliminary assumptions regarding study area size, boundary conditions, number of geologic layers, ground-water flow direction, recharge and discharge locations, etc. The first step in developing the conceptual model is to identify the extent and nature of the ground-water system (e.g., does the system consist of a single aquifer or combination of multiple aquifers). Using the preliminary conceptual model, an appropriate computer code can be selected and development of the numerical model can begin. The model is first used to synthesize the various data and then to test the validity of the conceptual model. From this stage, refinement of the conceptual model and calibration of the numerical model involve an iterative process that continues until the two are consistent with each other and the numerical model adequately reproduces observed data. When this is accomplished, the numerical model is ready for predictive simulations.

The implementation of this process to the development of the STP regional model is described below.

6.3.2 Data Compilation and Conceptual Model Development

Numerical model development and calibration require a variety of quantitative hydrogeologic data that can be classified into two groups (Boonstra and de Ridder 1981):

1. data that define the physical framework of the ground-water system, and
2. data that describe the system inflow and outflow.

Specific data types within each group are listed in Table 6.3.2-1.

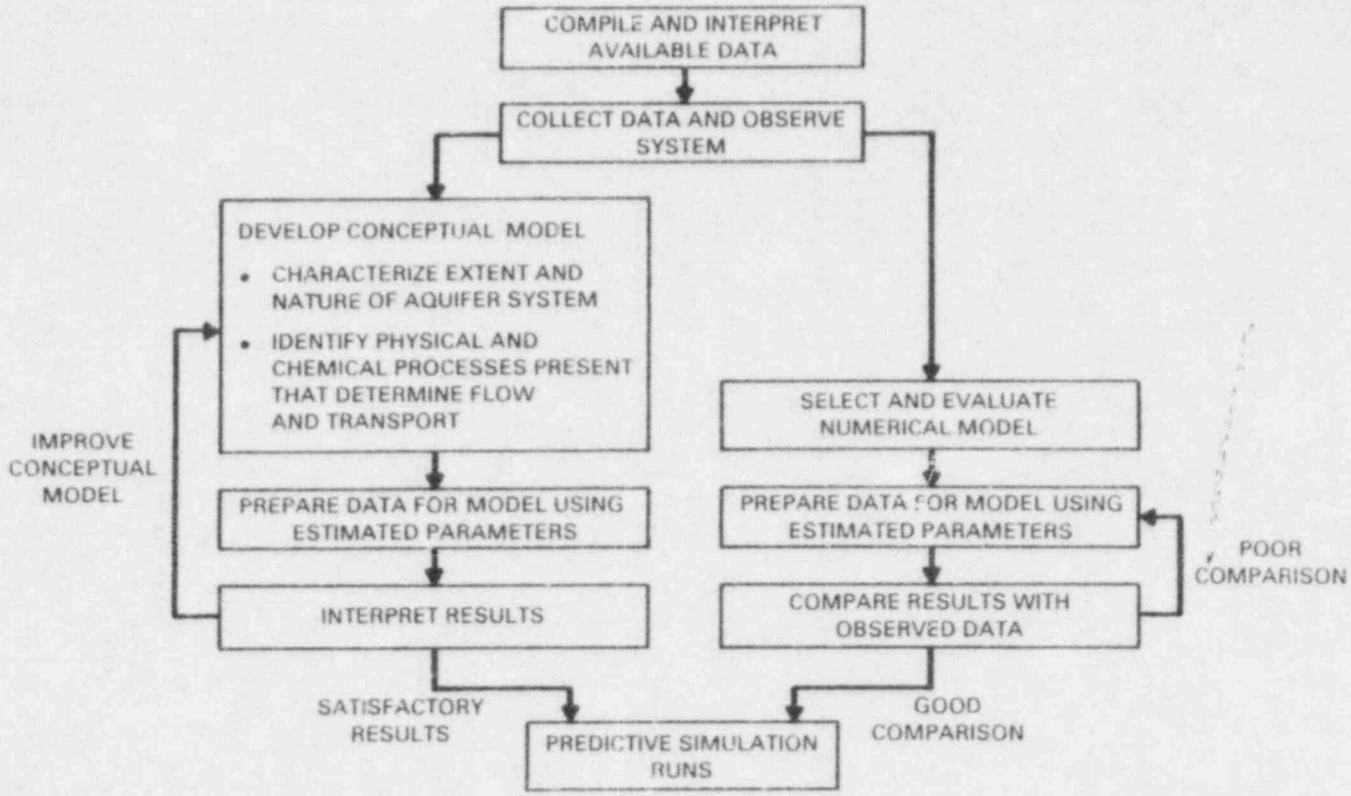


FIGURE 6.3.1-1. Major Steps in Ground-Water Model Applications (Source: Mercer and Faust 1980)

For a particular site there are many sources for the different types of data listed in Table 6.3.2-1. These sources include:

- Preliminary Safety Analysis Reports, Final Safety Analysis Reports, Environmental Reports, etc.
- Local water supply districts, well drillers, engineering consulting firms and other firms which deal with water problems.
- Local and regional Soil Conservation Service offices.
- State and county offices of natural resources, environment, health, or ecology.
- Local, state, regional and national offices of the U.S. Geological Survey (USGS), the U.S. Corps of Engineers, the U.S. Bureau of Reclamation, and the U.S. Environmental Protection Agency.
- Universities and colleges with programs in geology and hydrology.
- Local libraries.

In compiling data for the STP case study, readily accessible sources were used. Selected sections of the STP Final Safety Analysis Report (Houston Power and Light 1978) were relied upon as one of the principal data sources. Other key sources included the USGS and the Texas State Department of Water Resources. The remainder of this section presents the results of the STP hydrogeologic characterization. For each data type listed in Table 6.3.2-1, details of the data sources, analysis and interpretation are provided.

TABLE 6.3.2-1. Data Required for Ground-Water Modeling (Source: Boonstra and de Ridder 1981)

Physical Framework	Hydrologic Stress
1. Topography	1. Watertable elevation
2. Geology	2. Rate and extent of recharge areas
3. Types of aquifers	3. Rate and extent of point and and areal discharge
4. Aquifer boundaries	
5. Aquifer thickness and lateral extent	
6. Porous media material properties	

6.3.2.1 Topography

A basic requirement for conducting a ground-water study is a topographic map delineating surface water bodies, streams, man-made water courses and land surface elevation contours. Maps for the STP site were obtained from the USGS Map Distribution Center in Denver, Colorado. Figure 6.3.2-1 is a reduction of the topographic maps for the vicinity of the STP site. Features identified on

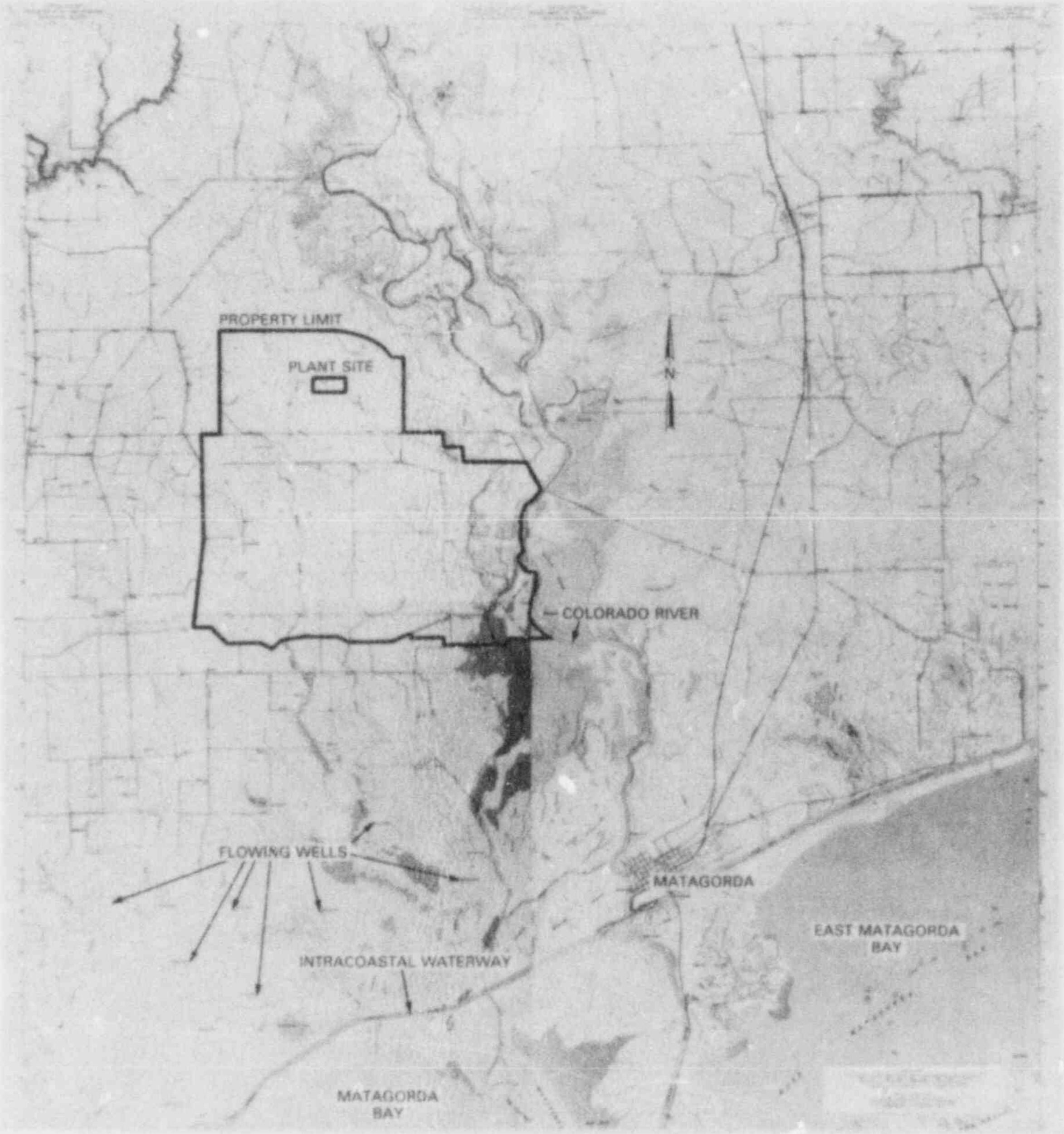


FIGURE 6.3.2-1. USGS Topographic Map for the Vicinity of the STP Site

the map include the approximate STP property limits and site location, Colorado River, Intracoastal Waterway and the Gulf of Mexico. In general, the topography is characterized by gently sloping terrain to the north of the site and flat swampy areas to the south. There are no structural geologic features discernable at the STP site. Local land-forms are subdued due to the gentle slope of the underlying geologic units, weathering and stream erosion.

6.3.2.2 Geology

The structural history the of Texas Gulf Coastal Plain on which the STP is located began in the late Jurassic Period. At this time, roughly 145 million years ago, the crust of the earth began to downwarp as a large regional feature known as the Gulf Geosyncline (Stokes 1966). Sediments eroded from the continental land mass to the north were transported by rivers and deposited in the geosyncline. These processes have accumulated over 50,000 feet of sedimentary material in the central depression of the geosyncline ranging in age from the Cretaceous Period to the present. The regular transportation of sediments in this region has resulted in the formation of the thick sedimentary units forming the Coastal Plain. There are two characteristic features of these geologic units: 1) the sediments are graded, that is, they become finer material (i.e., sand to clay) toward the center of the geosyncline; and 2) the layers of sediments become thicker toward the center of the geosyncline. This depositional formation has created extensive units known as sedimentary wedges that thicken and dip seaward. The southward dip of the older environment is greater than the more recent units because of the continued continental uplift inland and continued downwarp of the Gulf Geosyncline.

Deposition of the more coarse sediments occurred by alluvial processes along rivers and streams. As the rivers altered their channels and deposited additional material, the lateral accretion deposits (i.e., channel lag deposits, channel bar deposits, and point bar deposits) became vertically superimposed. The rivers continually migrated back and forth across the broad low relief depositional plain and created a series of coalescing alluvial and deltaic plains (Houston Power and Light 1978). This process formed geologic units of discontinuous interfingering beds which grade laterally over very short distances from clay to silt to sand to gravel (Hammond 1969). Taken in its entirety, the sediments are referred to as the Gulf Coast Aquifer (Baker and Wall 1976).

The near surface geologic units found in Matagorda County and their hydrologic significance are listed in Table 6.3.2-2. The STP is situated on the Pleistocene Beaumont Formation which extends at least 700 feet below the site. The base of the Beaumont Formation dips to the south at 10 to 20 feet/mile (Houston Power and Light 1978). The upper surface of the Beaumont Formation constitutes the present land surface. The formation is characterized as layers of clay, sandy clay, and thick sand units. The layers of sand are up to 100 feet thick and produce significant amounts of water for irrigation and mining (Hammond 1969). Clay layers of up to 150 feet thick hydraulically isolate the various sand layers.

TABLE 6.3.2-2. Geologic Description and Water-Bearing Properties of Stratigraphic Units Forming the Gulf Coast Aquifer (Source: Hammond 1969)

System	Series	Stratigraphic Unit	Estimated Thickness (ft)	Composition	Water-Bearing Properties and Distribution of Supply
Quaternary	Recent	Alluvium	0-200?	Silt, clay, fine to coarse sand and gravel with wooden debris and logs. Chiefly in eastern portions of Matagorda County.	Capable of yielding large amounts of fresh water. Highly permeable. All irrigation wells in extreme eastern Matagorda County and western Brazoria County are completed in this unit. Fresh water is underlain by saline water in coastal areas.
		Coastal Deposits	0-50?	Beach and dune sand and coastal marsh deposits.	Not capable of yeilding fresh water. Water present is highly mineralized.
		Beaumont Formation	250-900?	Sandy clay, clayey sand, calcareous, fine to medium sand often occurring in thick lenses, some shell beds and calcareous nodules.	Capable of yielding moderate to large amounts of fresh water. Fresh water is overlain and underlain by saline water in coastal areas.
		Montgomery Formation	40-80?	Medium to fine sand, silt and clay. Generally finer grained than underlying Bentley Formation.	Capable of yielding moderate to large amounts of fresh water. Fresh water is overlain and underlain by saline water in coastal areas.
	Pleistocene	Bentley Formation	400-1000?	Thickly bedded, fine to coarse sand and gravel interbedded with clay. Lense-like sand structure.	Capable of yielding large amounts of fresh water in most of the county with the exception of the coastal areas where formation contains highly mineralized water. Supplies water to irrigation wells in the north-central and northwestern portions of the county.
		Willis Formation	80-85?	Very fine to coarse sand and gravel, ferruginous, interbedded with clays.	Not capable of yielding fresh water. Water is highly mineralized except in extreme northwestern portion of the county.

The Beaumont Formation has been characterized in detail at the site of the South Texas Plant by drilling, bore hole logging and reflection geophysical profiling between bores. A hydrologic evaluation combining the geological evidence and piezometric data identified three major sand layers that are capable of transmitting large volumes of water. These sand layers are separated into two hydrostratigraphic units; a deep aquifer at depths greater than 300 feet, and a shallow aquifer consisting of an upper and lower unit ranging between 90 and 150 feet below land surface. A hydrostratigraphic unit is defined as a body of rock or series of formations with considerable lateral extent that compose a reasonably distinct hydrologic system. The distinction of the upper and lower units in the shallow aquifer is based on the presence of a 20 feet thick clay layer that locally separates the units and produces slightly different potentiometric levels. South of the site boundary this clay layer pinches out and the shallow aquifer becomes a single unit.

6.3.2.3 Types of Aquifers

An aquifer is defined as a geologic formation or group of formations that contain sufficient saturated permeable material to yield significant quantities of water (Boonstra and de Ridder 1981). Thus, determination of aquifer type(s) is accomplished by translating the known geologic and hydrologic information into terms of high yield waterbearing formations (aquifers), confining layers (very low permeability) or semi-confining layers (low permeability). Consecutive formations having similar water transmitting properties should be classified as a single aquifer system. For example, consecutive strata of clay, silty clay, sand clay, etc., though different in age and depositional conditions, represent a single layer having similar ground-water hydraulic properties. The three basic types of porous media aquifers; unconfined (watertable), confined (artesian) and semi-confined (leaky) are shown in Figure 6.3.2-2.

The primary sources of information for identifying the aquifer types in the vicinity of the STP site were the STP FSAR (Houston Power and Light 1978) and the Texas Water Development Board Report 91 by Hammond (1969). The aquifers in the site vicinity are found in the lower Gulf Coastal Plain, described as a thick composite of deltaic sediments extending locally to depths of as much as 2000 ft. These sediments are discontinuous, interfingering beds of clay, silt, sand and gravel seldom traceable over very appreciable distances. The different stratigraphic units in the Gulf Coastal Plain are described in Table 6.3.2-1. In Matagorda County, the Beaumont Formation supplies most of the usable ground water and extends from the ground surface to depths of about 700 feet in the area of the STP. Ground water in the Beaumont Formation is confined by an overlying zone of predominantly clay materials up to 150 feet thick. The main producing aquifer zone, designated as the deep aquifer zone, lies below depths of 200 to 300 feet in the site area.

Within the Beaumont Formation there is a shallow aquifer zone that occurs above depths ranging from 90 feet to 150 feet in the vicinity of the site. Based on geophysical and hydraulic tests as discussed in the STP FSAR (Houston Power and Light 1978), the upper zone is segmented into lower and upper confined units. Each unit is characterized by a different piezometric surface.

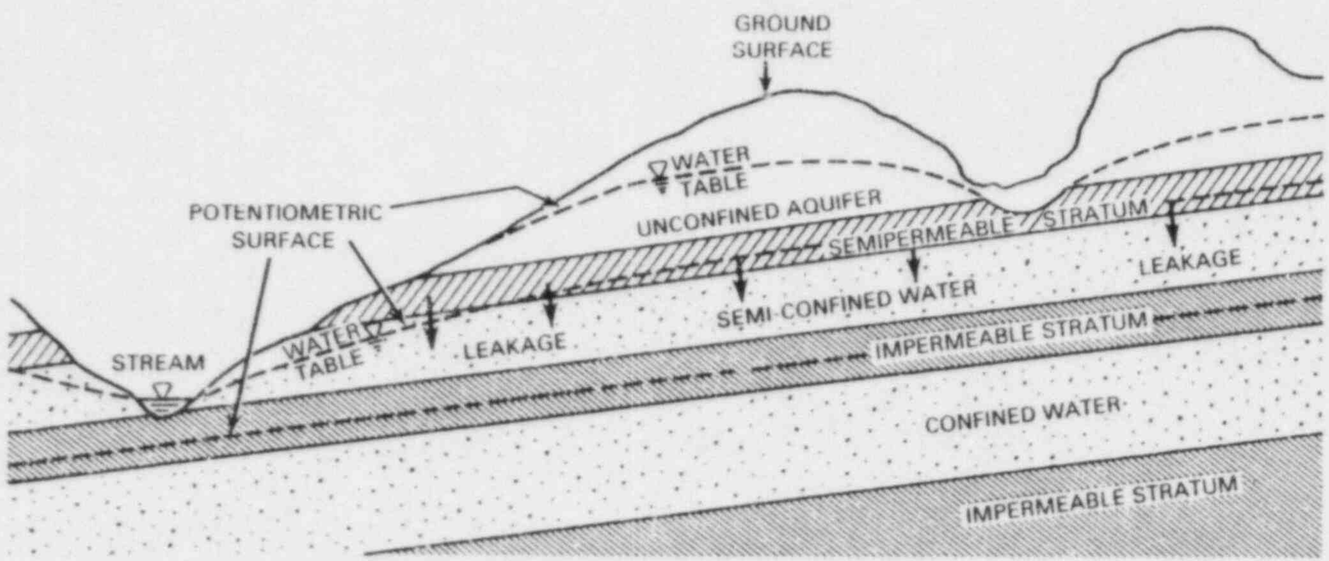


FIGURE 6.3.2-2. Illustration of the Three Basic Porous Media Aquifer Types

The regional geologic configuration in the vicinity of the site is illustrated by the geohydrologic-cross section shown in Figure 6.3.2-3. The cross-section clearly shows the deep aquifer zone overlain by the deep confining zone which ranges in thickness from over 150 feet to almost 250 feet. The shallow aquifer zone is located between the surface confining zone and the deep confining zone. The upper and lower units of the shallow aquifer zone are easily distinguishable, separated by a layer of predominantly impermeable material 25 feet to 50 feet in thickness. The piezometric levels shown on Figure 6.3.2-3 indicate both the shallow and deep aquifer zones are artesian units.

On the basis of the stratigraphy and relative locations of the aquifer units and the discussion of the postulated effects of a severe accident at the STP in Section 6.2.3.2, it is apparent that the molten core mass would penetrate to a depth corresponding to the lower unit of the shallow aquifer zone. Thus, flow and transport would occur under artesian conditions and, in the absence of significant inner-unit transfers, analysis can be limited to the lower unit of the shallow aquifer. This assumption precludes the need to perform detailed study of both the upper and lower units of the upper aquifer. Subsequent data analysis is conducted under this assumption. In an actual assessment of potential accident effects outside the context of a case study, this assumption would be subjected to extensive evaluation before proceeding.

6.3.2.4 Aquifer Boundaries

In addition to describing the thickness and lateral extent of the aquifer in question, the aquifer boundaries must also be properly defined. The different types of boundaries identified by Boonstra and de Ridder (1981) include:

- zero-flow boundaries.
- head-controlled boundaries.
- flow-controlled boundaries.
- free surface boundaries.

Since the free surface boundary is to be determined by the numerical model it will not be discussed here. These types of boundaries are illustrated in Figure 6.3.2-4 and briefly discussed below.

Zero-Flow Boundary

Conceptually, a zero-flow boundary is one across which flows are insignificant relative to flows in the main aquifer. Zero-flow boundaries can occur as either internal or external boundaries. For example, a massive unfractured crystalline formation along the outer edge of an aquifer or a ground-water divide would produce an external zero-flow boundary. A local outcrop of massive rock and an impermeable aquifer bottom would be representative of internal zero-flow boundaries. In developing a ground-water model of a basin, it is necessary to delineate the zero-flow boundaries on a map. Zero-flow is

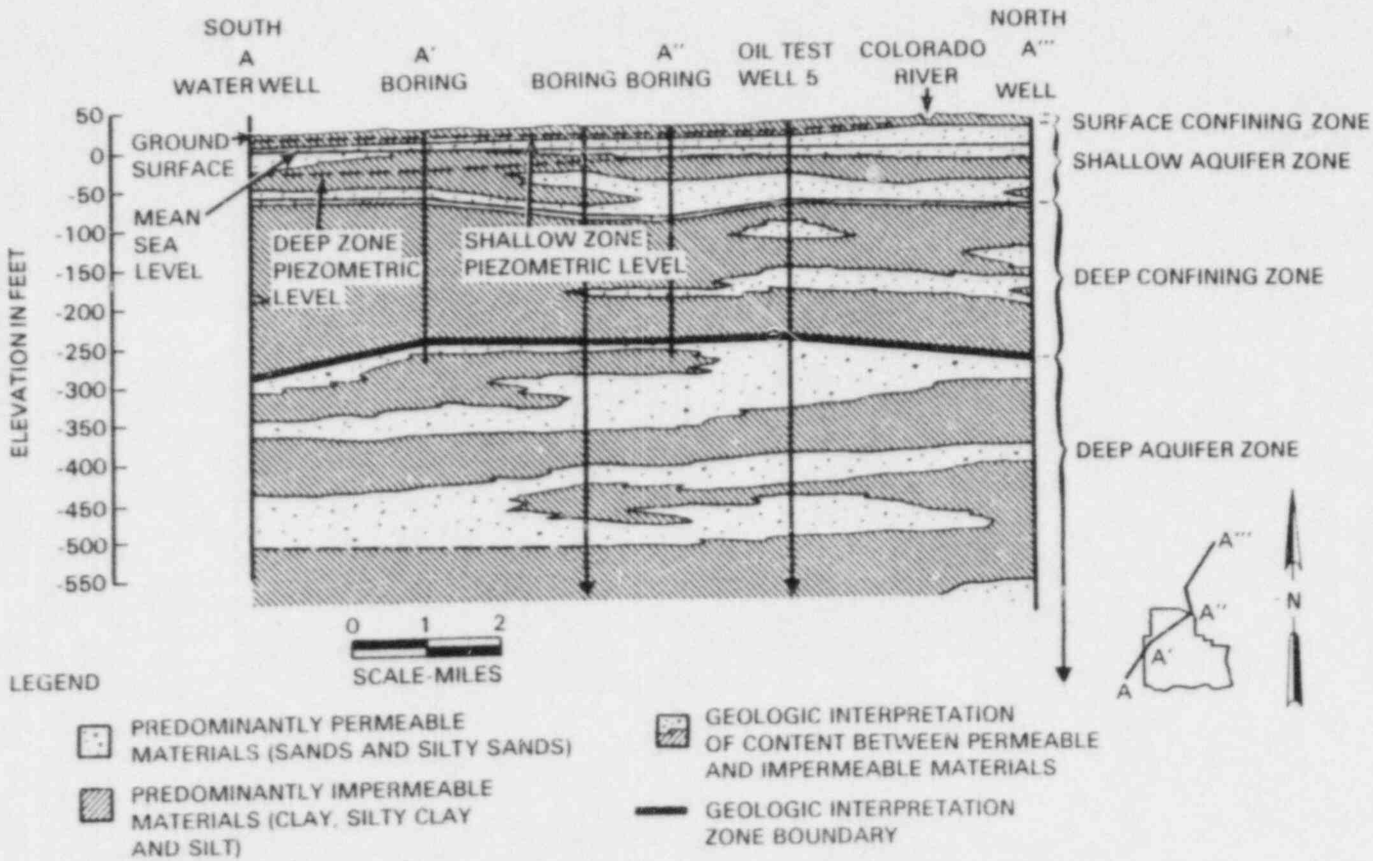


FIGURE 6.3.2-3. Geohydrologic Cross-Section A-A"' for the STP Site (From: Houston Power and Light 1978)

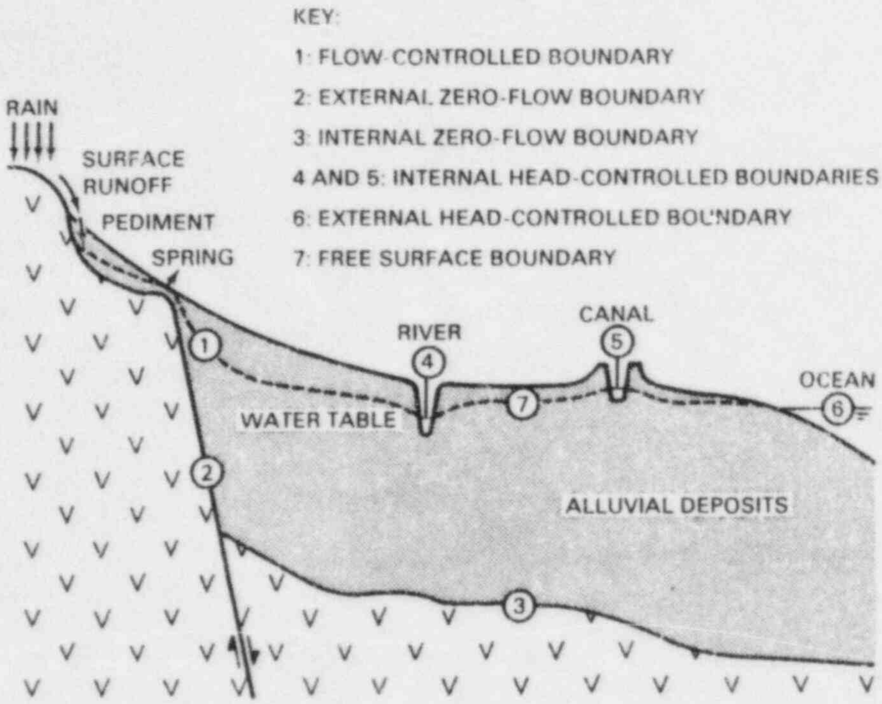


FIGURE 6.3.2-4. Illustration of Different Aquifer Boundary Types (Source: Boonstra and de Ridder 1981)

then achieved in the model by setting the hydraulic conductivity at the boundary equal to zero (Boonstra and de Ridder 1981).

Head-Controlled Boundary

A head-controlled boundary has a known hydraulic head which is either constant or varies with time and is not affected by potentiometric or permeability changes within the ground-water basin. Examples include large water bodies such as oceans and lakes or water courses with fixed water levels like irrigation canals. Similar to zero-flow boundaries, head-controlled boundaries can occur both internal and external to the aquifer. A stream in hydraulic contact with an aquifer inside its boundaries is an internal head controlled boundary while the ocean in direct contact with an aquifer is an external head-controlled boundary (Boonstra and de Ridder 1981).

Flow-Controlled Boundary

A flow-controlled boundary is a boundary through which ground water enters the aquifer at a certain rate from adjacent strata whose hydraulic head is not known. The volume of water transferred in this way is normally estimated by recharge based on rainfall and runoff data. The aquifer boundary may be a

zero-flow boundary but a portion of the incident precipitation may percolate into the colluvium and enter the aquifer at the boundary as ground-water flow (Boonstra and de Ridder 1981).

Selection of STP Boundary Conditions

During development of a ground-water model, it is advantageous in defining boundary conditions to select the external boundaries of the model so they coincide with head-controlled and/or zero-flow boundaries. If the model is being developed for only a portion of the basin, however, it might be necessary to arbitrarily choose a boundary where ground water flows into or out of the basin. In this case, the flow rate must be computed based on the boundary heads and hydraulic conductivity.

Selection of boundaries for the lower unit of the shallow aquifer in the STP site vicinity was based upon Hammond's (1969) general description of ground-water movement in Matagorda County and analysis of the observed hydraulic potentials in the vicinity of the site presented in the STP FSAR (Houston Power and Light 1978). As described by Hammond (1969), the ground water underlying Matagorda County moves continually from the principal areas of recharge, to the north in Wharton County, to the southeast toward the Gulf of Mexico where the primary discharge occurs. Hammond (1969) also points out that at times, though the Colorado River is completely dammed at a point below Bay City, its flow is partially resumed by ground-water seepage. Further, the STP FSAR (Houston Power and Light 1978) states: "shallow-zone discharge is into Matagorda Bay and the Colorado River estuary at least 5 miles to the southeast of the power station area." Thus, it was initially thought that the shallow aquifer discharges into Matagorda Bay except possibly where it is intercepted by the Colorado River. The pre-construction piezometric levels observed on March 14, 1974 for the upper unit of the shallow aquifer suggest this is occurring. The contours, presented in Figure 6.3.2-5, show a definite steepening in gradient as they approach the Colorado River, indicating the upper unit is hydraulically connected to the river. The contours for the lower unit, presented in Figure 6.3.2-6, show a tendency to align themselves with the river. However, the piezometric levels do not converge to the apparent water level of the river. This circumstance indicates that the lower unit only discharges a portion of its flow to the Colorado River as upward seepage through its confining layer.

On the basis of these observations, a regional study area was designed with the x-direction roughly parallel to the observed lower unit contours of equal hydraulic head. The rectangular area, partially outlined in Figure 6.3.2-7, superimposed on the site topography maps extends to the south into Matagorda Bay and East Matagorda Bay and to the north beyond the STP property limits. The exact regional boundaries were chosen arbitrarily at a distance far enough away from the STP site so that assigned boundary conditions would not greatly influence the local hydraulic conditions in the immediate vicinity of the STP. The complete grid, approximately 11.4 mi. by 13.3 mi. (2000 ft by 2000 ft grid elements) is shown in Figure 6.3.2-8.

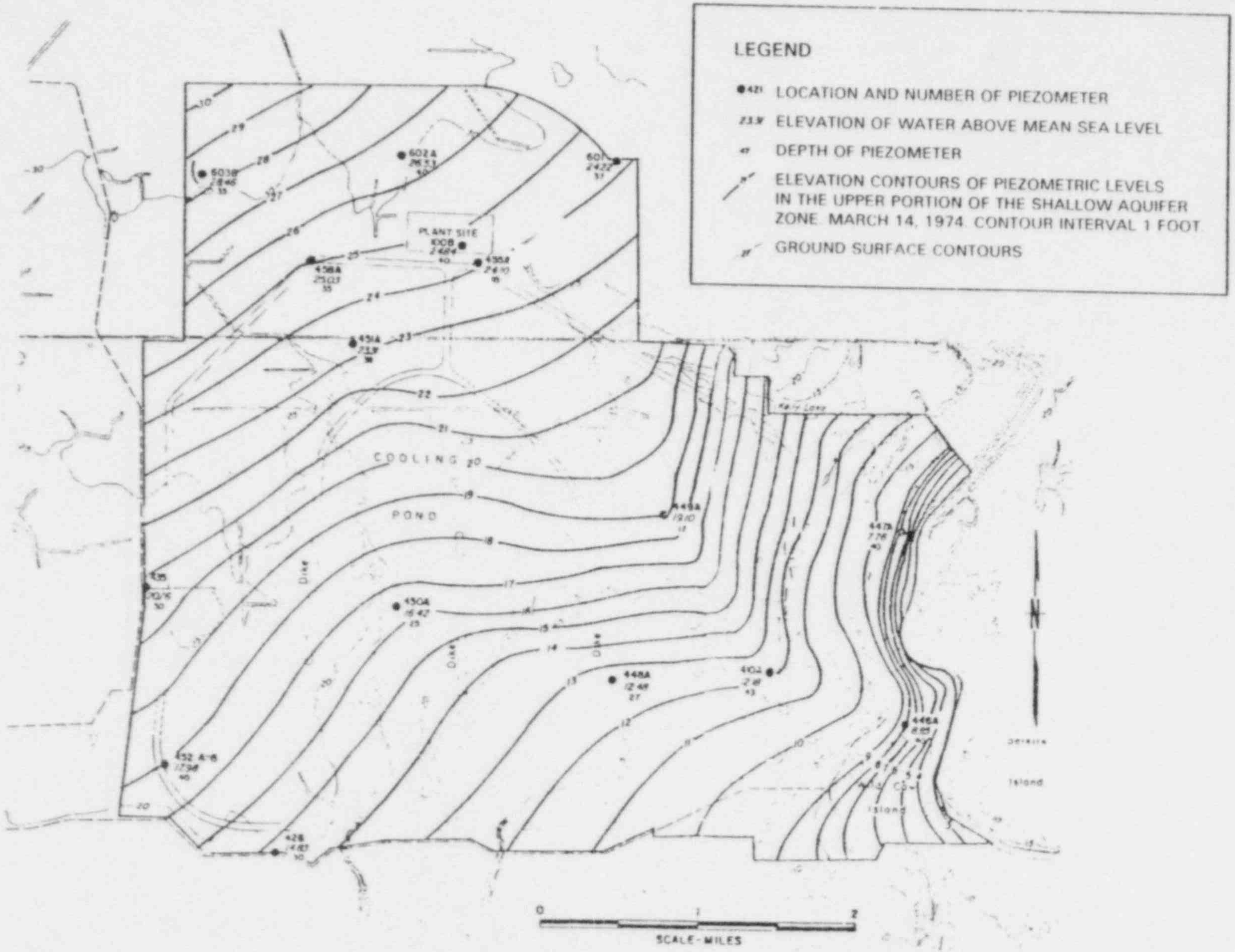


FIGURE 6.3.2-5. Observed Potential Contours for the Upper Unit of the Shallow Aquifer Zone (Source: Houston Power and Light 1978)

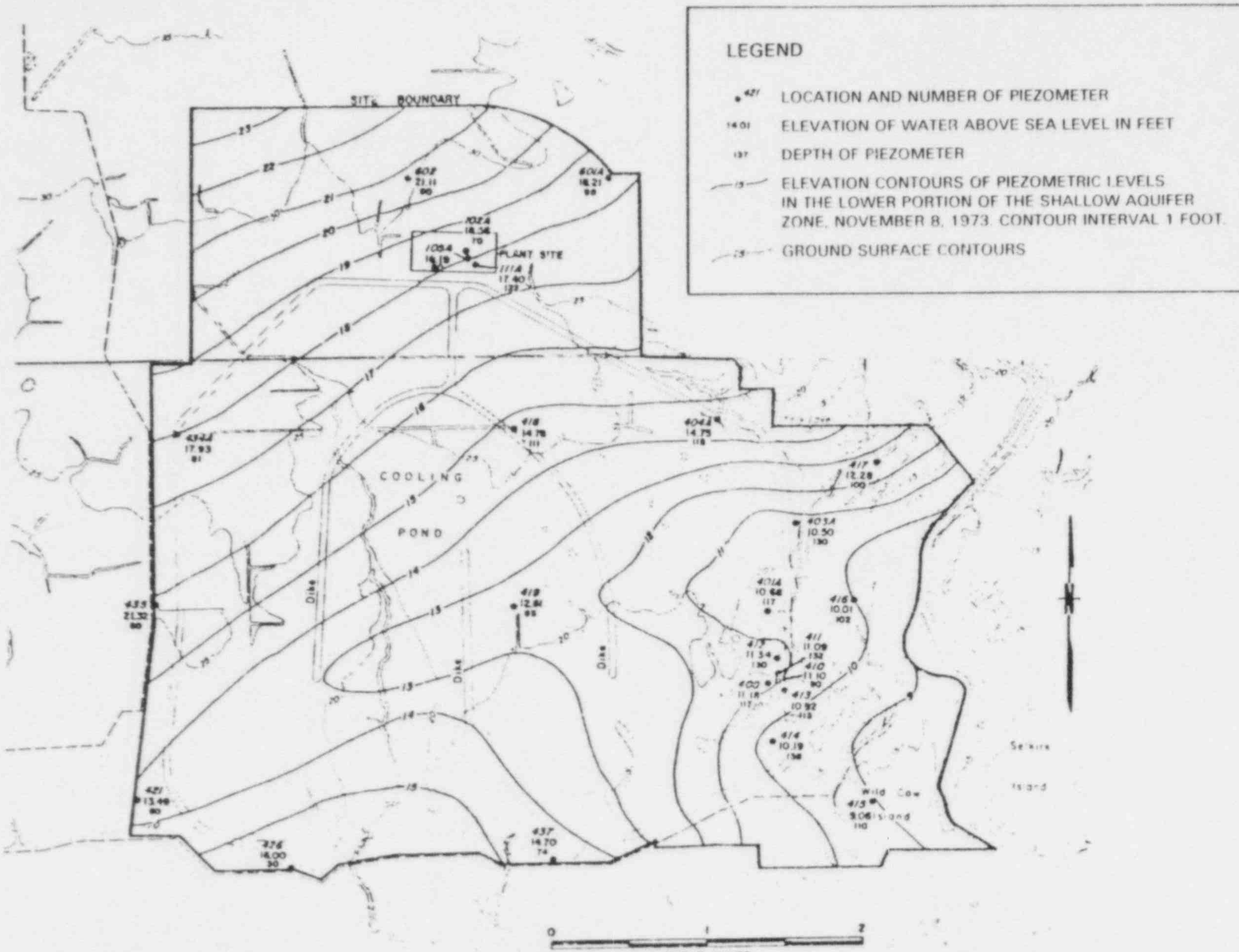


FIGURE 6.3.2-6. Observed Potential Contours for the Lower Unit of the Shallow Aquifer Zone (Source: Houston Power and Light 1978)

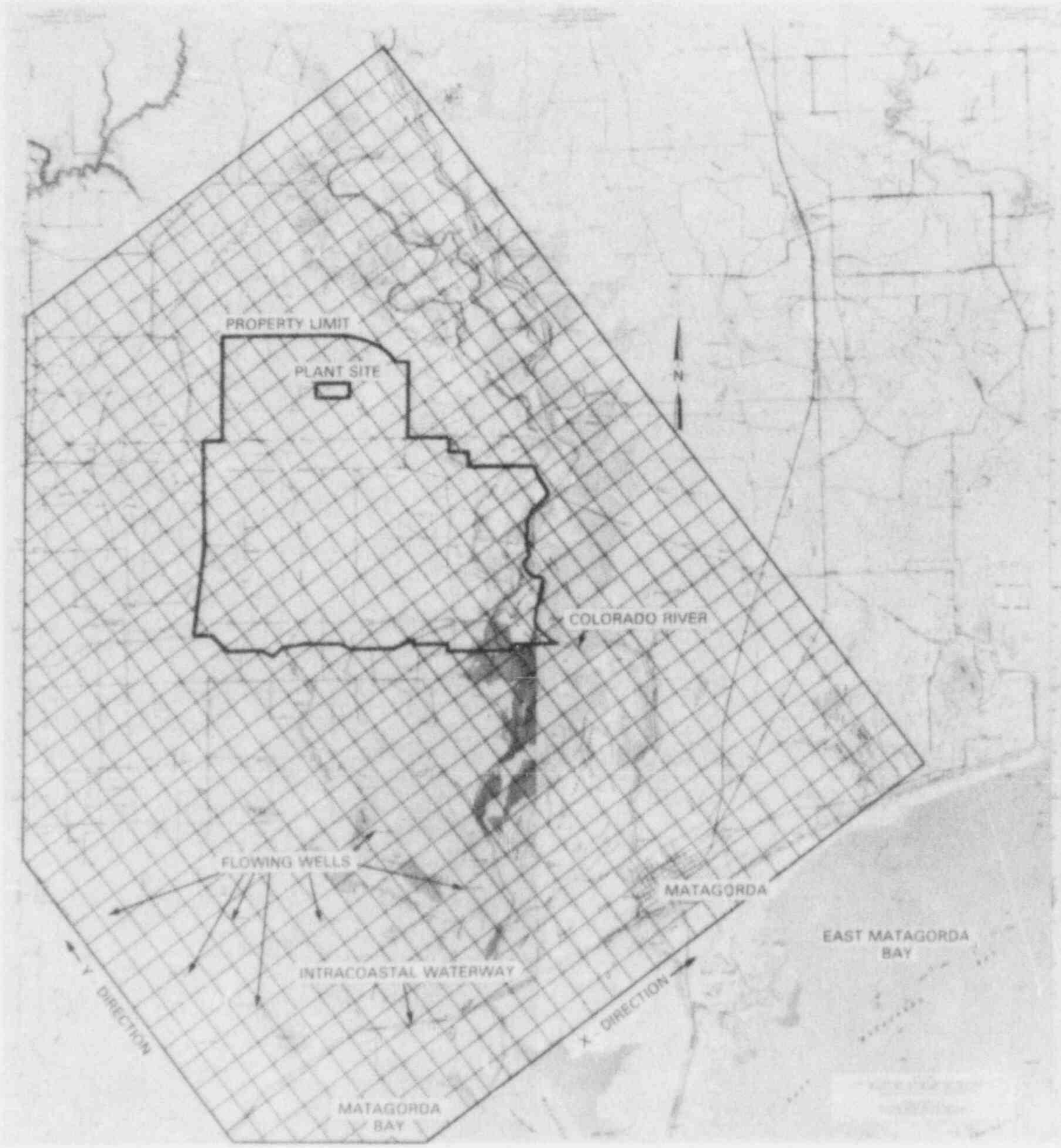


FIGURE 6.2.3-7. Partial Outline of Regional Study Area Superimposed on the STP Topographic Map

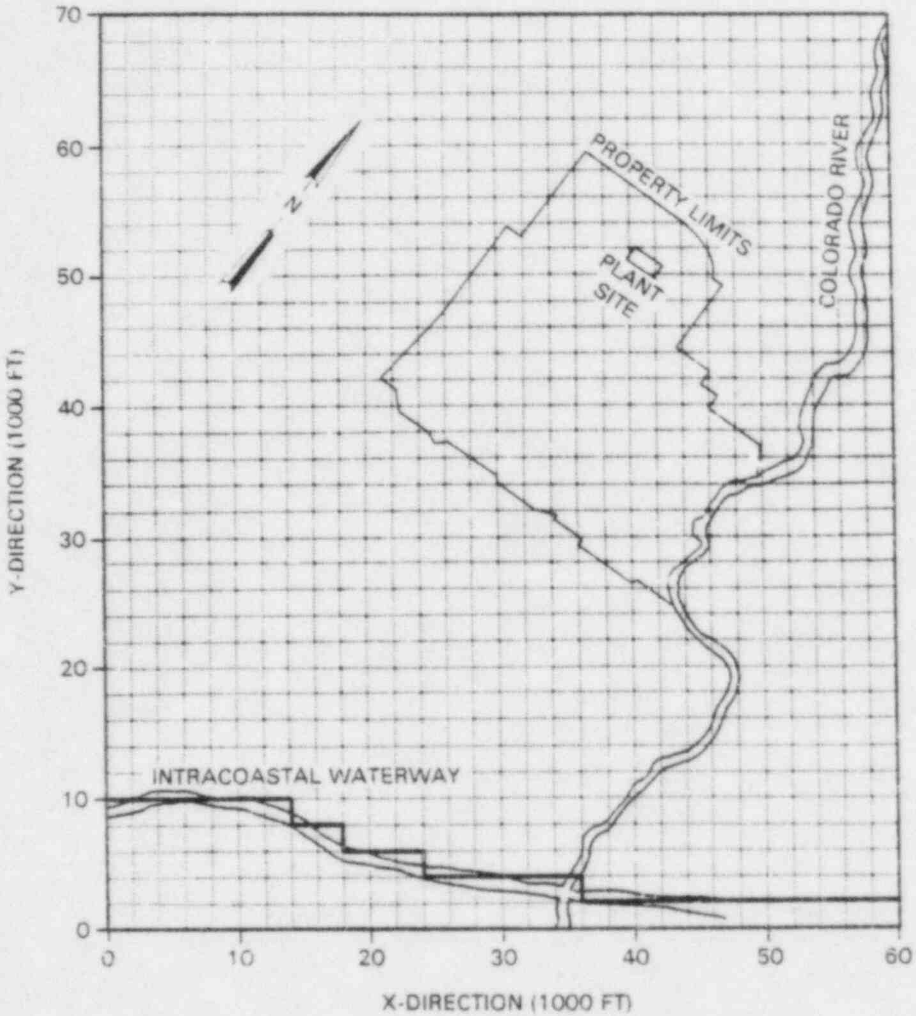


FIGURE 6.3.2-8. Complete Grid for the STP Regional Study Area

For the purposes of constructing the STP conceptual and numerical models it was necessary to determine the type and location of the aquifer boundaries for all four sides of the regional grid. The southern most boundary was assumed to be a head-controlled boundary coinciding with the Intracoastal Waterway. Tide gage records for the Waterway obtained from the USGS[a] indicate the mean tide level in the vicinity of Matagorda, Texas is approximately 1 foot MSL. The actual location of the 1 ft MSL constant head boundary is designated in Figure 6.3.2-8 by the heavy grid lines running along the Intracoastal Waterway.

Several of the observed contours of equal hydraulic head were extended the breadth of the regional grid. The contour extensions were based primarily on

(a) Letter from Robert K. Gabrysch, Chief, Houston Subdistrict, U.S. Geological Survey to Richard Skaggs, PNL.

the shape of the observed contours and additional facts such as the existence of a line of flowing wells at an approximate surface elevation of 6 ft MSL and an apparent ground-water mound in the center of the grid. Using the observed and estimated contours shown in Figure 6.3.2-9, hydraulic heads were estimated for each node in the regional grid using a 16-direction, steepest gradient, liner interpolation procedure. The results of the interpolation are presented in Figure 6.3.2-10, and designated as the "observed potential contours" for the STP regional area. Inspection of the interpolated contours shows an almost

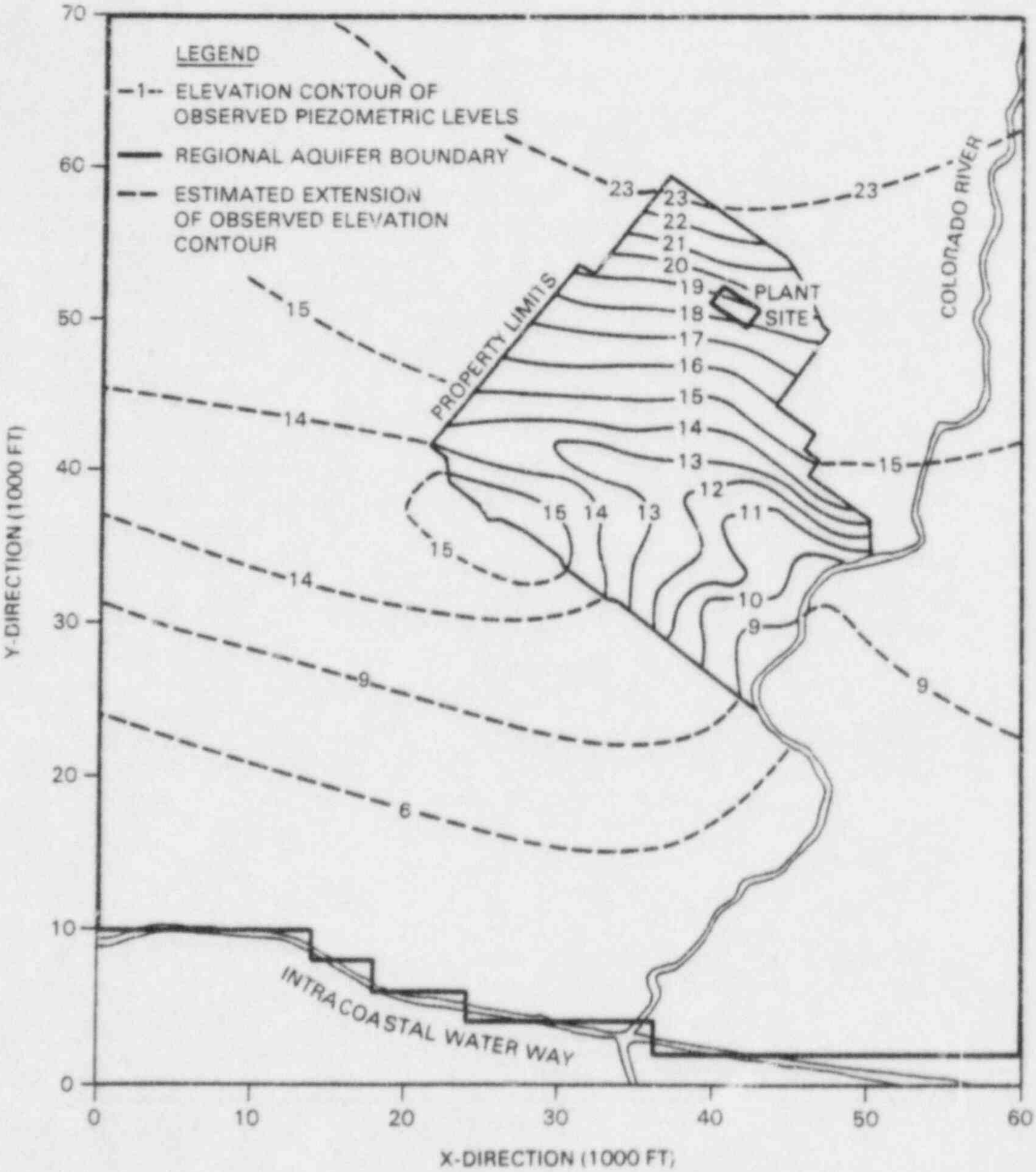


FIGURE 6.3.2-9. Observed and Estimated Potential Contours for STP Regional Study Area

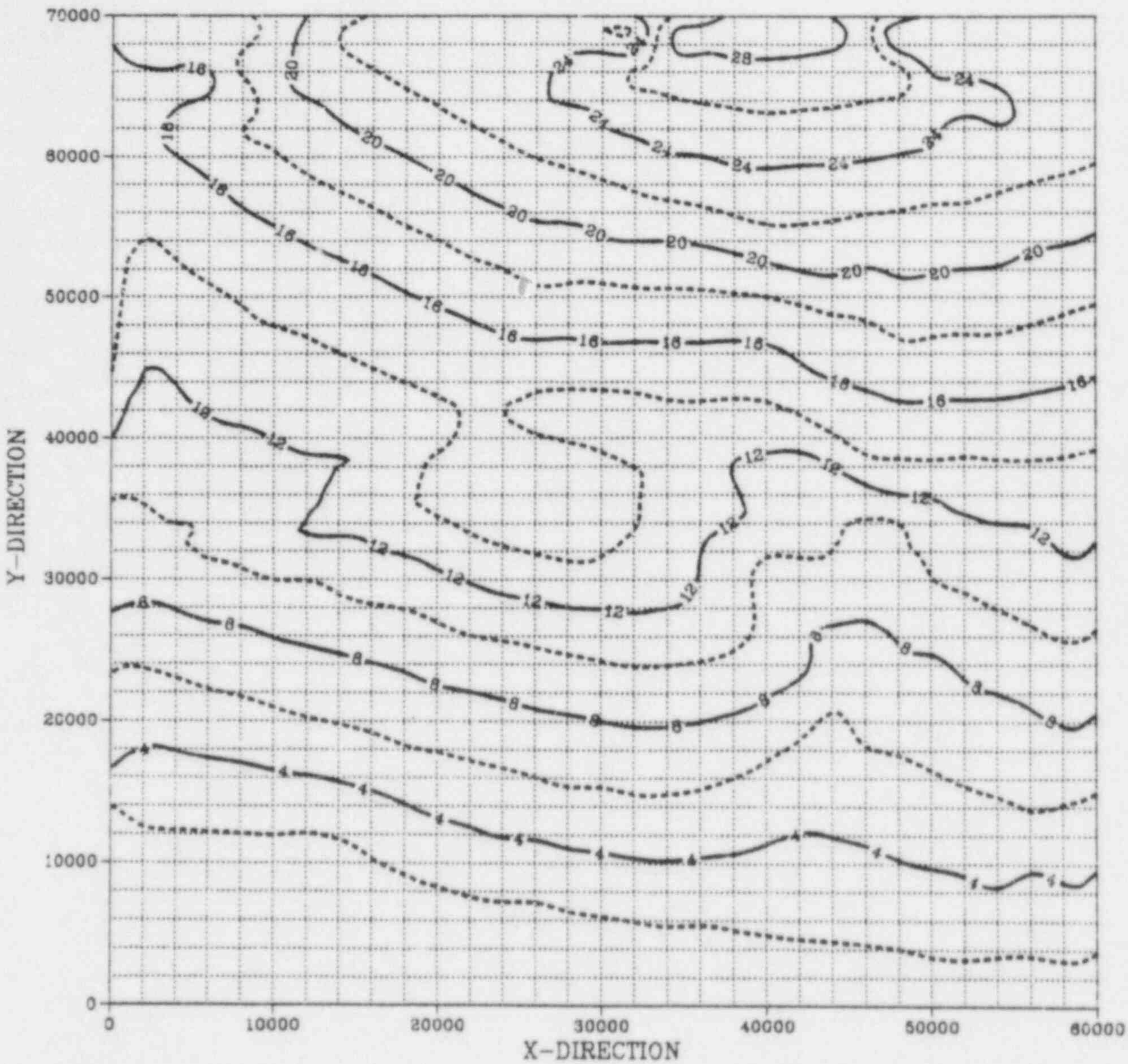


FIGURE 6.3.2-10. Observed Potential Contours for the STP Regional Area

exact match with the observed and estimated contours. However, due to the interpolation scheme used and the relatively large area over which the interpolation was made, minor peculiarities were produced. The 24, 26 and 28 foot contours appear to be closed indicating a ground-water mound that most likely does not exist. Also, the 12, 14, and 16 foot contours near the western edge of the area incorrectly bend upward. In both instances the net effect on the overall flow field, particularly in the vicinity of the STP site, is insignificant.

Inspection of the observed potentials shows that near the east and west grid boundaries the contours are approximately perpendicular to the y-direction. In the model, these boundaries are assumed to be head-controlled boundaries having constant head values equivalent to those shown in Figure 6.3.2-10.

The northern-most boundary is assumed to be a flow-controlled boundary where the head and flow at the boundary will be determined by the model.

6.3.2.5 Aquifer Thickness and Lateral Extent

Typically, the lateral extent and thickness of an aquifer vary considerably from one place to another. Fluvial basin aquifers commonly thin toward the rim of the basin while some basins show structural deformation due to downwarping and faulting. The primary sources of data for delineation of aquifer lateral extent are generally well and bore logs, and existing geologic maps. From these data, the aquifer top and bottom elevations are determined and the aquifer thickness calculated as the difference between the two.

The primary sources of information for the STP site were well and bore logging data presented in the STP FSAR (Houston Power and Light 1978). Useful information was also obtained from Hammond (1969). Over 100 oil or gas well electric logs, water well drilling logs, and soil borings were identified in the STP FSAR (Houston Power and Light 1978). Though the actual data from the logs and boring were not available for this study, three geohydrologic cross-sections interpreting the data were presented in the STP FSAR (Houston Power and Light 1978). Two cross-sections are shown in Figure 6.3.2-11 while the third is presented in Figure 6.3.2-3 above. The approximate locations of the three cross-sections relative to the STP site are indicated in Figure 6.3.2-12. It is clear from the figure that only a limited portion of the cross-sections extend outside the STP property limits. Therefore, for modeling purposes, it was necessary to infer from the cross-sections the top and bottom elevations of the lower unit for the study area.

The basic approach to accomplishing this is to compute values for the aquifer top and bottom at the node points of the regional grid using a fifth-degree polynomial interpolating function developed by International Mathematical and Statistical Libraries, Inc. (IMSL), (1980). Contour and surface maps showing the interpolated results for the top and bottom elevations are presented in Figures 6.3.2-13 and 6.3.2-14, respectively. The (0,0) coordinate for both figures corresponds to the lower left hand corner of the study grid.

To summarize data presented thus far, the conceptual model for the STP case study consists of the lower unit of the shallow zone aquifer. The lower unit is a confined or semi-confined aquifer that extends continuously over the study area. The aquifer interfaces at its bottom with the very low permeability deep confining zone and at its top with a zero or low permeability layer segregating the lower unit from the upper unit of the shallow aquifer.

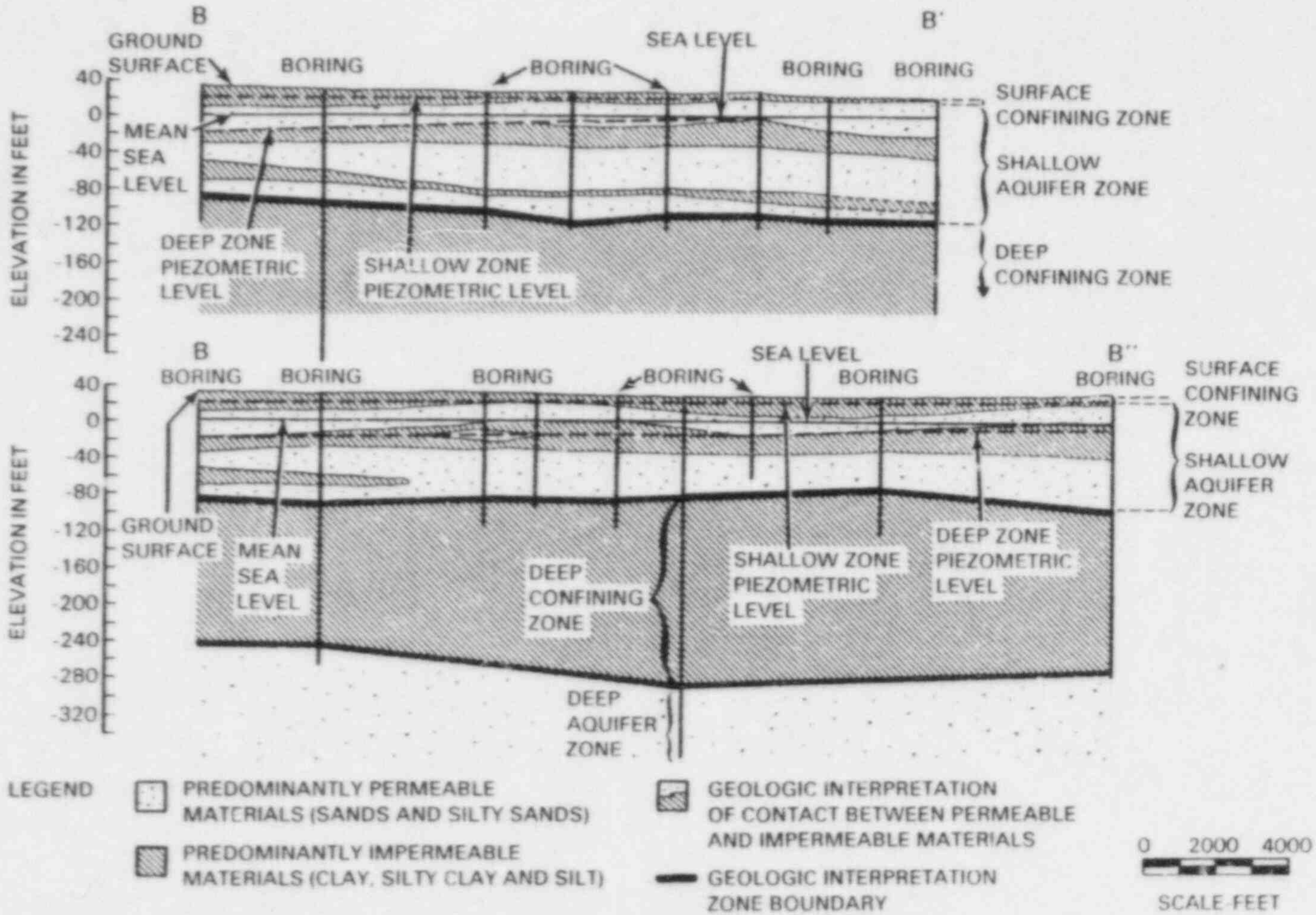


FIGURE 6.3.2-11. Geohydrologic Cross-Sections B-B' and B-B" for the STP Site (Source: Houston Power and Light 1978)

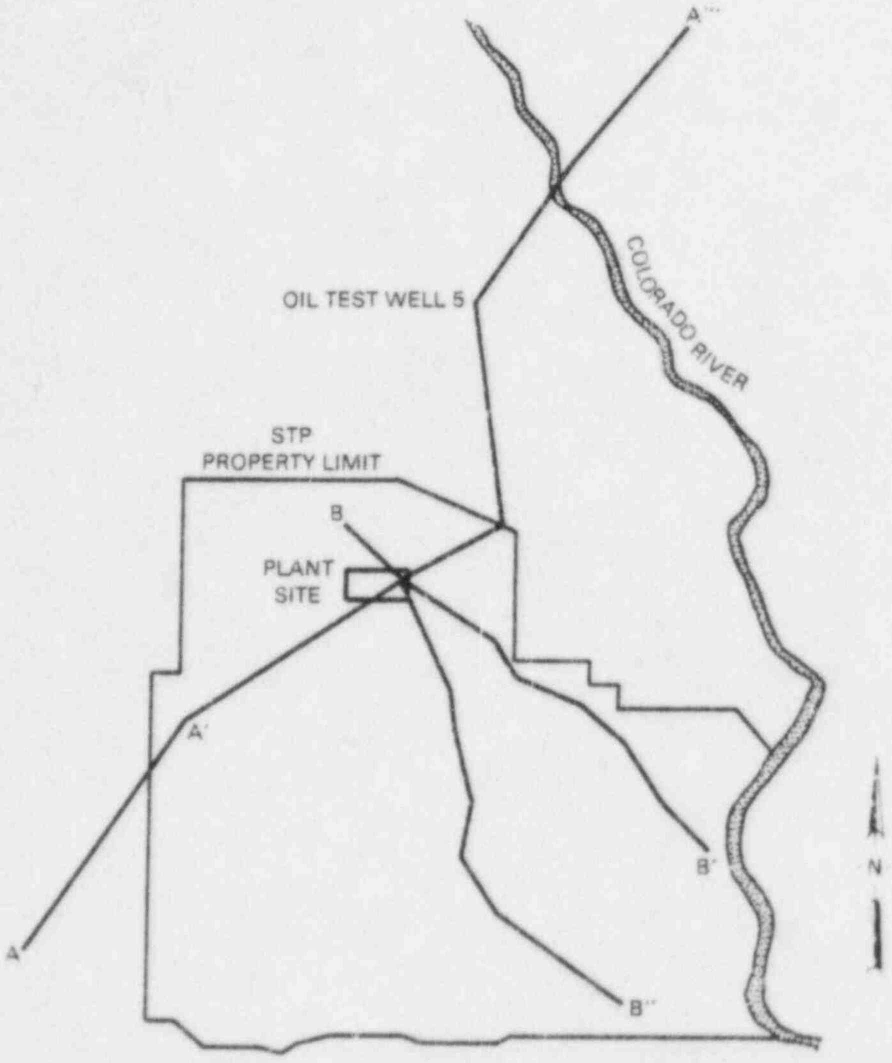


FIGURE 6.3.2-12. Locations of the Geohydrologic Cross-Section A-A''', B-B' and B-B'' (Source: Houston Power and Light 1978)

The bottom of the aquifer varies from approximately -60 feet mean sea level (MSL) to -120 feet MSL. Similarly, the top varies from about -30 feet MSL to -60 feet. Both the aquifer top and bottom dip from the northwest to the southeast which is consistent with the slope of the underlying Beaumont Formation as described by Hammond (1969). The thickness of the lower unit varies from about 28 feet to 62 feet. An overlay of the lower unit aquifer top and bottom illustrating the spatial distribution of the thickness is presented in Figure 6.3.2-15.

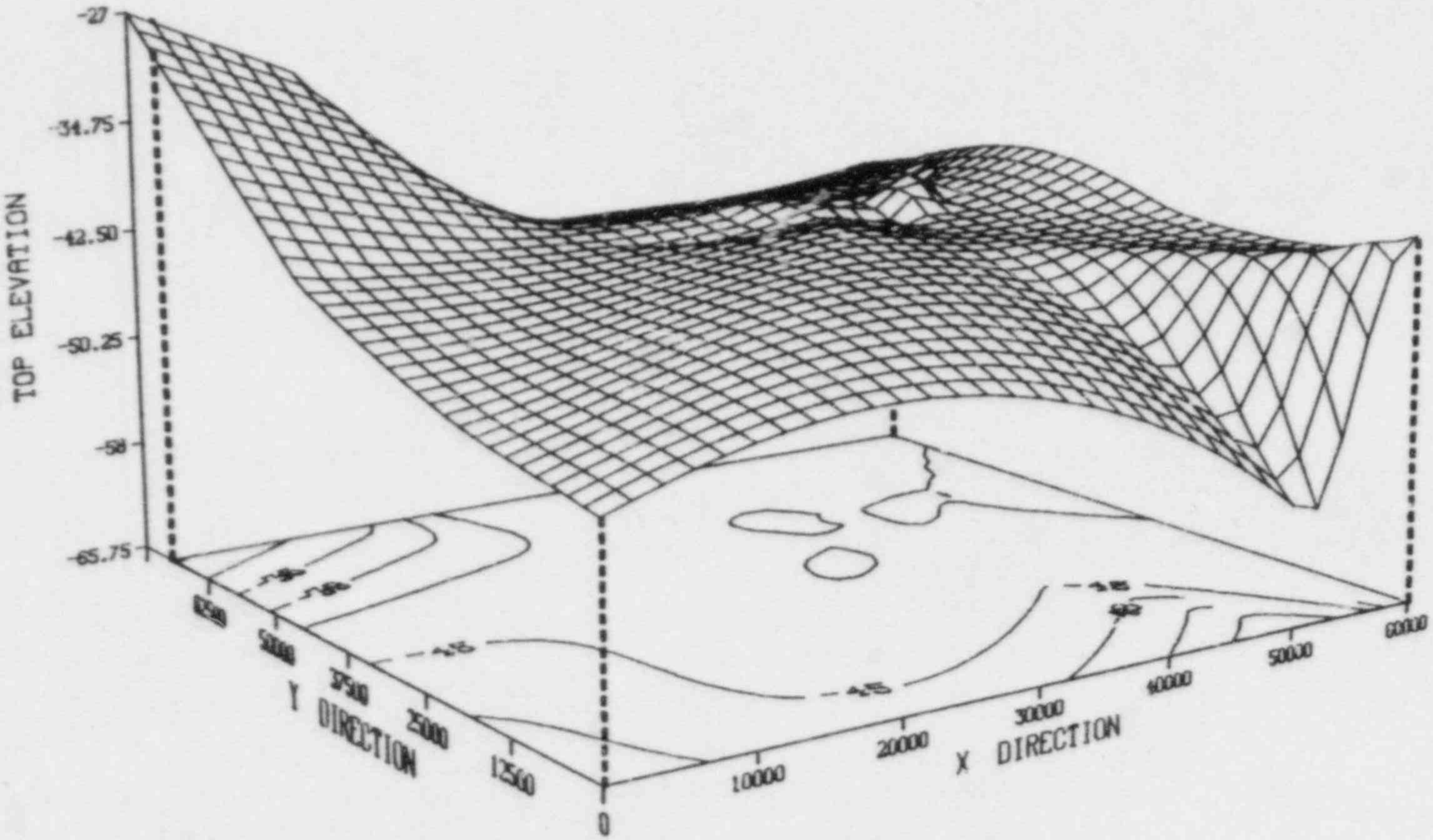


FIGURE 6.3.2-13. Top Elevation Contour and Surface Maps for the Lower Unit of the Shallow Aquifer Within the STP Regional Study Area

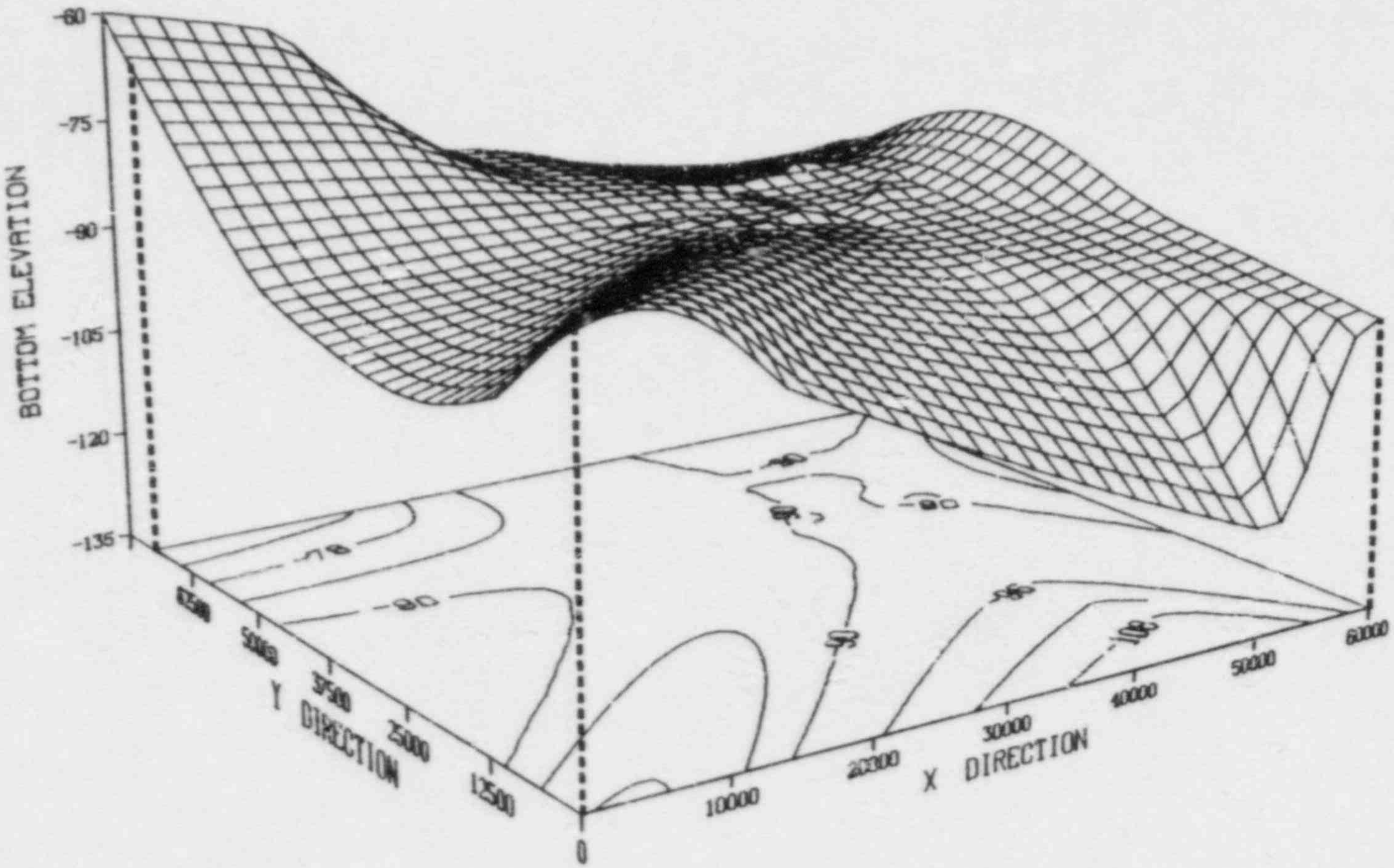


FIGURE 6.3.2-14. Bottom Elevation Contour and Surface Maps for the Upper Unit of the Shallow Aquifer Within the STP Regional Study Area

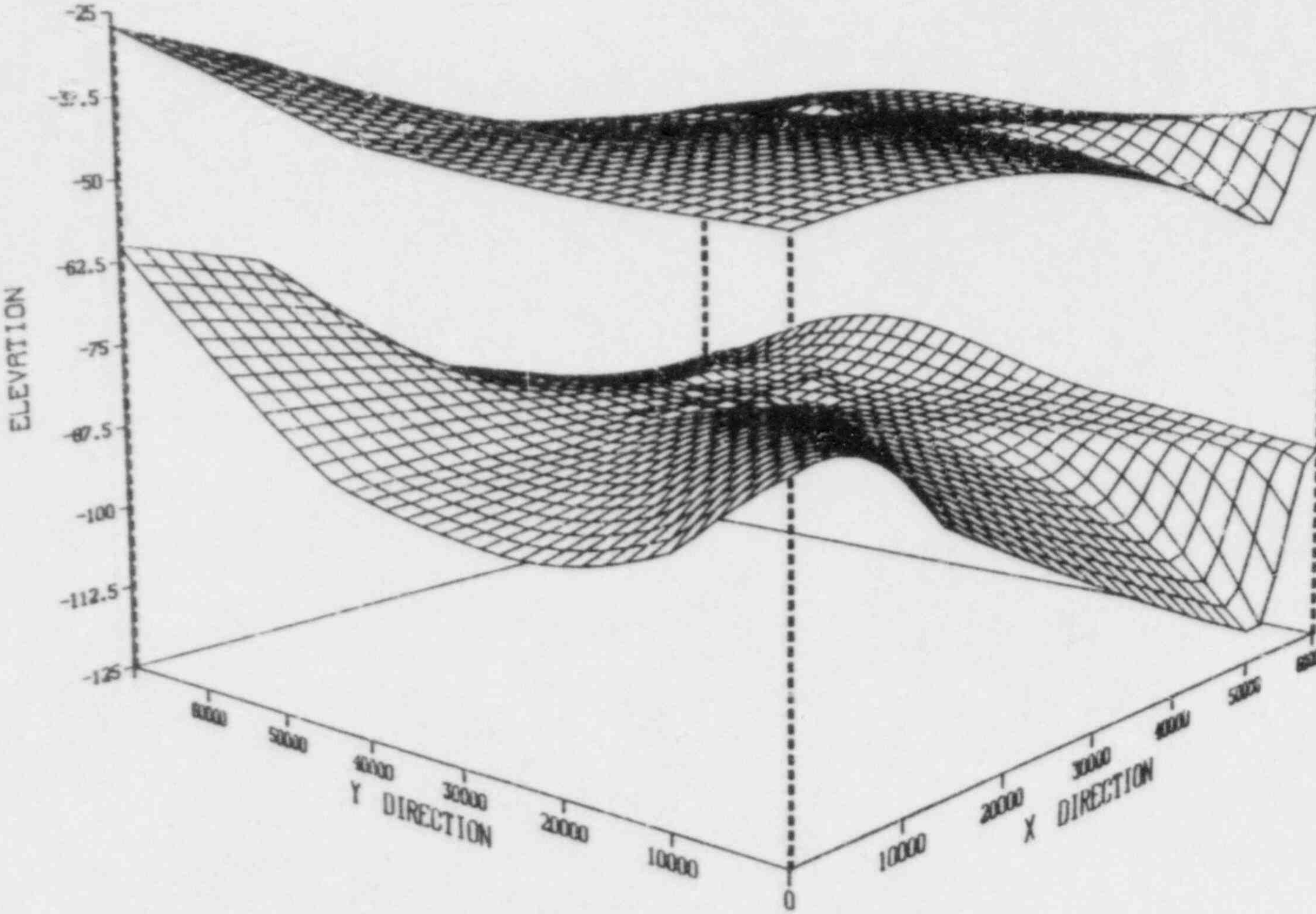


FIGURE 6.3.2-15. Overlay of the Top and Bottom Elevations for the Lower Unit of the Shallow Aquifer Within the STP Regional Study Area

6.3.2.6 Porous Media Hydraulic Properties

The magnitude and spatial distribution of a number of material properties must be specified for the aquifer under study. Table 6.3.2-3 presents some of the required properties for each of the aquifer types (Boonstra and de Ridder 1981). A variety of field, laboratory and numerical methods have been developed to determine the different hydraulic properties. For discussions of the various methods, the reader is referred to Bentall (1963), Krusemand and de Ridder (1970), Office of Water Data Coordination (1977) and Boonstra and de Ridder (1981).

The data and information concerning the hydraulic properties for the lower unit of the shallow aquifer zone were compiled primarily from the results of four pumping tests conducted in the shallow zone within the STP property limits (Houston Power and Light 1978). The depths of the tests and the test results are summarized in Table 6.3.2-4.

Based on the test depths, pump test 3 most likely measures the conditions in the upper unit while the remaining test depths coincide with the lower unit. Therefore, hydraulic conductivities in the lower unit are in the range of 400 to 600 gpd/ft^2 while storage coefficients vary from 0.00045 to 0.0007.

TABLE 6.3.2-3. Required Porous Media Hydraulic Properties

Property	Aquifer Type		
	Confined	Unconfined	Semi-Confined
Hydraulic Conductivity, K	X	X	X
Hydraulic Conductivity for Overlying Confining Layer, K'			X
Storage Coefficient, (for transient simulation only), S	X		X
Specific Yield, μ		X	
Porosity, n	X	X	X
Effective Porosity, n_e	X	X	X

X designates required property.

TABLE 6.3.2-4. Aquifer Test Summary (Source: Houston Power and Light 1978)

Pump Test Number	Test Depth (ft)	Transmissivity (gpd/ft)	Hydraulic Conductivity (gpd/ft^2)	Storage Coefficient
1	60-140	33,000	410	0.00071
2	59-83	13,000	600	0.00045
3	20-43	1,100	65	0.0017
4	30-45	10,500	420	0.0007

Additional information regarding hydraulic properties related to analyses of accidental radionuclide releases in the lower aquifer unit is presented in the STP FSAR (Houston Power and Light 1978). For the analysis discussed, the following properties were assumed:

- hydraulic conductivity 635 gpd/ft^2
- porosity: 37%.

Complementing the information obtained from the STP FSAR, Hammond (1969) describes the hydraulic characteristic of the Gulf Coast aquifer. Though Hammond's discussion emphasizes the heavily pumped deep aquifer zone, the information provides a framework for evaluating the FSAR pumping tests data. Pumping tests in Matagorda County and surrounding areas provide values of hydraulic conductivity for the sands of the Gulf Coast aquifer ranging from 103 to 3,950 gpd/ft^2 and averaging about 570 gpd/ft^2. Generally, the deeper sands, because of increased compaction and cementation, have lower hydraulic conductivities. Table 6.3.2-5 presents the results of the pump tests for the two wells in Matagorda County that are partially screened in the lower unit aquifer. The transmissivities for both wells are relatively high compared to 70,000 gpd/ft average value for the deeper wells.

The hydraulic conductivity value in Table 6.3.2-5 is also much higher than the pump test results presented in the FSAR (Table 6.3.2-4). This is explained in part by Hammond's observation that lower permeabilities are generally found near the coast due to finer grain size sediments. The sands away from the coast are part of the Colorado River alluvial deposits and tend to have coarser size distributions.

The storage coefficients of the STP pump tests 1, 2, and 3 were in the neighborhood of 5×10^{-4} to 7×10^{-4} (Houston Power and Light 1978). The single value for storage coefficient shown in Table 6.3.2-5 is similar in magnitude, having a value of 1.1×10^{-3}. These values are typical for a confined aquifer and confirm the conceptual model based on bore logs.

The values for porosity provided by Hammond (1969) are representative ranges for sedimentary material. These are presented below in Table 6.3.2-6. On the basis of the values shown, the value for porosity of 37% assumed in the

TABLE 6.3.2-5. Pump Test Results for Wells in Matagorda County (Source: Hammond 1969)

Well Number	Date	Screened Internal (ft MSL)	Hydraulic Conductivity (gpd/ft^2)	Transmissivity (gpd/ft)	Storage Coefficient
TA-65-58-107	10-04-66	75-202	---	176,000	1.1×10^{-3}
TA-65-58-803	05-01-66	91-215	3,950	399,000	---

TABLE 6.3.2-6. Representative Porosities for Sedimentary Material (Source: Hammond 1969)

Material	Porosity, %
Soils	50-60
Clay	45-55
Silt	40-50
Medium to coarse mixed sand	35-40
Uniform sand	30-40
Fine to medium mixed sand	30-35
Gravel	30-40
Gravel and sand	20-35
Sandstone	10-20
Shale	1-10

FSAR (Houston Power and Light 1978) is within the range for medium to coarse mixed sand (35-40%) and uniform sand (30-40%). The assumed value is slightly outside the 30% to 35% range for the fine to medium mixed sand.

6.3.2.7 Hydraulic Head (Ground-Water Potential)

Generally, a ground-water basin experience a number simultaneously occurring of hydrologic stresses (Boonstra and de Ridder 1981). These stresses include: infiltration of rainfall and/or irrigation water, streambed percolation, evapotranspiration, ground-water discharge by streams or springs and well pumpage. At any given time, the combined effects of the stresses are reflected in the configuration and fluctuation of the basin hydraulic heads. Consequently, the collection and evaluation of watertable data are important parts of ground-water model development (i.e., calibration by history matching). Simulated hydraulic heads are compared with measured values to ensure the ground-water model is representing the various stresses that are being exerted on the basin. The data requirements for quantifying the hydrologic stress and the results obtained for the STP site are discussed below.

The magnitude and distribution of the hydraulic head within a basin and at the boundaries are determined by observation wells and/or piezometers. For confined and unconfined aquifers, only observation wells screened in the aquifer of study are required. In a semi-confined aquifer, however, hydraulic head measurements are required in the study aquifer as well as in the aquifer overlying the permeable confining layer. Though not used directly, the heads for the overlying aquifer are required to compute recharge/discharge through the confining layer. Under normal conditions, the observation wells should be measured periodically (dependent on local conditions) for one to two years to establish temporal hydraulic head level fluctuation trends.

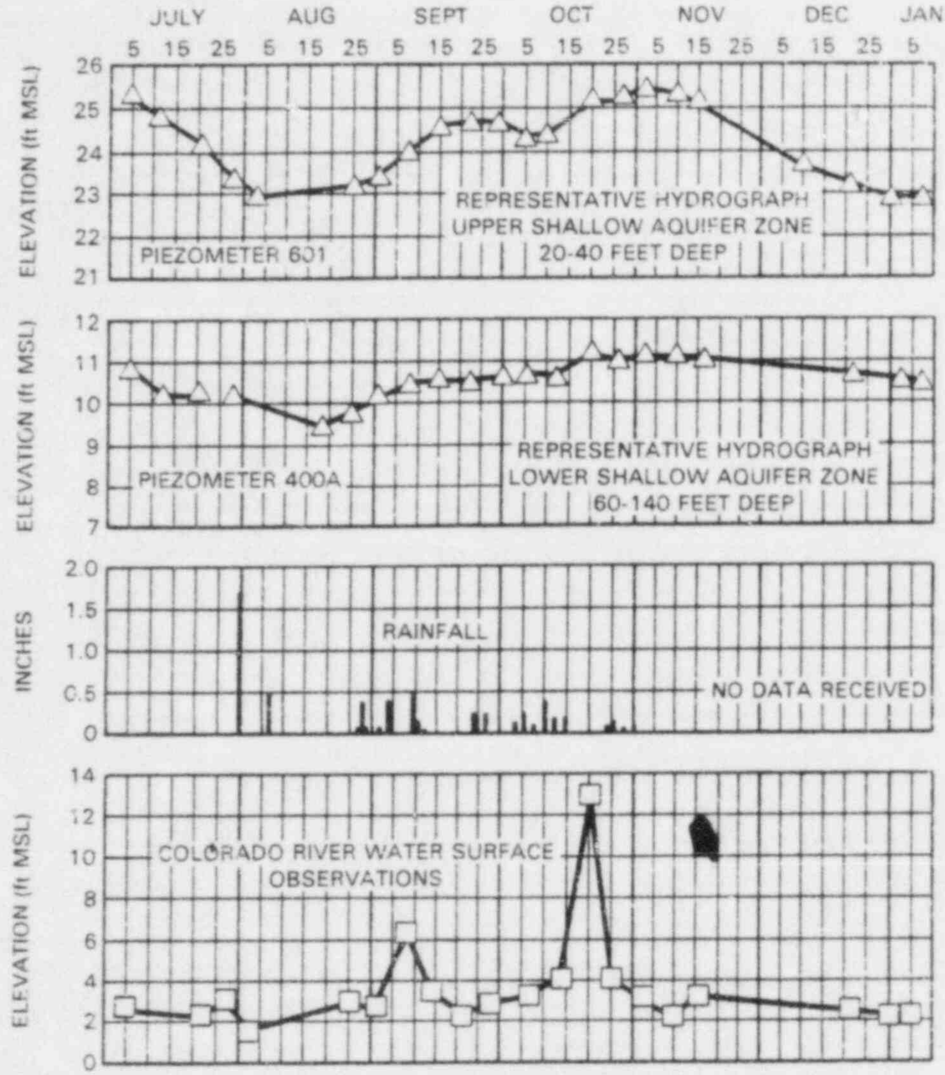
The results of the water level measurements are best presented in the form of maps of equal contours of hydraulic hgeads, hydrographs, and, if applicable, head-difference maps. The necessary map(s) must be generated for the beginning of the study period to establish the initial conditions for the basin. If sufficient change in the head contours occurs within the basin over time due to recharge, discharge, pumping, etc., then maps are also needed for each successive time period chosen for modeling. If steady-state conditions are assumed for the basin, the model is calibrated only for the specified initial conditions. For a semi-confined aquifer, two hydraulic head contour maps must be drawn; one for the study aquifer and one for the aquifer above the confining layer.

At the STP site, hydraulic head data are limited to those available in the STP FSAR (Houston Power and Light 1978). Contours of the observed data for the lower and upper units of the shallow aquifer zone are presented in Figures 6.3.2-5 and 6.3.2-6. As discussed in Section 6.3.2.4, the contours were extrapolated over the study region based upon the available information. The results of this process for the lower unit are presented in Figure 6.3.2-10. The same procedure is followed to extrapolate data for the upper unit. Since the available data are limited to measurements taken on a single day (i.e., May 14, 1974 from the upper unit and November 8, 1973 for the lower unit) the following assumptions are made:

1. post-construction potential levels rebounded to those measured,
2. steady-state conditions prevail in the upper and lower unit aquifers, and
3. the measured data are representative of steady-state conditions.

The primary reason for making these assumptions is that the data required to characterize dynamic conditions within the basin and the efforts of construction were unavailable.

The main source of temporal change within the basin is the Colorado River which under normal conditions can experience stage changes of up to 10 feet within a given year. Figure 6.3.2-16 shows the response of the Colorado River to typical rainfall period in 1973 and the associated response of the lower and upper units of the shallow aquifer zone. It can be seen that the response of the lower unit is attenuated significantly compared to that of the upper unit. With regard to the first assumption, the single greatest potential effect is due to construction of the Cooling Reservoir (see Figure 6.2.2-1).



NOTES

1. RIVER LEVEL MEASUREMENTS MADE AT FM ROAD 521 BRIDGE APPROXIMATELY TWO MILES EAST OF SITE.

2. RAINFALL MEASUREMENTS FROM SOUTH TEXAS PROJECT METEOROLOGICAL TOWER.

FIGURE 6.3.2-16. 1973 Water Level Observations in the Vicinity of the STP Site (Source: Houston Power and Light 1978)

As stated in the STP FSAR (Houston Power and Light 1978), the normal maximum operating level for the Cooling Reservoir is 49 feet MSL which is over 20 feet above the surrounding ground surface. Thus, seepage from the Cooling Reservoir will tend to raise local ground-water potentials. However, the reservoir embankment is designed to use compacted fill, to insure that the piezometric levels of the soil in the plant area remain below the ground surface. Since pre-construction levels in the upper aquifer were 2 to 3 feet below the ground surface, this would indicate the ground-water mounding will be less than 2 to

3 feet. Further, since the seepage from the Cooling Reservoir will discharge directly into the upper unit, the effect in the lower unit will be limited to the increased leakage that will occur due to the higher hydraulic head.

From the above discussion, it is apparent that changes in hydrologic stresses with time and changes due to the STP construction will modify the potentials measured for the upper and lower units of the shallow aquifer. However, based on the evidence presented it appears that the effects of these changes are not severe in the lower unit. Therefore, in light of the stated objectives of the case study, the limitations associated with making the stated assumptions regarding the use of available hydraulic head data are considered acceptable by the authors.

6.3.2.8 Aquifer Recharge/Discharge

A key element of any ground-water study is the estimation of the type and magnitude of aquifer recharge/discharge. For the case of unconfined aquifers Freeze and Cherry (1979) define ground-water recharge as the entry of water into the saturated zone at the water table surface accompanied by flow away from the watertable within the saturated zone. Unconfined aquifers having a deep watertable can be recharged by rainfall percolation, streambed percolation and percolation of irrigation water. In confined aquifers, recharge in the strict sence occurs along the rim of the basin where the aquifer intersects the land surface. In a manner similar to that for unconfined aquifers, the sources of recharge at the confined aquifer outcrop are rainfall percolation, streambed percolation and surface runoff from adjacent areas. Another important source of recharge for semi-confined aquifers may be water flow through over- or under-lying confining layers.

Freeze and Cherry (1979) similarly define discharge as the removal of water from the saturated zone across the watertable surface accompanied by flow within the aquifer toward the watertable surface. In both confined and unconfined aquifers, discharge can occur as spring flow, seepage into streams, evapotranspiration and through underlying or overlying confining layers.

The delineation of recharge and discharge areas may require information on topography, surface and subsurface geology, and climate. If the aquifer is semi-confined and exchange of water between aquifers occurs, data on hydraulic potential surfaces is necessary. Several methods can be used to estimate aquifer recharge and discharge. Examples include computations using Darcy's equation or analysis of measured stream flow hydrographs. These and others are described by Boonstra and de Ridder (1981).

According to the STP FSAR (Houston Power and Light 1978), recharge of the shallow-zone aquifer probably occurs within a few miles north of the site. The available data indicate that the upper confining layer prevents any appreciable recharge within or to the south of the site. Consequently, analysis of recharge for the lower unit of the shallow aquifer is limited to seepage from the upper unit. Discharge from the lower unit, as discussed previously regarding boundary conditions, is assumed to occur at the Intracoastal Waterway and

by vertical seepage, particularly at the Colorado River. The recharge and discharge of the lower unit were calculated by applying a form of Darcy's equation (Freeze and Cherry 1979):

$$Q = -K' \frac{(h - h')}{m'} \Delta x \, \Delta y$$

where

Q = recharge/discharge rate (gpd)
K' = vertical hydraulic conductivity in confining bed (gpd/ft)
h = hydraulic head in aquifer (ft MSL)
h' = hydraulic head in covering layer (ft MSL)
m' = thickness of confining bed (ft)
Δx = x-dimension of regional grid elements (ft)
Δy = y-dimension of regional grid elements (ft).

A negative value for $(h - h')$ indicates flow of water into the lower unit. Conversely, a positive value signifies water is being discharged by the lower unit as vertical upward seepage.

The initial evaluations of recharge and discharge were computed at each node of the regional grid based upon an assumed uniform vertical hydraulic conductivity, the previously determined potentials for the upper and lower units of the shallow aquifer, values of the confining layer thickness interpolated from the available geohydrologic cross-sections, and the values of Δx and Δy determined directly from the regional study grid. The vertical hydraulic conductivity was initially set at 0.005 gpd/ft^2 based on the range of 0.01 gpd/ft^2 to 0.00001 gpd/ft^2 provided by Freeze and Cherry (1979) for unconsolidated, unweathered marine clay. The result of the initial evaluation was a net recharge to the aquifer of 1.6×10^6 gpd or 3.7×10^{-4} gpd/ft^2 of aquifer.

According to Hammond (1969), in some parts of Matagorda County, available data indicate that sands in the deep zone aquifer are replenished by water from overlying shallower sands through the bore holes of idle water wells. Obviously, this is not directly accounted for in the recharge/discharge analyses. If this represents a significant form of water loss which has affected observed potentials, it will ultimately be reflected in the model through the calibration procedure by reducing the recharge from the overlying unit to achieve the necessary water balance. Another possibility not accounted for is downward vertical seepage from the lower unit shallow aquifer to the deep zone aquifer. Though there is significant head differential between the two aquifers, as much as 30 feet, it is believed that the 100 to 200 foot thick deep confining zone would limit the seepage to insignificant levels.

Hammond (1969) reports that most of the irrigation, public supply and industrial wells in Matagorda County produce water from the 200 to 700 foot depth interval. Due to the high salinity of the shallow zone aquifer, only minor pumpage for some domestic and livestock wells produce from shallower depths. A summary of estimated 1973 ground-water usage within a 10-mile radius of the STP site is shown in Table 6.3.2.7 (Houston Power and Light 1978). The total 130 acre-ft/yr withdrawal from the shallow-zone aquifer, equivalent to

TABLE 6.3.2.7. Summary of Estimated 1973 Ground-Water Use in the Vicinity of the STP (Source: Houston Power and Light 1978)

Ground-Water Use	Total Wells in Operation		Estimated Pumpage (acre-ft/yr)	
	Shallow Aquifer Zone	Deep Aquifer Zone	Shallow Aquifer Zone	Deep Aquifer Zone
Irrigation	-	17	-	1,750
Industrial	-	1	-	160
Public and Municipal	-	7	-	100
Stock and Domestic	49	69	100	140
Drilling Supply	1	1	30	30
TOTAL	50	95	130	2,180

1.2×10^5 gal/day, is less than 7% of the computed net recharge rate. The pumpage was not directly accounted for in developing the ground-water model of the lower unit of the shallow aquifers.

6.3.2.9 Conceptual Model

The preceding sections describe a majority of the data obtained and analyzed for the STP case study. In the sections following, the numerical model development, calibration and simulations are presented. The bridge linking the two efforts is the conceptual model of the ground-water flow system that determines transport of radionuclide releases due to a postulated severe accident at the STP site.

A conceptual model, defined by Simmons and Cole (1985) is the modeler's perception of the physical behavior of a ground-water system. Conceptual model development is simply the process by which a preliminary description and understanding of a ground-water system is obtained based on available data, experience and fundamental hydrologic principles. The conceptual model thus becomes, a simplified composite picture of what is known about the study area, hydrogeologic boundaries and boundary conditions, geologic and stratigraphic layers, ground-water flow directions and quantities, flow barriers, recharge/discharge areas, time dependencies of the flow system, etc.

A general rule suggested by Simmons and Cole (1985) for developing a conceptual model is: "a model should be made as simple as possible and only the detail necessary to explain the available data and observed phenomena relevant to the study objectives should be included." Further, as depicted in Figure 6.3.1-1, development of the conceptual model is a continuing process that

occurs parallel to the numerical model development. As new data and new understanding about the system are gained, the conceptual model is updated as required throughout the study.

The objective of the STP case study requires the development a ground-water model to simulate the transport of radionuclides released by a severe accident. Given the postulated accident scenario, it is assumed the molten reactor core mass would ultimately rest approximately 35 ft below the STP containment building basemat within the lower unit of the shallow-zone aquifer. On the basis of this assumption, development of the conceptual model focused on the lower unit. The key features of the conceptual model are listed below and illustrated in Figure 6.3.2-17:

- The lower unit of the shallow-zone aquifer is a semi-confined aquifer situated between the deep confining zone and a semi-permeable confining layer that divides the shallow aquifer.
- The general flow direction within the lower unit of the shallow aquifer is the southwest toward Matagorda Bay, its primary discharge point.
- From available lower unit aquifer tests the average hydraulic conductivity is assumed to be 600 gpd/ft^2. The vertical hydraulic conductivity through the upper confining layer is assumed to be 0.005 gpd/ft^2.
- The lower unit aquifer top ranges in elevation from approximately -30 ft MSL to -60 ft MSL, the bottom varies from -60 ft MSL to -120 feet MSL, and its from 30 to 60 ft thick.
- Over most of the study region, the lower unit aquifer receives recharge as vertical seepage from the upper unit aquifer. The exception to this, where discharge occurs, is just west of the river all the way to the east study boundary. Within the study region, the net recharge to the lower unit is estimated to be 1.6 x 10^6 gpd.

6.4 REGIONAL MODEL DEVELOPMENT

A two-stage modeling approach is used to characterize the ground-water system at the STP site. The first stage consists of developing a coarse grid regional hydrologic flow model, whereas the second stage involves developing flow and contaminant transport models for the immediate vicinity of the STP site.

The purpose of the regional model is to establish boundary conditions for the local model. The local model then simulates the ground-water system in the immediate area of the plant in greater detail. The usual procedure for determining boundary conditions is to extend the regional model from the area of interest to where the conditions are known (i.e., constant head along a river, no flow along a ground-water divide, etc.). The problem, however, is that if one goes too far from the area of interest, the resolution around the area of

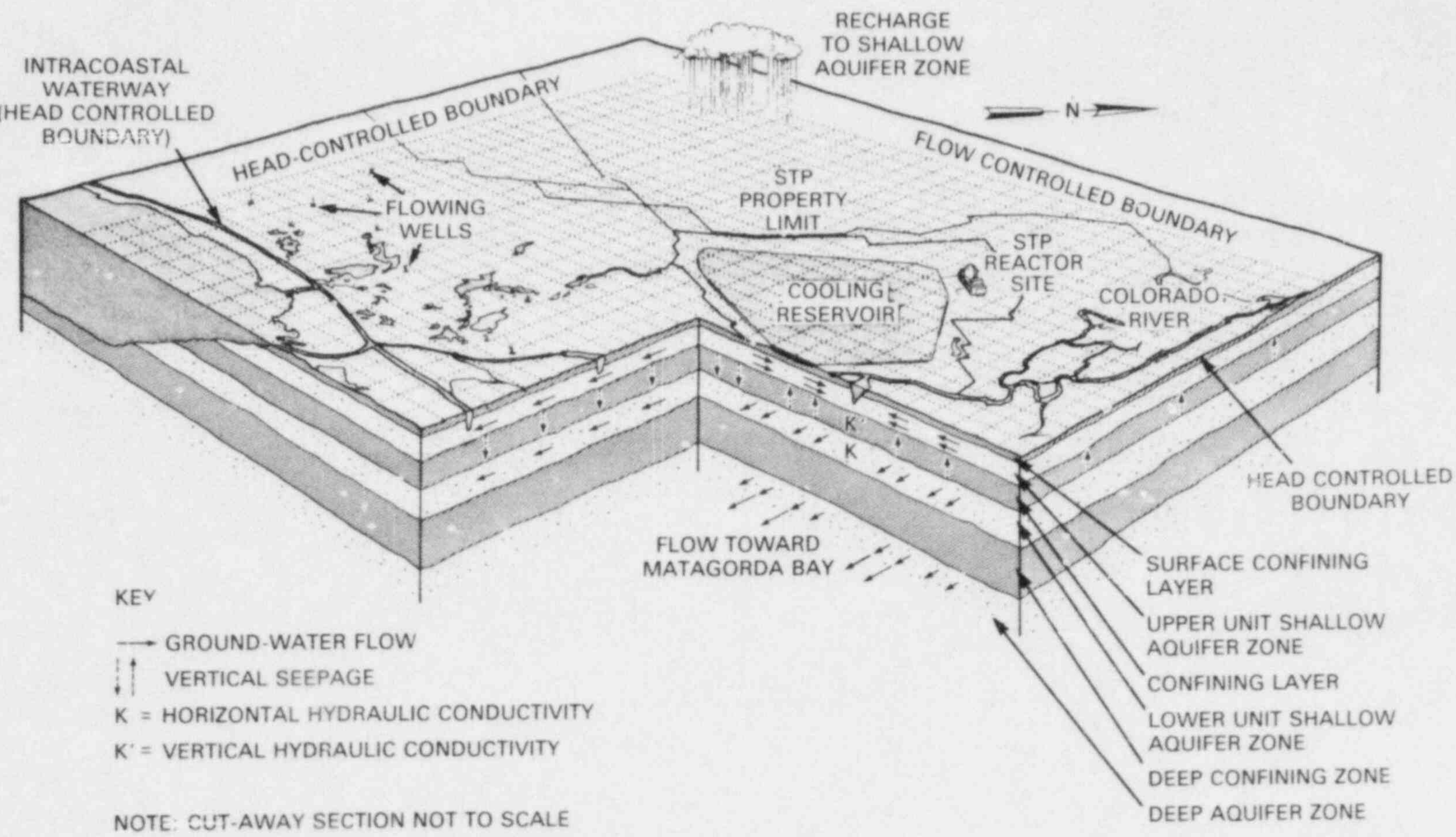


FIGURE 6.3.2-17. Illustration of the Conceptual Model for the STP Site

interest is reduced because of numerical model size restrictions (i.e., the number of nodes). This problem can be alleviated by using a two-stage modeling approach. The regional model should be coarse enough to enable its boundaries extend out to either where the hydrologic conditions were "known", and/or to where the boundary is far enough away that it would have little influence on the area of immediate interest (i.e., local model area).

Again, referring to Figure 6.3.1-1, and subsequent to data collection and initial conceptual model development, construction of a ground-water model involves three additional steps:

1. selection of an appropriate computer code,

2. preparation of data for model using determined parameters (Initial Model Development), and

3. compare results with observed data (Model Calibration).

6.4.1 Code Selection

A computer code is the numerical implementation of a set of equations that describe, in simplified form, the important physical processes acting within a ground-water system (Simmons and Cole 1985). Quite often and incorrectly, the computer code is said to be the model and model development is thought to be merely the selection of an appropriate code. However, the two processes are quite distinct. There are a number of different codes that may solve the same equations; but selecting an appropriate code for the analysis of a site specific ground-water flow and transport problem requires analyses of several factors. It is unlikely that there is a "best" code for all study purposes and objectives. For example, a code that is appropriate for flow in a porous medium may be totally inappropriate for similar analyses in a fractured environment. Codes should be selected on the basis of demonstrated numerical accuracy and faithful description of the dominant physical processes. Further, the selected code should have the necessary regional simulation capabilities (e.g., representation of spatial variations in hydraulic conductivity).

Preliminary code selection for the STP case study is based on a study of available ground-water flow and transport code capability summaries (Kincaid et al. 1983). The specific criteria used in the final code selection include:

Is the theoretical basis of the code technically sound, is it well documented, and has it been previously implemented and verified?

Does the code simulate the dominant hydrologic and transport processes identified in the preliminary STP conceptual model?

Does the code provide for varying structure of the hydrologic flow and transport system?

Will the code accept spatially varying parameters (e.g., hydraulic conductivity, transmissivity, etc.)?

- Can the code accept the appropriate boundary conditions (head-controlled and flow-controlled), interior impermeable boundaries, recharge, and withdrawal/injection?

From the results of the review, the TRANS code was selected for use in the STP case study local and regional modeling. TRANS is an extensively documented, generalized computer code which simulates the effects of convection, dispersion and to a limited extent, chemical reaction. Ground-water flow is computed using a variable grid, finite difference formulation. Solutions are included for nonsteady/steady flow problems in heterogeneous aquifers under confined, unconfined or leaky aquifer conditions. The code also accounts for time-varying withdrawal and injection, ground-water recharge and evapotranspiration. The solute transport portion of the code is based on a Lagrangian particle technique for the convective mechanisms and a random-walk technique for the dispersion. With its many features, TRANS provides the means to evaluate the performance of both active and passive interdiction methods that may be implementable at the STP with reasonable accuracy and efficiency. For a detailed description of the code formulations, the reader is referred to Prickett and Lonnquist (1971), and Prickett, Naymik and Lonnquist (1982). The basic equations for flow and transport in the TRANS code are described in Appendix C.

6.4.2 Initial Regional Model Development

Development of the initial ground-water model is essentially the process of translating the conceptual model into a discretized form consistent with the input format of the selected code. This process can be streamlined considerably if in the data collection and analysis stage of the study, the need for discretization is anticipated. If so, the data analysis can be done node by node and the results converted directly into the appropriate code format. Such was the case in constructing the conceptual model for the lower unit of the shallow zone aquifer at the STP site. All aquifer system parameters that required spatial discretization were discretized using the previously selected regional grid (Figure 6.3.2-8). The primary TRANS input data requirements and the source of the initial data values used are summarized in Table 6.4.2-1.

The results of the initial model steady-state simulation are presented in Figure 6.4.2-1 which shows the simulated potential contours. Comparison of the simulated potentials to the observed potentials, general slopes are similar and they match at the boundaries, (Figure 6.3.2-10), reveals quite a difference, even though general slopes are similar and they match at the boundaries. The observed ground-water mound just to the left of the grid center is missing in the simulated results. The observed low potential extending from the lower right corner of the grid toward the mound is also not simulated. In general, the simulated potentials on the left half of the grid are too high.

6.4.3 Regional Model Calibration

Before the initial model of the study area can be used to predict hydraulic potentials or contaminant concentrations resulting from implementation of various mitigative action alternatives, it must be calibrated.

TABLE 6.4.2-1. TRANS Input Requirements and Source of Data for Initial Regional Model

Parameter (Required for Each Node)	Source
Aquifer top and bottom elevations	Interpolated from available geohydrologic cross-sections
Hydraulic Conductivity, (K)	Uniform value (600 gpd/ft^2) selected from well test results
Storage Coefficient, (S)	Uniform value (0.00045) selected from well test results
Effective Porosity, (n_e)	Uniform value (0.37) from the STP FSAR
Actual Porosity, (n)	Uniform value (0.37); assumed n_e equals n for sandy aquifers
Vertical Hydraulic Conductivity for Confining Layer, (K')	Uniform value (0.005 gpd/ft^2) from Freeze and Cherry (1979)
Recharge/Discharge Rate	Computed directly from K' and aquifer thickness (top elevation minus bottom elevation)
Initial Hydraulic Heads	Observed potential contours

Calibration means that a check is made to determine how well the model can correctly generate the past behavior of hydraulic potentials (hydrologic flow model) and/or contaminant movement (contaminant transport model) as they are established from historical records. Adjustments are then made in model parameters until an acceptable re-creation of historical patterns is achieved.

The calibration procedure begins by selecting a period of time for which historical records are available. The required modeling information is then input to the model (as discussed in the previous section on initial model development) and an initial potentiometeric surface is calculated. The predicted values are then compared with values observed (measured) in the field for the period of historical record. Typically, there is a discrepancy between the simulated and the observed.

Because geologic and hydrologic information can be interpreted in a number of different ways, and because of inherent measurement errors which are incorporated in historical records or incompleteness in historical records, the input parameters must be adjusted to a certain extent. A re-evaluation of the hydrogeologic information and/or the historical records is required to formulate a new input data set for the model. The historical period is then

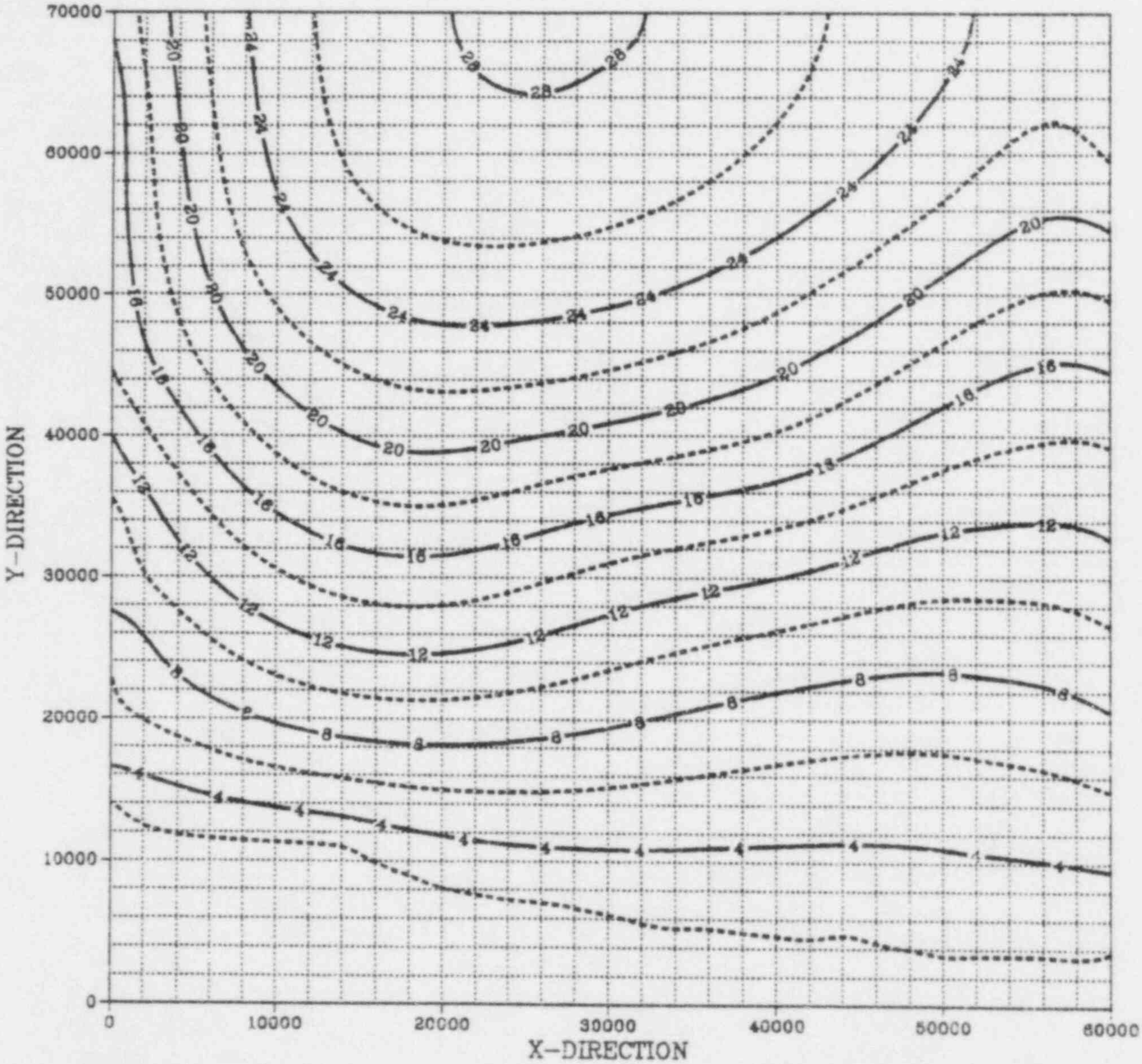


FIGURE 6.4.2-1. Potential Contours Simulated by the Initial STP Region Model

simulated with the new data set and the results compared to observed potentials. The process is repeated until an adequate fit of simulated results to observed behavior is reached.

At the STP site little data were available from available literature regarding the hydraulic potentials or the hydraulic properties of the lower unit of the shallow-zone aquifer. Because of the lack of data and associated

uncertainties adjustments must be made in the input data set in order to adequately calibrate the model. Because of these uncertainties the model is helpful in understanding the ground-water flow system. Many changes in parameter values can be made relatively quickly and inexpensively in the model. This facilitates comparison of model response to changes in parameter values and appreciation of model sensitivity to these changes. The adjustments made in the calibration process are based on refinements/improvements in the data set and in the understanding of the ground-water flow system. The model only indicates that changes are needed. It does not describe the changes. Arbitrary changes not supported by data or bound by understanding should be avoided.

It is often difficult to define a satisfactory match. Obviously, the longer the historical record of hydrologic measurements/observations used for calibration, the better the results will be. Since long-term hydrologic records are seldom available, models are usually calibrated with data covering only a relatively short time period. For example, at the STP site, because of data limitations it was necessary to calibrate the model to a single set of hydraulic potential measurements covering a limited portion of the study region.

Calibration is often the most difficult and one of the most time consuming aspects of ground-water modeling. However, calibration is of utmost importance. Depending on the desired accuracy and the difficulties experienced with scarcity of data, tens of runs can be required to obtain a satisfactory match. However, a model becomes a reliable prediction tool on which to base decisions once it is properly calibrated (Boonstra and de Ridder 1981).

The procedure used to calibrate the STP regional flow model was to run the initial model, compare observed with model-predicted results, make the appropriate changes in the initial data set and rerun the model. This process continued for a number of runs until an acceptable match between simulated hydraulic potentials and observed potentials was achieved. Because the available hydraulic potential data are limited, the focus in the calibration had to be duplication of the hydraulic head within the STP property limits.

The parameters adjusted in the regional model calibration process (i.e., those parameters to which were most sensitive) were the hydraulic conductivity, (K), and the recharge/discharge rates. After several iterations, an acceptable match was achieved between observed and model predicted potentials. The original hydraulic conductivities (i.e., uniformly 600 gpd/ft^2) were adjusted as shown in the three-dimensional plot presented in Figure 6.4.3-1. The relative adjustments made were guided by the differences in the observed potential contours and those simulated by the initial model. For example, simulation of the observed ground-water mound left (or west) of the study area center required relatively low hydraulic conductivities within and down-gradient of the mound (540 gpd/ft^2) and a high hydraulic conductivity immediately up-gradient of the mound (4200 gpd/ft^2). Similarly, the trough (i.e., low potential levels) in the lower right corner (or southeast) of the study area was simulated by increasing hydraulic conductivities to 2340 gpd/ft^2.

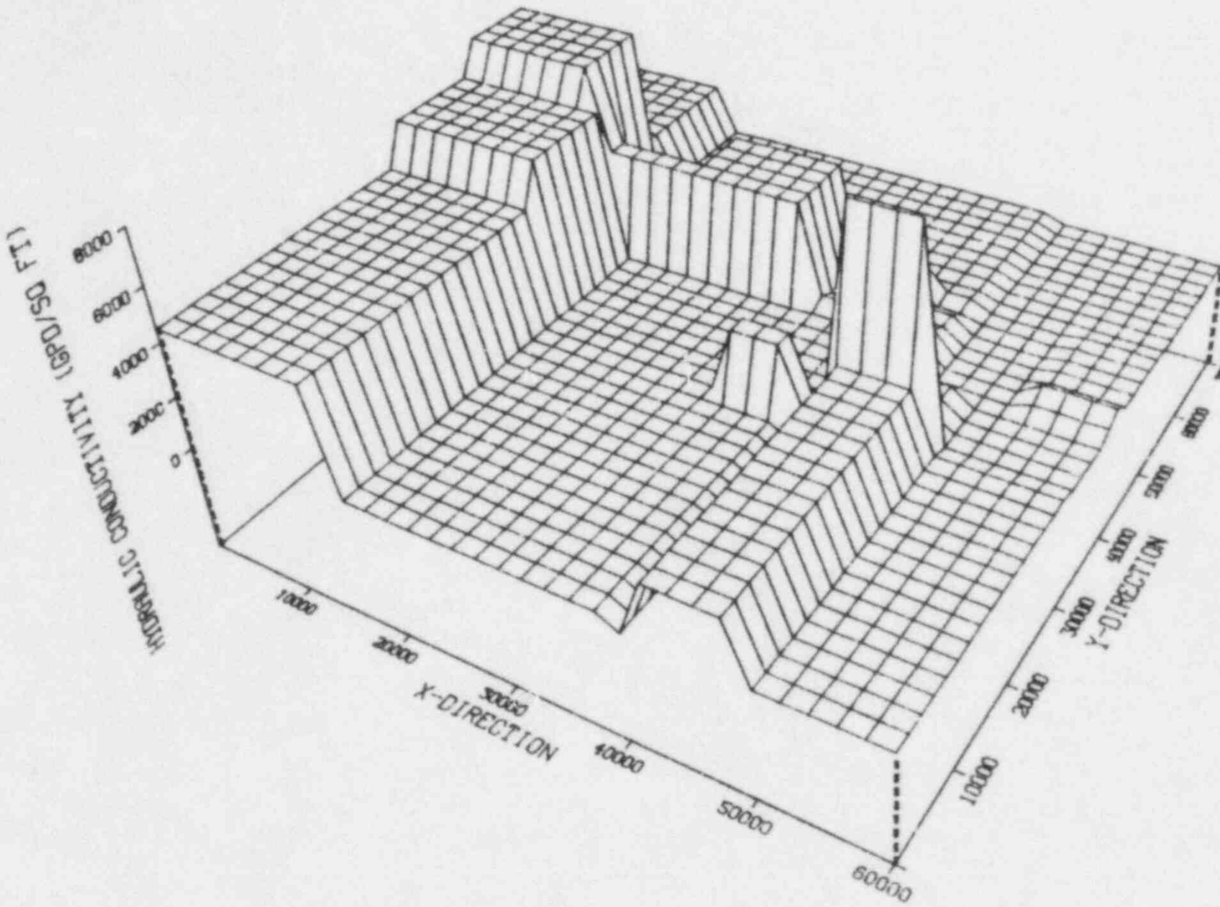


FIGURE 6.4.3-1. Calibrated Hydraulic Conductivities for the STP Final Regional Model

The recharge/discharge rate adjustments were accomplished by adjusting the vertical hydraulic conductivity (K'). Adjustments were made primarily to simulate the water mound to the left of the grid center which is characteristic of increased recharge, and the trough which is characteristic of decreased recharge. The final recharge/discharge distribution is shown in Figure 6.4.3-2. Overall, the recharge to the study region was increased above initial estimates by about 70% to 2.8×10^7 gpd.

Potential contours from the final STP model simulation are presented in Figure 6.4.3-3. Though the match is not exact, the key features which would most effect flow paths and velocities (i.e., general trends in the potential gradient, the ground-water mound and trough) are acceptably reproduced. To further verify the reasonableness of the final model results, Figure 6.4.3-4 shows streamlines beginning at the approximate location of the STP (coordinate 42000 ft, 51000 ft) based on the observed and the simulated potentials. The streamlines, which closely approximate the contaminant trajectory, are approximately the same.

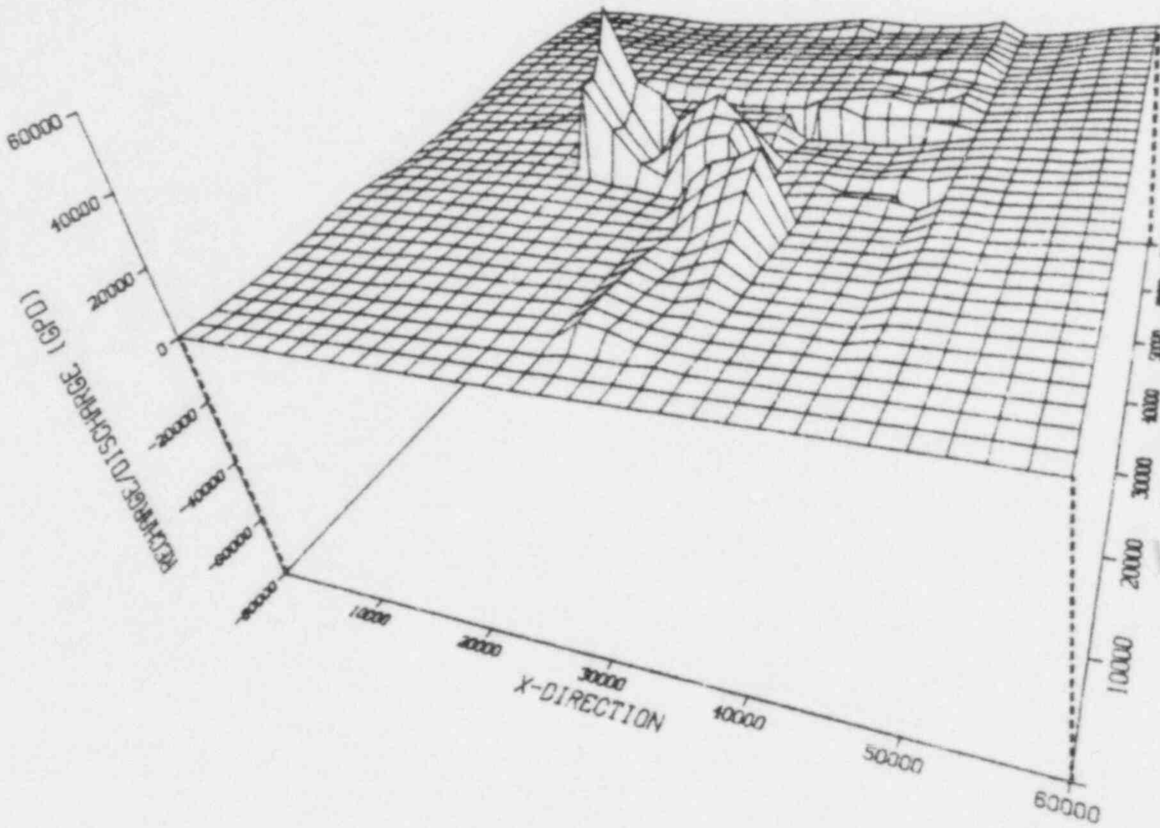


FIGURE 6.4.3-2. Calibrated Recharge/Discharge for the STP Final Regional Model

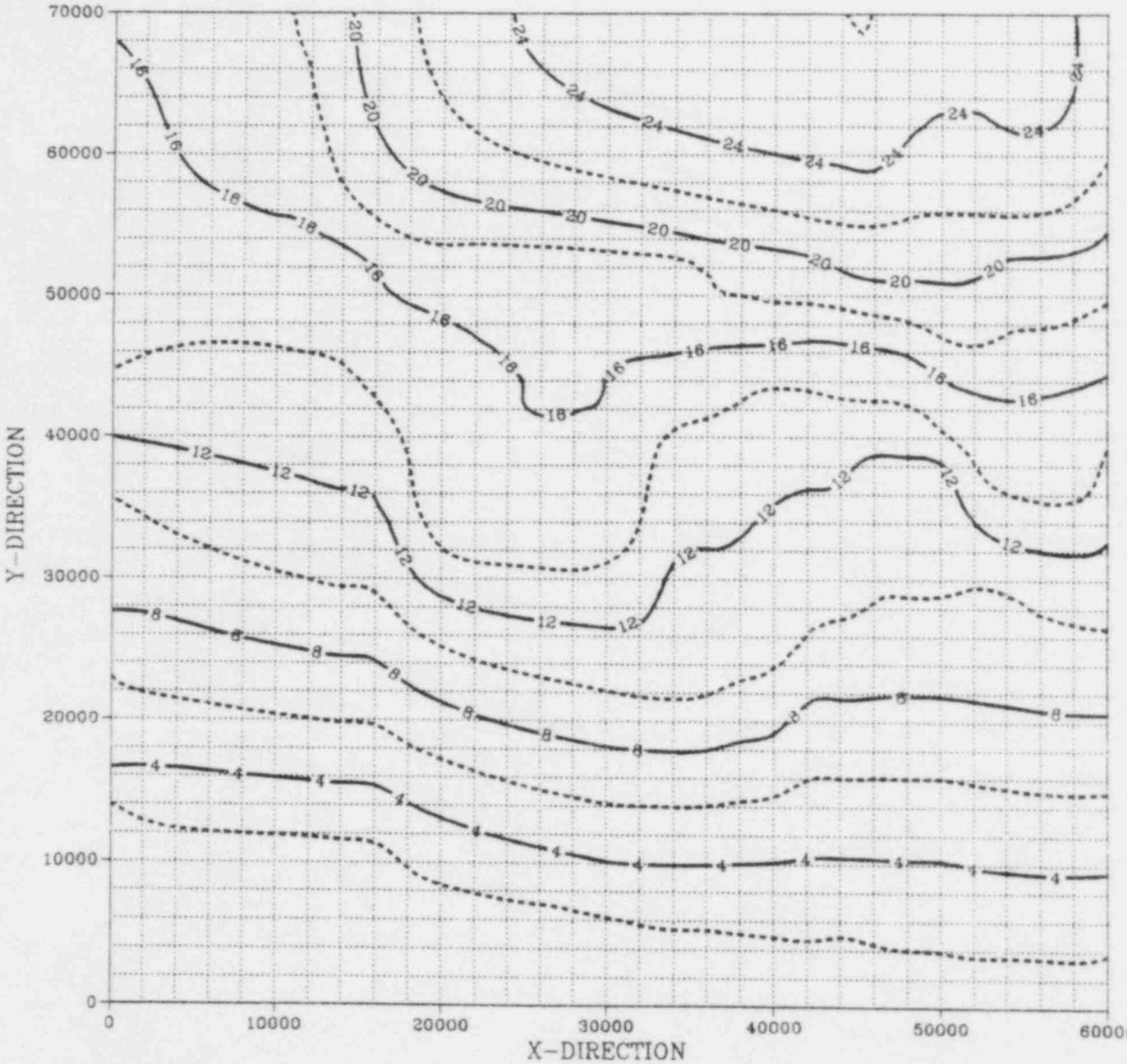


FIGURE 6.4.3-3. Potential Contours Simulated by the Final STP Regional Model

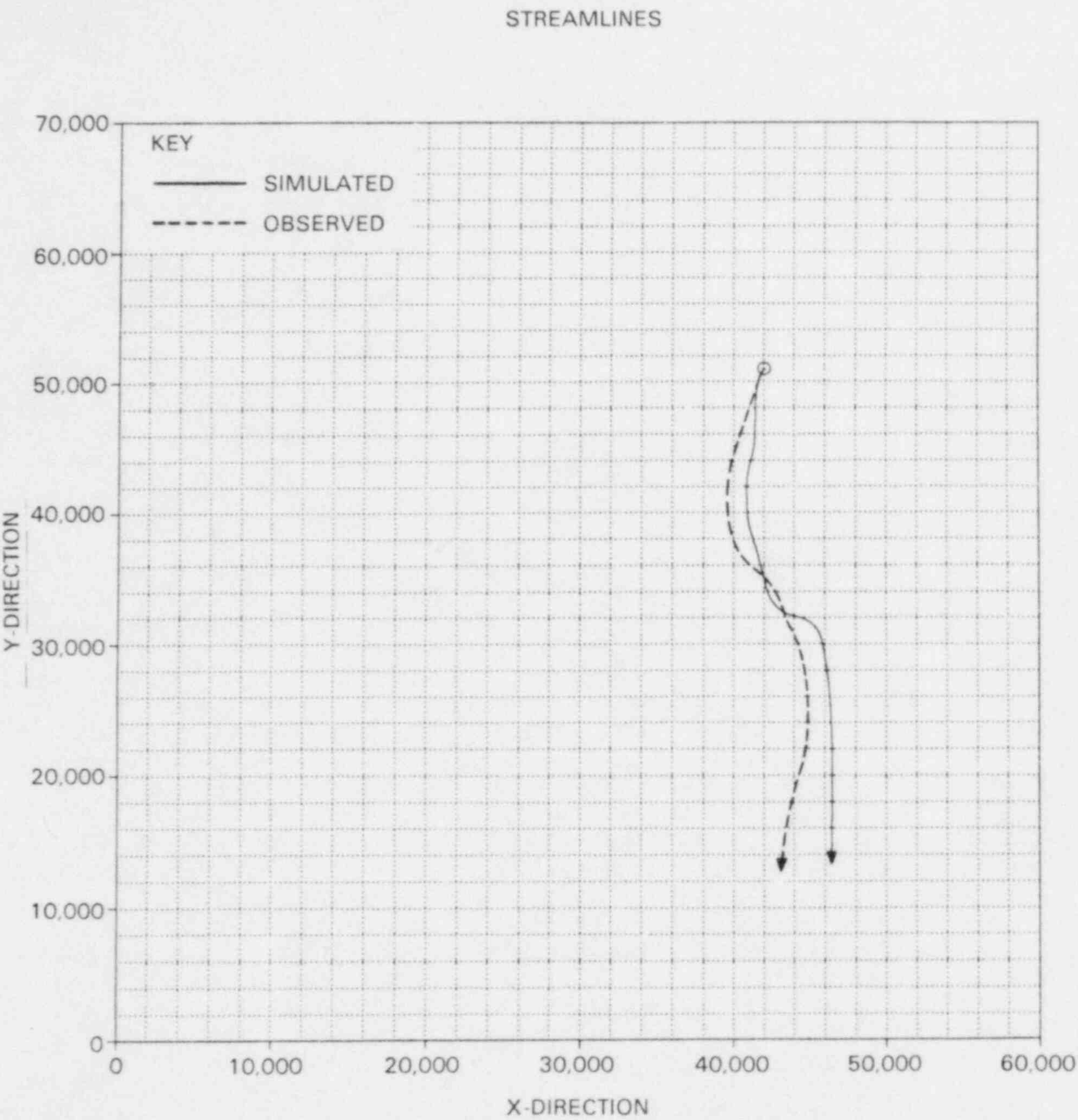


FIGURE 6.4.3-4. Estimated Streamlines From the STP Site Based on Observed and Simulated Potentials

6.5 LOCAL MODEL DEVELOPMENT

6.5.1 Local Area Size and Boundary Conditions

The local model covers an area of 11.2 square miles within the regional study area as shown in Figure 6.5.1-1. The model boundaries were chosen to encompass the STP site and the area down-gradient to the Colorado River.

The boundaries of the local model are located far enough away from the plant site such that implementation of the various mitigative strategies in the vicinity of the plant would not greatly affect the flow field at the boundaries. At the same time the local study area was made small enough to allow a detailed look at flow and transport from the plant (i.e., use a fine mesh) without using an excessive number of nodes. The irregular local model grid, was designed to minimize grid cell size around the STP (coordinate 42000 ft, 51000 ft) and have increasing cell size away from the plant. This was done to allow flexibility in locating and sizing barriers and injection/withdrawal wells for the evaluation of the performance of various mitigative strategies.

As previously discussed, the boundary conditions for the local model are determined directly from the regional model. The procedure for implementing the two models (i.e., regional and local) for pre-mitigative and post-mitigative analyses is to first run the regional model, set the local boundary conditions from the regional results, then run the local model. The boundary conditions in the local model are head-controlled for all boundaries.

The structural top and bottom of the lower shallow-zone aquifer were defined the same as in the regional model as were also the hydraulic conductivities and the recharge/discharge rates. The transfer of these properties from the regional model grid to the irregular local grid was accomplished using an IMSL interpolation routine (IMSL 1980).

The observed and simulated potential contours for the local area are presented in Figures 6.5.1-2 and 6.5.1-3, respectively. It can be seen that the observed trends in the gradient and contour shapes are basically reproduced by the local model. This is particularly true immediately down-gradient of the approximate plant location (i.e., coordinate 42000 ft, 51000 ft), indicating an acceptable model calibration.

6.6 PRE-MITIGATIVE LOCAL FLOW AND TRANSPORT MODELING

Ideally, development of a transport model parallels that for the flow model. Beginning with the final local flow model, initial values of the transport parameters are estimated from available data and subsequently calibrated to obtain a reasonable match between field-measured and model-predicted contaminant concentrations. However, when developing a model to evaluate mitigative techniques for the control of radionuclide contaminants due to severe nuclear power plant accidents, radionuclide contaminant field data are not likely to exist. Consequently, one must rely on initial estimates of the transport parameters (e.g., primarily the effective porosity, retardation

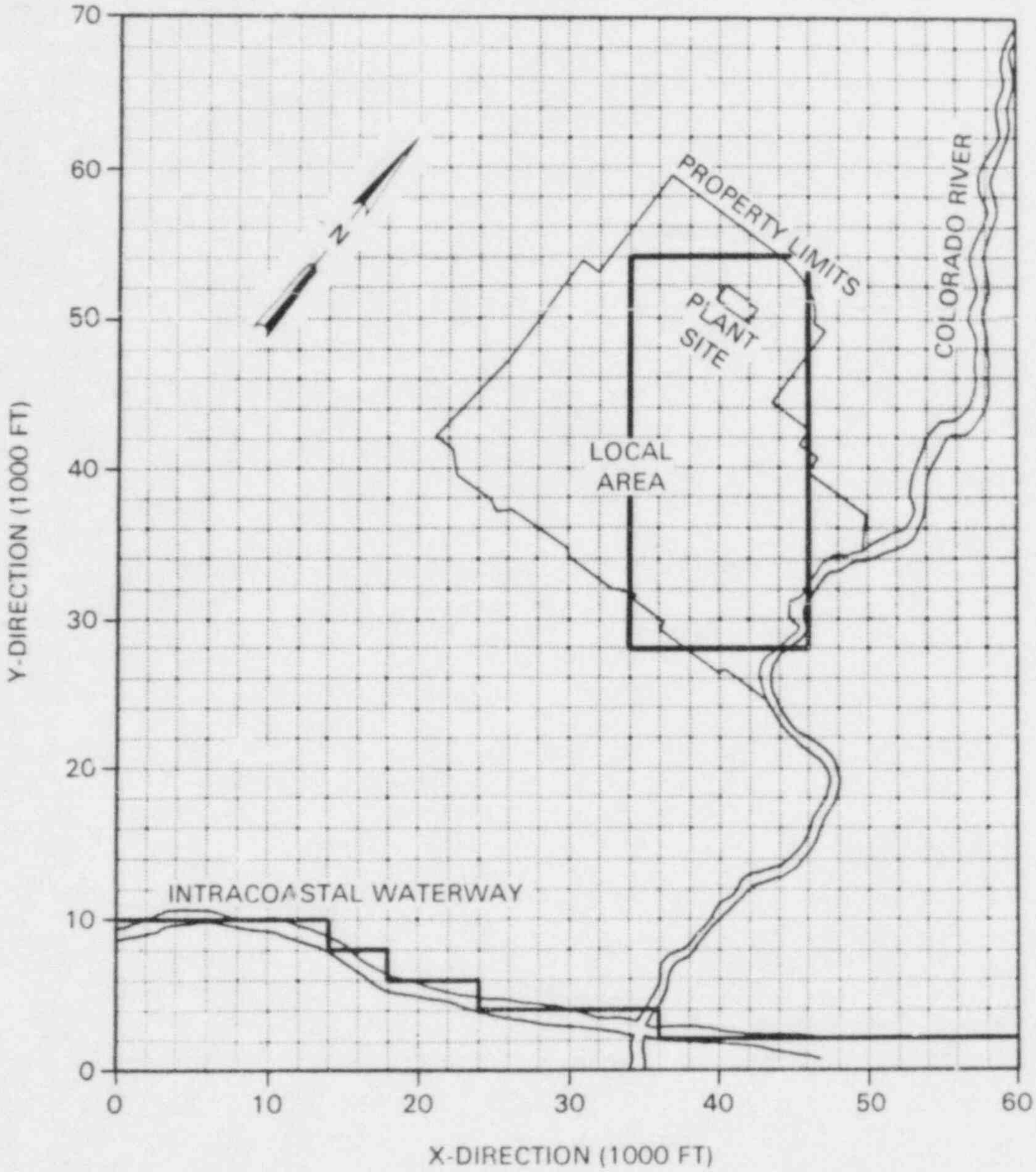


FIGURE 6.5.1-1. STP Local Model Study Area

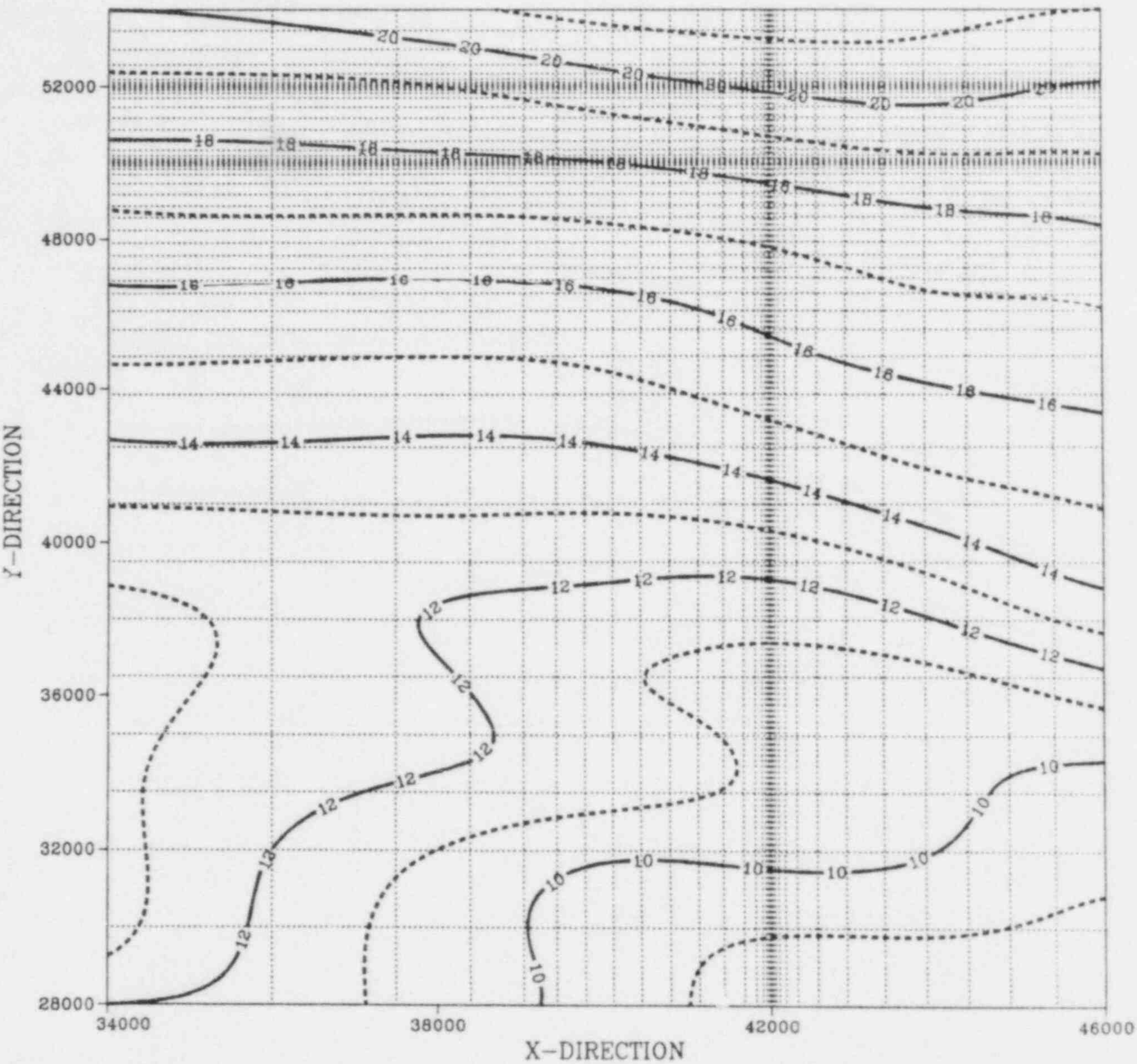


FIGURE 6.5.1-2. STP Local Area Observed Potential Contours

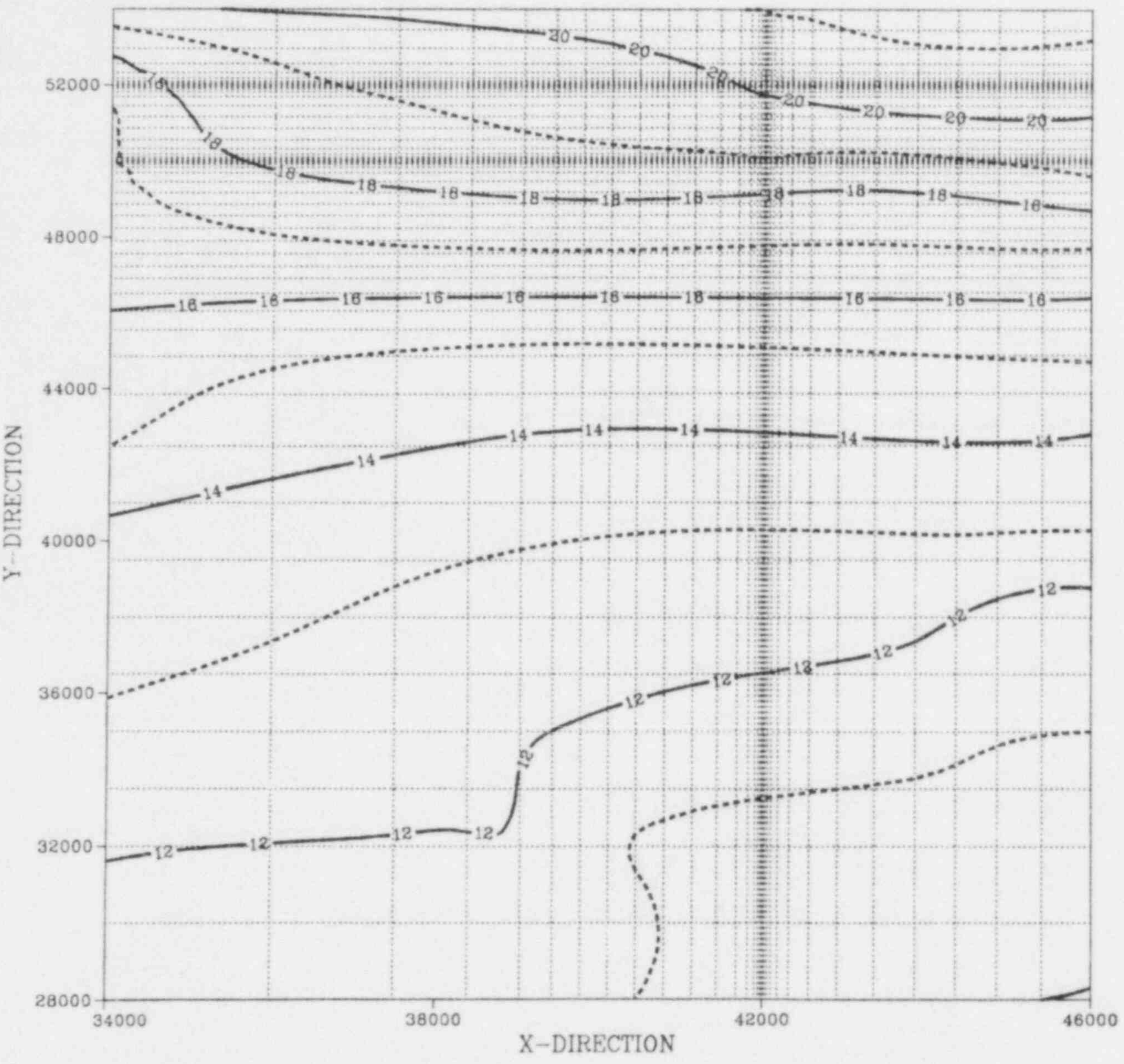


FIGURE 6.5.1-3. STP Local Model Simulated Potential Contours

factor and dispersivity coefficients), based on available information. Generally, once satisfactory estimates are obtained, these same values are used for the baseline pre-mitigative simulations as well as the post-mitigative performance comparisons.

6.6.1 Transport Parameter Estimations

The data and parameters required to simulate transport are:

1. longitudinal and transverse dispersivity coefficients,
2. contaminant source characteristics including leach rate and contaminant decay constant,
3. adsorption data such as distribution coefficient, bulk density and effective porosity.

The determination of the required parameters and data for the pre- and post-mitigative transport analysis is discussed below.

6.6.1.1 Dispersivity Coefficients

Simmons and Cole (1985) describe dispersion in a ground-water system as a combination of molecular diffusion, which occurs even under conditions of no flow, and hydrodynamic dispersion. Hydrodynamic dispersion results from variation in the local water velocity within the medium with respect to its average value as described by convection. This variation in velocity exists at any scale, from microscopic (in the pores) to macroscopic (due to uncharacterized heterogeneity of the medium) and even megascopic (due to large scale variations in medium properties like fractures). Dispersion is important because it produces mixing and spreading both longitudinally and transversely with respect to the flow direction of the transported contaminants. Estimation and measurement of hydrodynamic dispersion for field conditions is presently a topic of intensive research (Molz et al. 1983; Wang and Anderson 1982; Simmons 1982). The problems in dealing with spatial variability of hydrologic properties and field-scale dispersion processes have resulted in questions about the adequacy of the classical convective-dispersive approach with its inherent assumptions regarding dispersion. However, as long as the appropriate effective dispersion coefficients can be defined the convective-dispersion approach is considered applicable.

In TRANS contaminant transport is computed on the basis that the distribution of contaminant concentrations in the ground water can be represented by a finite number of discrete particles. Each particle represents a fraction of the total mass of contaminant involved and is assumed to move with the ground-water flow. The technique, designated by Prickett, Naymick and Lonnquist (1981) as the "random-walk" method, is founded on the concept that dispersion in porous media is a random process such that particles move through an aquifer with two types of motion. One motion is that of the mean flow along computed streamlines. The other type is random motion governed by scaled probabilities related to flow length and the longitudinal and transverse dispersion coefficients. The "random-walk" concept is illustrated in Figure 6.6.1-1.

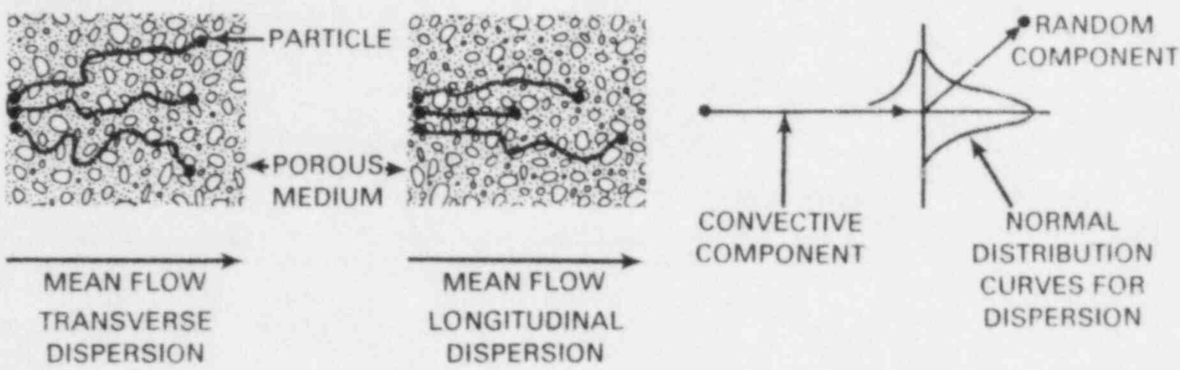


FIGURE 6.6.1-1. Illustration of "Random-Walk" Concept Employed by TRANS (Source: Prickett, Naymik and Lonnquist 1981)

Because of a total lack of data related to dispersivity in the study aquifer, initial estimates of longitudinal (D_L) and transverse (D_T) dispersivity coefficients were made based on information in the literature. One source (Yeh 1981) provided estimates of D_L for various materials which are presented in Table 6.6.1-1.

The geologic material in the study aquifer most likely fits in the sandy silt to sand range resulting in a valve equal to 25 m to 50 m or approximately 80 ft. to 165 ft. According to Fried and Combarnous (1971) based on laboratory studies D_T is equal to about 1/20 D_L. Therefore, the equivalent range for D_T is 4 ft. to 8 ft. Data presented by Gelhar and Axness (1981), shown in Figure 6.6.1-2, demonstrate that the value of D_L is a function of scale.

TABLE 6.6.1-1. Estimated Longitudinal Dispersivity (D_L) for Various Geologic Materials (Source: Yeh 1981)

Geologic Material	Estimated Longitudinal Dispersivity (D_L), m
Clay-Silt	1
Silty Clay	5
Silty Marl	10
Sandy Silt	25
Sand	50

Though there is considerable scatter in the data it shows a distinct increase in D_L with travel distance. For reference, the line for D_L equal to one-tenth the travel distance is shown. The estimated down-gradient travel distance from the STP to the Colorado River is about 5 miles (8000 m). For distances of this magnitude measured dispersivities shown in Figure 6.6.1-2 range from about 5 m to 500 m. The values suggested by Yeh (1981) falls well within this range and would be at about the midpoint on the log-log plot. Consequently, the value for sand (50 m) is used for the initial transport analysis.

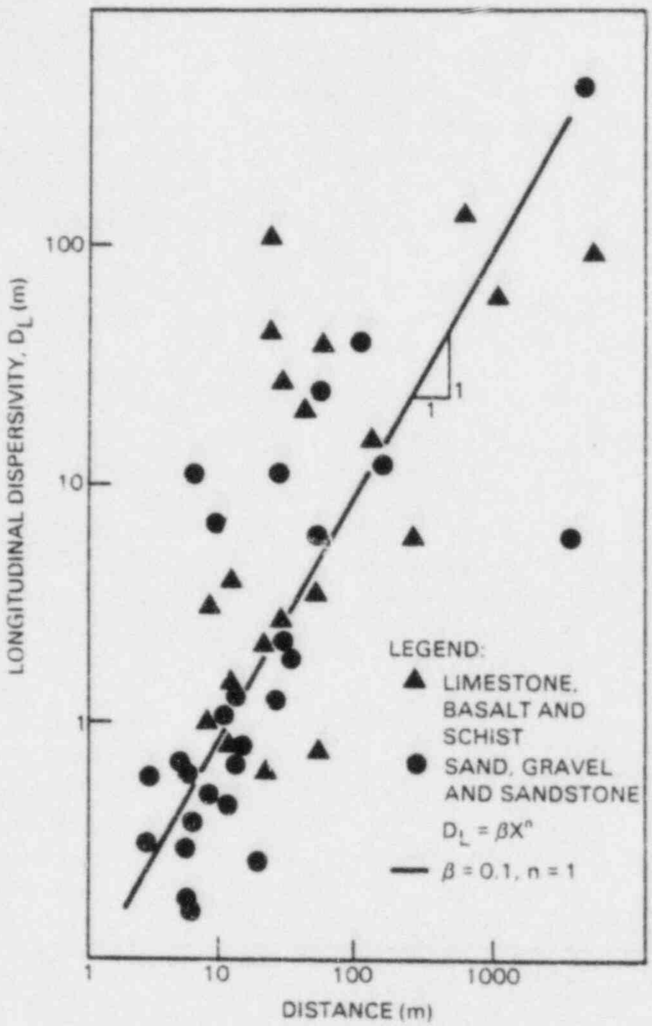


FIGURE 6.6.1-2. Field Observations of Longitudinal Dispersivity (After: Gelhar and Axnes 1981)

6.6.1.2 Source Characteristics

Data requirements for characterization of a radionuclide contaminant source are the source leach rate and decay constants for the radionuclides present. As noted in Section 6.2.3 this case study and methodology demonstration will address the leaching of radionuclides from the core melt debris following a postulated severe reactor accident at the STP. Further, strontium-90 will be used as the indicator radionuclide for the pre-mitigative transport modeling.

The hypothesized leach release of strontium-90, shown in Figure 6.2.3-1, decreases exponentially from approximately 4 x 10^{15} pCi/year at the time leaching begins to 2 x 10^{3} pCi/year after 1000 years. With TRANS, the source must be input as a sequence of slug releases, each occurring over a specified time period (i.e., the time step used for the transient transport simulation). Given the strontium-90 half-life of 10,519 days and the large amounts of radionuclide hypothesized to be released, simulation times of several hundred years are anticipated. Therefore, the release rate curve was integrated in 100-year intervals to obtain an equivalent curve consisting of a series of constant release rates. The resulting stepped curve (Figure 6.6.1-2) decreases from about 4 x 10^{14} pCi/year for the first 100 years to 1 x 10^{4} pCi/year at the end of 1000 years.

6.6.1.3 Adsorption Related Parameters

Distribution Coefficient

In TRANS, reversible equilibrium controlled adsorption is described in terms of a retardation coefficient. In turn, the retardation coefficient is related to the distribution coefficient, K_d, of the concentration isotherm for a particular chemical species and the bulk density and effective porosity of the aquifer medium. In effect the retardation coefficient represents the reduction in solute travel velocity relative to the ground-water flow velocity such that an adsorbed contaminant moves more slowly, but without being permanently affixed to the medium (Simmons and Cole 1985).

Simmons and Cole (1985) state that the estimation and use of distribution coefficients is currently at issue within the technical community. Measurements of K_d under static laboratory conditions may not be comparable to those obtained from a dynamic field experiment. This brings into question transferability of laboratory measurements to field conditions. Usually discrepancies can be explained in terms of violation of chemical equilibrium assumptions. However, retardation coefficients may have phenomenological meaning when estimated inversely to match a field-scale tracer test, even though values may not conform to laboratory conditions. In this light, an estimate of K_d for strontium-90 which was obtained from the representative K_d's is presented in Table 3.3.2-2. Values in the table are 10 ml/g for porous silicate and 50 ml/g for porous silicate containing clay and silt. Conservatively, a value of 10 ml/g is selected for the analysis.

Effective Porosity

The available information regarding effective porosity for the study area was limited to an assumed value of 37% obtained from the STP FSAR (Houston Power and Light 1978) and the representative porosity values for various sedimentary material shown in Table 6.3.2-6 (Hammond 1969). The ranges given are 30% to 40% for uniform sand and 30% to 35% for fine to medium mixed sand. The 37% value assumed in the STP FSAR (Houston Power and Light 1978) falls within or near these ranges and thus appears to be reasonable.

Bulk Density

In addition to effective porosity, determination of the retardation factor requires a known bulk mass density which can be determined from the relationship (Freeze and Cherry 1979):

$$n_e = 1 - \frac{\rho_b}{\rho_m}$$

where

n_e = effective porosity
ρ_m = particle mass density (2.65 g/cm^3 for most mineral soils), and
ρ_b = bulk mass density.

From this relationship, assuming an effective porosity of 37%, the bulk mass density for the lower unit is 1.7 g/cm^3 or 104 lb/ft^3.

6.6.1.4 Additional TRANS Parameters

As previously discussed, the total mass of contaminant is represented in TRANS by a discrete number of particles, with the idea that as the number of particles approaches the molecular level the exact solution to the convective-diffusion equation would be obtained. Prickett et al. (1981) suggest that relatively few particles (less than 5000) are needed to obtain an acceptable solution for most engineering applications. Measured contaminant data necessary to test their assertion were not available. However, in initial model tests several transport runs were made with different numbers of particles. The results indicated that simulation results did not change appreciably due to increasing the number of particles from 5000 to 7000, though computer simulation time increased significantly. Consequently, the number of particles was set equal to 5000 for all simulations in this case study.

The final parameter values determined for the pre-mitigative transport simulations and their sources are summarized in Table 6.6.1-1.

6.6.2 Pre-Mitigative Local Transport Results

There are two primary objectives in performing a pre-mitigative transport analysis:

1. quantitatively assess the need for mitigation following a severe accident release of radionuclides to the ground-water system, and

2. when mitigation is found to be necessary, provide a baseline for evaluating the effectiveness of selected mitigation techniques.

To meet these objectives for the case study, TRANS was used to simulate radionuclide transport from the STP for a 1000-yr period from the time leach releases begin. The transient simulation was made using 100 year time steps and the discretized leach rate curve shown in Figure 6.6.1-3. The radionuclide

TABLE 6.6.2-1. Summary of TRANS Transport Parameter Values for STP Transport Simulations

Parameter	Value	Source
Longitudinal Dispersivity (D_L)	50 m (164 ft)	Gelhar and Axness (1981)
Transverse Dispersivity (D_T)	2.5 m (8 ft)	Computed; $D_L/20$
Distribution Coefficient (K_d)	10 ml/g	Table 3.3.2-2
Effective Porosity (n_e)	37%	STP FSAR (Houston Power and Light 1978)
Bulk Density (ρ_b)	104 lb/ft^3	Computed
Retardation Factor (R_d)	46	Computed
Number of Particles (NP)[a]	5000	Prickett et al. (1981)

(a) TRANS specific parameter

release is assumed to be a point source at spatial coordinates 42,000 ft in the x-direction and 51,000 ft in the y-direction. The basis for assessing the need for mitigation is the potential for significant release of radionuclides to the Colorado River.

The results of the pre-mitigative transport simulation indicate that the strontium-90 plume, because of the high retardation, would migrate less than 2400 ft from the STP in 1000 years. Three-dimensional plots of the distribution of strontium-90 concentrations at 100 years and 1000 years are shown in Figures 6.6.2-1 and 6.6.2-2. At 100 years the maximum concentration is approximately 1×10^{5} pCi/ml while at 1000 years the maximum has decreased to 2.5×10^{-5} pCi/ml, well below the maximum permissible concentration of strontium-90 of 0.3 pCi/ml according to 10 CFR/Part 20 (USNRC 1978).

With regard to the first objective of the pre-mitigative transport analysis, based on the simulations using strontium-90 as an index radionuclide, significant contamination of the Colorado River would not occur. Consequently, mitigation of radionuclide movement in ground water following a severe accident at the STP would not be necessary.

Though mitigation would not actually be required at the STP, the ground-water model for the site still provides an excellent vehicle for demonstrating site specific evaluation of mitigative techniques. To this end, the following sections describe a detailed analysis of feasible mitigative techniques. The analysis is based upon the STP local and regional models and a hypothetical contaminant that is assumed to move only advectively with the ground water (i.e., the distribution coefficient and dispersivities are equal to zero). In Figure 6.6.2-3, the pathline for ground-water flow from the STP to the Colorado

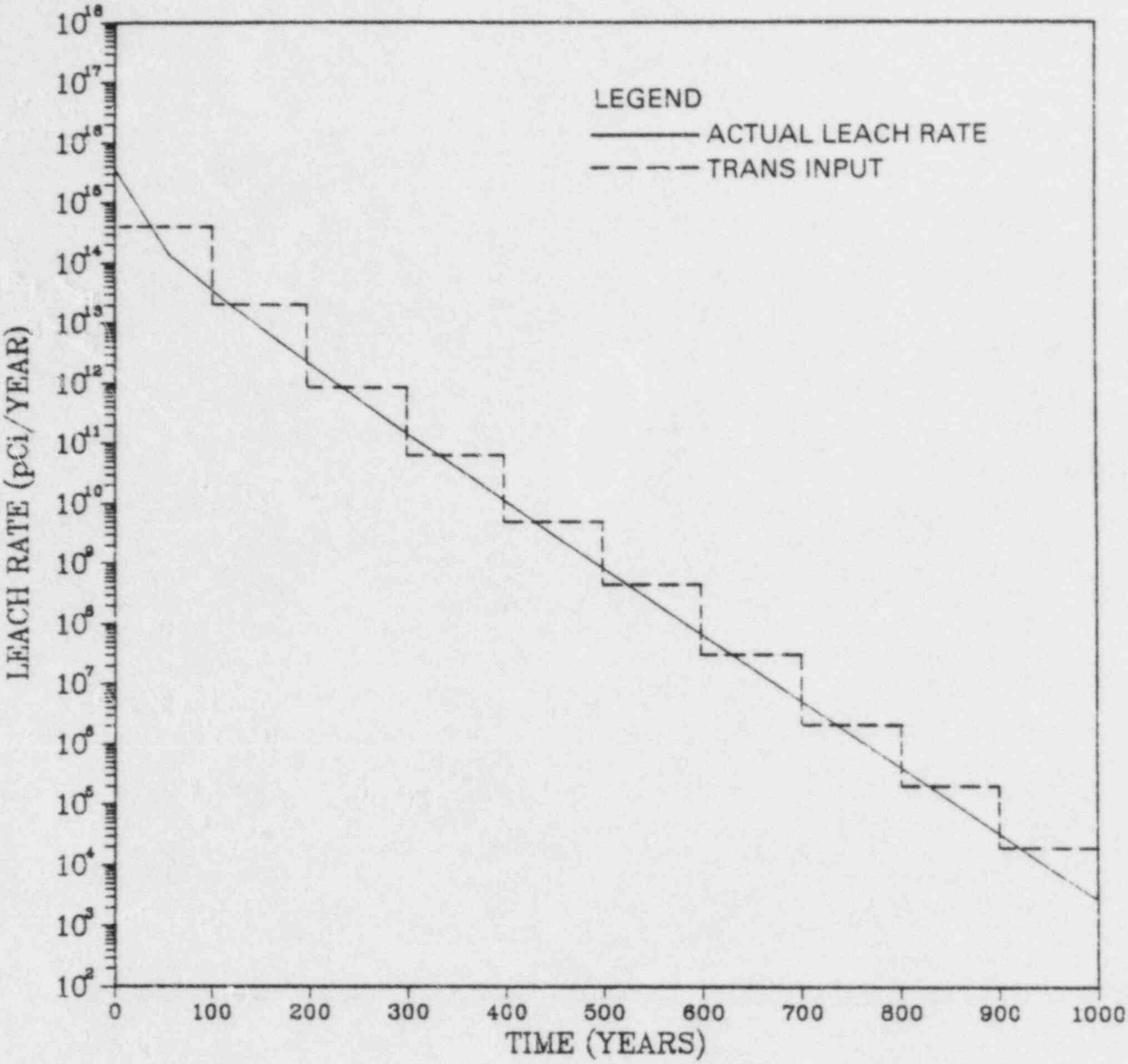


FIGURE 6.6.1-3. Stepped Source Leach Rate Curve for Strontium-90

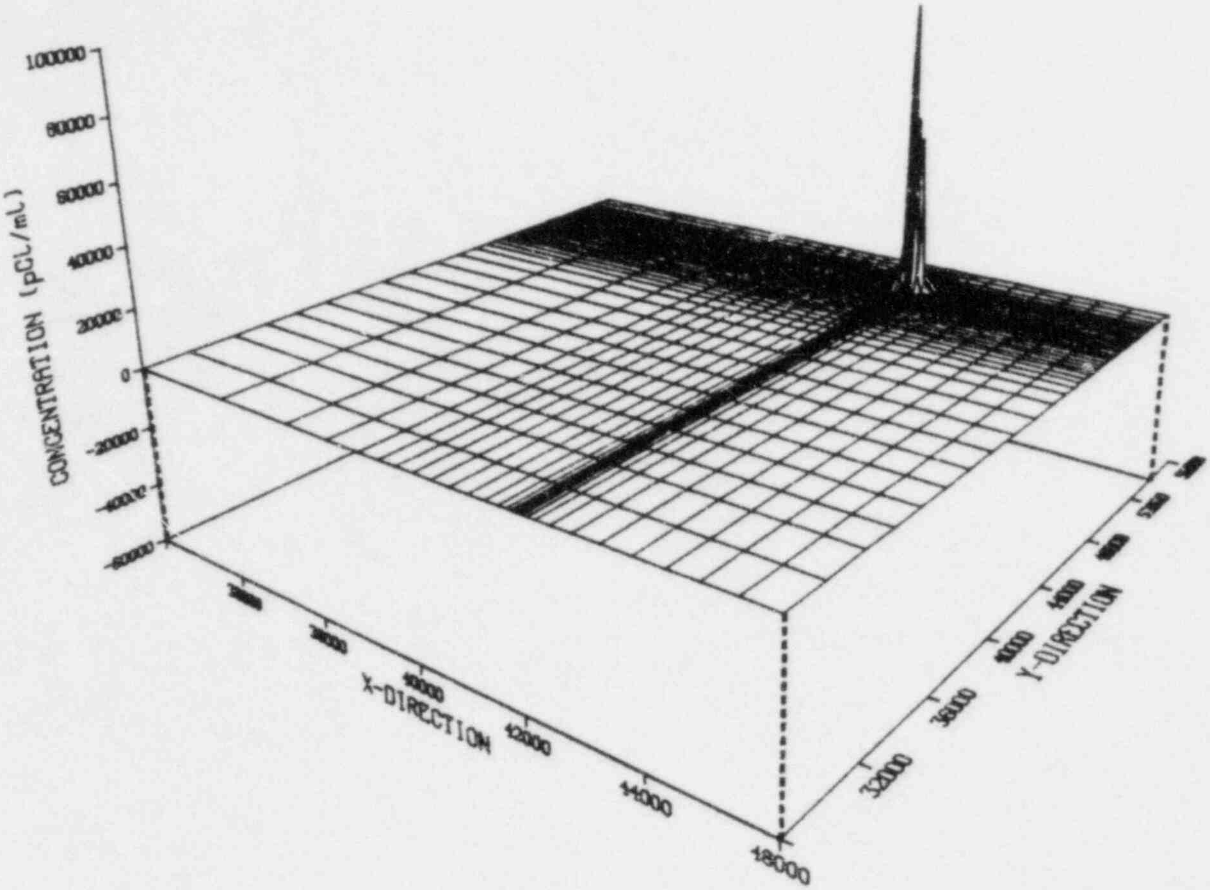


FIGURE 6.6.2-1. Simulated Pre-Mitigation Strontium-90 Concentrations at 100 Years

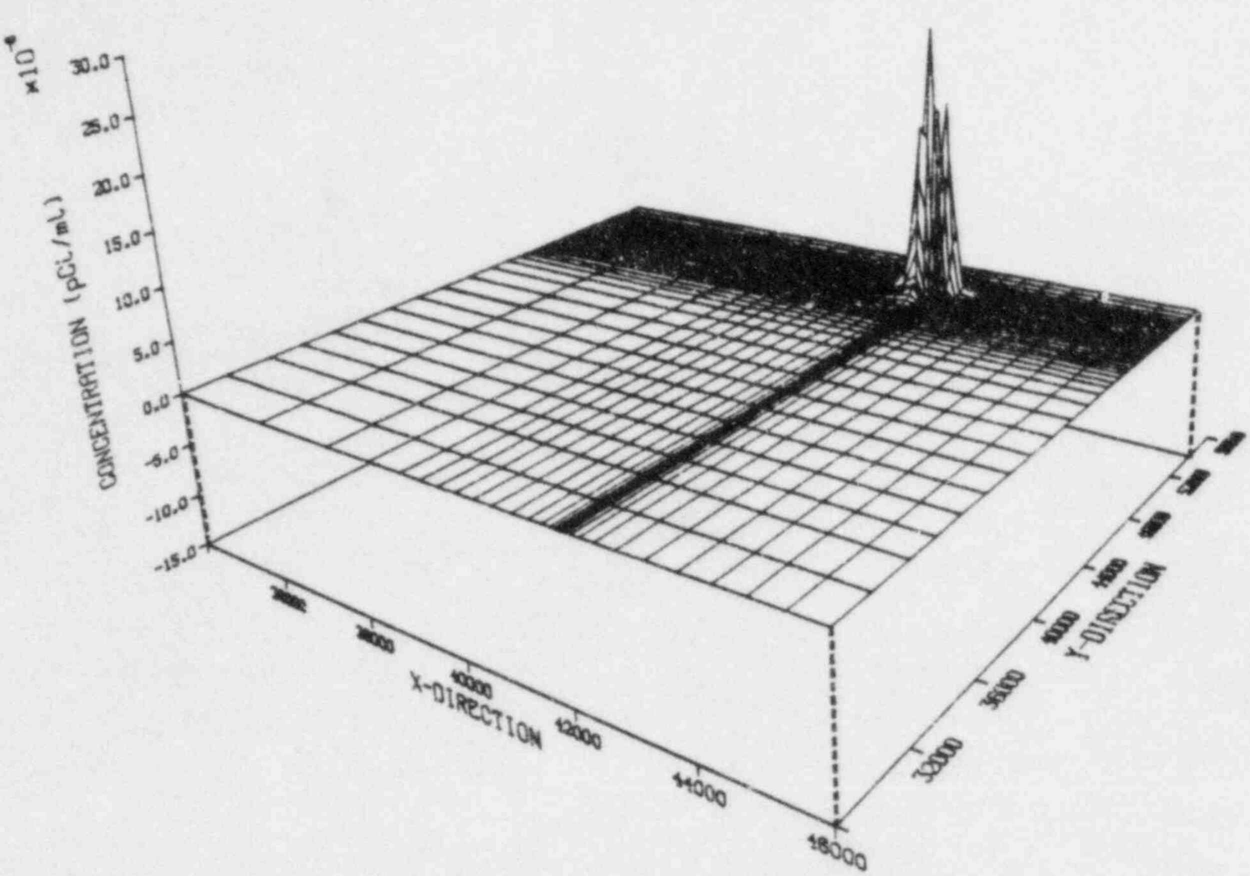


FIGURE 6.6.2-2. Simulated Pre-Mitigation Strontium-90 Concentrations at 1000 Years (Note Vertical Scale Exaggeration as Compared to Figure 6.6.2-1)

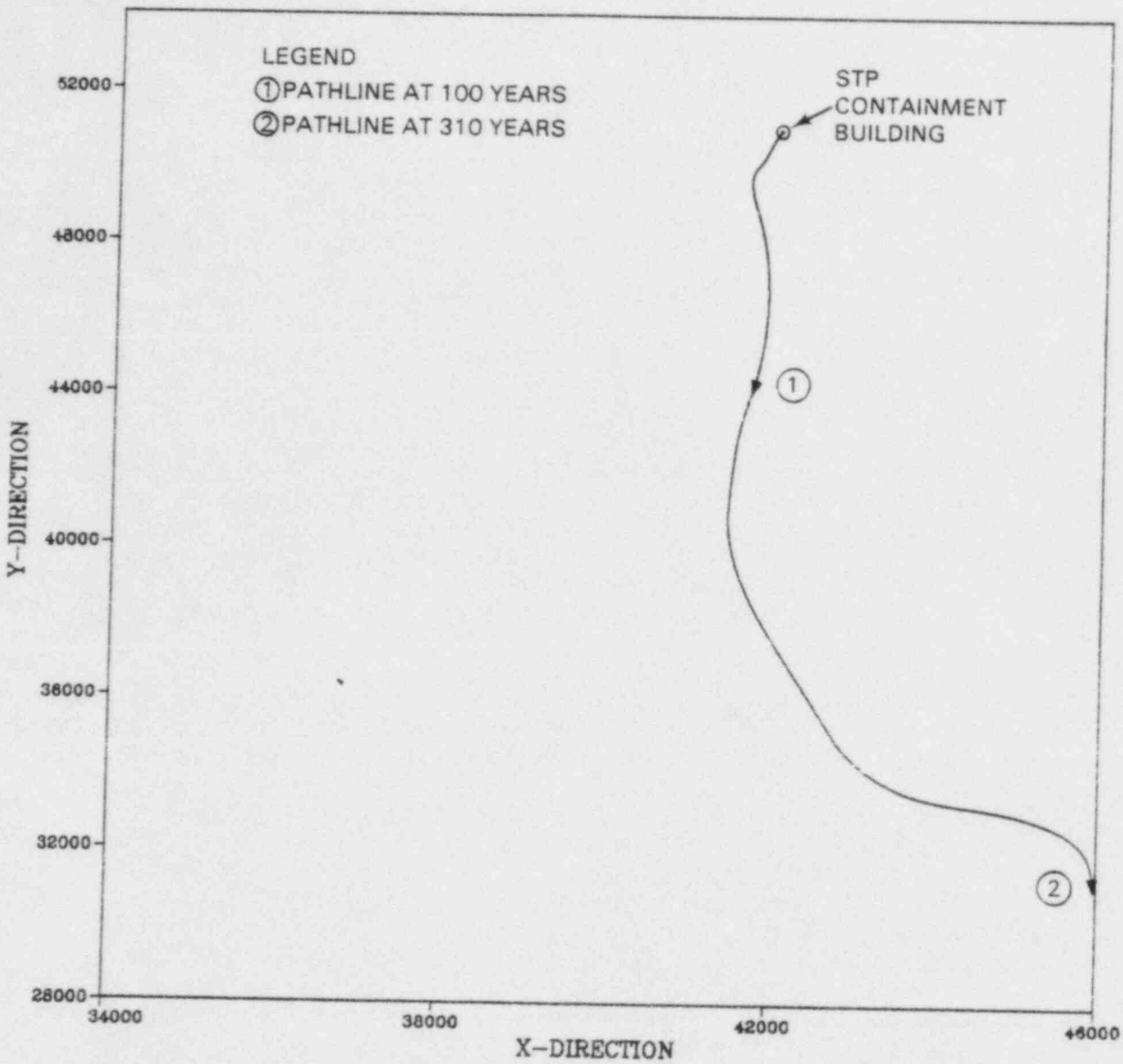


FIGURE 6.6.2-3. Pre-Mitigation Pathline From the STP

River is shown. The estimated time of arrival is 310 years. This value will serve as the baseline for evaluating the effectiveness of the mitigation techniques to be considered.

6.7 EVALUATION OF MITIGATIVE TECHNIQUES

The previous sections of this chapter discuss the procedures for developing a mathematical model for analysis of flow and contaminant transport at porous media nuclear power plant sites and then demonstrates the procedures by applying them to the STP. This pattern is continued in this section wherein the emphasis is in demonstrating the general approach to evaluate mitigative techniques while giving only limited attention to the specific characteristics and configuration of the STP.

6.7.1 Approach

The first step in the evaluation of mitigative techniques involves preliminary screening of those methods that clearly are not feasible given site-specific hydrogeologic conditions. Engineering judgment and an understanding of the design, construction and performance considerations of the different techniques are usually sufficient to determine those methods most likely to be applicable. The review of mitigation techniques presented in Chapter 4.0 provides information necessary for this screening phase.

Following the initial screening, the remaining mitigative techniques are subjected to a detailed assessment in terms of their effectiveness in achieving the desired level of mitigation and in formulating a their preliminary design. Recent studies by Silka and Mercer (1982); and Cole et al. (1983) demonstrate the usefulness of computer modeling in the feasibility study process. While it may be straightforward to determine that grouts are applicable to a relatively porous hydrogeologic unit, mathematical models are required to assess the performance of a grout cutoff as a function of location (i.e., up-gradient or down-gradient) and orientation and dimension. The trade-offs in performance lead one to conceptual design specifications for a technique. Once a model is developed for a site, any number of alternatives can be evaluated with minimal additional effort. Further, predictions of contaminant concentrations can be obtained for any location of interest such as site boundaries, surface water discharge points, or drinking water wells (Boutwell et al. 1984).

6.7.2 Screening of Mitigative Techniques

Within the generic site classification scheme discussed in Chapter 3.0, the STP site is categorized as porous unconsolidated silicate. As such, a wide range of mitigative techniques or strategies are potentially feasible (see Table 5.6.2-1). However, due to site specific data obtained from the hydrogeologic characterization and assessment performed on the STP the range of feasible strategies can be narrowed.

Both particulate (i.e., cement-based) and more probably non-particulate (i.e., chemical) grouts may be feasible for construction of ground-water cutoffs to the migration of radionuclides from the STP. The general material properties of the shallow zone aquifer indicate that both the upper and lower shallow zone aquifers could be successfully grouted with the intervening clay layer acting as a key-in and a natural ground-water flow barrier. The average permeability of the host material at the STP is approximately 85 ft/day (0.03 cm/sec) which falls in the middle of the "easy" to grout range of permeabilities listed in Table 5.6.2-1. The relatively high permeability coupled with the low average ground-water velocity (i.e., roughly 0.3 ft/day) facilitates successful chemical grouting of ground-water cutoffs. According to data presented in Table 4.3.1-3 permeation grouting with sodium silicate, lignochrome gel, colloidal solutions, or prepolymer grouts would be recommended. In granular materials, such as those present at the STP, there may be a filtration of cement-based grouts, thus diminishing their suitability for development of a low permeability barrier. However, detailed laboratory analyses would be required to estimate the degree and overall effect of the filtration as a constraint on the feasibility of implementing cement-based grouts. The soil size limitations on grout permeation presented in Figure 4.3.1-1 suggest that silicate grouts may be most suitable for the STP. Silicate grouts may also be somewhat more resistant to the potential prolonged exposure to saltwater that may be possible at the STP depending on the eventual location of the cutoff. Finally, the normal range of pH (i.e., roughly 6 to 8) observed for the groundwater in vicinity of the STP (Hammond 1969) would not prohibit the use of a silicate grout.

Because of the depth to which a constructed barrier would have to be placed to be effective in mitigating the consequences of contamination of the lower shallow zone aquifer, the use of steel sheet piling is infeasible and the construction of slurry trench cutoff walls would be impractical. A realistic value for the depth of cutoff, including key-in to the underlying confining bed, that would be required for the STP is roughly 100 ft below MSL. This depth coupled with a surface elevation of 25 ft would require excavation of over 125 ft of material. The depth of excavation could be reduced by locating the slurry trench in an area where the bottom of lower shallow zone aquifer is closer to the ground surface. However, this approach may result in a grossly non-optimal placement of the cutoff. The alternate-slot method would be recommended for construction of a slurry trench cutoff at depths approaching 125 ft. Because of structural integrity, the depth of the wall and the relatively high permeability of the site lean-concrete would most likely be preferred for the construction of the cutoff. Also, because of the depth and looseness of the host material, trench cave-in may prohibit the construction of a slurry trench cutoff.

Creation of a hydraulic barrier to the specific path or trajectory of the contaminant plume resulting from a possible severe accident at the STP would also be a feasible mitigative strategy for the STP site. Due to the conceptual nature of these dynamic strategies, their feasibility, which is directly related to performance, must be addressed via some form of model evaluation of the effects of varying withdrawal and/or injection rates at various locations on ground-water flow and contaminant transport. Since the aquifer for which

potential surface modification would be sought is deep (˜125 ft) and confined, deep wells would most likely be installed that have high pumping capacities. The wells would only be screened in the lower shallow-zone aquifer in order to prevent avoidable contamination of the upper shallow zone aquifer. Strategies that involve injection would be preferable to a significant amount of withdrawal because of the potential for contamination of the surface environment. Readily available sources of injection water could be obtained from the Colorado River although filtration would be required for efficient injection. An alternative to filtering Colorado River water would possibly be to locate a high volume discharge well a suitable distance away so that drawdown did not appreciably undermine the creation of a hydraulic barrier. The well discharge would then be used as injection water. Such a scheme would require development of an overland pipeline and small retention storage. The STP cooling reservoir could also be useful as both a source of injection water and/or a storage basin for water withdrawn from the aquifer, depending upon the degree to which it would be contaminated by atmospheric releases of radionuclides. According to Davis and DeWiest (1966) water yields of between 200 and 300 gpm are normally associated with coastal plain aquifers. Therefore, ground-water withdrawal rates should not exceed this limit by any appreciable amount. Acceptable injection rates should be achievable because of the relatively high porosity (i.e., 0.37) and high permeability in the vicinity of the STP.

Interceptor trenches and permeable treatment beds would not be practically feasible at the STP site for two specific reasons. First, the depth limitations on excavation for development of slurry trenches also apply to development of interceptor trenches. Second, the characteristics of ground-water flow and contaminant transport in the shallow zone aquifer prohibit implementation of collection systems. Because of the very flat hydraulic gradient associated with the lower shallow zone aquifer and the relatively high effective porosity ground-water velocities are very low. Consequently, dispersive behavior may contribute to the spread of a contaminant plume much more so than advective transport of radionuclides. Therefore, to be efficient any collection system (i.e., interceptor trenches or permeable treatment beds) would require artificial inducement (possibly through withdrawal and injection) to counter the effect of dispersion in intercepting the contaminant. Because of the depth and confined nature of the lower shallow zone aquifer and the dispersive influence of the hydraulic properties of the aquifer, collection systems would be impractical.

Ground-water freezing would also be impractical for construction of a barrier to ground-water flow because of the obvious limitations of cost and climate. Thermal erosion due to the warm climate and frequent precipitation could continually prohibit closure of a frozen cutoff. Air injection would not be recommended due to the lack of experience in designing and implementing air injection systems as barriers to ground-water flow.

In summary, the ground-water contaminant mitigative techniques that appear most suitable for implementation at the STP based on the reconnaissance level hydrogeologic characterization and the pre-mitigative ground-water flow and contaminant transport analysis are:

1. a fully penetrating and properly keyed grouted cutoff, and
2. a hydraulic barrier to ground-water flow and transport created by injection and/or withdrawal.

6.7.3 Assessment of Feasible Alternatives

The two basic mitigation techniques identified in the screening phase as most feasible for the STP site are a grouted cutoff wall and the development of a hydraulic barrier by ground-water injection and withdrawal. In reality there are any number of implementable conceptual designs for each of these techniques, either individually or in combination. It is not the purpose of this case study to attempt to evaluate all of these possibilities or even to identify the "best" or most effective design. The intent here is to provide information regarding the use of quantitative methods to make such an evaluation and to demonstrate their use for selected alternatives.

The approach used to assess the above alternatives involves quantifying their effectiveness in increasing ground-water travel time from the STP to the Colorado River and, hence, enhancing natural decay of radionuclides. Selected model parameters and inputs for the local transport model were adjusted to simulate the impact each alternative would have on ground-water flow. Steady-state simulations of ground-water flow were then made and the results were compared with the pre-mitigated results to obtain a measure of effectiveness.

6.7.3.1 Cutoff Walls

Cutoff walls are vertical barriers emplaced to either prevent contaminated ground water from migrating away from the site, or to divert incoming ground water away from the contaminant source. There are several alternative cut-off design configurations including (Boutwell et al. 1984):

1. an up-gradient cutoff extending to an impermeable layer,
2. a down-gradient cutoff extending to an impermeable layer,
3. both up-gradient and down-gradient cutoffs, and
4. a cutoff that extends completely around the site.

For this case study, up-gradient and down-gradient cutoffs having lengths of 2000 ft, 3000 ft and 4000 ft are considered. The effectiveness of each cutoff is simulated by introducing a line of nodes having zero permeability 1000 ft up-gradient or down-gradient, from the STP. The 2000 ft cutoffs are centered relative to the plant as illustrated in Figure 6.7.3-1 and Figure 6.7.3-2. The 3000 ft and 4000 ft cutoffs are simply increased in length by adding to the west. The results of the simulations show that ground-water travel times are increased from 310 years for the pre-mitigative case to 540 years, 530 years and 525 years for the 2000 ft, 3000 ft, and 4000 ft cutoffs, respectively. Figure 6.7.3-3 compares the pathline produced by the 2000 ft cutoff with that for the pre-mitigated case. It can be seen that there is very little difference in the two paths, but due to the lowering of the gradient below the plant, the cutoff reduces the distance traveled in the first 100 years substantially. The pathlines for the longer cutoff designs, not shown, are practically identical to that for the 2000 ft cutoff. From these

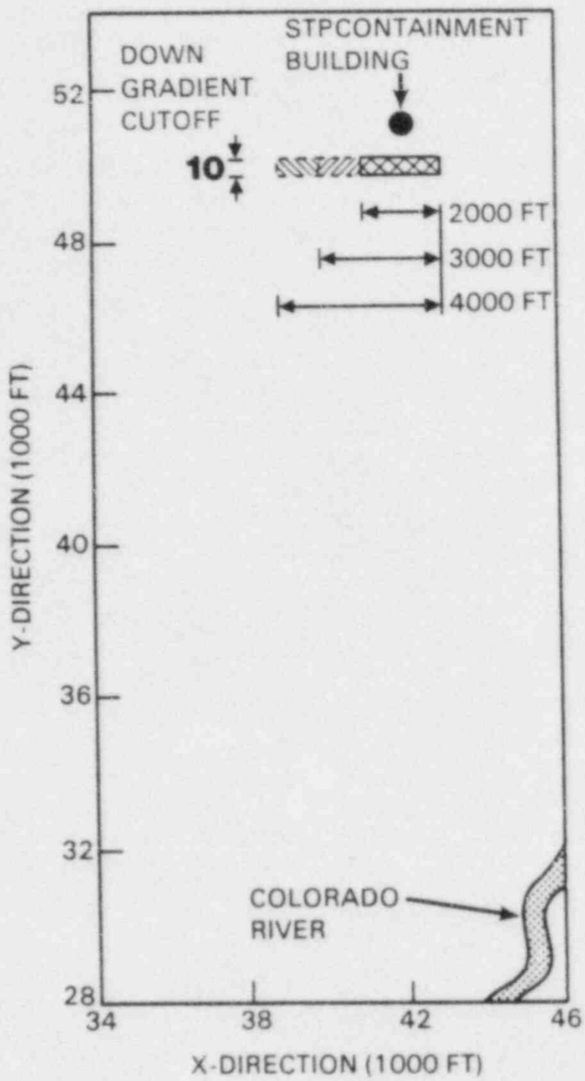


FIGURE 6.7.3-1. Location of Down-Gradient Cutoff Wall

results, it is apparent that for the flow field at the STP, once the up-gradient cutoff is sufficiently long to divert up-gradient flows and flatten the gradient at the site, there is no advantage to increasing the cutoff length to the west.

The ground-water mounding effects produced by the 2000 ft cutoff are illustrated by the potential surface in Figure 6.7.3-4. The maximum hydraulic head increase behind the cutoff, relative to the pre-mitigated condition, is about 1 ft. Thus no significant "bathtub effect" would be evidenced in the confined lower shallow zone aquifer.

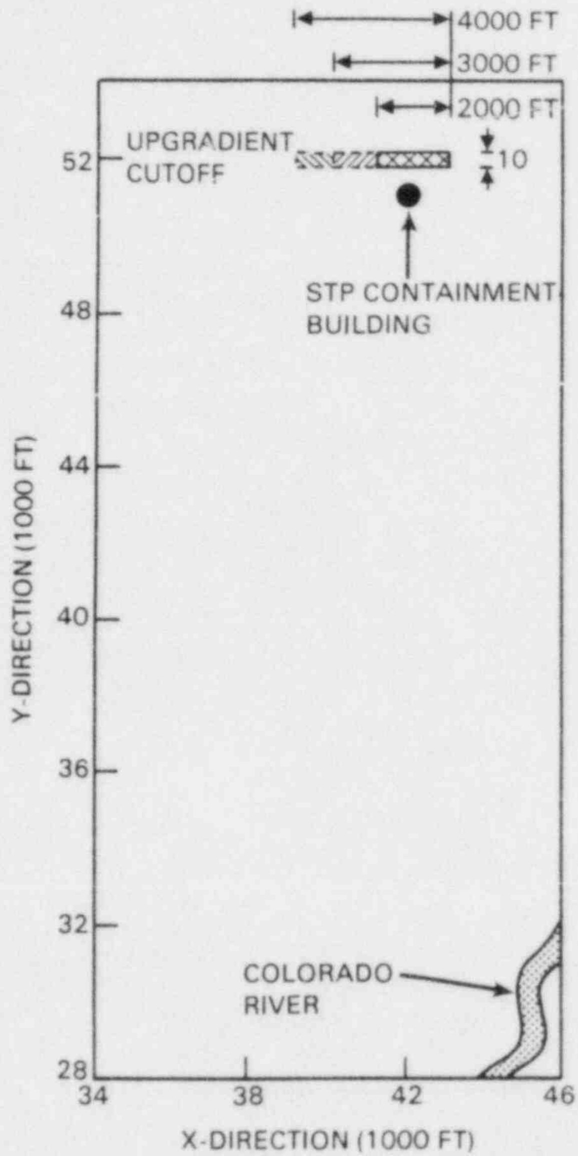


FIGURE 6.7.3-2. Location of Up-Gradient Cutoff Wall

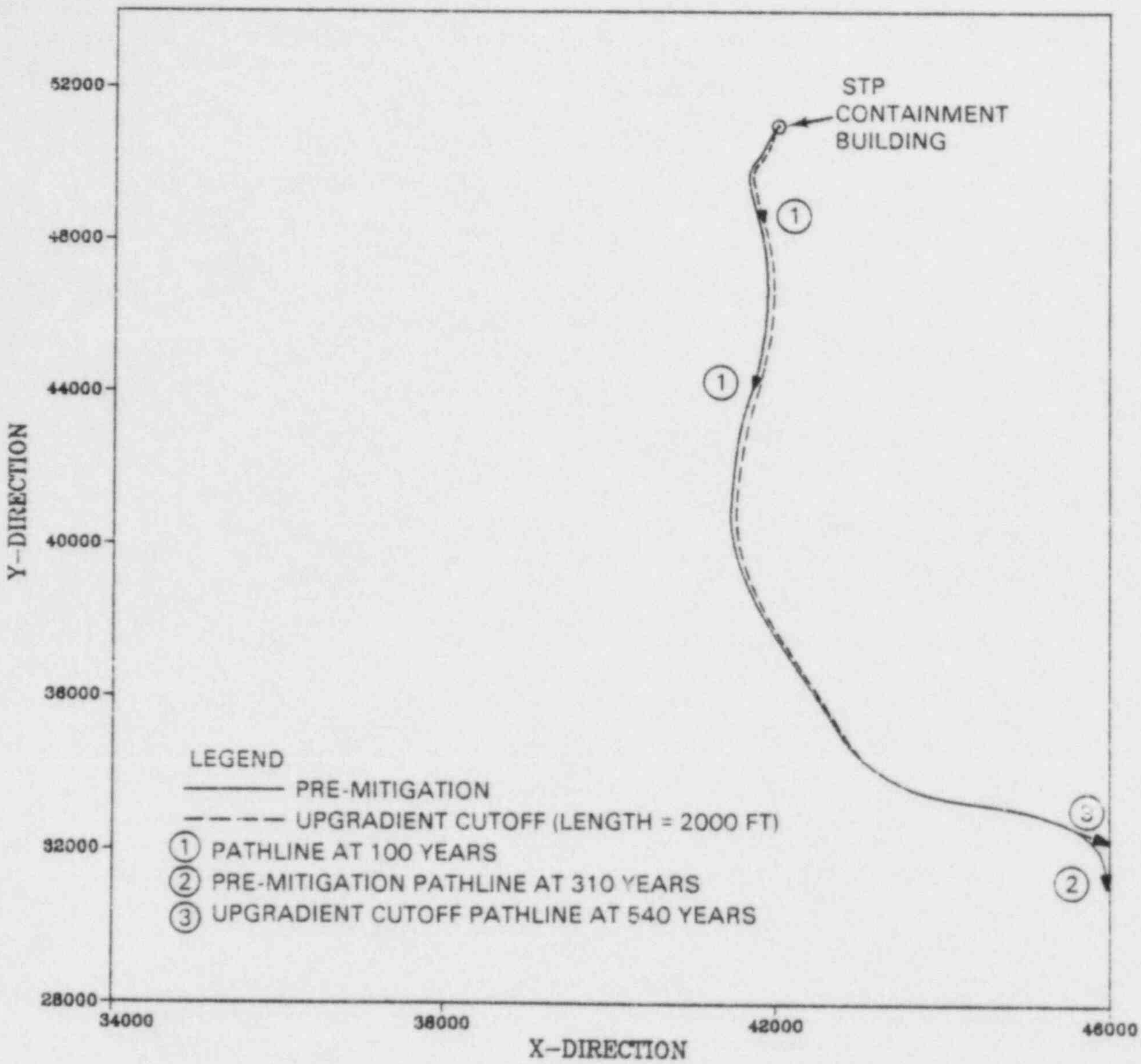


FIGURE 6.7.3-3. Pathline from the STP With 2000 ft Up-gradient Cutoff

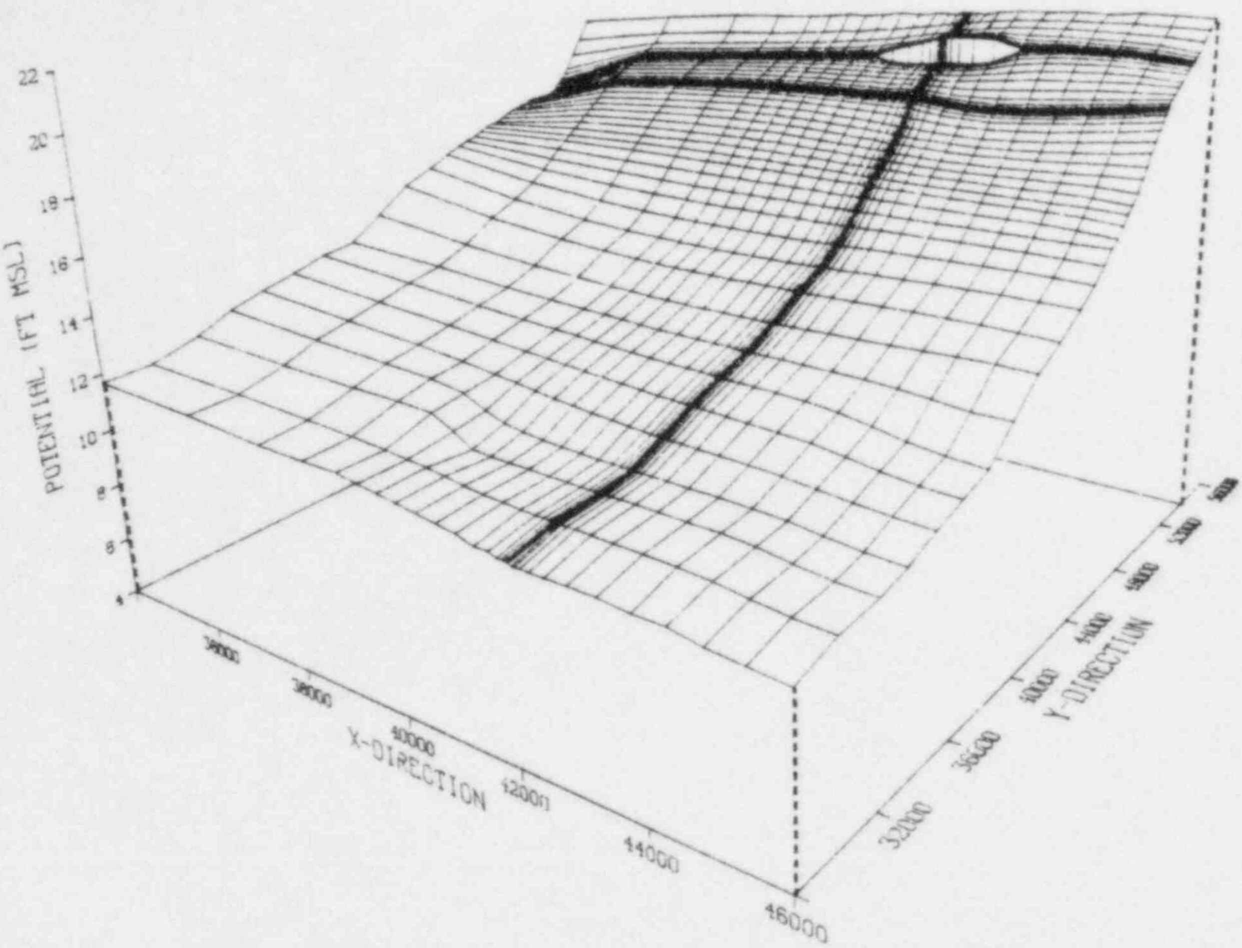


FIGURE 6.7.3-4. Simulated Potential Surface: With a 2000 ft Up-gradient Cutoff

The simulated travel time results for the down-gradient cutoffs, with increasing length, are 450 years, 475 years, and 565 years, respectively. The pathlines for the three cases shown in Figure 6.7.3-5, clearly demonstrate the circuitous routes produced by directly obstructing the pre-mitigative ground-water flow. In contrast to the up-gradient cutoffs, increasing the down-gradient cutoff length directly increases the path length and, tnerefore, increases the travel time.

Ground-water mounding effects produced by the 4000 ft down-gradient cutoff wall are illustrated by the potential surface presented in Figure 6.7.3-6. The maximum hydraulic head increase behind the cutoff is about 2 ft.

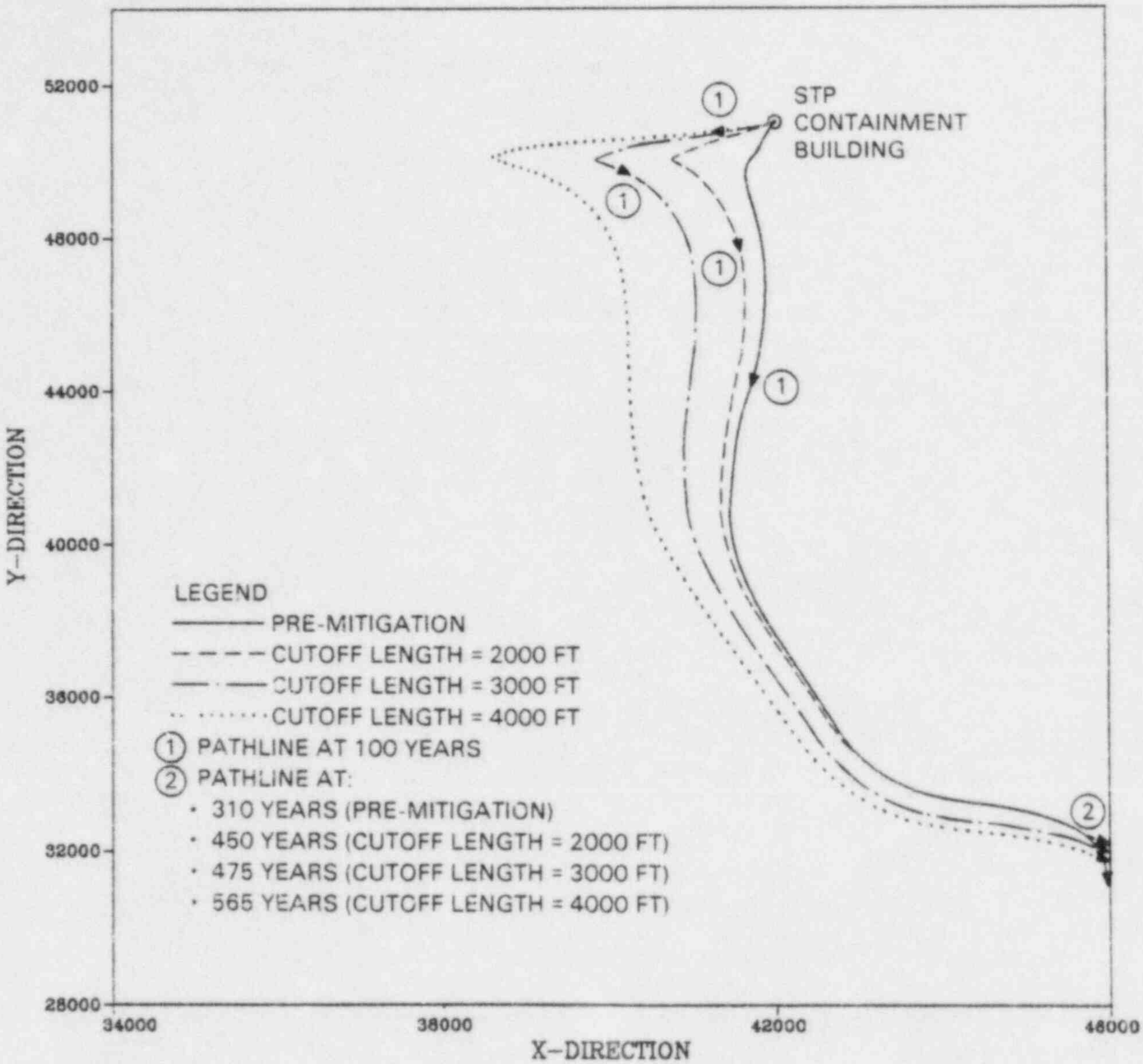


FIGURE 6.7.3-5. Simulated Pathlines from the STP: With Down-gradient Cutoffs

A summary of results for the six cutoff design evaluations is presented in Table 6.7.3-1. The table contains the design length, travel time and percentage increase in travel time for each cutoff design. In addition, to provide some perspective on how these designs would effect transport of radionuclides, an estimate is made of the radioactivity remaining in the ground water at the time of arrival at the Colorado River assuming an initial release at time zero of 1 x 10^{16} pCi of a hypothetical radionuclide having a 10,000 day half-life. In all cases, the increased travel times produced by the cutoffs results in a 2 to 3 orders of magnitude reduction in pCi remaining at the time of arrival.

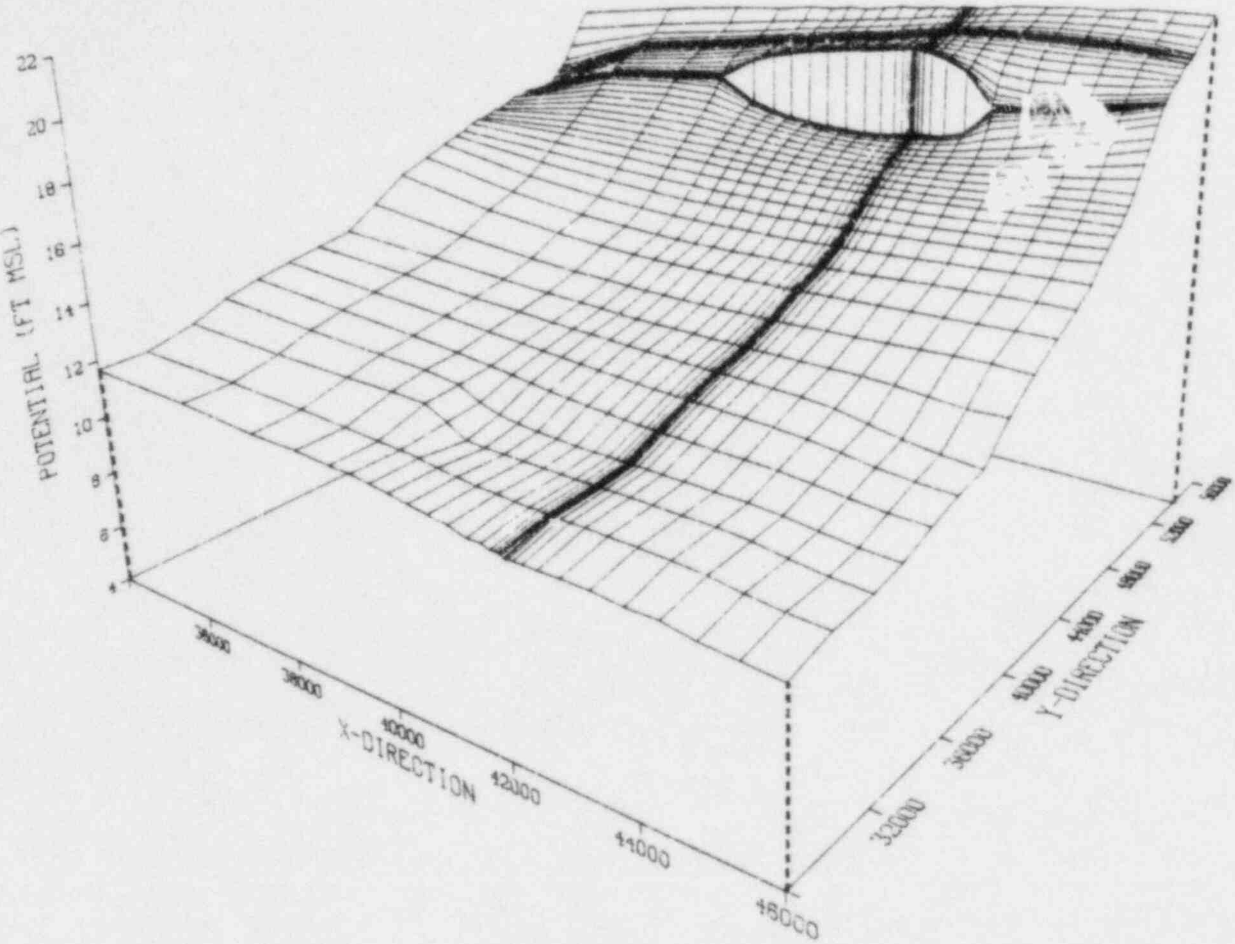


FIGURE 6.7.3-6. Simulated Potential Surface: With a 4000 ft Down-gradient Cutoff

6.7.3.2 Hydraulic Barriers

Hydraulic barriers to ground-water contaminant transport can often be created by ground-water withdrawal and injection which changes the potental surface in some advantageous way. The strategy selected for the STP was development of a hydraulic barrier to divert ground-water flow toward a less potentially hazardous route, that is a longer travel distance (thus greater decay) to the Colorado River. One of the main objectives is to divert the flow without withdrawing contaminated ground water and consequently creating a treatment or disposal problem.

In complicated ground-water flow regimes, where multiple wells may be installed to implement a hydraulic barrier, it can be very difficult to determine pumping rates that achieve the required level of mitigation through control of the trajectory of the contaminant plume. The method used to determine optimum steady-state pumping rates necessary to achieve a specific contaminant

TABLE 6.7.3-1. Summary of Cutoff Design Evaluations

Design	Cutoff Length, ft	Travel Time to Colorado River, yr	Increase Relative to Pre-mitigated Case	Radioactivity[a] Remaining, pCi
Pre-mitigation	--	310	--	3.9×10^{12}
Up-gradient Cutoff	2000	540	74%	1.2×10^{10}
	3000	530	71%	1.5×10^{10}
	4000	525	69%	1.7×10^{10}
Down-gradient Cutoff	2000	450	45%	1.1×10^{11}
	3000	475	53%	6.1×10^{10}
	4000	565	82%	6.2×10^{9}

(a) Assuming an arbitrary half-life of 10,000 days and a release of 1×10^{16} pCi at time zero.

control objective once the number and location of the wells have been established is as follows. A nonlinear optimization procedure is coupled with a two-dimensional, steady-state ground-water flow model (similar to TRANS) and a transient, advective contaminant transport model. The optimization algorithm drives the flow modeling component which in turn provides the hydraulic gradient information for the transport analysis. The whole process iterates on pumping rates until the rates that cause the contaminant trajectory and/or travel time to best meet a desired trajectory and/or travel time are determined.

The objective used in the above approach can be any sort of mathematical statement that describes the desired or mitigated contaminant transport. The objective can be in terms of the arrival location of the contaminant travel path and/or the arrival time of a contaminant trajectory, as long as the locations and times can be stated mathematically. This approach can be used to determine the pumping rates necessary to influence the potential surface (which controls contaminant transport) in a manner that will cause the contaminant to be diverted from the unmitigated (and potentially hazardous) travel path into a different and less hazardous trajectory.

Two general injection/withdrawal schemes for hydraulic barrier development at the STP are evaluated:

1. Near-field injection/withdrawal, and
2. Far-field injection/withdrawal.

The results of the evaluations are discussed below.

Near-field Injection/Withdrawal

Initially, a near-field scheme consisting of 10 wells, located as shown in Figure 6.7.3-7, was input to the optimization procedure. The objective stipulated for the optimization in simple terms was that the flow from the STP be directed away from the Colorado River while minimizing the injection rate. The result of the optimization indicated that only two of the ten injection wells are necessary to satisfactorily divert the contaminant trajectory. The specified injection rates were 34 gpm and 2 gpm, respectively. Considering the average yields for irrigation wells in Matagorda County of 1,955 gpm, and the high aquifer transmissivities and porosities, this appears to be an easily achievable injection rate. Further, 34 gpm could easily be withdrawn from the Colorado River and filtered. In this regard the cooling reservoir could also be used as a settling and/or storage basin.

The pathline for ground-water flow from the STP produced by the near-field scheme is shown in Figure 6.7.3-8. It can be seen that the travel time to the Colorado River is over 1300 years. The potential surface resulting from implementation of the near-field scheme is shown in Figure 6.7.3-9. The spike produced by the scheme represents a maximum increase in potential relative to the pre-mitigated case of approximately 2 ft.

Far-field Injection/Withdrawal

The initial scheme for the far-field case included three wells at the locations shown in Figure 6.7.3-10. Again the target was to divert ground-water flow away from the Colorado River with a minimum combined injection rate. In this case the optimization procedure results stipulated that Well No. 1 pump with a steady-state injection rate of 31 gpm. The simulation results show that the travel time to the model boundary is also about 1300 years, a factor of three times the pre-mitigated travel time to the Colorado River. Figure 6.7.3-11 shows the travel path with this scheme which exits the model region along the X-axis. The modified of the potential surface, as illustrated in Figure 6.7.3-12, though more pronounced than that for the other schemes, represents only a 4 ft increase in head relative to the premitigated case.

A summary of results for the injection scheme evaluations is presented in Table 6.7.3-2. The table contains the total injection rate, travel time to the Colorado River and percentage increase in travel time relative to the pre-mitigated case. Similar to what was presented for the cutoff designs, an estimate is made of the radioactivity remaining in the ground water at the time of arrival at the Colorado River assuming a hypothetical release of 1×10^{16}pCi of a radionuclide having a 10,000 day half-live. Both schemes produced a greater than three-fold increase in the travel time, reducing the remaining radioactivity by 10 orders of magnitude.

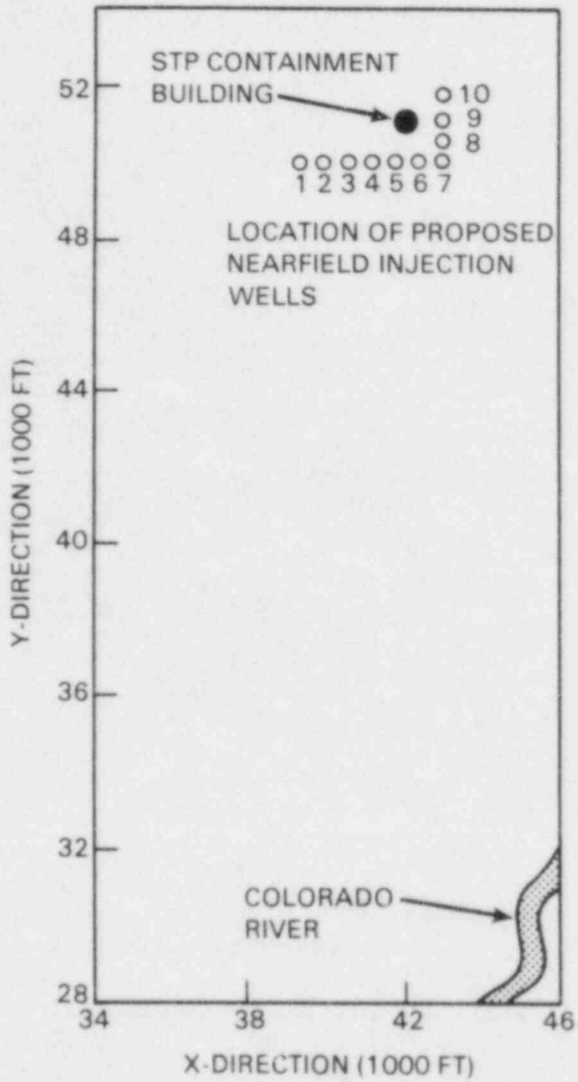


FIGURE 6.7.3-7. Proposed Near-Field Injection Scheme

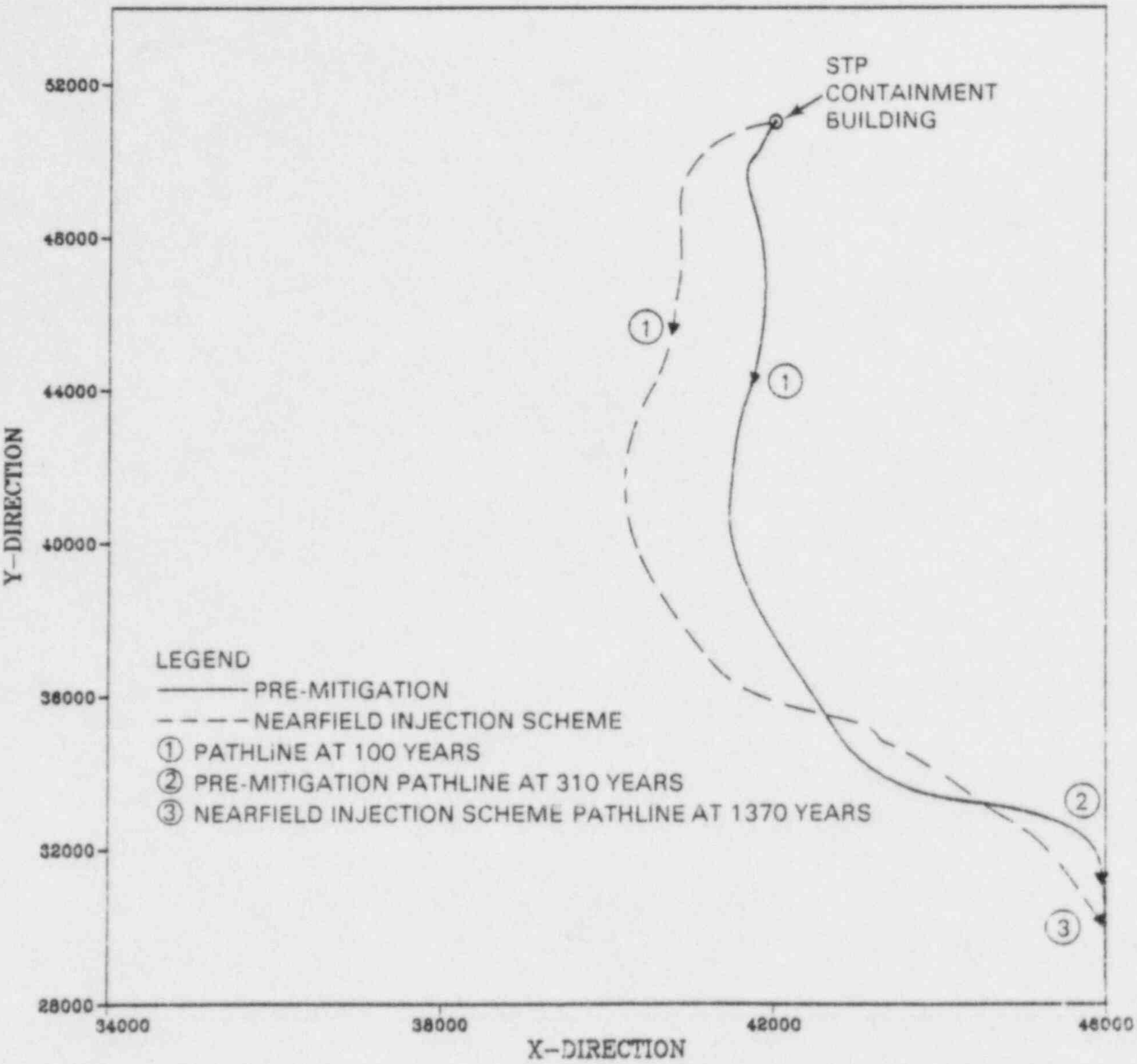


FIGURE 6.7.3-8. Simulated Pathline from the STP: With the Near-Field Injection Scheme

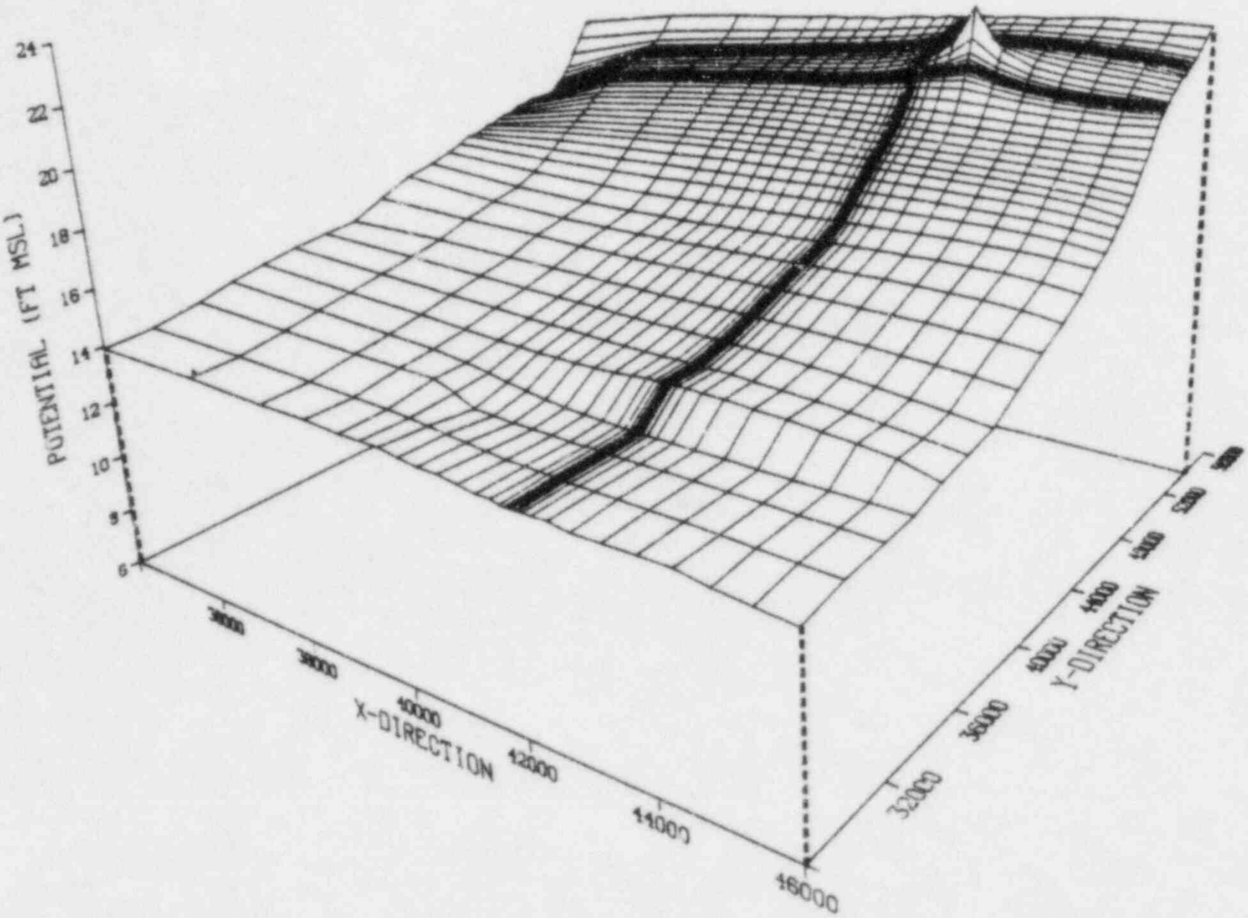


FIGURE 6.7.3-9. Simulated Potential Surface: With the Near-Field Injection Scheme

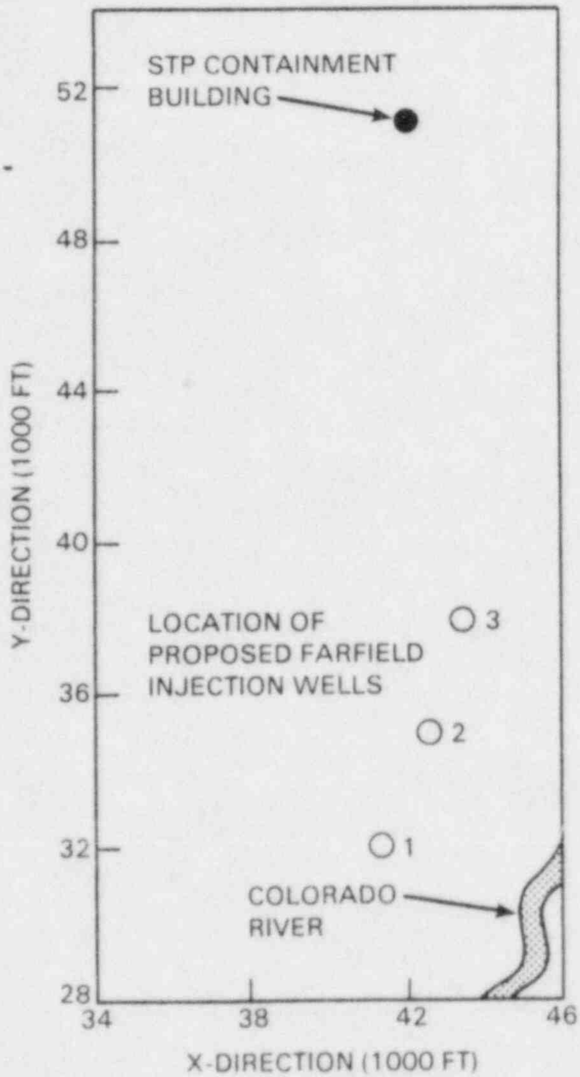


FIGURE 6.7.3-10. Proposed Far-Field Injection Scheme

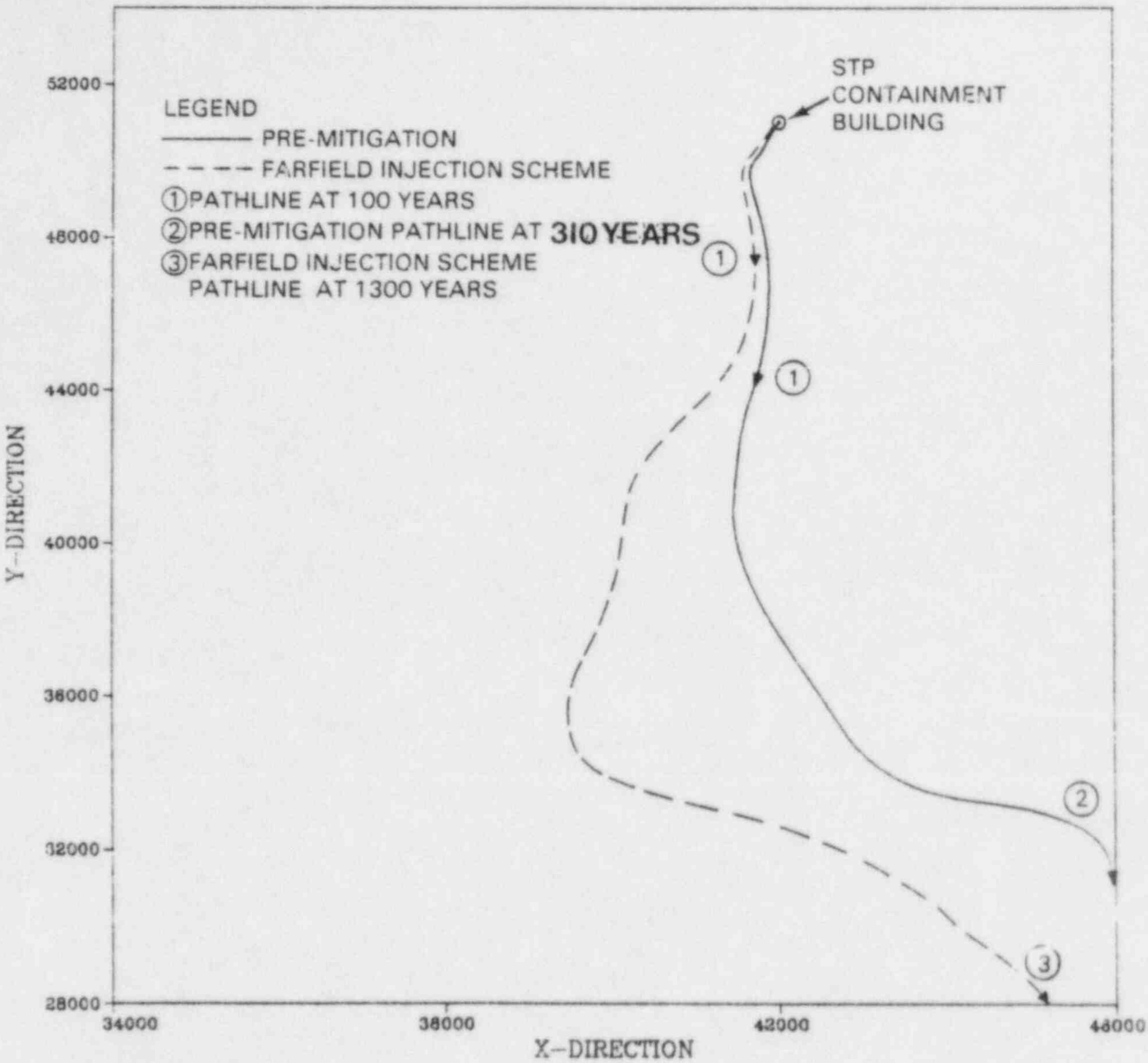


FIGURE 6.7.3-11. Simulated Pathline from the STP: With the Far-Field Injection Scheme

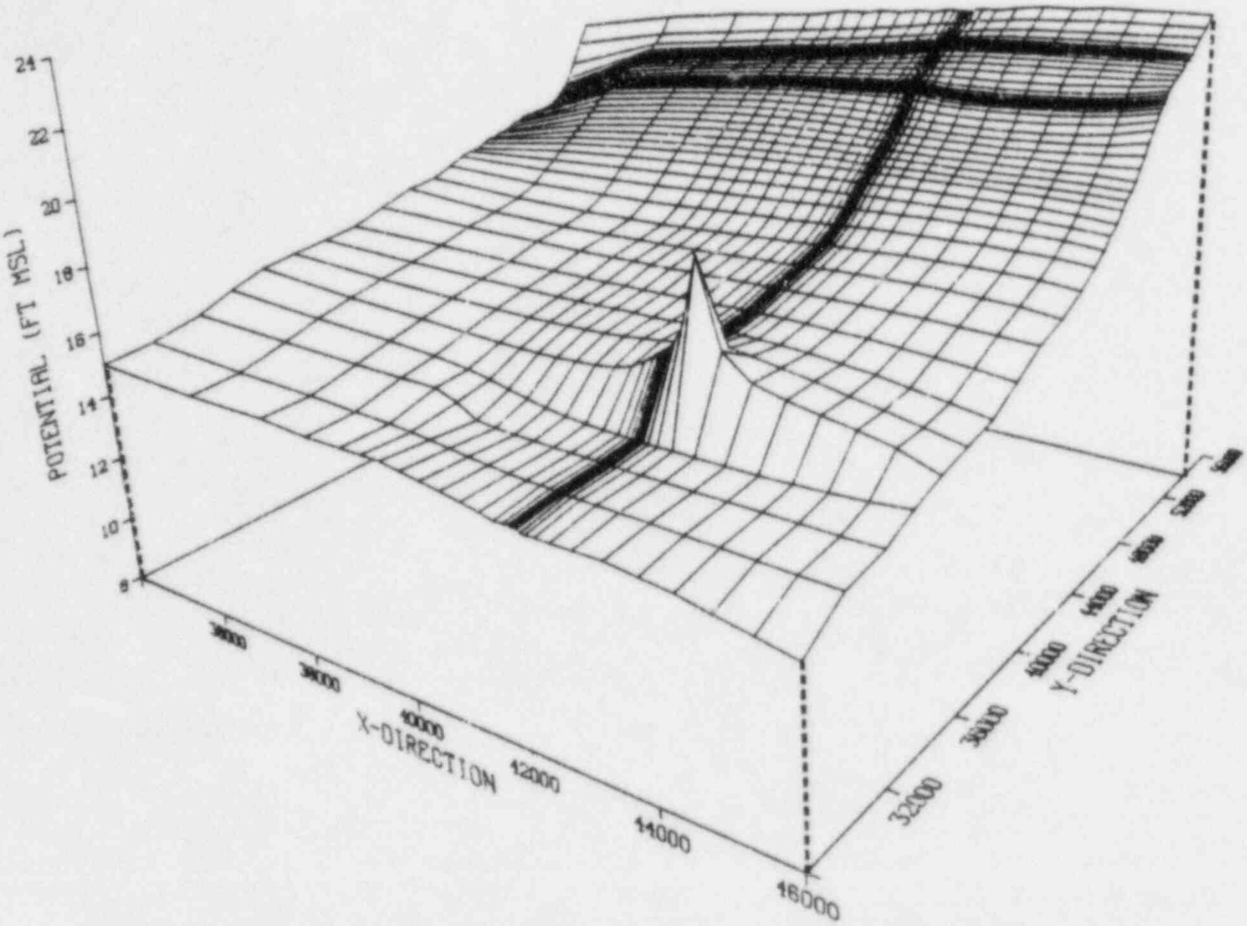


FIGURE 6.7.3-12. Simulated Potential Surface: With the Far-Field Injection Scheme

TABLE 6.7.3-2. Summary of Injection Scheme Evaluations

Scheme	Total Injection Rate, gpm	Travel Time to Colorado River, yr	Increase Relative to Pre-Mitigative Case	Radioactivity[(a)] Remaining, pCi
Pre-Mitigative	--	310	--	3.2×10^{12}
Near-field	36 gpm	1370	342%	8.9×10^{0}
Far-field	31 gpm	1300	320%	5.2×10^{1}

(a) Assuming an arbitrary half-life of 10,000 days and a release of 1×10^{16} pCi at time zero.

6.7.4 Conclusions

The primary objective of the STP case study is to develop and demonstrate general methodology for evaluating the desirability and feasibility of implementing ground-water contaminant mitigation strategies following a severe nuclear power plant accident. The study was conducted with readily available data sources including the STP Final Safety Analysis Report, regional hydrology reports, and the open literature. The level of technical detail attained in the case study results is commensurate with a reconnaissance or better level of analysis. The STP case study results include:

1. a detailed hydrogeologic characterization of a Texas Gulf Coastal Plain aquifer,

2. a complete discussion of data requirements, sources and procedures for the hydrogeologic characterization,

3. a two-dimensional ground-water flow and contaminant transport numerical model development based on the hydrogeologic characterization,

4. a baseline pre-mitigative analysis of radionuclide transport, and

5. a limited evaluation of the effect of selected engineered barriers and hydraulic barriers on radionuclide transport.

Major conclusions from the study results are the following:

1. flow and transport model simulation results show that following a severe accident at the STP ground-water radionuclide concentrations would be well below maximum permissible concentrations, therefore, mitigative action would not be necessary,

2. for the STP, all mitigation techniques evaluated significantly increased ground-water and contaminant travel times, and

3. model evaluations indicate that hydraulic and constructed barriers could prove to be effective in mitigating radionuclide discharges at the STP.

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7.0 SOUTH TEXAS PLANT - CASE STUDY NO. 2

7.1 INTRODUCTION

The South Texas Plant (STP) case study presented in this chapter is a continuation of analyses and results of Case Study No. 1 discussed in Chapter 6.0. The first case study focused on the hydrogeologic characterization and ground-water flow analysis for the STP. The study served to demonstrate the methods, procedures, and analyses necessary to evaluate, based on available information, the impact of mitigation on the ground-water flow regime of a specific site. Building on these results, Case Study No. 2 illustrates a more comprehensive (though not exhaustive) performance evaluation and trade-off analysis of mitigative strategy conceptual designs. Also provided is a discussion of the STP site configuration and the accompanying constraints the layout of plant facilities could have on the design, construction and implementation of mitigative measures. Referring to Figure 1.5-2, the STP case studies in composite are an illustrative example of a site-specific, reconnaissance level analysis and evaluation of strategies to mitigate the migration of radionuclides in a porous, unconsolidated geologic formation following a severe nuclear accident. It should be noted by the reader that much of the background information concerning the STP site characterization, data sources, etc., included in Chapter 6.0 are only summarized here. When appropriate, the reader is referred to specific sections in Chapter 6.0 if more detail is desired.

7.1.1 Case Study Objectives

The objectives of STP Case Study No. 2 are to utilize the conceptual and numerical models developed for STP Case Study No. 1 to accomplish the following:

- evaluate performance of an extensive array of mitigation alternatives including upgradient and downgradient engineered barriers (linear, L-shaped and U-shaped) and hydraulic barriers,

- assess the sensitivity of mitigation measure performance to design characteristics such as length, distance from the source, and effective barrier permeability,

- investigate the effects of hydrogeologic characteristics (e.g., hydraulic conductivity, retardation, dispersivity, etc.) on mitigation,

- consider the importance of the STP facilities' spatial configuration on mitigation measure design, and

- discuss cost as a factor in the evaluation and selection of mitigative strategies.

7.1.2 Case Study No. 2 Approach and Limitations

The two STP case studies were conducted based on the general methodology outlined in Section 6.1.3. Briefly, this methodology consists of four main steps:

Step 1. Survey of regional ground-water hydrogeologic characteristics and regional flow analysis to determine local boundary conditions.

Step 2. Pre-mitigative local ground-water flow and transport analysis.

Step 3. Performance evaluation of feasible mitigative techniques based on ground-water and contaminant transport simulation.

Step 4. Sensitivity analyses of contaminant transport to hydrogeologic parameters.

Part of the focus of Case Study No. 1 was to accomplish Step 1 above for the STP site. The available hydrogeologic data were reviewed and analyzed leading to the development of conceptual and numerical models for the STP site regional ground-water flow system (described in Sections 6.2, 6.3, and 6.4). Also within the first case study, a pre-mitigative local transport analysis and limited evaluations of selected mitigative techniques were conducted.

Taking advantage of the site characterization conducted for the first case study, Case Study No. 2 begins with a pre-mitigative flow and transport analysis and continues with a comprehensive performance evaluation of numerous mitigation alternatives and the sensitivity of their performance to specific hydrogeologic parameters. The TRANS two-dimensional ground-water flow and transport code (Prickett et al. 1981) was used throughout the two studies. A brief description of the capabilities and the main governing of the TRANS code are provided in Appendix C. As discussed in Section 6.1.4, only previously published data are used. Required data that are unavailable are estimated based on the best information available and/or engineering judgment. Other limitations to the analysis discussed in Section 6.1.4 apply equally to Case Study No. 2.

7.2 DEFINITION OF CASE STUDY NO. 2

As noted above, the reader is referred to Chapter 6 for a detailed description of the STP site location, reactor design and underlying ground-water system. In this chapter it is sufficient to present a brief summary of this information.

7.2.1 Physical Setting and Site Description

The STP is located in south-central Matagorda County, Texas adjacent to the Colorado River approximately 10.6 mi inland from Matagorda Bay, which opens into the Gulf of Mexico. The surrounding area consists of low relief, abandoned river valleys and marshes, and it is within the humid subtropical region of Texas. The plant is situated on the Pleistocene Beaumont Formation

which extends at least 700 ft below the site. The formation consists of clay, sandy clay, and thick sand units. Available data indicate there are two hydrostratigraphic units underlying the site: a deep aquifer at depths greater than 300 ft, and a shallow aquifer consisting of upper and lower units ranging between 90 ft and 150 ft below the land surface. The two units of the shallow aquifer are separated locally by a 20-ft thick clay layer that pinches out south of site (Houston Power and Light 1978). For the convenience of the reader Figure 6.3.2-17, an illustration of the conceptual model for the STP site, is included here as Figure 7.2.1-1. The figure depicts the underlying geologic units and the general directions of ground-water flow in the vicinity of the plant.

7.2.2 STP Facilities Description and Configuration

For several reasons, the general configuration of the STP facilities would be an important factor in the design and implementation of possible mitigative actions subsequent to a severe accident. The power station is composed of two identical pressurized water reactors (PWR). Several of the plant structures important to safe operation and shutdown of the plant (located in Figure 7.2.2-2) are shared by both units including the cooling reservoir, makeup pumping station, spillway and blowdown facilities, essential cooling pond, emergency transformer and switchyard (Houston Power and Light 1978). If continued operation of one unit is important, a key element of any mitigation design would be maintenance of essential functions such as reactor cooling and power transmission. For example, engineered barriers would have to be located outside the cooling reservoir and essential cooling pond to maintain their integrity as reliable sources of cooling water.

Plant structures also serve as physical obstacles that would influence location of engineered barriers (e.g., it might be difficult to construct a grout curtain or slurry wall within the switchyard). Another consideration would be the likelihood that water impounded by the cooling reservoir and essential cooling pond will be contaminated by atmospheric fallout. To prevent release of contaminated, reservoir water, it would be necessary to avoid damaging the impoundment embankments during mitigation construction.

Each of the principal site structures and their functions are discussed below. The influence each of these structures would have on design of specific mitigation measures is discussed in Section 7.5.

The STP site occupies approximately 12,220 acres with principal structures placed as shown in Figure 7.2.2-1. Each of the two units has a reactor core-rated thermal power of 3,800 MWt and net electrical power output of 1,250 MWe. The two units, located on a detailed plan view of the plant area in Figure 7.2.2-2, are approximately 600 ft apart. The reactor containment structures for both units are post-tensioned concrete cylinders with steel liner plates, hemispherical tops and flat bottoms. The cylinders have inside diameters of 150 ft, are 166 ft 3 in. high and 4-ft thick walls. The basemat is 18 ft thick. The containments are designed to withstand the internal pressure and temperature associated with the mass and energy release of a loss of coolant accident (Houston Power and Light 1978).

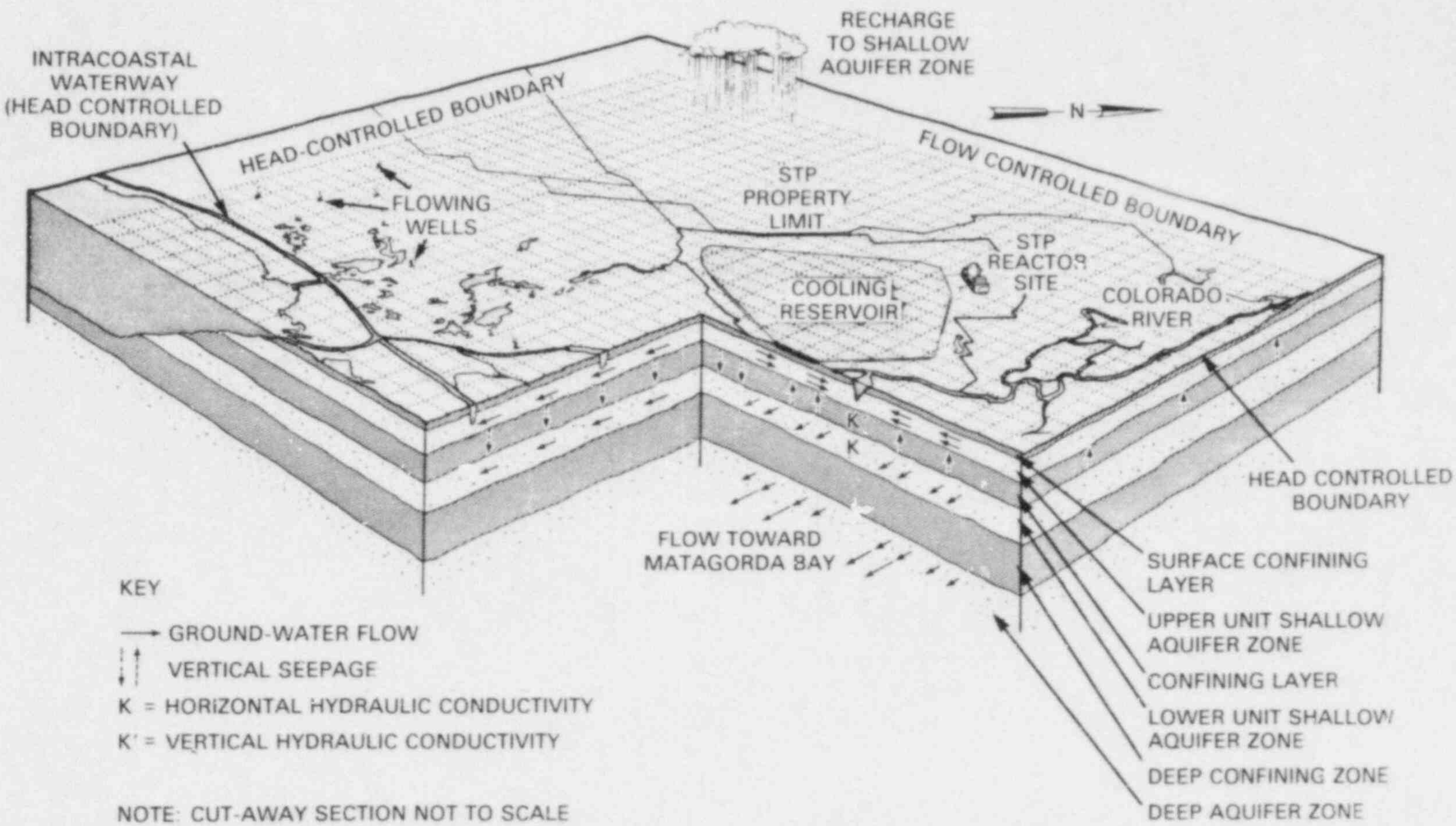


FIGURE 7.2.1-1. Illustration of the Conceptual Model for the STP Site.

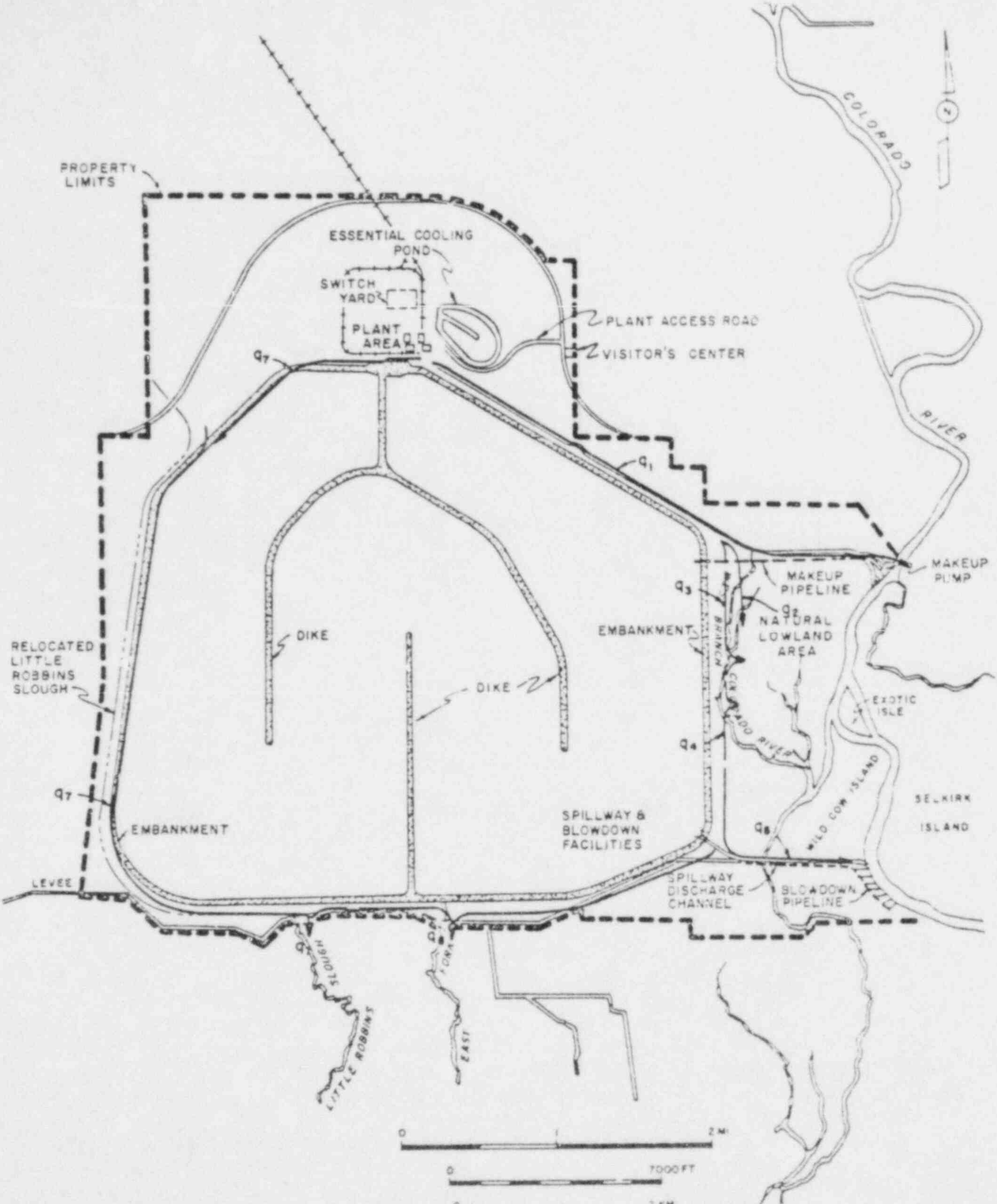


FIGURE 7.2.2-1. STP Plant Area (Source: Houston Power and Light 1978).

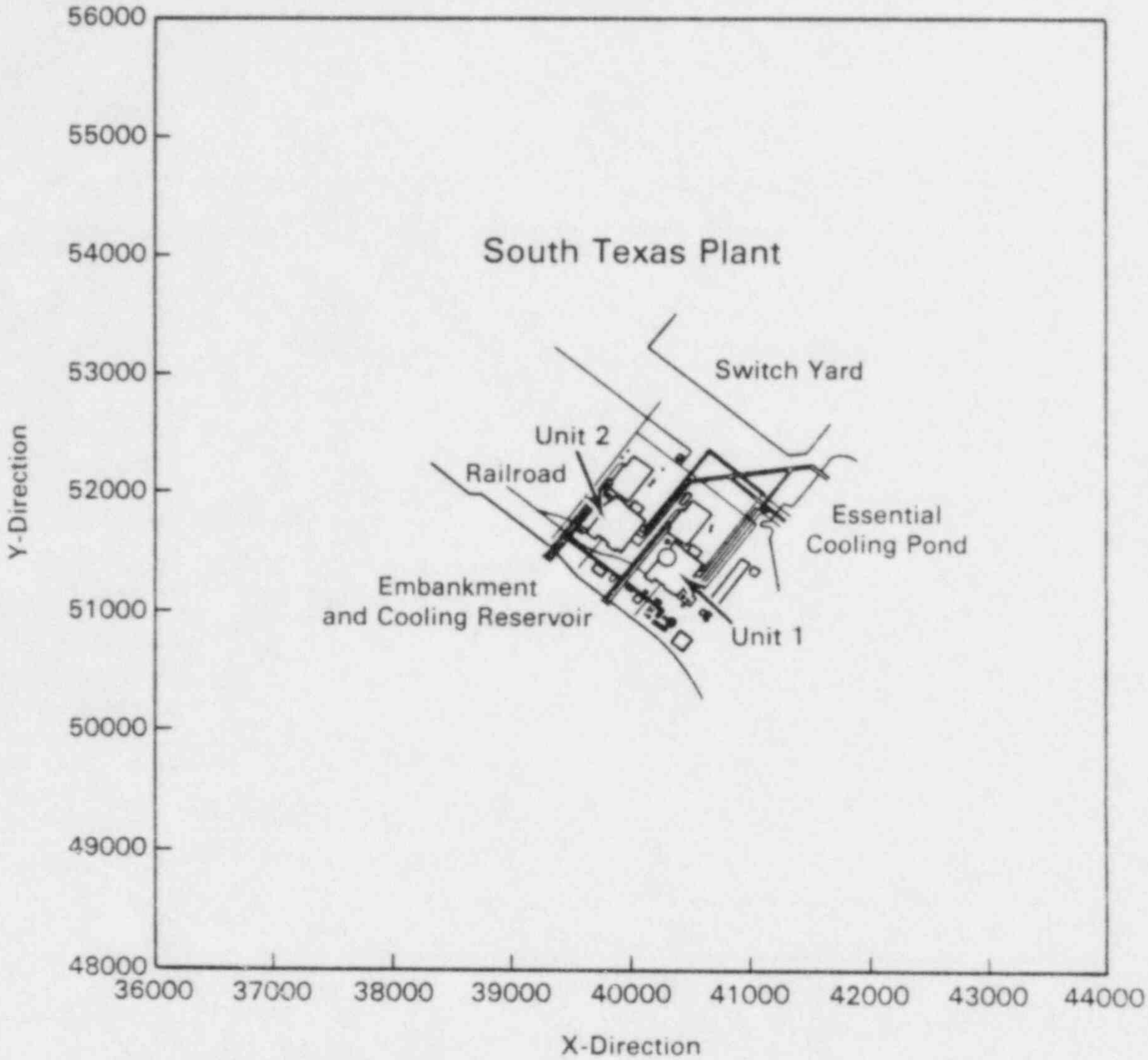


FIGURE 7.2.2-2. Detailed Plan View of STP Configuration.

The STP cooling reservoir covers an area of 7,000 acres and is contained within an 65,500-ft embankment having a crest elevation of 66.25 ft MSL (plant grade elevation is 28 ft MSL). The reservoir is over 3 mi long extending from its north embankment about 550 ft south of the reactor containment buildings to the south end of the STP property limit. The purpose of the reservoir is to dissipate the excess heat from the circulating cooling water. Condenser cooling water is discharged into the western half of the reservoir and directed along a circuitous path by a baffle system to the power plant intakes located in the eastern half of the reservoir. Water for the reservoir is supplied from the Colorado River. Blowdown to control reservoir water quality is discharged into the Colorado River near the southeast corner of the reservoir. The essential cooling pond is a 48-acre offstream impoundment located east of the plant area that derives water from the cooling reservoir and has a 530-gpm

backup well. The pond supplies cooling water for safe shutdown of the plant during normal operations and serves as the ultimate heat sink under postulated accident conditions. The STP's transmission system facilities, located north of the plant area, include a 345-kV switchyard and a 138-kV emergency transformer. The switchyard covers a rectangular area of about 12 acres. When power is unavailable from the main generator, the system provides reliable power for simultaneous normal shutdown of both units or concurrent shutdown of one unit and a design basis accident in the other (Houston Power and Light 1978).

7.2.3 Definition of Accident Scenario

The STP is a PWR incorporating a double loop for removal of heat from the reactor core. As discussed in Section 6.2.3, the postulated severe accident for the STP occurs when insufficient heat is removed from the reactor and the core materials overheat to the melting point. The hot core materials could then melt through the concrete basemat of the reactor containment building, allowing radioactive debris to enter geologic materials below the plant (USNRC 1975). The debris containing nuclear fuel, steel and liquefied geomaterials would begin cooling and solidifying once it entered the substratum. A period of approximately one year would be required for sufficient cooling of the debris to occur to allow ground water to flow through and around the melt mass and begin transporting contaminant away from the site (Niemczyk et al. 1981).

In addition to core debris, another possible source of contamination following a severe accident at the STP is the cooling water and water used in emergency spray systems. This water (referred to as sump water) could collect in the containment building sump, become contaminated in the accident process, and be released to the stratum beneath the plant.

The depth of penetration of the core melt into the earth below the containment basemat is a function of the accident sequence, size of the reactor and the chemical composition of the geologic materials. The clay and sand units underlying the STP are composed primarily of silicic minerals. The shape of the core melt penetration into silicate material has been calculated by Niemczyk et al. (1981): the core debris would be approximately cylindrical with a radius of roughly 29 ft at a depth of 35 ft below the basemat. At the STP site, this depth would coincide with an elevation of about 80 ft below MSL. Thus, the core debris would reside in the lower unit of the shallow-zone aquifer (see Section 6.3.2 for a detailed characterization of the STP site hydrogeology). The deep aquifer, which is the source of fresh water in the region, would be isolated from the core melt by a 150-ft-thick clay confining layer. Thus, it is assumed that the lower unit of the shallow aquifer would transport the majority of the radionuclides away from the site. Figure 7.2.3-1 is a schematic showing the final configuration of the core melt debris relative to the reactor basemat and the underlying units.

For the purposes of pre- and post-mitigation contaminant transport analysis following a severe accident at the STP, the focus is on radionuclides that: 1) have long half-lives and would not decay to low levels very soon after an accident, or 2) are in large quantities that are not strongly sorbed and

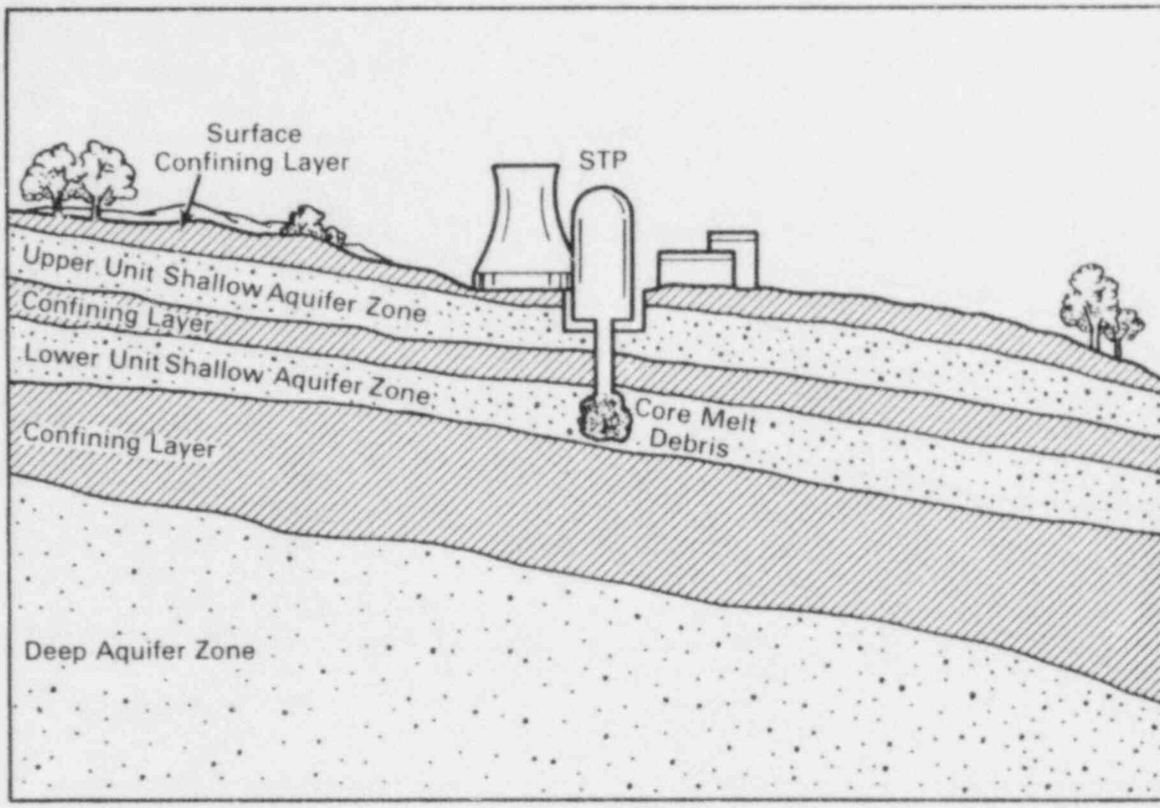


FIGURE 7.2.3-1. Assumed Location of the Core Melt Debris Subsequent to a Severe Reactor Accident at the STP.

thus could potentially migrate away from the site in high concentrations. As discussed in Section 6.2.3, strontium-90 was selected as the most appropriate indicator radionuclide on which to base pre- and post-mitigation transport analyses at the STP. The hypothesized leach release rate of strontium-90 into the shallow aquifer as a function of time following the initiation of leaching (approximately one year after the accident) is plotted in Figure 7.2.3-2. This curve includes only the core melt debris leach release and was determined assuming 4.53×10^{18} pCi of strontium-90 are initially present following a severe accident for a single unit of the STP and that 89% of the strontium-90 is contained in the core melt debris leach release and the remaining 11% in the sump water release.

7.3 MODEL DEVELOPMENT

The ground-water flow and transport analyses for the STP site are accomplished using a two-stage modeling approach. The first stage utilizes a course grid regional hydrologic flow model. The purpose of the regional model is to establish boundary conditions for the local model under both pre- and post-mitigation conditions. The local model is then used to simulate the ground-water system in the immediate area of the plant in greater detail.

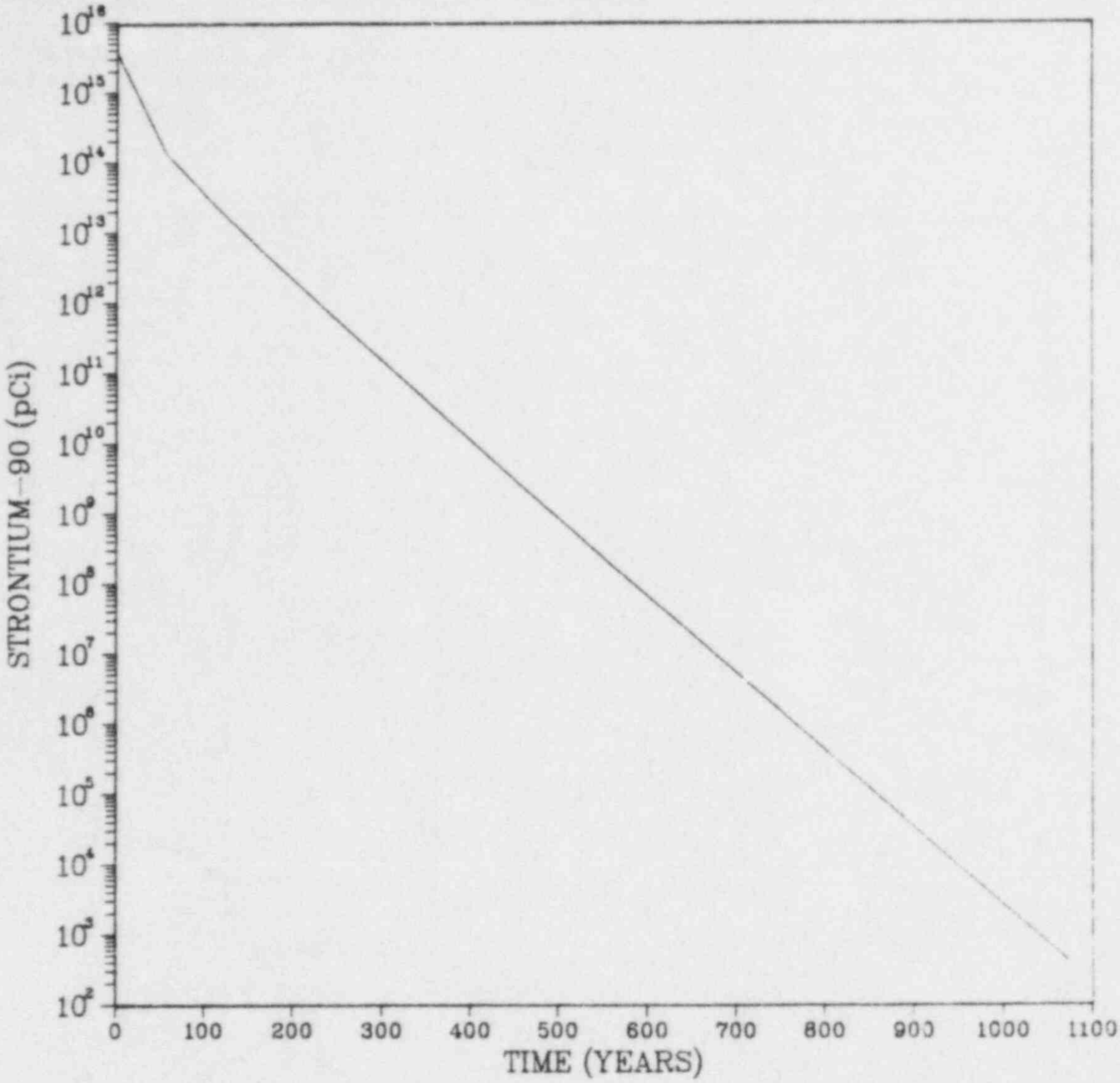


FIGURE 7.2.3-2. Hypothesized STP Leach Release of Strontium-90.

While the regional model used in this case study is the same as that developed for STP Case Study No. 1 without modification, a modified, higher resolution local model is employed.

Development of the regional model is discussed in Section 6.4, including: use of available data, model parameter values selected, and model calibration results. The new local model allows consideration of the STP plant facilities configuration and provides greater flexibility in siting mitigation measures (i.e., engineered and hydraulic barriers) within the model grid. This is accomplished by reducing the areal extent of the local model from 11.2 square miles (12,000 ft x 26,000 ft) to 2.3 square miles (8,000 ft x 8,000 ft) and by increasing the number of grid nodes from 33 x 58 to 55 x 63. The location and extent of the new local study area relative to the regional model grid and

original local study area is delineated in Figure 7.3-1. Figure 7.3-2 shows the STP facilities superimposed on the new local grid. The reader is reminded that the model grids are oriented such that the general ground-water flow direction is approximately parallel to the y-direction.

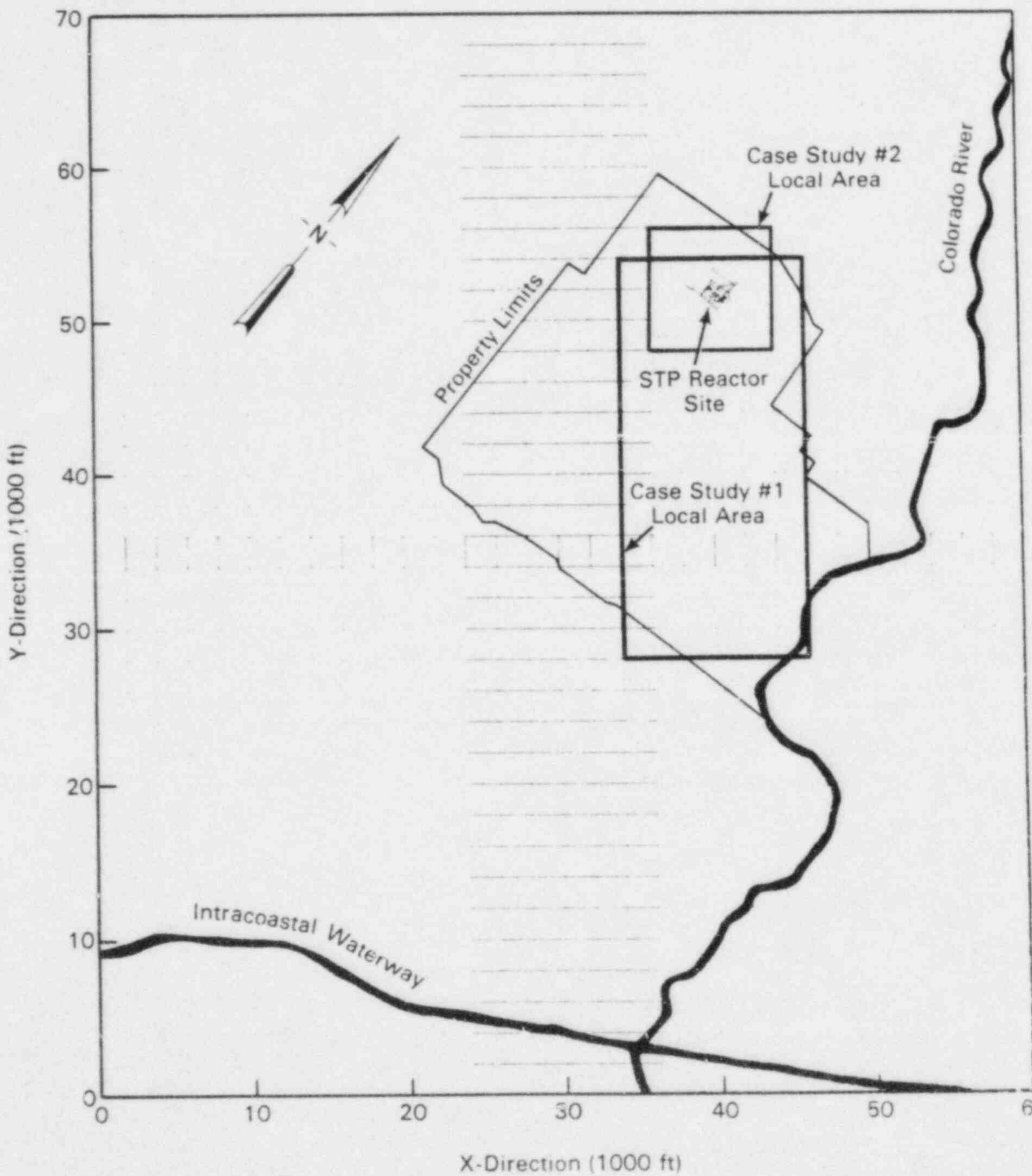


FIGURE 7.3-1. STP Case Study No. 2 Local Study Area.

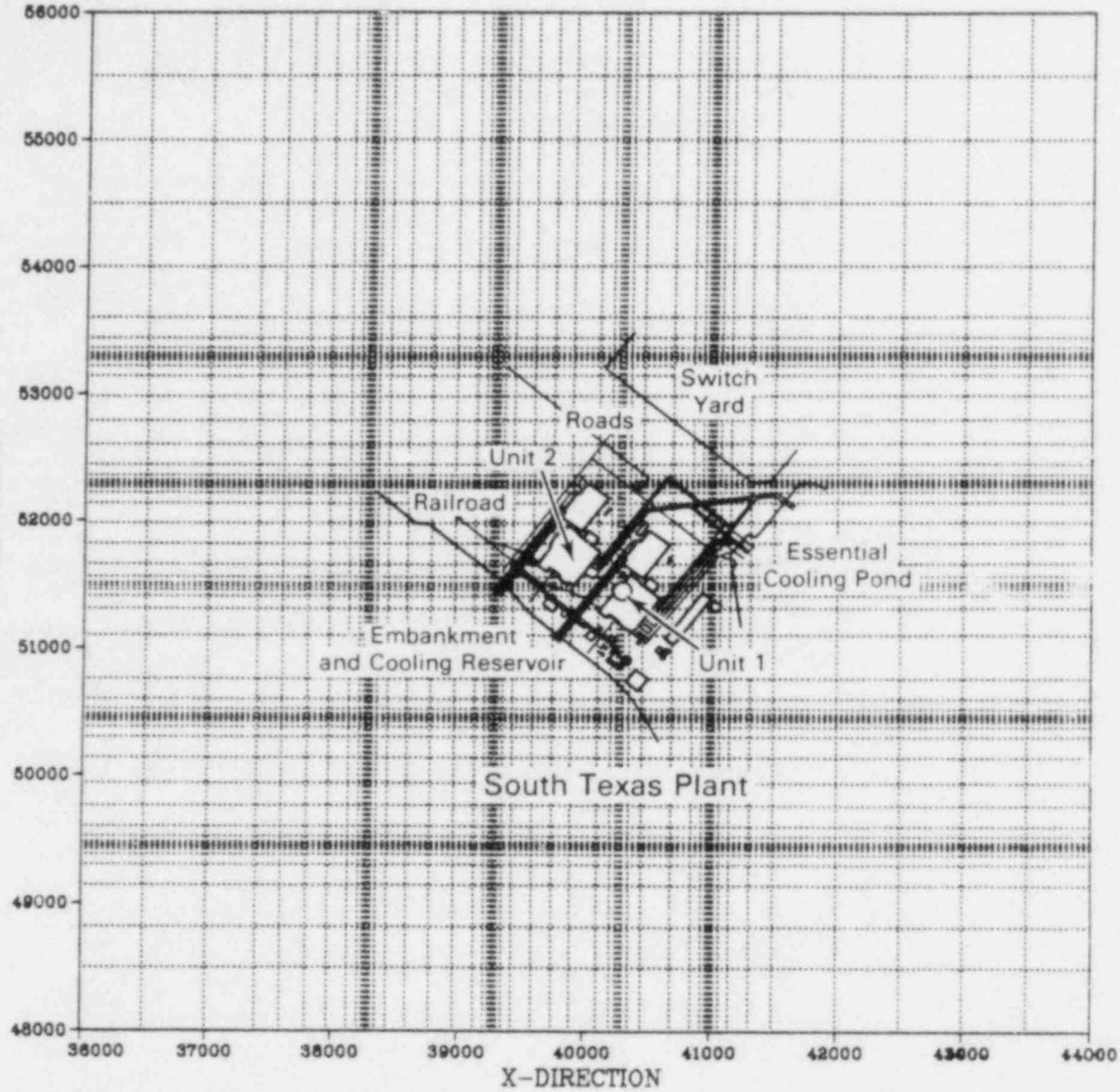


FIGURE 7.3-2. STP Case Study No. 2 Local Model Grid.

The boundary and initial conditions for the new local model are determined directly from the regional model. The procedure followed is to first run the regional model and then determine the potentials for the local model boundary from the regional model simulation results. Other local model parameters were also interpolated directly from the regional model including the lower shallow-zone aquifer top and bottom, hydraulic conductivities and the recharge/discharge rates from/to the upper shallow-zone aquifer. Potential contours for the local study area (plotted in Figure 7.3-3) vary somewhat uniformly from about 22 ft MSL at the upper boundary to 17 ft MSL at the lower boundary, producing a relatively flat average gradient of about 0.0006 ft/ft. The spatial distribution of hydraulic conductivities is illustrated by the

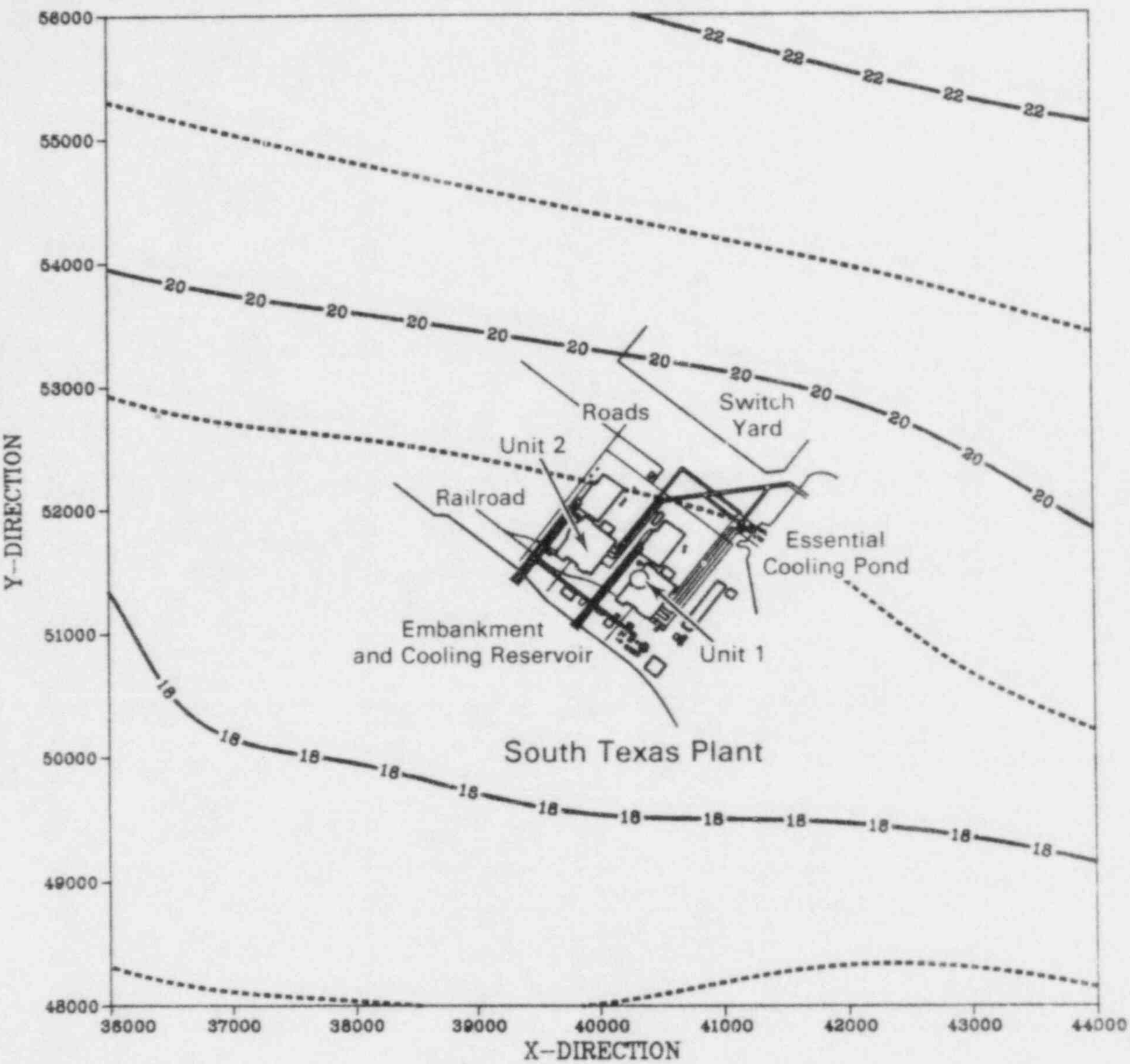


FIGURE 7.3-3. Observed Potential Contours For the Local Study Area.

three-dimensional surface plot presented in Figure 7.3-4. In contrast to the regional model distribution in Figure 6.4.3-1 which show distinct discontinuities (as determined through calibration), the surface for the local hydraulic conductivities includes the minor smoothing effects due to the transfer to the local model by interpolation. The variation in conductivities in the local area is characterized by a region of high values of about 1,500 gpd/sq ft in the center of the local area extending to the west boundary. Values are lower over the rest of the area, generally ranging from about 200 to 500 gpd/sq ft. In the immediate vicinity of the plant (noted in Figure 7.3-4) the conductivity is high and decreases in the down-gradient direction. Vertical ground-water movement within the local area is from the upper zone of the shallow aquifer

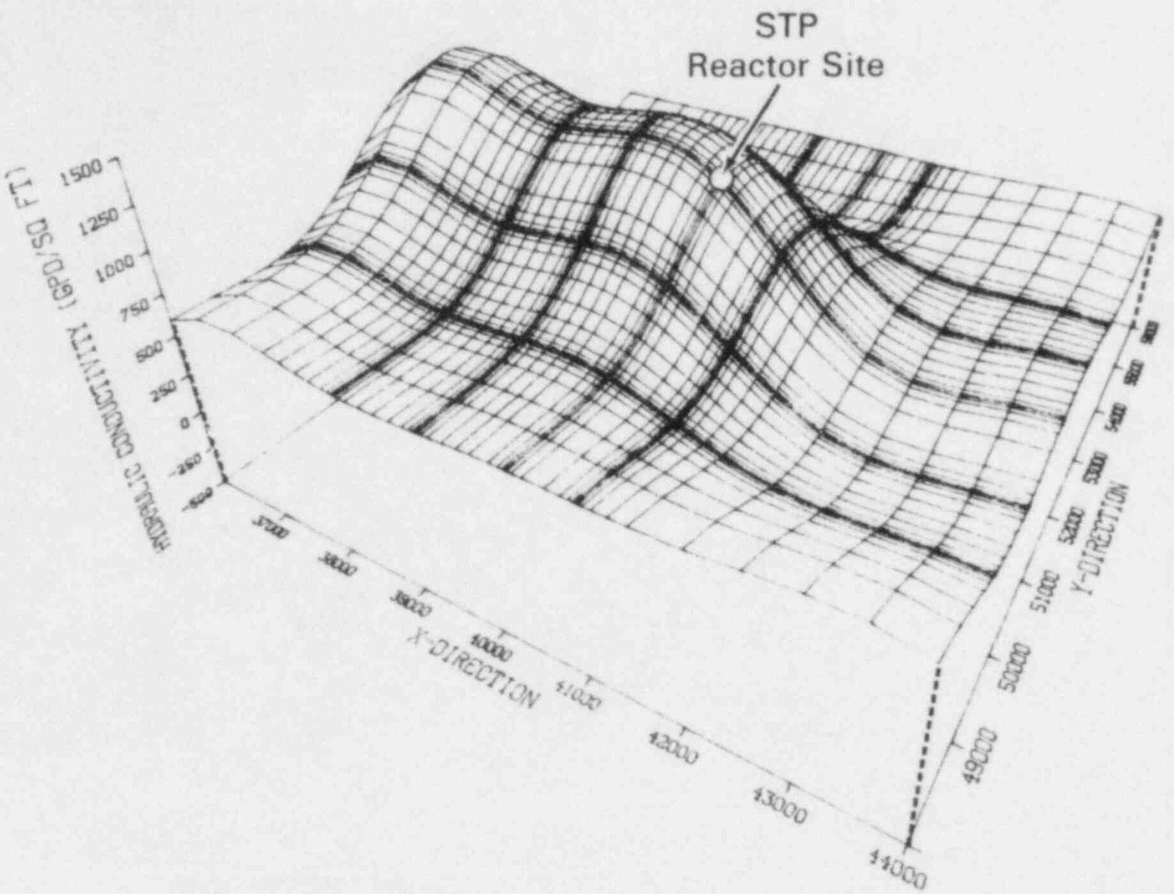


FIGURE 7.3-4. Hydraulic Conductivities for the STP Local Model.

into the lower zone (i..e., recharge to the study aquifer) and is spatially fairly uniform. The estimated average recharge rate over the local area, based on calibration of the regional model (see Section 6.4.3) is approximately 0.0005 gpd/sq ft or about 0.3 in/year.

As a check on the interpolation process used in transferring parameters and aquifer properties from the regional model to the local model, the potential contours were simulated by the local model and compared to the observed values. The simulated potentials are plotted in Figure 7.3-5. The greatest difference between the two sets of contours occurs in the center of the model area where the simulated results are about 0.5 ft lower than the observed. Overall the simulated and observed values compare favorably, providing a psuedo-verification that the local model adequately represents the ground-water flow system and can be useful in evaluating pre- and post-mitigation analyses.

Because of the total lack of observed radionuclide transport data at the STP site, estimates of transport modeling parameters (i.e., soil bulk density, effective porosity, retardation factor, and dispersivity coefficients) are

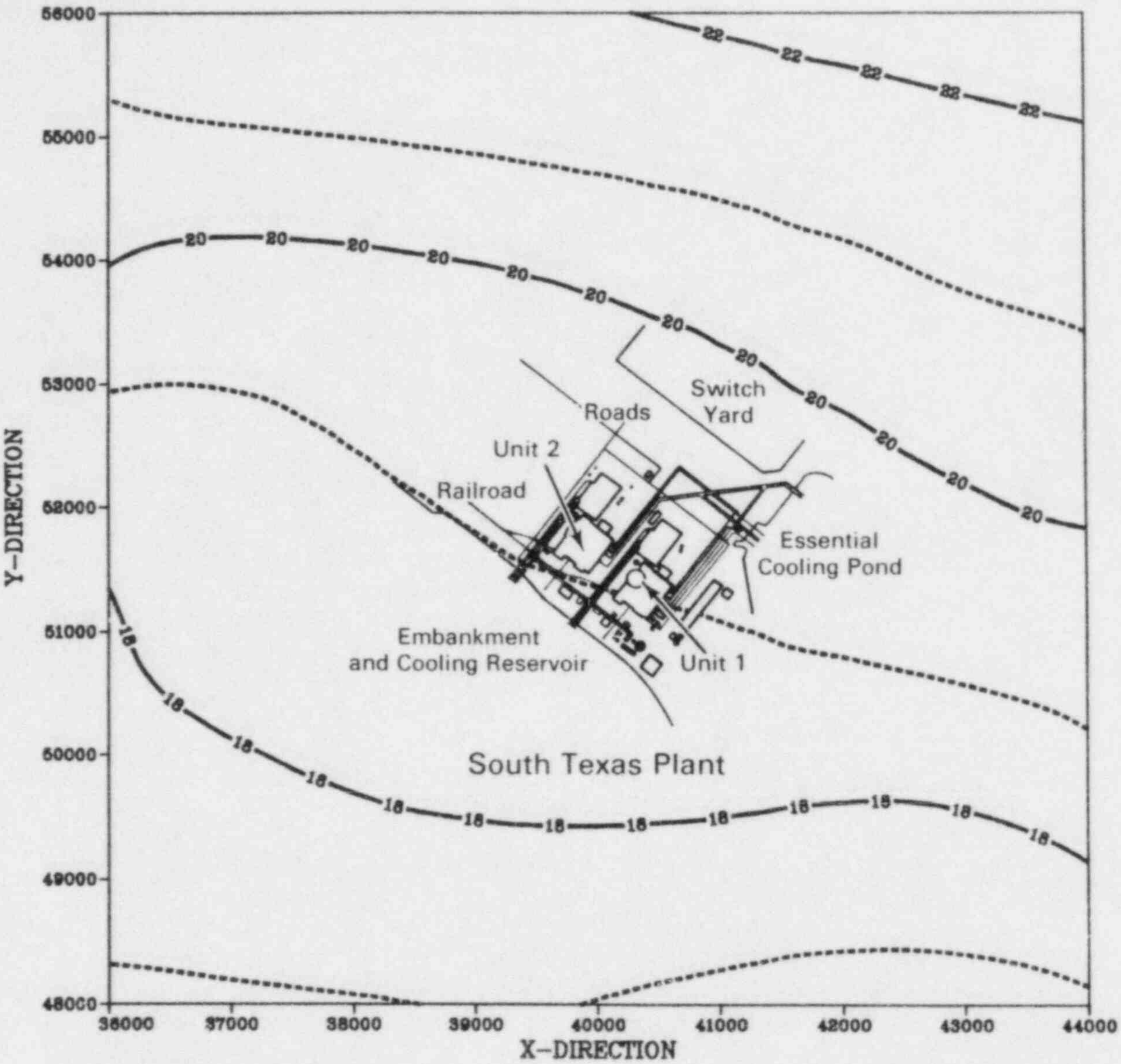


FIGURE 7.3-5. Simulated Potential Contours for the Local Study Area.

based entirely on previously published information. The values used for the local model utilized in Case Study No. 2 are the same as those used in the first case study with one exception. The values used and the information source for each are discussed in Section 6.6.1 and listed in Table 7.3-1. The only change to this list is the increase of the number of particles (NP) from 5,000 to 10,000; therefore, individual particles represent half as much contaminant, providing greater resolution for analyzing the transport simulation results. A sensitivity analysis of these parameters and their effect on mitigation performance is presented in Section 7.5.

TABLE 7.3-1. Summary of TRANS Transport Parameters for STP Case Study No. 2 Transport Simulations

Parameter	Value	Source
Longitudinal Dispersivity (D_L)	164 ft	Gelhar and Axness (1981)
Transverse Dispersivity (D_T)	8 ft	Computed; DL/20
Distribution Coefficient (K_d)	10 ml/g	Table 3.3.2-2
Effective Porosity (n_e)	37%	STP FSAR [Houston Power and Light (1978)]
Bulk Density (ρ_b)	10^4 lb/ft^3	Computed
Retardation Factor (R_d)	46	Computed
Number of Particles (Np)	10,000	Prickett et al. (1981)

7.4 PRE-MITIGATIVE LOCAL TRANSPORT RESULTS

The purpose of the a severe accident pre-mitigative transport analysis is twofold:

1. quantitatively assess the need for mitigation, and
2. when mitigation is found to be necessary, provide a baseline for evaluating relative mitigation performance.

On the basis of Case Study No. 1 results it was determined that after 1000 years subsequent to the assumed severe accident, transport of significant quantities of radionuclides was limited to a distance of approximately 2400 ft. The results also indicate that by 1000 years dilution and natural decay would reduce the maximum concentration within the contaminant plume to about 20 x 10^{-4} pCi/ml, a level well below the maximum permissible concentration of 0.3 pCi/ml set for strontium-90 by 10 CFR Part 20 (USNRC 1978).

However, as previously noted, the STP still provides a vehicle far analyzing mitigation performance. Conclusions regarding the need for mitigation in Case Study No. 1 were based on the prevention of significant radionuclide releases to the Colorado River. The new local model, which encompasses a smaller area with greater spatial resolution, facilitates incorporation of an alternative objective for possible mitigation measures, (i.e., containment of radionuclide contamination within, or close to the immediate plant area). Such would be the case if it were determined that the ultimate course of action in response to a severe accident was site restoration (i.e., removal and safe storage of contaminated material). It would then be desirable to minimize, through mitigation, the quantity of geologic material contaminated by migrating radionuclides. In light of this possibility, the pre-mitigation radionuclide transport for the STP was reanalyzed using the new local model to determine the extent of the unmitigated contaminant migration

with time and to provide the basis for evaluating the relative benefits (i.e., reduction in contaminant flux) derived from selected mitigation measure designs. The approach used to assess the pre-mitigated condition at the STP was to simulate the steady-state flow condition and the transient leach release and transport of the strontium-90.

The results of the pre-mitigation analysis are consistent with those of Case Study No. 1. Three-dimensional plots showing the simulated spatial distribution of strontium-90 concentration at 100 years and 1000 years following the accident are presented in Figures 7.4-1 and 7.4-2, respectively. Similar to the results obtained from the Case Study No. 1 local model, these plots illustrate significant reduction in concentrations with time due to dilution and natural decay. In addition, both models indicate that the combination of fairly flat potential gradient in the study aquifer and the sorption of strontium-90 limit the extent of migration to approximately 2000 ft downgradient of the reactor site.

In analyzing the pre-mitigation simulation results as the baseline for evaluation of mitigation alternatives, the objective of the mitigation is very important. As discussed above, the assumed purpose of the mitigation measures in Case Study No. 2 is to limit the areal extent of radionuclide migration to allow future contaminant removal and site restoration at minimal risk to the environment and minimal cost. Figures 7.4-1 and 7.4-2 illustrate qualitatively the concentration levels and lateral extent of the pre-mitigative radionuclide movement at their respective times. These plots do not provide for quantitative comparison between pre-mitigative and post-mitigation results. Therefore, the approach used to assess mitigation performance is to determine the resulting contaminant flux as a function of time at a common downgradient location. The section 800 ft downgradient from the reactor site was selected as the location for the flux determinations for all cases, and is referred to as the "breakthrough section." This section is sufficiently distant from the site while being upstream of the downgradient mitigation measures to be evaluated. The effectiveness of a given mitigation measure in reducing contaminant flux serves as an index to its performance in contamination containment. For example, Figure 7.4-3 shows superimposed on the source release curve, the pre-mitigated flux (pCi/yr) of strontium-90 at the breakthrough section. The travel time for strontium-90 to the breakthrough is greater than 200 years. The maximum flux rate is about 6.2×10^{9} pCi/yr and decays to less than 3×10^{5} pCi/yr by the year 1000. The main implication from the pre-mitigated results is that on the order of 200 years are available for implementation of mitigation. Also, if site restoration were desirable prior to that time the contamination would be limited to a distance of less than 500 ft from the plant.

Implementation of mitigation measures to reduce the flux rate at the breakthrough section, in effect, also meets the proposed mitigation objective of decreasing downgradient migration. Furthermore, impeding movement of the radionuclides serves as an in situ treatment process by allowing more time for natural decay to occur close to the source. The significance of decay in analyzing radionuclide transport is illustrated by the two curves in Figure 7.4-4. The lower curve, pre-mitigated flux 800 ft downgradient of the source, is the same as shown in Figure 7.4-3. The upper curve is the flux that

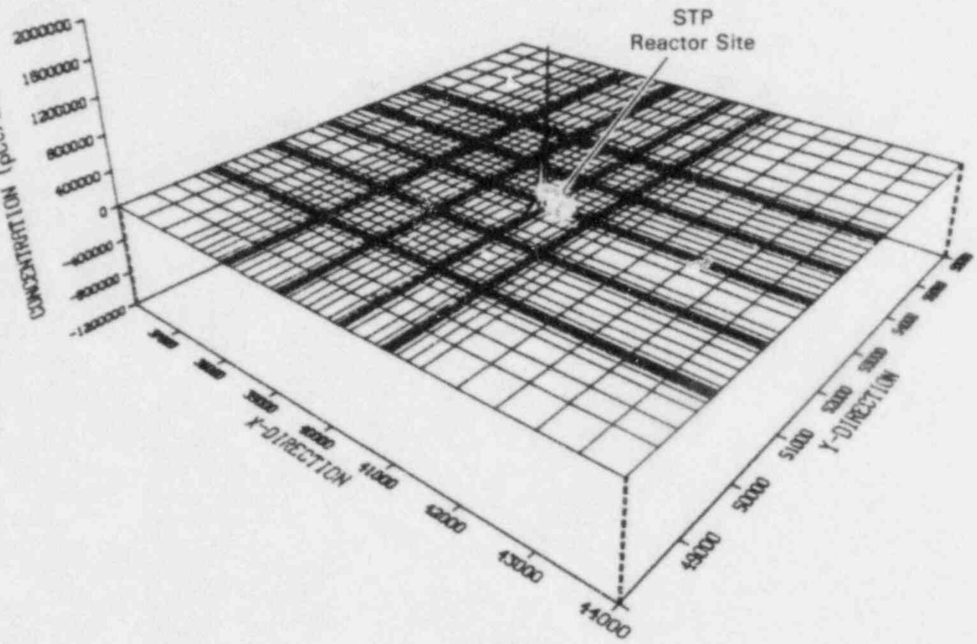


FIGURE 7.4-1. Simulated Pre-Mitigation Strontium-90 Concentrations at 100 Years.

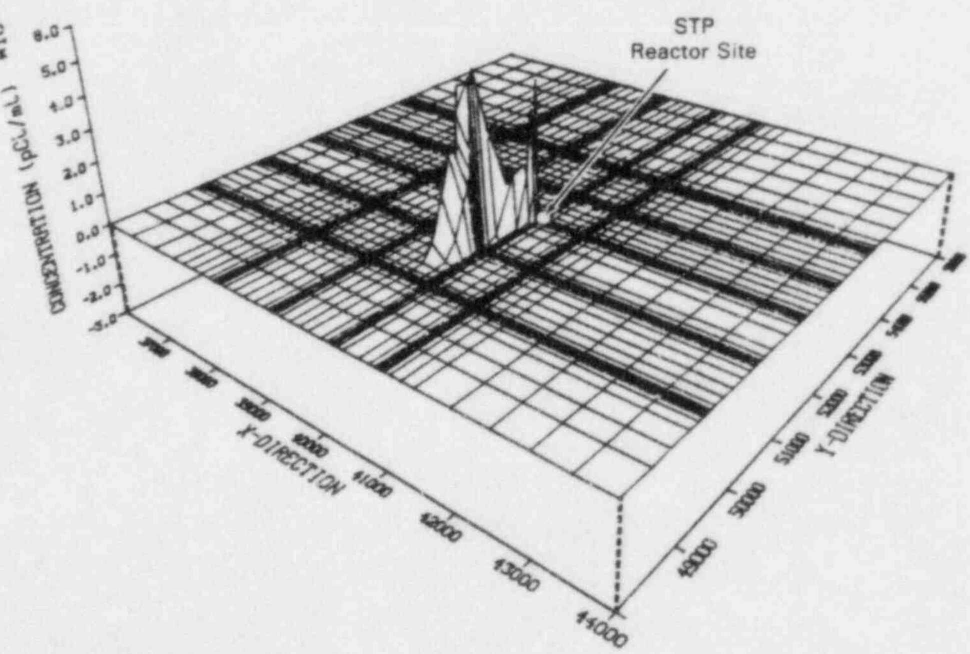


FIGURE 7.4-2. Simulated Pre-Mitigation Strontium-90 Concentrations at 1000 Years.

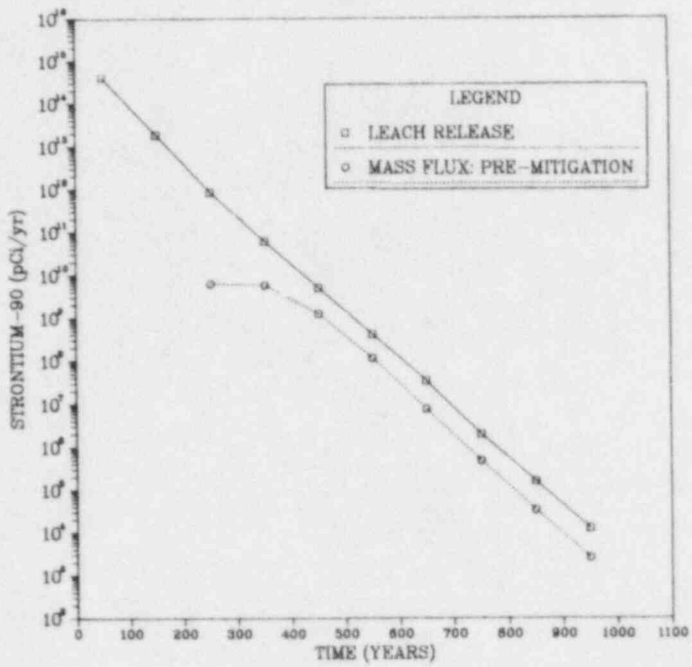


FIGURE 7.4-3. Pre-Mitigation Strontium-90 Flux Rate with Time 800 ft Down-Gradient of STP Reactor Site.

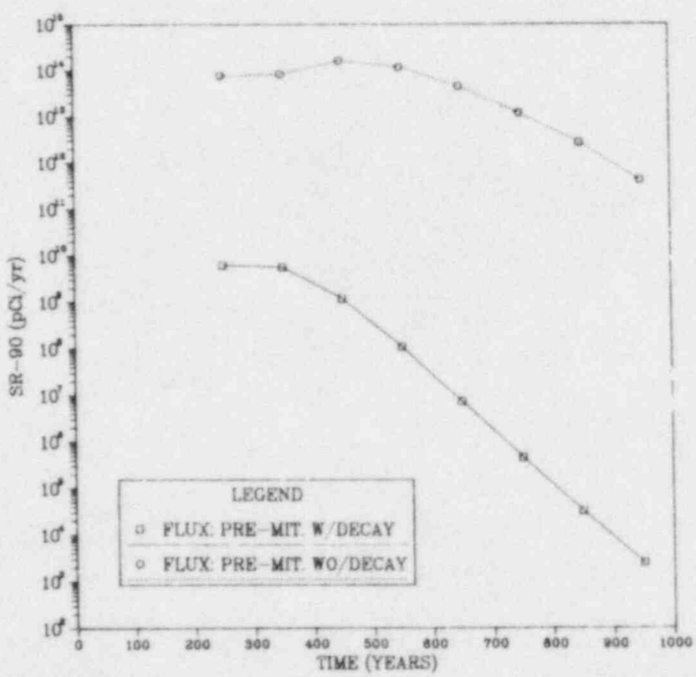


FIGURE 7.4-4. Comparison of Pre-Mitigation Strontium-90 Flux Rate With and Without Natural Decay.

would occur assuming the same release curve, flow velocities, retardation, etc., but without including natural decay with time. The non-decaying curve shows the first arrival of the plume, the passing of the peak and the effects of the continually decreasing source release rate. The curve with decay on the other hand, is much lower, produces maximum values early and continually decreases at a rate slightly greater than the release curve. Clearly the beneficial effects of natural decay point to an overall mitigation strategy that takes advantage of decay by delaying transport of radionuclides away from the source, limiting the areal extent of contamination and reducing the flux levels from the site.

7.5 EVALUATION OF MITIGATIVE TECHNIQUES

The first step in evaluation of mitigative techniques involves preliminary screening of those methods that clearly are not feasible given site-specific hydrogeologic conditions. A broad spectrum of measures may be considered for a particular site including grout cutoffs, slurry cutoffs, hydraulic barriers or interceptor trenches and treatment beds. The applicability of each of these approaches to the STP site is discussed in Section 6.7.2. To summarize, the ground-water contaminant mitigative techniques that appear most suitable for implementation at the STP, based on the reconnaissance level hydrogeologic characterization and pre-mitigative ground-water flow and transport analysis in Chapter 6.0, are:

1. a fully penetrating and properly keyed grouted cutoff, and

2. a hydraulic barrier to ground-water flow and transport created by injection.

The general material properties of the shallow zone aquifer indicate that the upper- and lower-zone aquifers could be successfully grouted with the intervening clay layer acting as a key-in and a natural ground-water flow barrier. A schematic illustrating an in-place grout cutoff such as might be used at the STP is presented in Figure 7.5-1. The average permeability of the host material at the site is 85 ft/day which falls in the middle of the "easy" to grout range of permeabilites listed in Table 5.6.2-1. Combined with the low average ground-water velocity (less than 0.1 ft/day), the relatively high permeability facilitates successful chemical grouting of ground-water cutoffs. The soil size limitations on grout permeation presented in Figure 4.3.1-1 indicate that silicate grouts may be most suitable for the STP site.

Acceptable injection rates should be achievable at the STP due to the relatively high porosity (0.37) and high permeability. Because the lower unit of the shallow aquifer is relatively deep (125 ft) and confined, deep wells with high capacities would most likely be installed. The wells would be screened only in the lower shallow aquifer zone to prevent contamination of the upper unit. The Colorado River serves as a readily available source of injection water. Other alternatives include location of a high volume

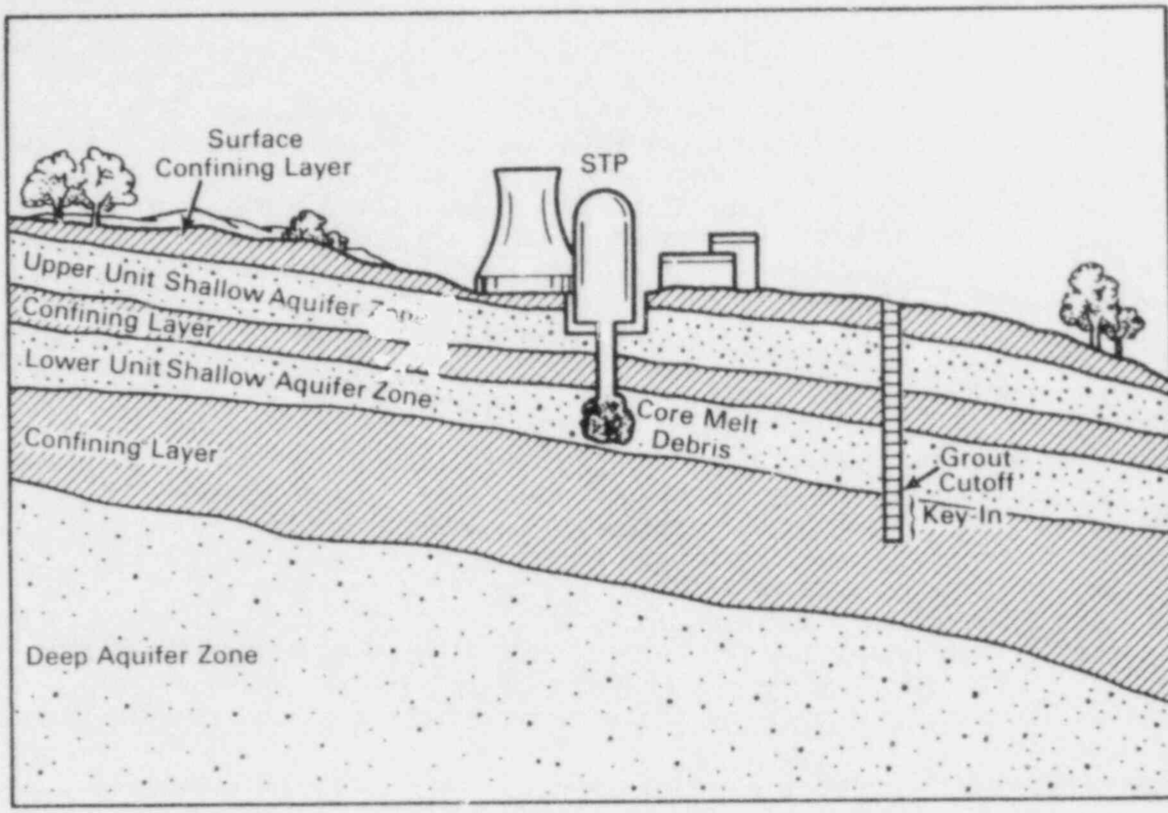


FIGURE 7.5-1. Schematic of Typical Grout Curtain Placement for the STP.

discharge well a suitable distance away from the STP or use the STP cooling reservoir as both a source of injection water and storage basin, depending on the level to which it would be contaminated by atmospheric releases of radionuclides. Injection strategies would be preferable to withdrawal because of the need for disposal of contaminated water discharged by withdrawal schemes.

While grout cutoffs and hydraulic barriers are identified as the most feasible techniques for the STP, there are an unlimited number of alternative designs utilizing these techniques individually or in combination. It's beyond the scope of this effort to determine a "best" design alternative. However, 28 different designs are considered in this case study including upgradient and downgradient cutoffs, one combination design and a limited number of downgradient hydraulic barriers. Keeping the mitigation objective in mind (i.e, minimizing contaminant migration from the site vicinity) the purposes of the evaluations presented are to:

1. evaluate the general effectiveness of selected mitigation alternatives in limiting contaminant migration from the immediate reactor site,

2. investigate the relative importance of specific design parameters including barrier length, distance from the site, orientation to the

site (i.e., centered or offset), shape (linear, L-shaped, U-shaped), permeability of grout cutoffs and upstream vs. downstream location, and

3. consider the sensitivity of mitigation performance to hydrogeologic and transport parameters such as hydraulic conductivity, dispersivity, and retardation factor.

The selection of conceptual designs to be considered, while somewhat arbitrary, was based on several factors. Because of the potential for high levels of surface and subsurface contamination in the immediate vicinity of the reactor due to atmospheric fallout, it was assumed that barriers would have to be placed approximately 1000 ft away from the reactor site. A sufficient number of alternatives were considered to provide insight into the performance characteristics of a wide range of design variations. Both downgradient and upgradient cutoffs were considered while only downgradient hydraulic barriers were assessed. Also, one combination design was included which utilizes both an upgradient and a downgradient barrier. Costs, while discussed in a later selection, played no role in the initial design selections.

The approach to analyzing the flow and transport for the mitigation conditions was to adjust the local flow model parameters (i.e., hydraulic conductivity) to simulate the impact each alternative would have on groundwater flow. The transport of strontium-90 was then simulated under these conditions.

The format for discussion of the mitigation conceptual designs and their performance is the following. First, each of the individual designs and their performance (i.e., reduction in contaminant flux) relative to the pre-mitigated case are discussed as part of a set. For example, one set of downgradient designs includes four separate grout cutoffs 1000 ft from the reactor site, with centered orientation, and having lengths of 500, 1000, 2000 and 3000 ft. Second, mitigation performance as a function of design parameters is considered wherein designs from more than one set are compared to one another. Finally, the results of the parameter sensitivity studies will be presented. Throughout the discussions, graphics are used liberally to present simulation results and to avoid overly repetitious descriptions of plume shapes, potential surfaces, etc.

7.5.1 Downgradient Mitigation Measures

Each of the downgradient designs and their performance relative to the pre-mitigated case are discussed in this section. Sketches are provided to illustrate the location, shape, and length of each design. Also, example three-dimensional plots of resultant potential surfaces and contamination distributions are provided. At the end of the section Table 7.5.1-1 is included which summarizes the alternatives evaluated, their general design parameters and their performance. Specifically, for each alternative the summary table lists the design shape, length, and permeability. The table also includes the average potential gradient created by the design, approximate contaminant travel time and peak and total contaminant flux at the breakthrough section.

7.5.1.1 Downgradient Plant Configuration Design Considerations Alt. #1, Alt. #2 and Alt. #3 (with Cooling Reservoir)

The primary plant feature in the downgradient direction from the plant which might effect design and placement of mitigation measures is the cooling reservoir. As discussed in Section 7.2.2, there may be certain circumstances wherein, following a severe accident, the cooling capacity for the second unit would have to be maintained. Or, if the reservoir water is heavily contaminated by atmospheric fallout, for a period of time following a severe accident it might not be feasible to drain it. If either of these possibilities were the case, construction of a grout curtain in the shallow aquifer through and beneath the reservoir might be infeasible. Worker safety considerations might preclude drilling activities in and around contaminated reservoir water, and the presence of the reservoir water might make the drilling and injection operations associated with grout cutoff construction much more difficult.

In light of these concerns, the first set of design alternatives are conceptualized assuming grout cutoff construction beneath the cooling reservoir is precluded. Three designs are assessed: having lengths of 500, 1000, 2000 ft (designated as Alt. #1, Alt. #2, and Alt. #3, respectively) each offset to the east such that the west end of the cutoff terminates at the reservoir embankment. The cutoffs are simulated assuming they have zero permeability, penetrate to the confining layer beneath the shallow aquifer and are 10-ft wide. An illustration showing their approximate location relative to the STP is presented in Figure 7.5.1-1.

The relative effect the three cutoffs have on the local model potential surface is illustrated in Figures 7.5.1-2, 7.5.1-3, and 7.5.1-4. Each of the figures show the offset position of the cutoffs relative to the reactor site. The drop across the cutoffs range from 0.5 ft for the 500-ft cutoff to 1.75 ft for the 2000-ft cutoff. In general, the cutoffs produce a very flat gradient immediately upgradient and immediately downgradient of the cutoff center. At the cutoff edges, gradients steepen toward the cutoff centers. The flux rates produced by the cutoffs are compared to the pre-mitigated flux in Figure 7.5.1-5. For the pre-mitigated case flux begins at about year 250. Results for the mitigated cases show flux begins at about 150 years. In all three cases the flux rate is increased by the cutoffs; in fact, the increase is directly related to cutoff length. Over the 1000-year simulation period, the total pre-mitigated flux is increased from 1.3×10^{12} pCi to 3.5×10^{12}, 4.4×10^{12}, and 3.4×10^{12} pCi, respectively for the three cutoff designs. The cause for the negative impact of the cutoffs can be seen clearly in the potential surface plots. The reactor site is located almost directly upgradient of the west end of each of the cutoffs. Thus, as the contaminant plume approaches the vicinity of the cutoff it moves into an area of steep gradient and increased velocity. Consequently, the contaminant is transported rapidly toward and past the cutoff locations, producing markedly increased flux rates. The surface plot of strontium-90 concentrations for the 2000-ft cutoff at 1000 years (Figure 7.5.1-6) further illustrates the effect of the offset grout cutoffs. Compared to the pre-mitigated concentrations in Figure 7.4-2, the front of the main plume has advanced approximately 500 ft farther after the same period of time.

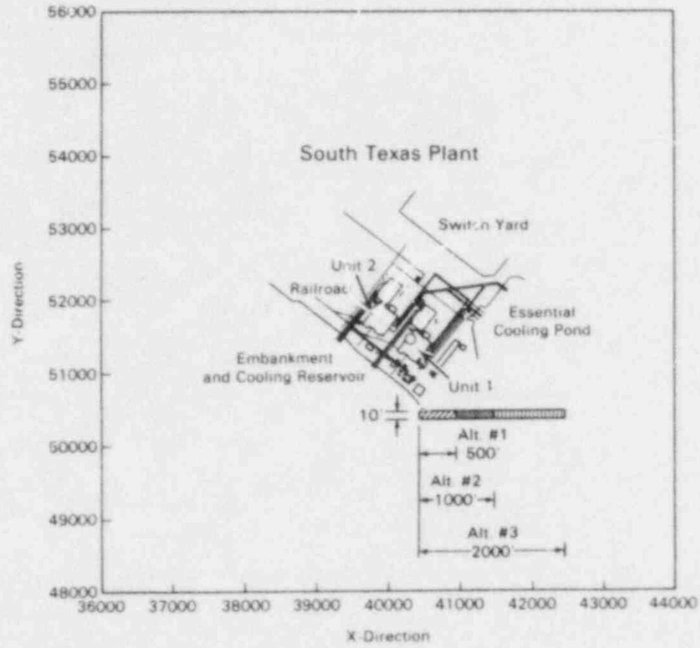


FIGURE 7.5.1-1. Location of Alt. #1 (L=500 ft), Alt. #2 (L=1000 ft) and Alt. #3 (L=2000 ft).

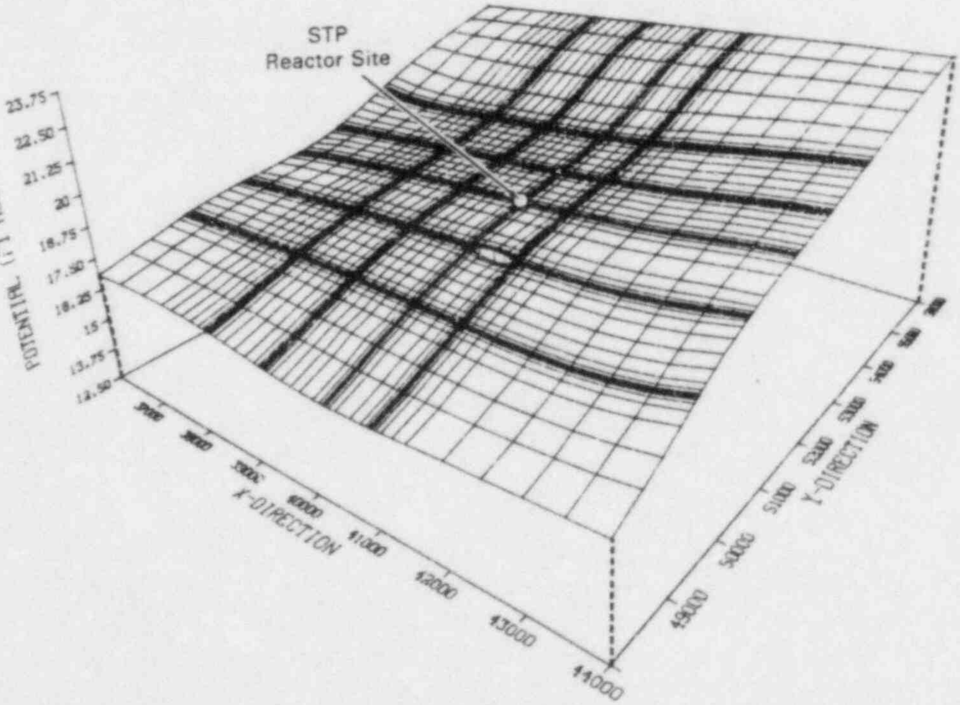


FIGURE 7.5.1-2. Simulated Potential Surface Alt. #1: 500-ft Cutoff, East of the Cooling Reservoir, 1000 ft Downgradient.

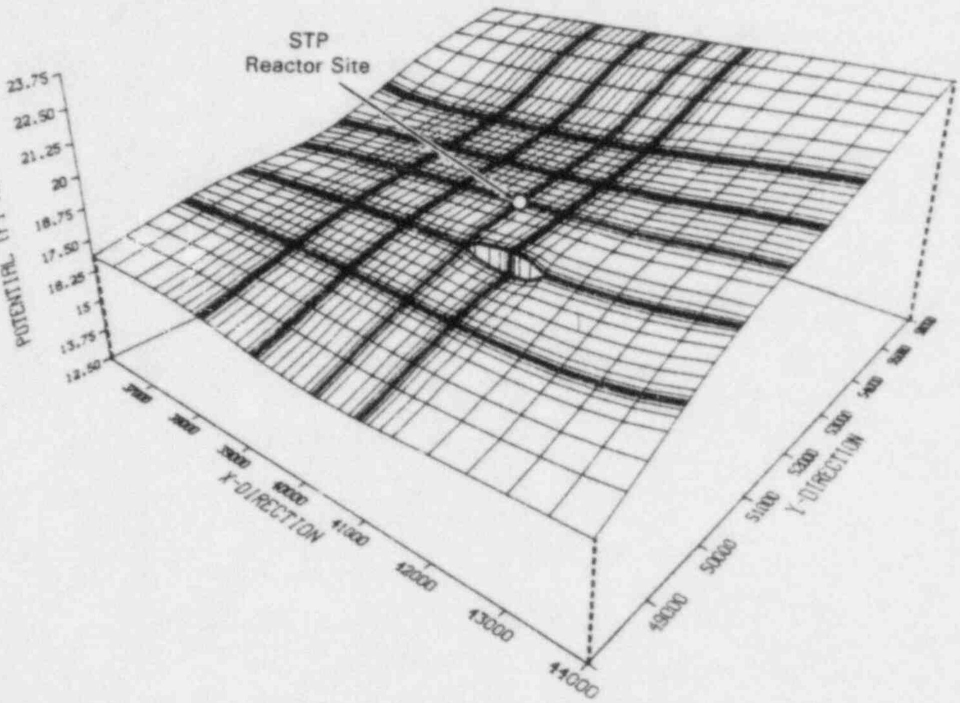


FIGURE 7.5.1-3. Simulated Potential Surface Alt. #2: 1000 ft Cutoff, East of the Cooling Reservoir, 1000 ft Downgradient.

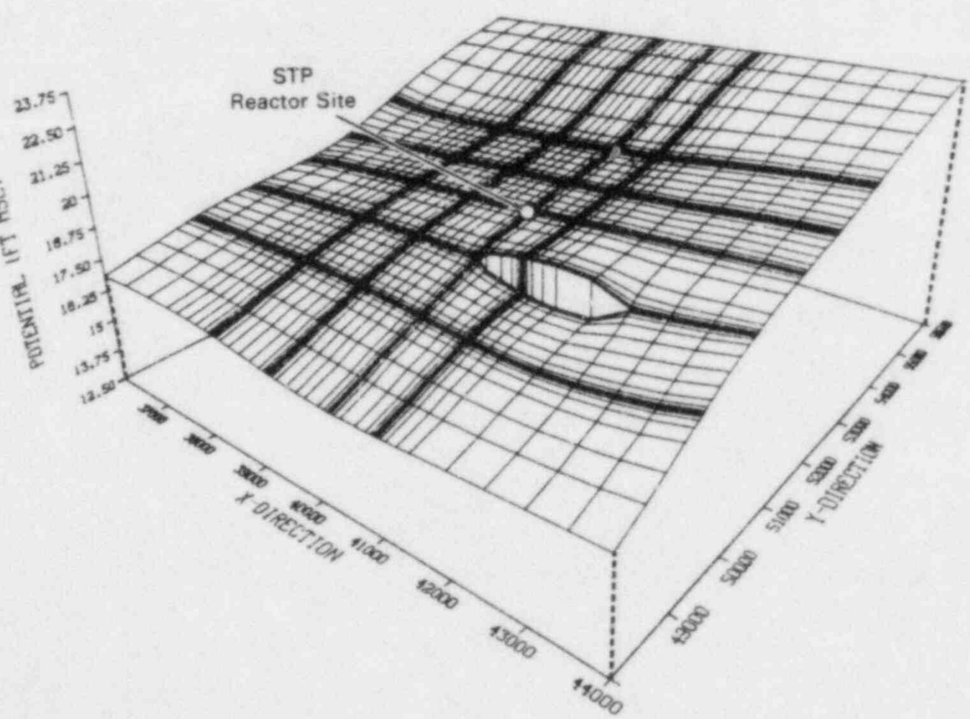


FIGURE 7.5.1-4. Simulated Potential Surface Alt. #3: 2000 ft Cutoff, East of the Cooling Reservoir, 1000 ft Downgradient.

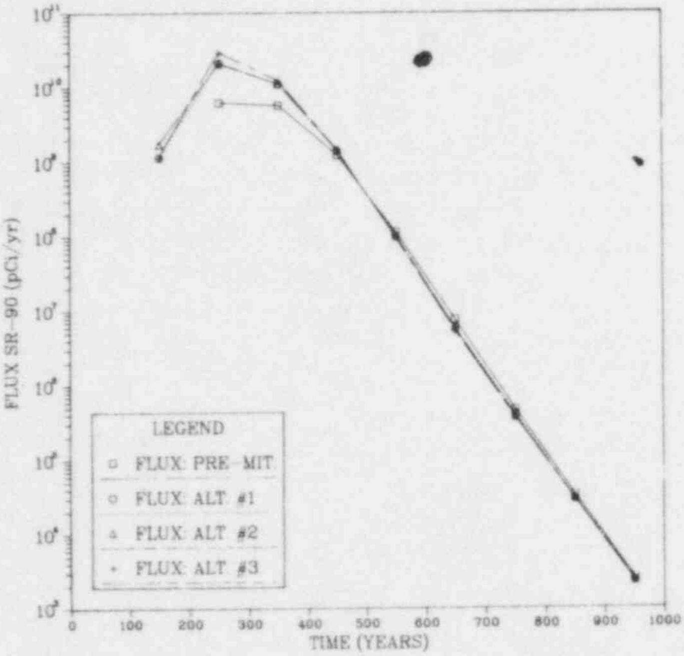


FIGURE 7.5.1-5. Simulated Flux Rates: Alt. #1 (L=500 ft), Alt. #2 (L=1000 ft), and Alt. #3 (L=2000 ft).

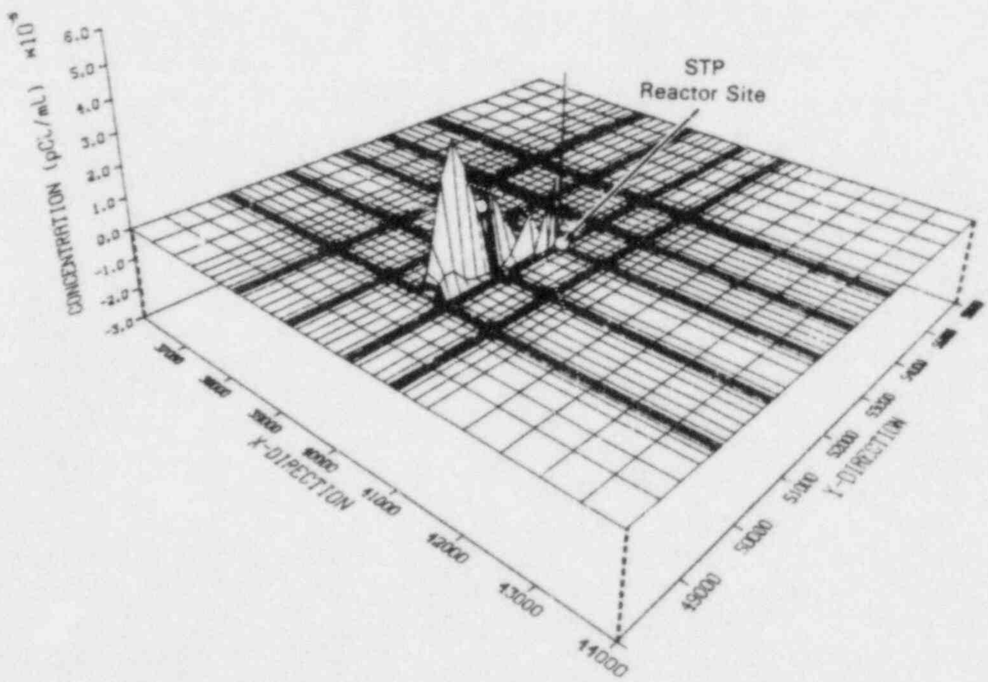


FIGURE 7.5.1-6. Simulated Strontium-90 Concentrations at 1000 years: Alt. #3 (L=2000 ft).

On the basis of these results, downgradient cutoffs, if constructed outside the cooling reservoir, provide no benefit and actually increase transport of radionuclides from the STP site. Therefore, if grout cutoffs are to be constructed in the downgradient direction it will be necessary to locate them with a more centered orientation relative to the reactor site. If mitigation were delayed sufficiently to allow the potentially contaminated water in the reservoir to be disposed of or to evaporate such that dangerously high surface contamination is removed, grout cutoffs could be effectively implemented downgradient of the reactor site. Assuming this to be the case, the remainder of the downgradient grout cutoffs evaluated are conceptualized without the constraint of remaining outside the reservoir area.

7.5.1.2 Additional Downgradient Grout Cutoff Design Evaluations Alt. #4, Alt. #5, Alt. #6, and Alt. #7 (without Cooling Reservoir)

The next set of alternatives considered consists of four linear cutoffs. These cutoffs are centered relative to the reactor site and located 1000 ft downgradient. As noted above, it was assumed that the closest possible location for cutoff construction following a severe accident was about 1000 ft from the reactor. This assumption is largely arbitrary; however, it was made recognizing that in reality a limit will exist. Contrary to the previous designs, this set is intended to maximize, per unit length of linear barrier, the impact on the transport of radionuclides from the reactor site. Therefore, they are located at the assumed minimum distance from and centered relative to the reactor. Four designs are evaluated at this location, including lengths of 500, 1000, 2000 and 3000 ft. The locations are shown in Figure 7.5.1-7. As examples, the resulting potential surfaces with the cutoffs in place are shown for the 500-ft and 3000-ft cutoffs in Figures 7.5.1-8 and 7.5.1-9. The 500-ft cutoff acts merely as a small obstruction to flow, again creating a drop of only 0.5 ft and only minor backwater effect. The 3000-ft barrier on the other hand produces a 2.1-ft drop and significant backwater effects, reducing the gradient at the site to 0.7×10^{-4} ft/ft.

The resultant flux with time for the cutoff designs are compared in Figure 7.5.1-10. The simulations show that the 500-ft design actually increases the flux rate with arrival of strontium-90 occurring at the breakthrough point after less than 200 years while the 1000-ft design only marginally reduces the flux rate. The results for the two longer cutoffs demonstrate that, for increasing lengths beyond 1000 ft, there is substantial increase in first arrival time. Consequently, because of natural decay, the flux rates are reduced. The total flux of strontium-90 for the 1000-year simulation period is reduced from 1.3×10^{12} pCi for the pre-mitigated case to 8.4×10^{11}, 1.7×10^{10}, and 1.2×10^{8} pCi for the 1000-, 2000- and 3000-ft designs, respectively. An example of the spatial distribution of strontium-90 concentration for the centered cutoff is presented in Figure 7.5.1-11. For this case, the 3000-ft design at 1000 years, the contaminant is totally contained behind the barrier and lateral spreading at the plume front is increased. Overall concentrations are approximately the same as in the pre-mitigated case.

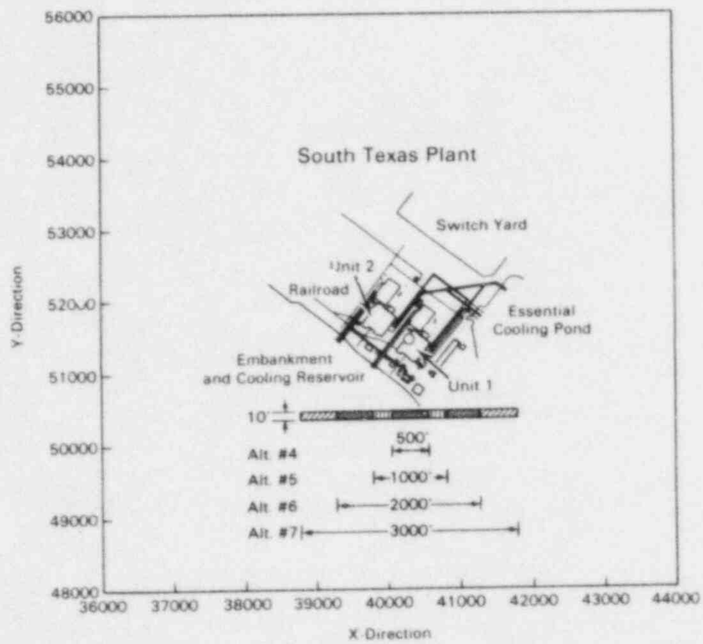


FIGURE 7.5.1-7. Location of Alt. #4 (L=500 ft), Alt. #5 (L=1000 ft), Alt. #6 (L=2000 ft), and Alt. #7 (L=3000 ft).

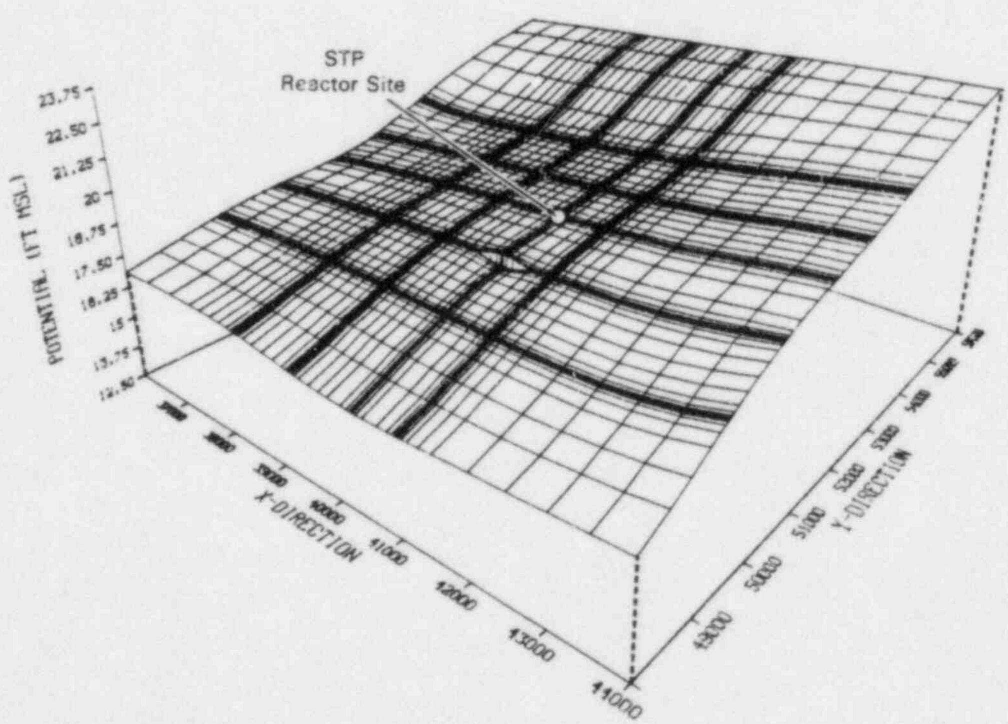


FIGURE 7.5.1-8. Simulated Potential Surface Alt. #4: 500-ft Cutoff, Centered, 1000 ft Downgradient.

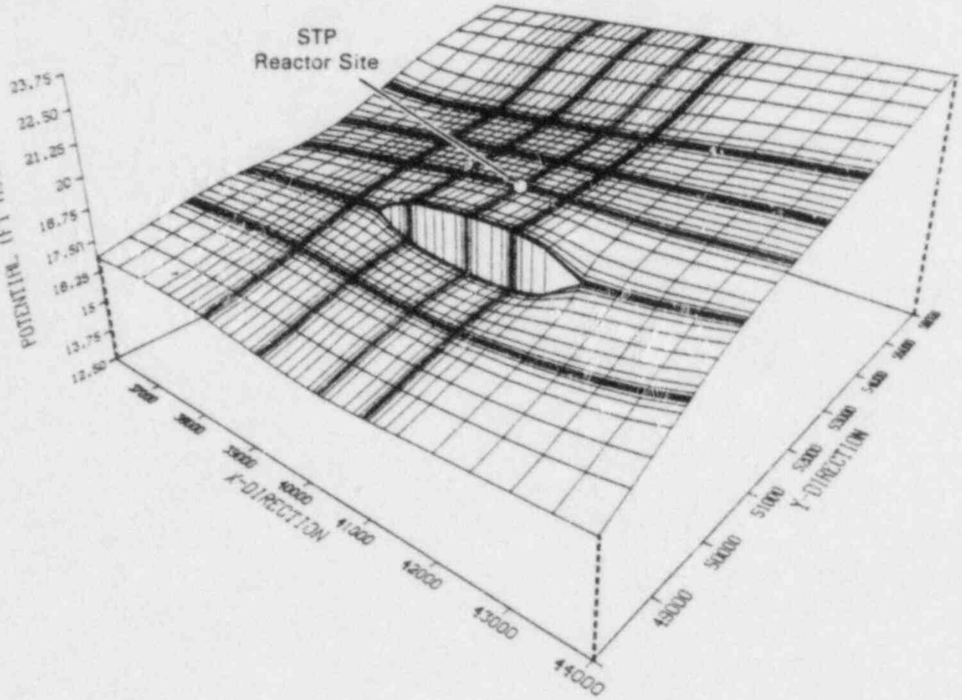


FIGURE 7.5.1-9. Simulated Potential Surface Alt. #7: 3000-ft cutoff, Centered, 1000 ft Downgradient.

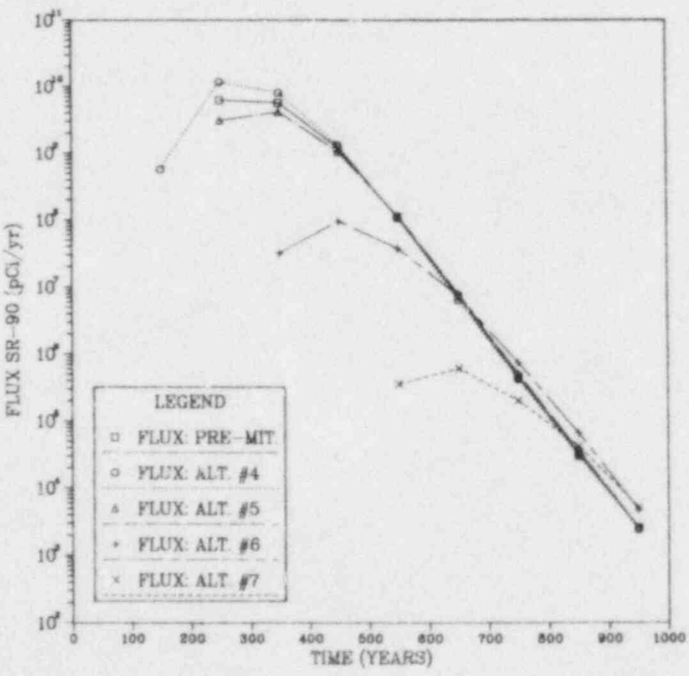


FIGURE 7.5.1-10. Simulated Flux Rates: Alt. #4 (L=500 ft), Alt. #5 (L=1000 ft), Alt. #6 (L=2000 ft), and Alt. #7 (L=3000 ft).

Alt. #8 and Alt. #9 (Offset)

To investigate the possible advantages and disadvantages of partially offsetting the cutoffs relative to the reactor location, Alt. #8 and Alt. #9 are 3000 ft in length and located 1000 ft downgradient. As can be seen in Figure 7.5.1-12, relative to the reactor site, the centers are offset 1000 ft to the east and west, respectively. The simulated potential surfaces for the designs are presented in Figures 7.5.1-13 and 7.5.1-14. It's noteworthy that the drop across Alt. #8 is greater due to the lower hydraulic conductivities in the eastern portion of the study area (see Figure 7.3-4). The maximum drop across Alt. #8 is 3.2 ft compared to 2.5 ft for Alt. #9. Though the higher drop results in greater backwater effect in the y-direction, it also produces an increased gradient in the x-direction from east to west. The effect of the lateral gradient can be seen by comparing the 1000-year concentration surface plots for the two designs in Figures 7.5.1-15 and 7.5.1-16. The lateral gradient produced by Alt. #8 allows the strontium-90 to escape around the western end of the cutoff and continue to move away from the site. Alt. #9, on the other hand, completely contains the plume. This difference is also evident in the simulated contaminant flux for the two designs plotted in Figure 7.5.1-17. In both cases the flux is significantly reduced relative to the pre-mitigated case; however, for the same length of cutoff, Alt. #9 exhibits much better performance. Total flux for Alt. #8 is 3.3×10^{10} pCi compared to 2.0×10^{10} pCi for Alt. #9.

Alt. #10 (2000 ft Downgradient)

Intuitively, the closer a barrier is placed to the contaminant source, the better its performance will be in terms of decreasing potential gradients and flow velocities and increasing travel distances. However, conditions could exist such that a barrier would have to be placed a greater distance away. To provide a measure of the impact increased distance would have on cutoff performance, Alt. #10 is evaluated, a 3000 ft long, centered cutoff located 2000 ft downgradient from the reactor (shown in Figure 7.5.1-18). The potential surface for Alt. #10 is shown in Figure 7.5.1-19. The drop across the cutoff is about 2.5 ft and the backwater effect extends a considerable distance upgradient. The contaminant plume after 1000 years, presented in Figure 7.5.1-20, is completely contained within 1000 ft downgradient of the reactor. The mitigated flux, compared to the pre-mitigated flux in Figure 7.5.1-21, is substantially reduced and the first arrival is delayed approximately 100 years. These results indicate that even at greater distances from the reactor successful mitigation can be achieved without dramatically increasing cutoff size.

Alt. #11, Alt. #12, and Alt. #13 (L- and U-Shaped)

These designs, two L-shaped cutoffs and a U-shaped cutoff, are evaluated to investigate possible advantages of modifying cutoff shape. All three designs have a total length of 3000 ft and are located 1000 ft downgradient of the reactor. The L-shaped cutoffs, illustrated in Figure 7.5.1-22, consist of a 2000-ft leg in the x-direction and a 1000-ft leg in the y-direction. The potential surfaces produced by the L-shaped cutoffs are plotted in

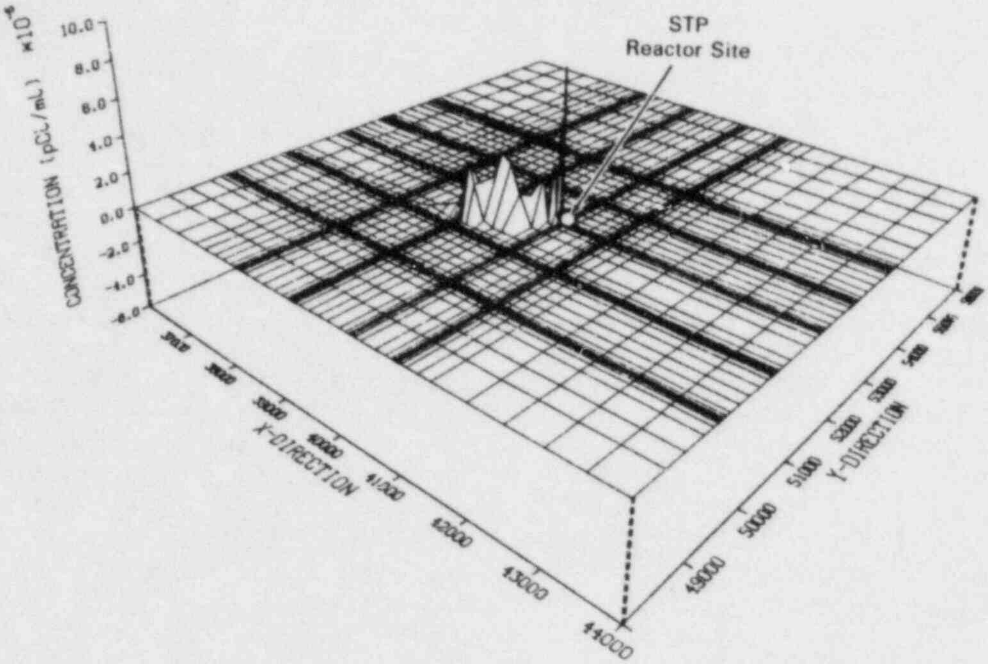


FIGURE 7.5.1-11. Simulated Strontium-90 Concentrations at 1000 years: Alt. #7 (L=3000 ft).

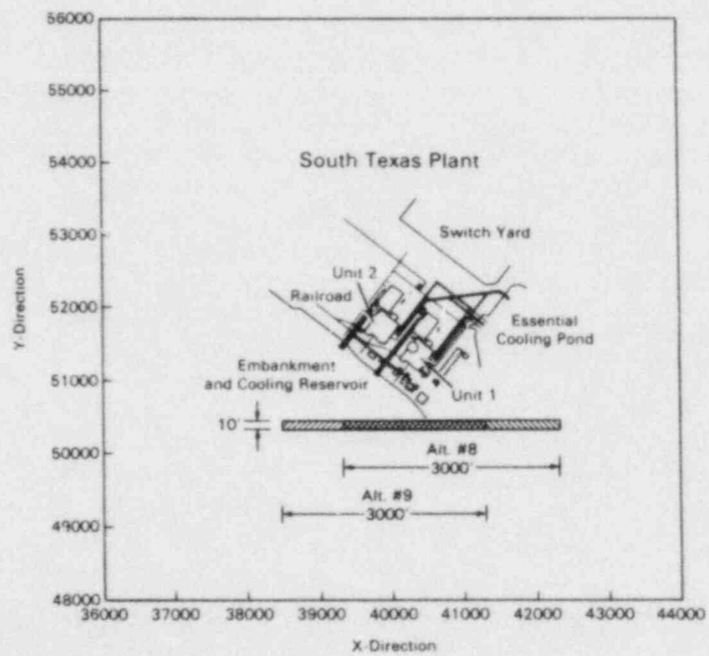


FIGURE 7.5.1-12. Location of Alt. #8 (L=3000 ft, Offset-East) and Alt. #9 (L=3000 ft, Offset-West).

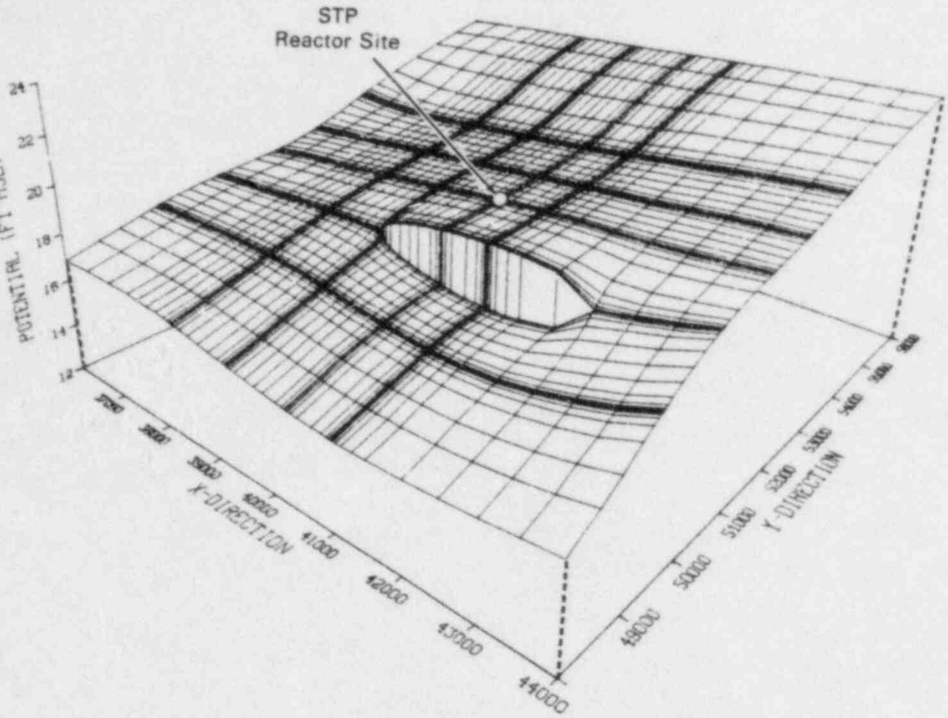


FIGURE 7.5.1-13. Simulated Potential Surface, Alt. #8: 3000 ft Cutoff, Offset-East, 1000 ft Downgradient.

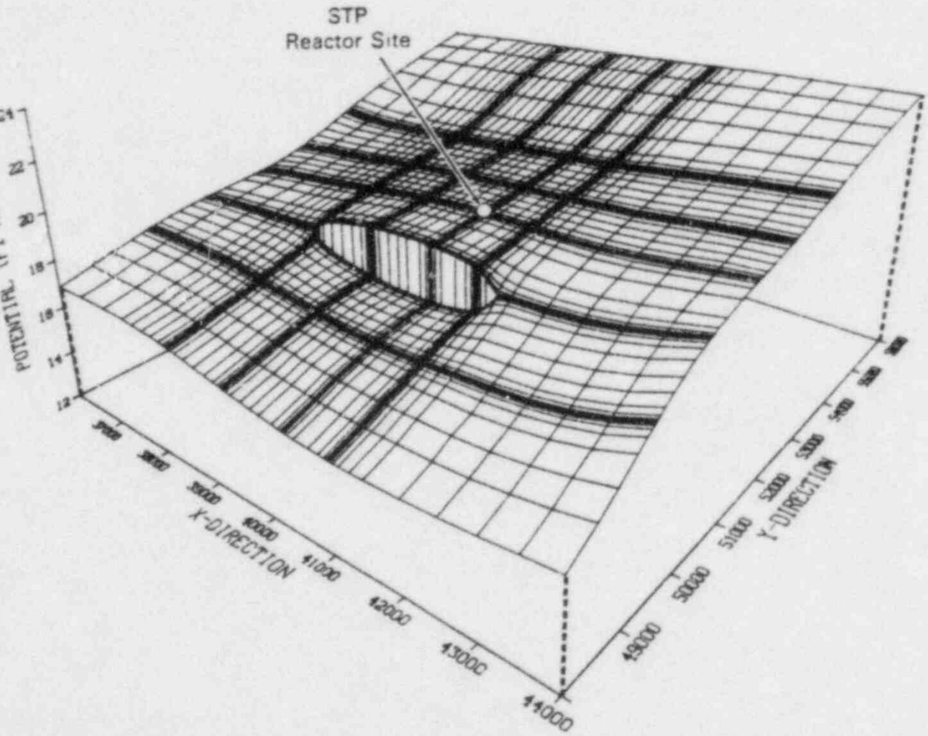


FIGURE 7.5.1-14. Simulated Potential Surface, Alt. #9: 3000 ft Cutoff, Offset-West, 1000 ft Downgradient.

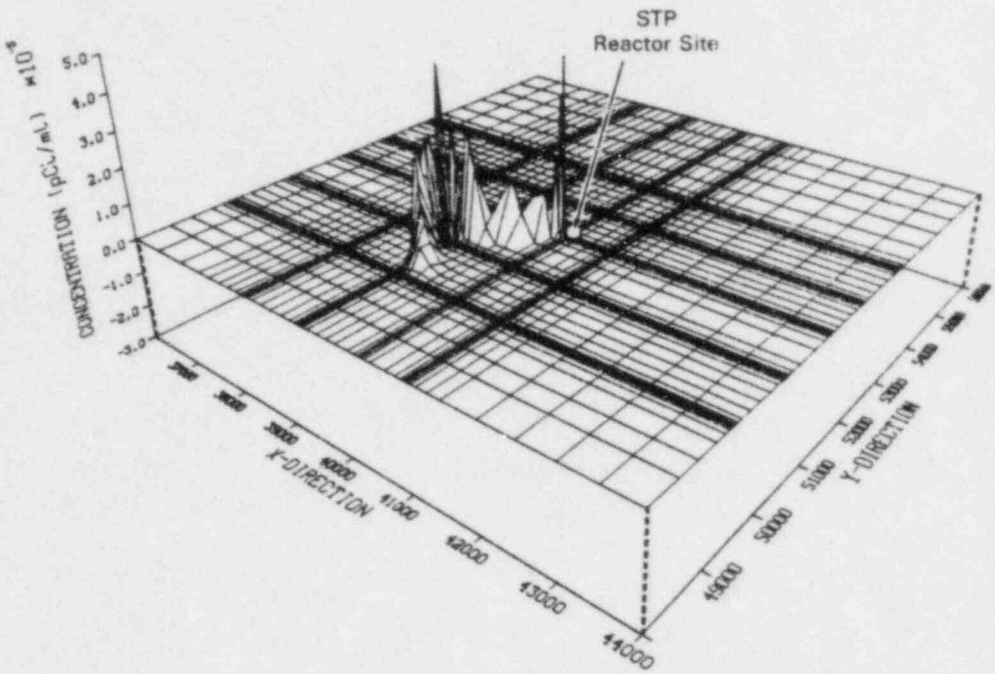


FIGURE 7.5.1-15. Simulated Strontium-90 Concentrations at 1000 Years: Alt. #8 (L=3000 ft, Offset-East).

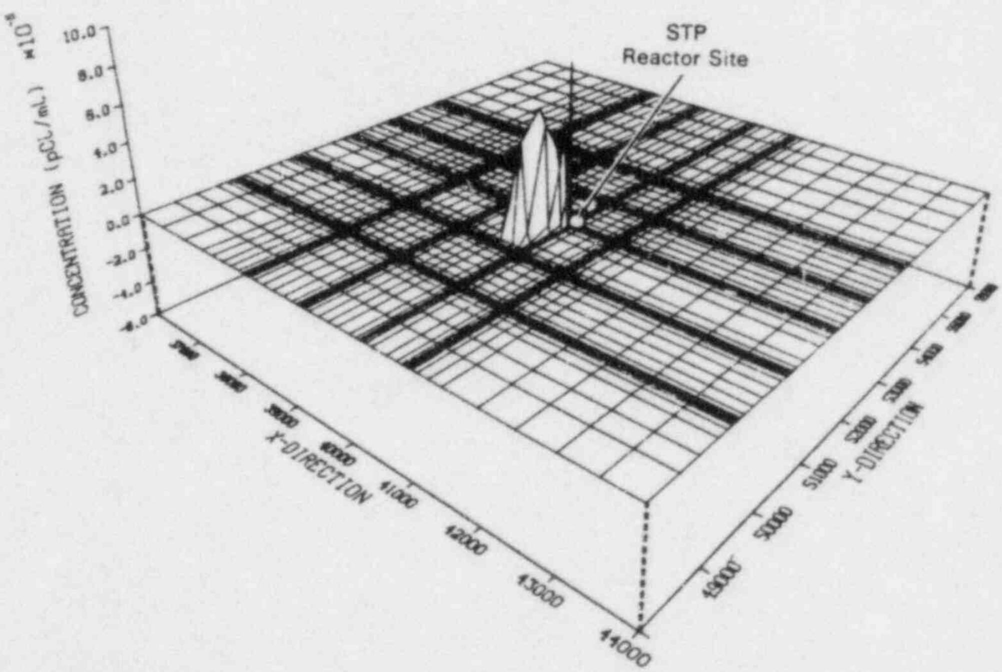


FIGURE 7.5.1-16. Simulated Strontium-90 Concentrations at 1000 Years: Alt. #9 (L=3000 ft, Offset-West).

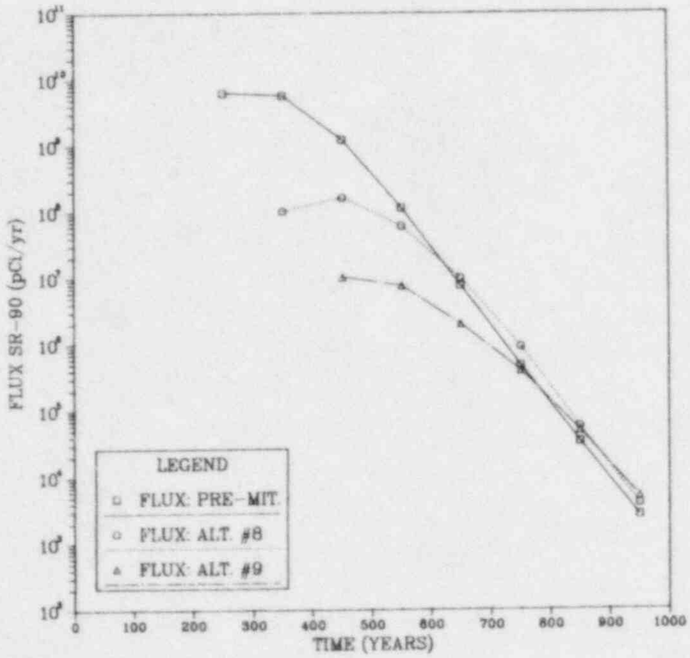


FIGURE 7.5.1-17. Simulated Flux Rates: Alt. #8 (L=3000 ft, Offset-East) and Alt. #9 (L=3000 ft, Offset-West).

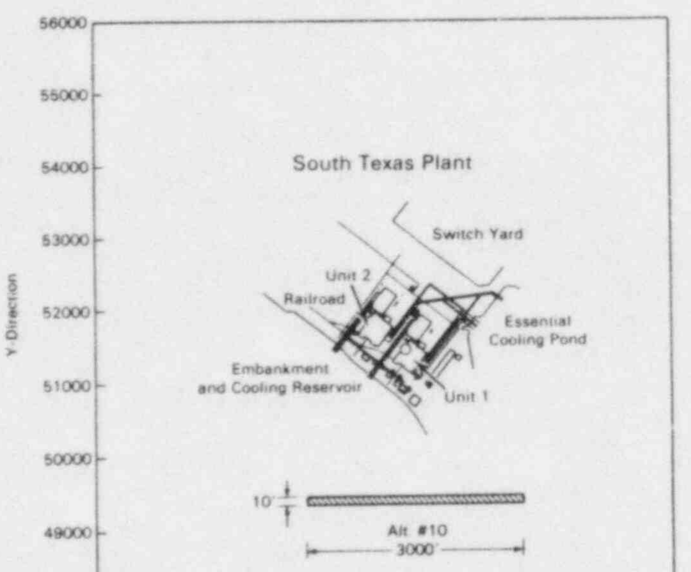


FIGURE 7.5.1-18. Location of Alt. #10 (L=3000 ft), 2000 ft Downgradient.

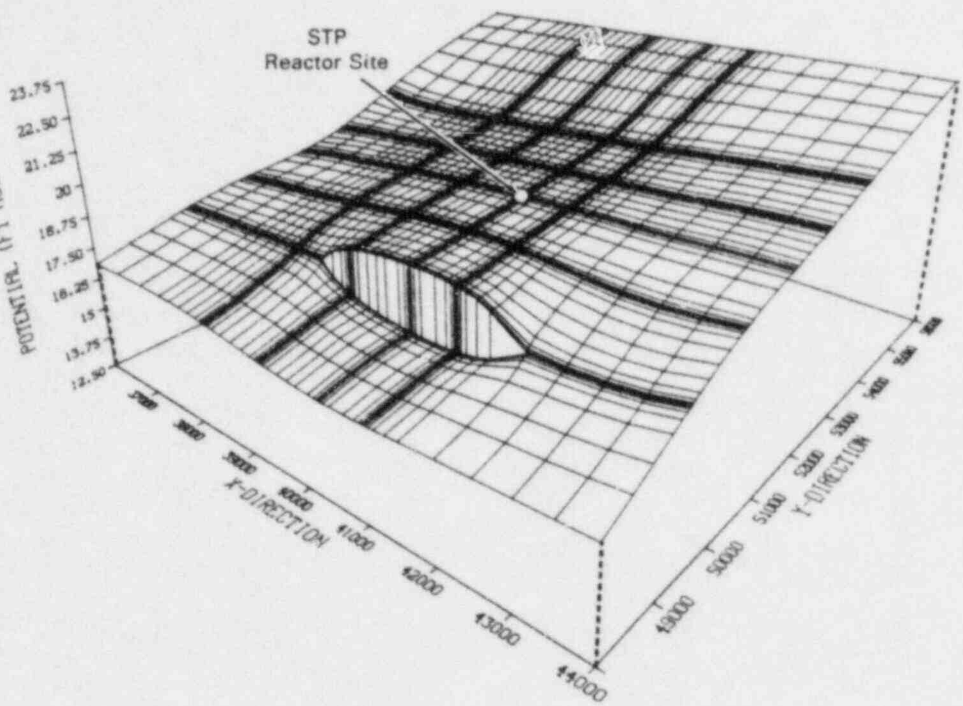


FIGURE 7.5.1-19. Simulated Potential Surface, Alt. #10 (L=3000 ft), 2000 ft Downgradient.

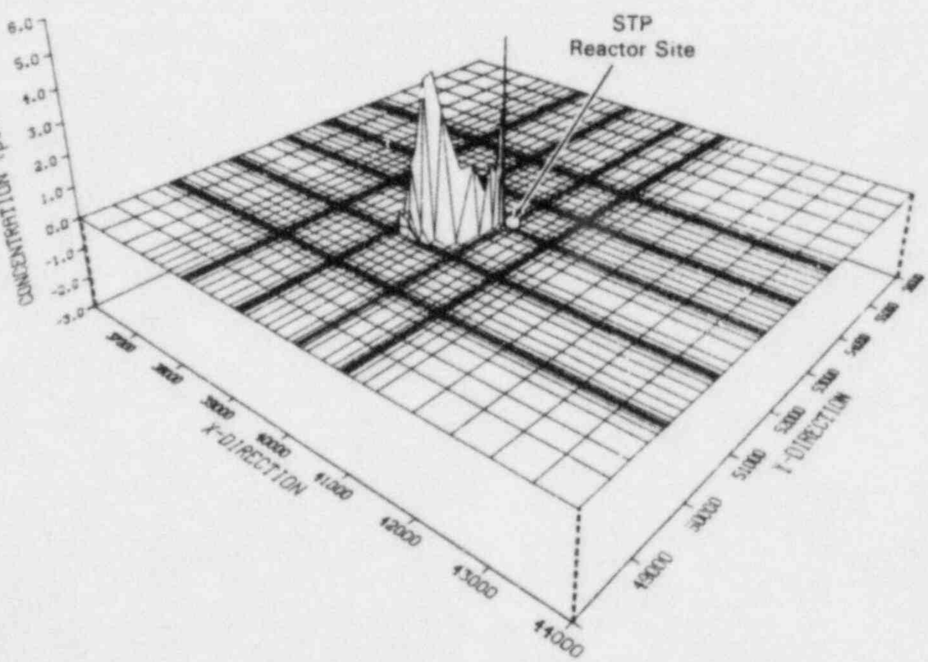


FIGURE 7.5.1-20. Simulated Strontium-90 Concentrations at 1000 years: Alt. #10 (L=3000 ft), 2000 ft Downgradient.

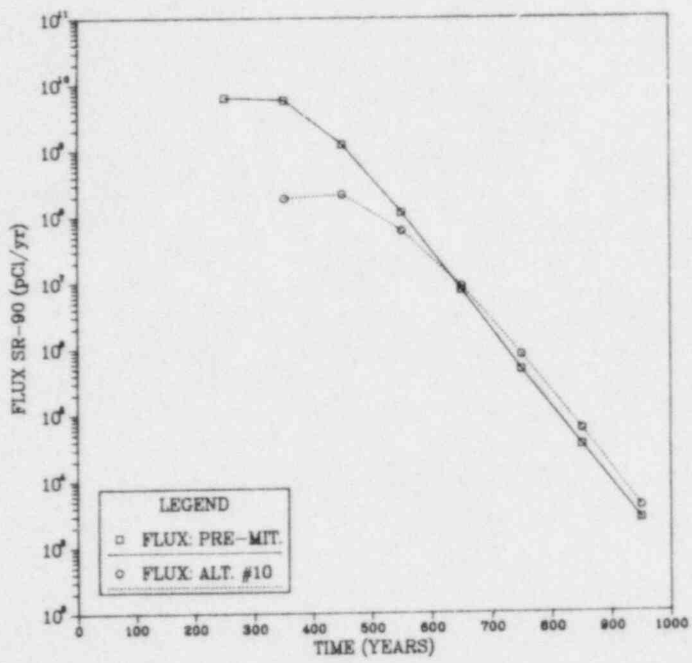


FIGURE 7.5.1-21. Simulated Flux Rates: Alt. #10 (L=300 ft), 2000 ft Downgradient.

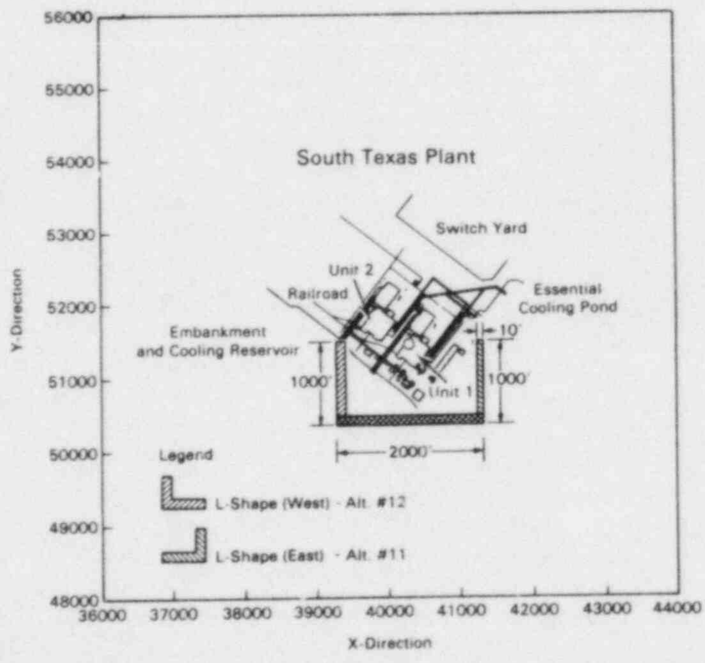


FIGURE 7.5.1-22. Location of Alt. #11 (L-Shaped-East) and Alt. #12 (L-Shaped-West).

Figures 7.5.1-23 and 7.5.1-24. The surface generated by Alt. #11 is very similar to that for Alt. #6, a 2000-ft linear cutoff, indicating the 1000-ft leg has little effect. Because of the low conductivities in the eastern portion of the study area, the lateral velocities are low and are not greatly affected by the cutoff leg. In contrast Alt. #12, with the leg added to the western end, produces observable drop and backwater in the x-direction which contributes to the reduction in potential gradient at the site. The gradient at the reactor created by Alt. #12 is 0.7×10^{-4} ft/ft compared to 1.4×10^{-4} ft/ft for Alt. #11. The difference in potentials produced by the two designs is clearly manifested in the resulting concentration distributions presented in Figures 7.5.1-25 and 7.5.1-26. The lateral velocities generated by Alt. #11 transport the strontium-90 laterally along the upstream face of the cutoff toward the west, allowing it to escape around the end. The 1000-ft leg on Alt. #12, on the other hand obstructs both lateral and longitudinal flow and transport, and effectively contains the contaminant plume near the reactor site.

The U-shaped design of Alt. #13, shown in Figure 7.5.1-27, utilizes the same length of cutoff as the L-shaped designs but consists of a centered 2000-ft section placed 1000-ft downgradient of the reactor, with 500-ft legs attached at each end. Similar to Alt. #12, the design significantly reduces the gradient at the reactor to 0.8×10^{-4} ft/ft and creates observable potential drop in the x-direction (see Figure 7.5.1-28). The resultant distribution of contaminant at 1000 years, shown in Figure 7.5.1-29, is totally contained upgradient of the cutoff. However, the lateral spreading is greater than that for Alt. #12. As indicated by Figure 7.5.1-30, all three designs reduce flux relative to the pre-mitigated case, but Alt. #12 and #13 are clearly better designs. In fact, the attachment of the cutoff leg to the western end as is done for both designs, almost completely eliminates flux from the reactor area within the 1000-year simulation period. The total flux for Alt. #12 and #13 are 6.3×10^{3} and 1.8×10^{4} pCi, respectively, eight orders of magnitude less than pre-mitigated flux. Because of their effectiveness in reducing gradients, the two designs delay the first arrival of contaminant at the breakthrough section until after 800 years. Several conclusions can be drawn from these results. First, the attachment of the leg in the area of low hydraulic conductivity (east) does not significantly increase performance. Installation of barriers to flow in both the x- and y-directions greatly enhances performance. The 500-ft leg (U-shape) on the western end performs nearly as well as the 1000-ft leg (L-shape), leading to the fact that if costs are important, a shorter leg for the L-shape design might provide adequate mitigation. Finally, a more general conclusion is that the spatial distribution of hydraulic conductivity plays an important role in the relative performance of specific designs and should be considered accordingly in the site characterization phase of the study.

Alt. #14, Alt. #15, and Alt. #16 (Injection Scheme)

The final set of downgradient designs evaluated are three injection schemes. The schemes consist simply of an injection well (or wells) located directly 1000-ft downgradient of the reactor site at the location shown in Figure 7.5.1-31. The total injection rates for the three schemes are 20, 30,

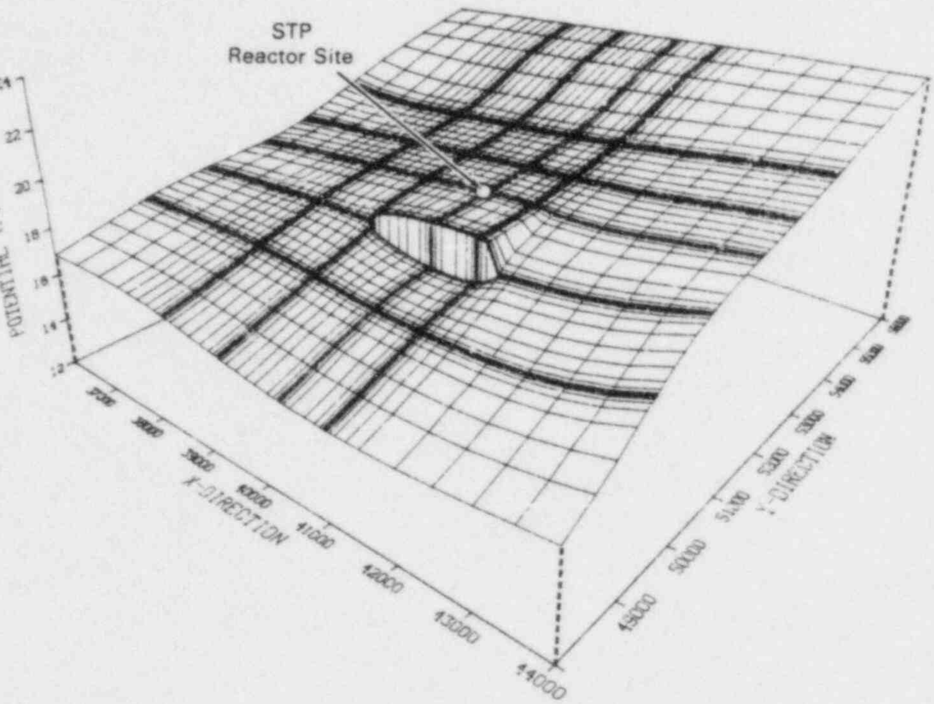


FIGURE 7.5.1-23. Simulated Potential Surface, Alt. #11 (L-Shaped-East).

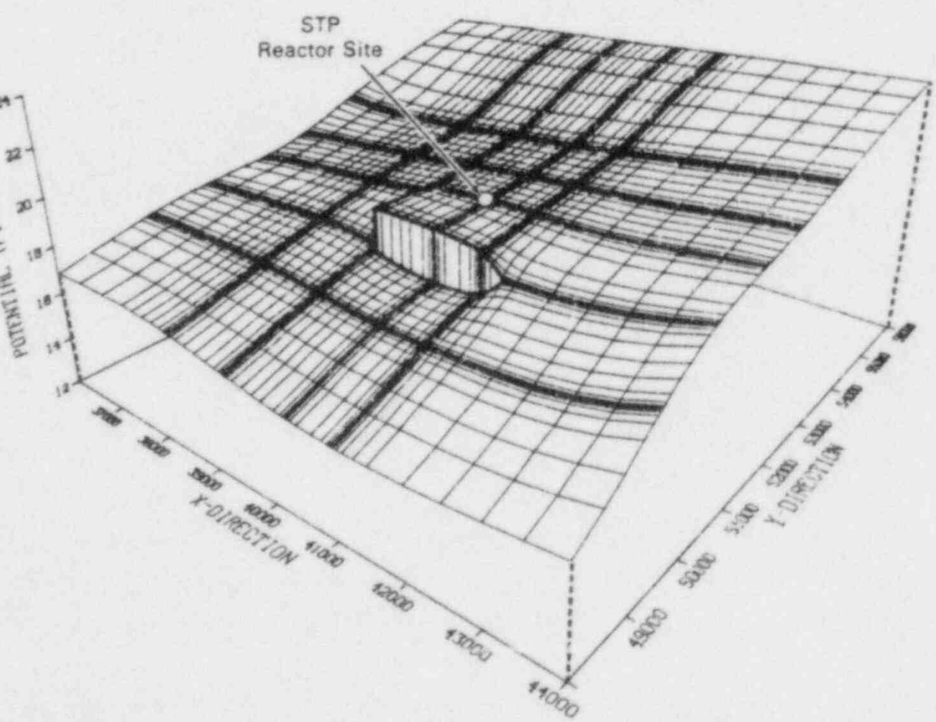
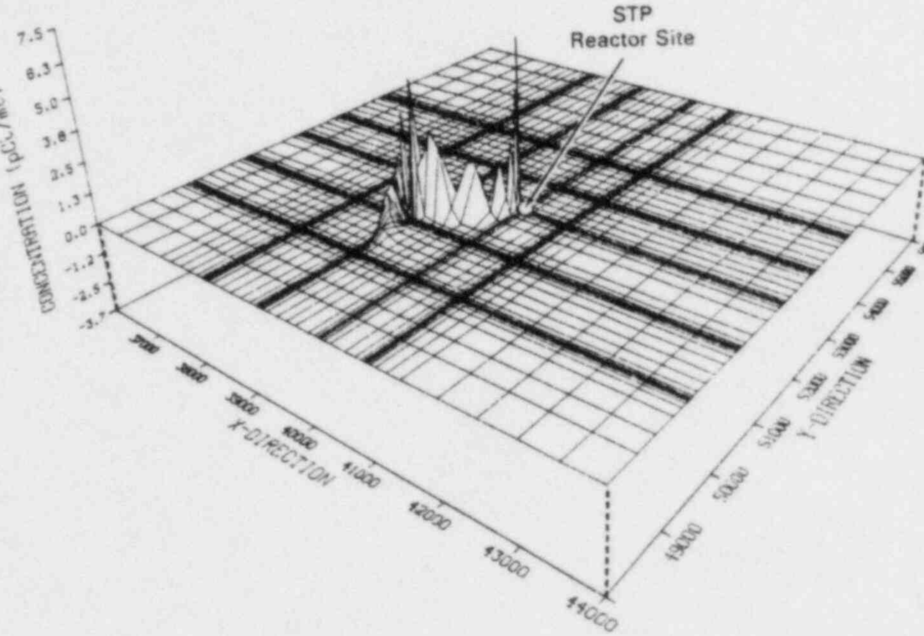
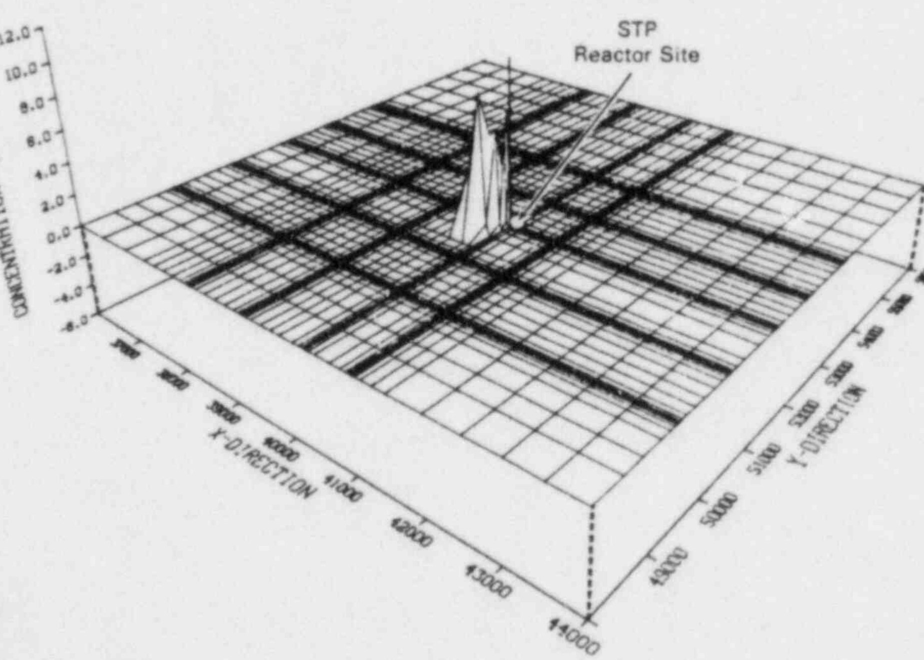


FIGURE 7.5.1-24. Simulated Potential Surface, Alt. #12 (L-Shaped-West).



<u>FIGURE 7.5.1-25.</u> Simulated Strontium-90 Concentrations at 1000 years: Alt. #11 (L-Shaped-East).



<u>FIGURE 7.5.1-26.</u> Simulated Strontium-90 Concentrations at 1000 years: Alt. #12 (L-Shaped-West).

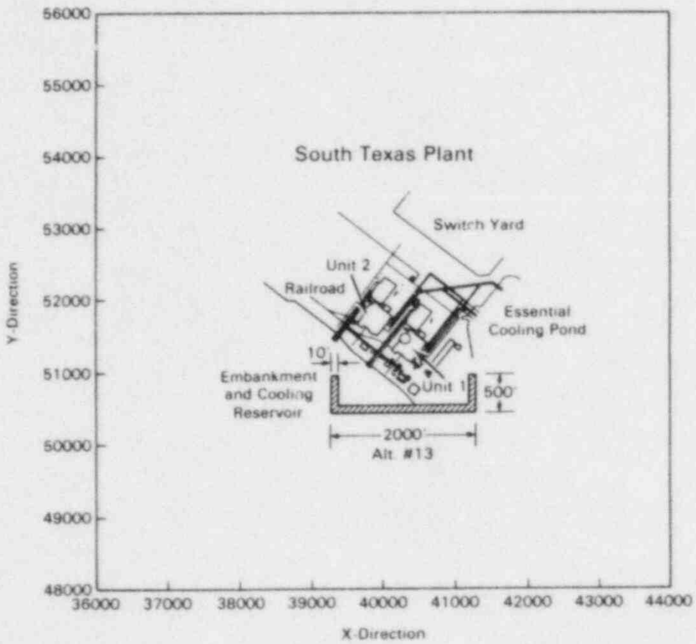


FIGURE 7.5.1-27. Location of Alt. #13 (U-Shaped).

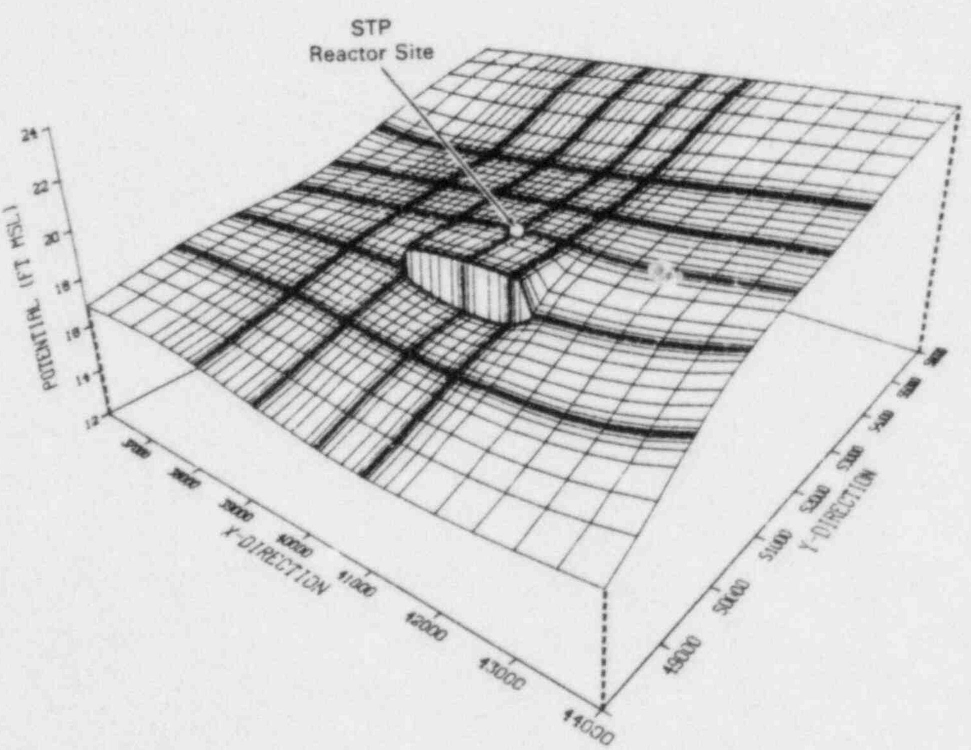


FIGURE 7.5.1-28. Simulated Potential Surface, Alt. #13 (U-Shaped).

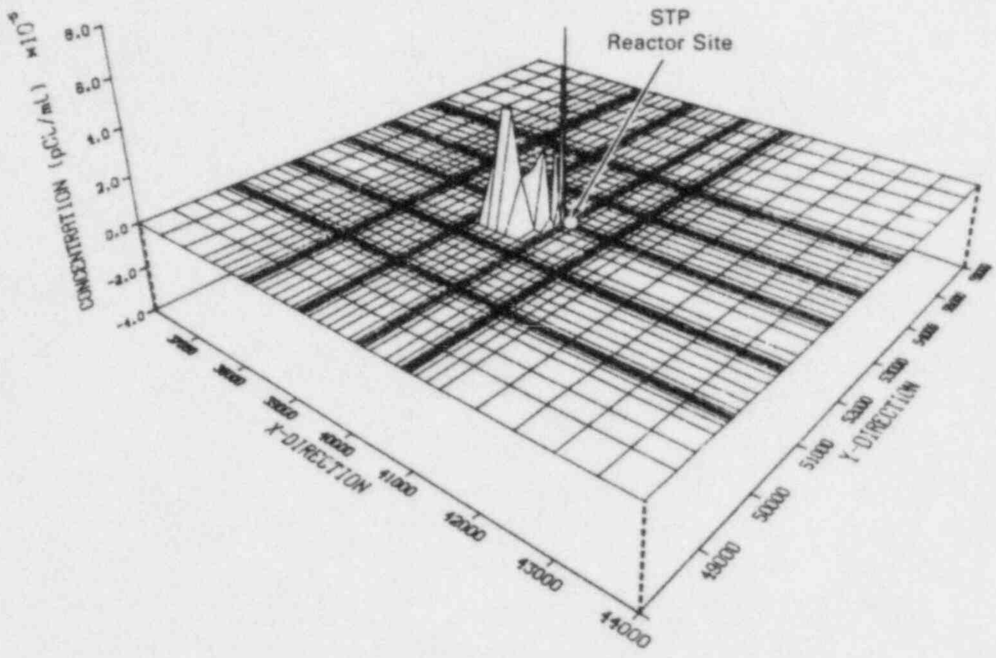


FIGURE 7.5.1-29. Simulated Strontium-90 Concentrations at 1000 years: Alt. #13 (U-Shaped).

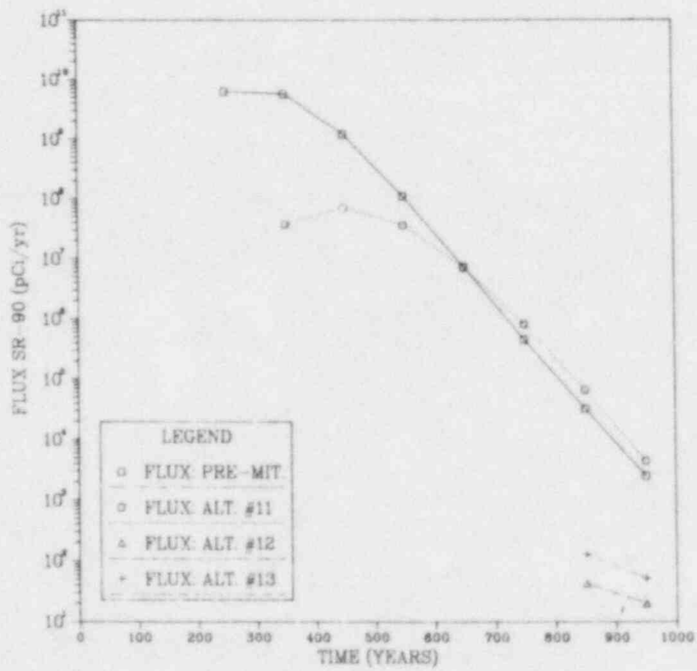


FIGURE 7.5.1-30. Simulated Flux Rates: Alt. #11 (L-Shaped-East), Alt. #12 (L-Shaped-West), and Alt. #13 (U-Shaped).

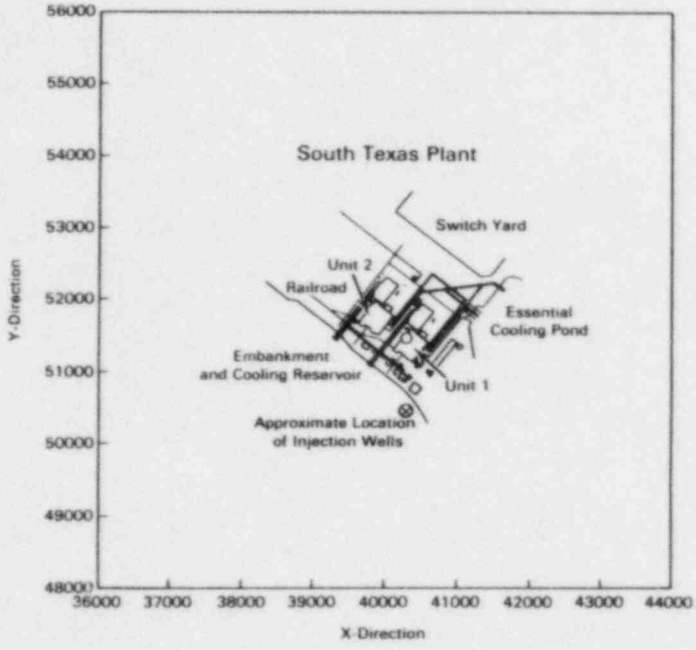


FIGURE 7.5.1-31. Location for Injections Wells: Alt. #14, Alt. #15, and Alt. #16.

and 40 gpm, respectively. The impact of injection on the potential field is creation of a mound which increases in magnitude and areal extent with increasing injection rate. This is illustrated by the potential surface plots for the 20-gpm and 40-gpm schemes in Figures 7.5.1-32 and 7.5.1-33. The mound for the 20-gpm injection scheme is barely discernible while that for the 40-gpm scheme is quite prominent. Similar to the cutoff designs, the preferred direction for contaminant movement is around the hydraulic barrier created by the injection, to the west. This can be most clearly seen in Figure 7.5.1-34, the strontium-90 distribution at 1000 years for the 20-gpm scheme. The majority of the plume is upgradient of the injection location; however, the leading edge is advanced a few hundred feet beyond that point. For the 40-gpm case, though some spreading has occurred, the majority of the plume is contained. The contaminant flux rates resulting from the injection schemes are compared to the pre-mitigated case in Figure 7.5.1-35. In general the curves are similar to those from the cutoff designs, having high initial values and then showing the effects of the decreasing source term and natural decay. The curves also show the increasing effectiveness in containing the strontium-90 with increasing injection rate. All three schemes reduce flux relative to pre-mitigation. Doubling the injection rate from 20- to 40-gpm delays the first arrival from about 350 years to about 450 years, decreases the maximum flux rates by over two orders of magnitude and reduces the total flux from 3.8×10^{10} pCi to 2.8×10^{8} pCi.

A summary of design characteristics and performance for the downgradient alternatives is provided in Table 7.5.1-1.

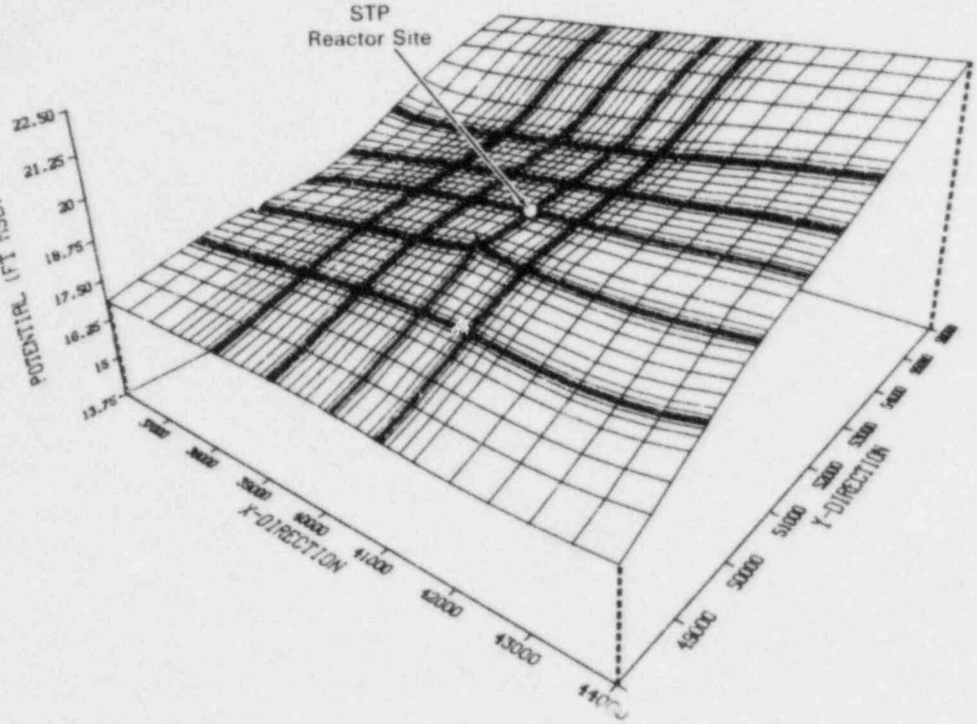


FIGURE 7.5.1-32. Simulated Potential Surface, Alt. #14 (20 gpm).

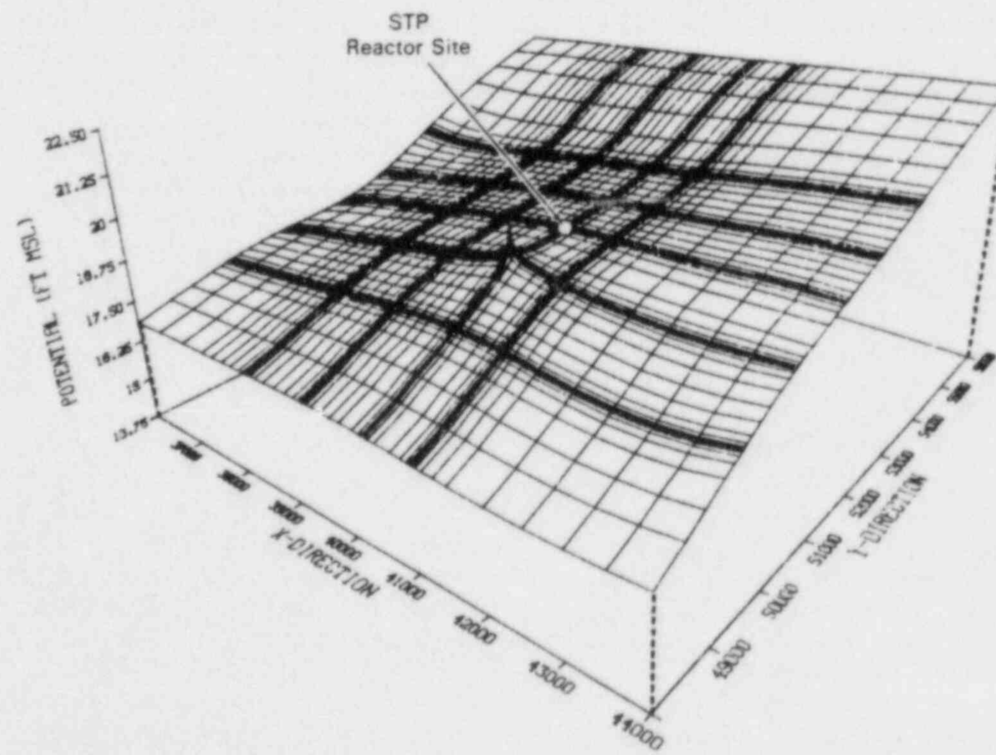


FIGURE 7.5.1-33. Simulated Potential Surface, Alt. #16 (40 gpm).

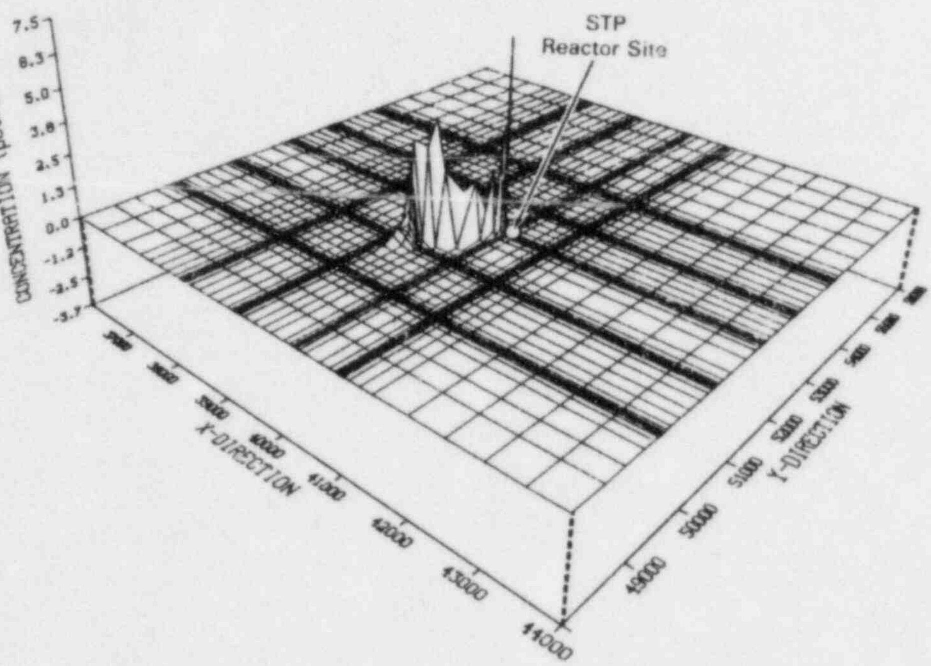


FIGURE 7.5.1-34. Simulated Strontium-90 Concentrations at 1000 years: Alt. #14 (20 gpm).

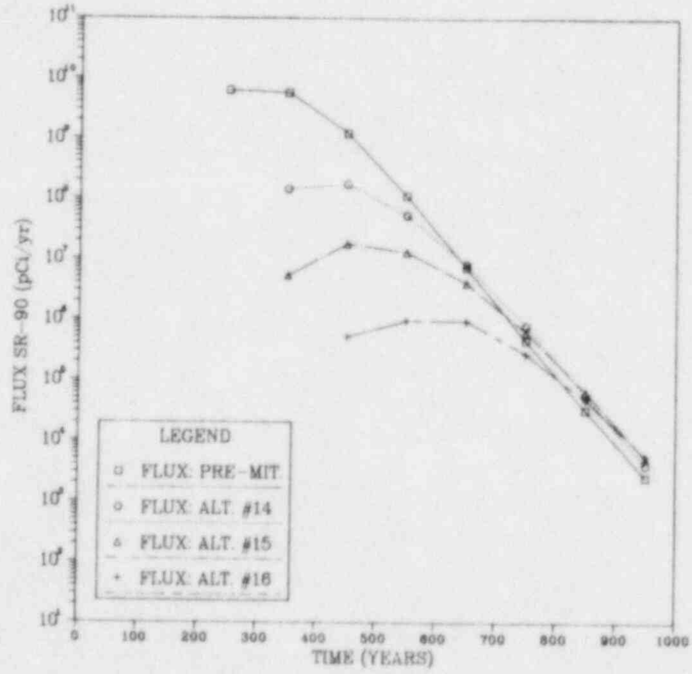


FIGURE 7.5.1-35. Simulated Flux Rates: Alt. #14 (20 gpm), Alt. #15 (30 gpm), and Alt. #16 (40 gpm).

TABLE 7.5.1-1. Summary: Downgradient Design Parameters and Performance

Alternative	Type/Shape	Distance from STP Reactor (ft)	Length (ft)	Cutoff Permeability (gpd/ft^2)	Maximum Heat Drop (ft)	Potential[a] Gradient at the STP Reactor (ft/ft)	Approximate Strontium-90[b] First Arrival Time (yr)	Maximum Flux Rate[b] of Strontium-90 (pCi/yr)	Total Flux[c] of Strontium-90 (pCi)
Pre-Mitigation	-	-	-	-	-	4.0×10^{-4}	250	6.2×10^{9}	1.3×10^{12}
#1	Cutoff/Linear (Offset)	1000	500	0.0	0.4	5.4×10^{-4}	150	2.0×10^{10}	3.5×10^{12}
#2	"	1000	1000	0.0	0.8	5.6×10^{-4}	150	2.0×10^{10}	4.4×10^{12}
#3	"	1000	2000	0.0	1.8	6.3×10^{-4}	150	2.9×10^{10}	3.4×10^{12}
#4	Cutoff/Linear (Centered)	1000	500	0.0	0.7	2.8×10^{-4}	150	1.2×10^{10}	2.2×10^{12}
#5	"	1000	1000	0.0	1.4	1.7×10^{-4}	250	4.1×10^{9}	8.4×10^{11}
#6	"	1000	2000	0.0	2.2	1.1×10^{-4}	350	9.5×10^{7}	1.7×10^{10}
#7	"	1000	3000	0.0	3.2	0.7×10^{-4}	550	6.1×10^{5}	1.2×10^{8}
#8	Cutoff/Linear (Offset-East)	1000	3000	0.0	3.2	0.8×10^{-4}	350	1.6×10^{8}	3.3×10^{10}
#9	Cutoff/Linear (Offset-West)	1000	3000	0.0	2.5	1.5×10^{-4}	450	1.0×10^{7}	2.0×10^{10}
#10	Cutoff/Linear (Centered)	2000	3000	0.0	2.5	1.3×10^{-4}	350	2.1×10^{8}	4.6×10^{10}
#11	Cutoff/L-East (Centered)	1000	3000	0.0	1.9	1.4×10^{-4}	350	7.1×10^{7}	1.5×10^{10}
#12	Cutoff/L-West (Centered)	"	3000	0.0	2.0	0.7×10^{-4}	850	4.3×10^{1}	6.3×10^{3}
#13	Cutoff/U-Shape (Centered)	"	2000	0.0	2.0	0.8×10^{-4}	850	1.3×10^{2}	1.8×10^{4}
#14	Injection (20 gpm)	1000	-	-	-	-	350	1.7×10^{8}	3.8×10^{10}
#15	Injection (30 gpm)	1000	-	-	-	-	350	1.8×10^{7}	4.1×10^{9}
#16	Injection (40 gpm)	1000	-	-	-	-	450	9.7×10^{5}	2.8×10^{8}

(a) Average potential gradient over the distance 1000 ft downgradient of the STP reactor site.
(b) At the breakthrough section 800 ft downgradient of the reactor site.
(c) During the 1000 year simulation period.

7.5.2 Upgradient Mitigation Measures

It is the intent of the above section on downgradient cutoff design evaluations to illustrate several of the many possible variations that could be employed in the design of contaminant mitigation measures. To avoid duplication, the focus of this section is to provide a basis for comparison of upgradient vs. downgradient cutoff design performance. Therefore, only a few examples are presented in detail. One in particular, is a combination design which utilizes concurrently an upgradient and a downgradient cutoff. Injection schemes are not included since they are not applicable as upgradient measures. While withdrawal schemes might be functional in this regard, they are not considered because of the potential danger of capturing and discharging contaminated water to the surface environment. In addition to those designs that are discussed, the characteristics and performance for a large number of additional upgradient designs are summarized for the interested reader in Table 7.5.2-1 at the end of this section.

7.5.2.1 Upgradient Plant Configuration Design Considerations

The upgradient plant features that may directly influence mitigation design at the STP, with the exception of the buildings in the immediate plant area, are the essential cooling pond and the switchyard. The concerns associated with the cooling reservoir are equally applicable to the pond (i.e., the water may become highly contaminated due to atmospheric deposition and/or it may be necessary to keep the pond functional in the near term following a severe accident). Similarly, the the operability of the switchyard facilities may be required under post-accident conditions or the facilities and their foundations may simply be obstacles to convenient or expedient cutoff construction. Therefore, because of these concerns, the primary location for upgradient cutoffs was selected just upgradient of the essential cooling pond (approximately 800 ft upgradient of the reactor), maintaining its integrity and minimizing interference with the switchyard. A secondary location approximately 1800 ft upgradient of the reactor was also evaluated. At these locations there are no constraints on cutoff placement, thus designs analogous to Alt. #1, #2, and #3 were not evaluated.

While downgradient cutoffs serve to increase the travel path length and reduce the potential gradient, upgradient cutoffs accomplish only the latter. However, they do offer an advantage in that because they are upgradient, the possible hazards associated with contaminated ground water are greatly reduced. For the same reason it might also be possible to initiate upgradient cutoff construction sooner than downgradient alternatives following an accident.

7.5.2.2 Upgradient Design Evaluations

Alt. #17, Alt. #18, Alt. #19, Alt. #20, and Alt. #21 (Centered)

This set of upgradient cutoffs, shown in Figure 7.5.2-1, is centered and located 800 ft from the reactor. Lengths for the five designs are 500, 1000, 2000, 3000, and 4000 ft, respectively. Referring to Figure 7.3-4, this location has hydraulic conductivities of about 1,100 gpd/sq ft, values considerably

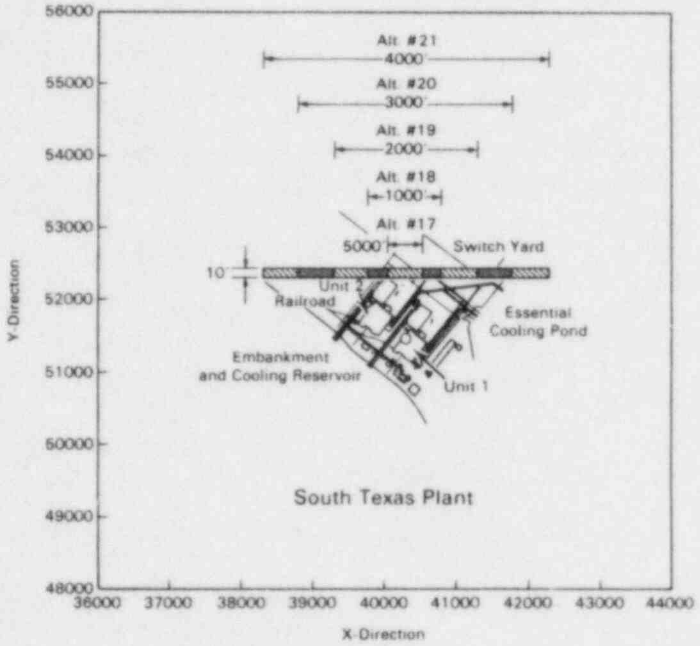


FIGURE 7.5.2-1. Location of Alt. #17 (L=500 ft), Alt. #18 (L=1000 ft), Alt. #19 (L=2000 ft), Alt. #20 (L=3000 ft), and Alt. #21 (L=4000 ft).

higher than where the downgradient cutoffs were located (e.g., 600 gpd/sq ft at y coordinate 50,450 ft). Consequently, the impact of the cutoffs on hydraulic gradients, for a given length, is attenuated relative to the downgradient barriers. The potential surfaces for the 500-ft and 4000-ft cutoffs are shown in Figures 7.5.2-2 and 7.5.2-3 to illustrate the range of effects produced by the five designs. Taking the 500-ft cutoff as an example, it only produces a maximum potential drop of 0.26 ft in contrast to a drop of 0.70 ft for a downgradient cutoff of the same length. Likewise, resultant gradients are higher, 4.5×10^{-4} ft/ft compared to 3.6×10^{-4} ft/ft.

In terms of performance, as with the downgradient cutoffs, flux from the reactor site was actually increased for cutoff lengths less than 1000 ft. Likewise, for longer cutoffs, increased length produces decreased flux (see Figure 7.5.2-4). When compared to the flux rates for downgradient cutoffs in shown Figure 7.5.1-9, it is seen that the upgradient cutoff flux rates are higher for a given length indicating greater downstream migration of the contaminant plume. From this limited analysis it appears that for a given length, downgradient cutoffs are more effective than upgradient cutoffs in reducing contaminant flux from the site. However, with regard to future efforts to remove all contaminated soil, it's noteworthy that because the upgradient barriers do not obstruct the flow (i.e., they simply reduce gradient and velocity), less lateral spreading of contaminant occurs. This is clearly illustrated in Figure 7.5.2-5 which shows the strontium-90 plume at 1000 years

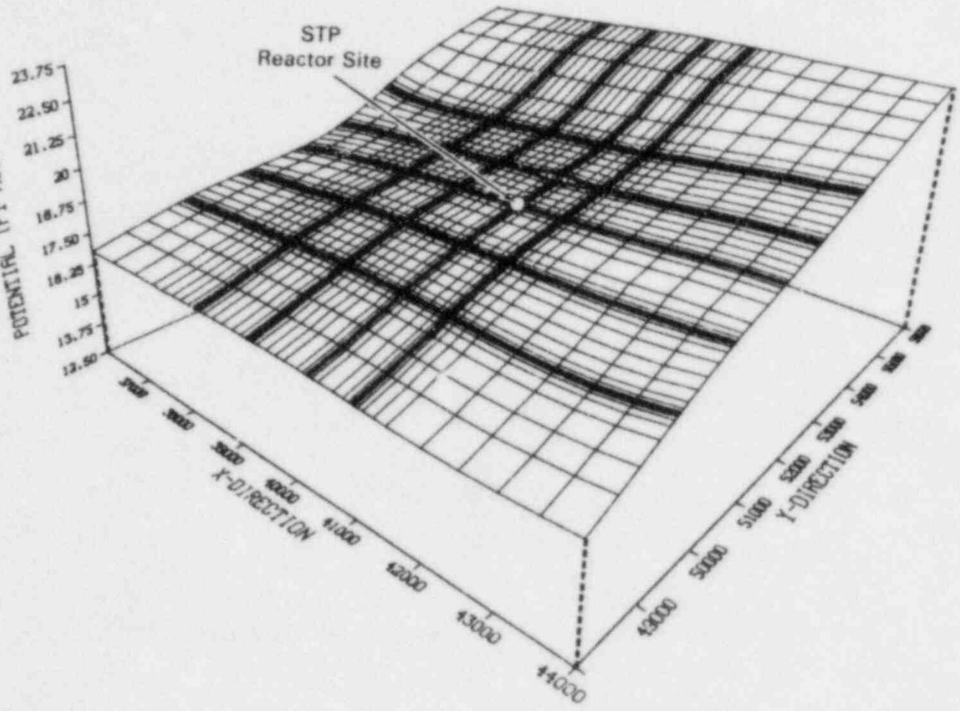


FIGURE 7.5.2-2. Simulated Potential Surface, Alt. #17 (L=500 ft).

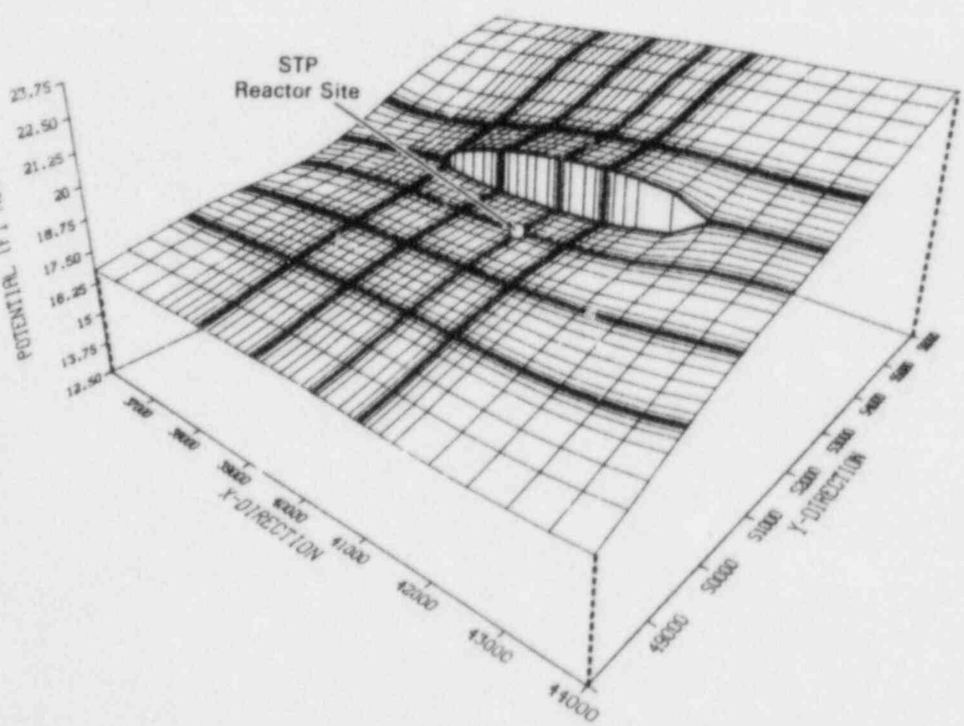


FIGURE 7.5.2-3. Simulated Potential Surface, Alt. #21 (L=4000 ft).

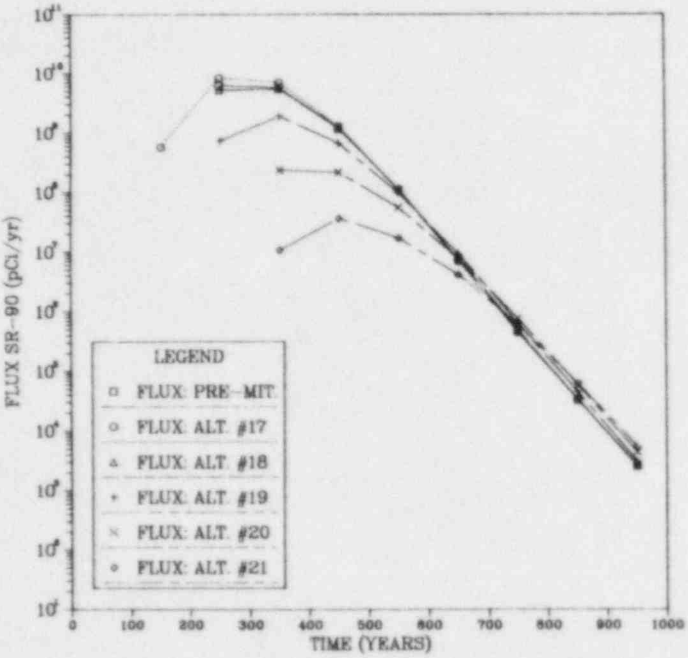


FIGURE 7.5.2-4. Simulated Flux Rates: Alt. #17 (L=500 ft), Alt. #18 (L=1000 ft), Alt. #19 (L=2000 ft), Alt. #20 (L=3000 ft), and Alt. #21 (L=4000 ft).

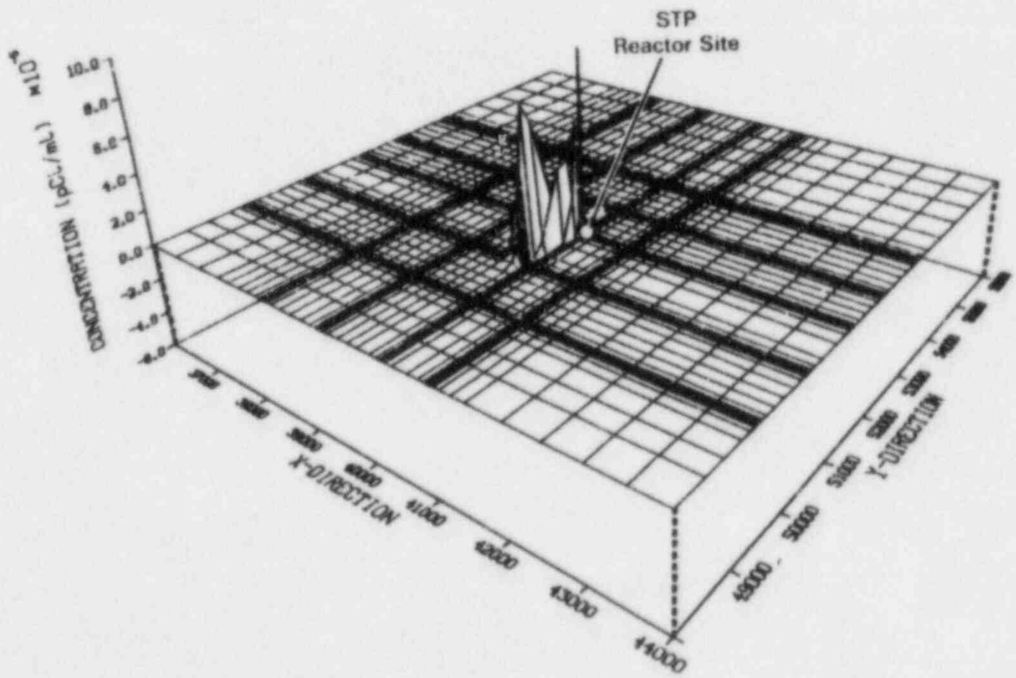


FIGURE 7.5.2-5. Simulated Strontium-90 Concentrations at 1000 years: Alt. #20 (L=3000 ft).

for the 3000-ft cutoff. The plume width is only about 300 ft compared to a width of over 1000 ft for the corresponding downgradient cutoff (see Figure 7.5.1-11).

Alt. #22

The only combination design evaluated consists of a 1500-ft cutoff located 1000 ft downgradient of the reactor and a 1500-ft cutoff placed 800 ft upgradient (see Figure 7.5.2-6). The potential field produced by the two cutoffs is plotted in Figure 7.5.2-7. The two barriers, though the same length, produce different head drops, demonstrating the influence of hydraulic conductivity on design performance. The maximum head drop across the upgradient cutoff is about 0.5 ft. In contrast, the downgradient cutoff, located in the a region with relatively low conductivities, produces a drop of almost 0.9 ft. The combined effect of the two cutoffs is to produce a gradient at the reactor site of 1.5×10^{-4} ft/ft. The resultant flux is compared to the pre-mitigated flux in Figure 7.5.2-8. The first arrival is delayed to between 400 and 500 years and the peak flux rate is reduced over two orders of magnitude. The concentration distribution, plotted in Figure 7.5.2-9, shows that the plume is largely contained to within 1000 ft of the reactor and has spread to a width of about 1000 ft.

A summary of the upgradient alternatives, including those analyzed but not discussed is provided in Table 7.5.2-1.

7.5.3 Sensitivity Analysis: Cutoff Design Parameters

One of the keys to identifying a "best" design, given specific design objectives, is to develop insight into how performance changes with variations in major design characteristics. Mathematical models are ideally suited for accomplishing this in that once a model is developed for a site, any number of designs can be simulated without appreciable additional cost. Though there is no attempt to select an optimum design in this study, for demonstration purposes a limited sensitivity analysis of cutoff design parameters is conducted. The approach used is simply to compare mitigation performance of different alternatives as a function of selected design parameter values. The parameters evaluated include length, distance from the reactor, cutoff permeability, shape and orientation. Performance is measured on the basis of the strontium-90 flux at the breakthrough section 800 ft downgradient of the reactor. In an actual mitigation design situation, many more simulations than are presented here would be conducted, perhaps in conjunction with optimization techniques, to select the most appropriate design.

7.5.3.1 Cutoff Length

The general effect of increasing cutoff length is to decrease velocities in both the upgradient and downgradient directions. It follows that velocities approaching zero could be achieved with a sufficiently long cutoff. However, infinitely long cutoffs are not necessary or practical. As the simulation results have shown (see Figure 7.5.1-10), significant reductions in flux can be achieved with cutoffs of several thousand feet. The question then is how long

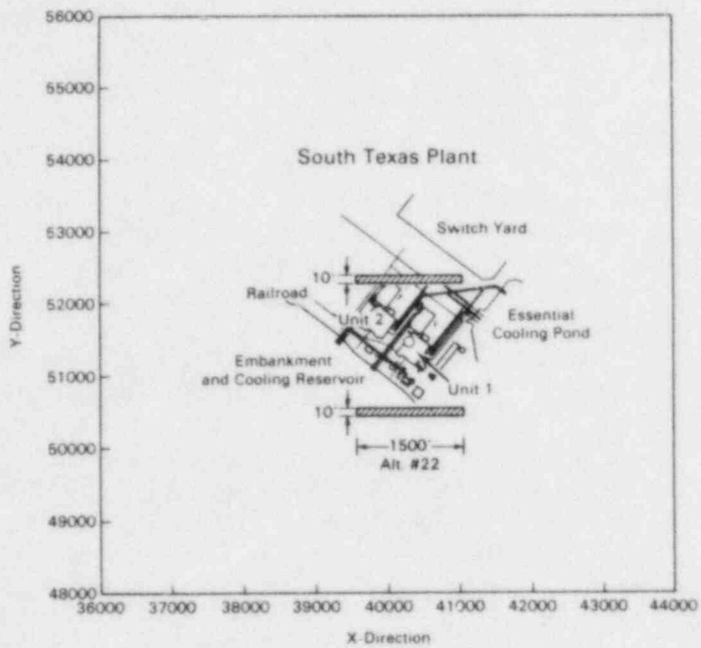


FIGURE 7.5.2-6. Location of Alt. #22 (Combination Design).

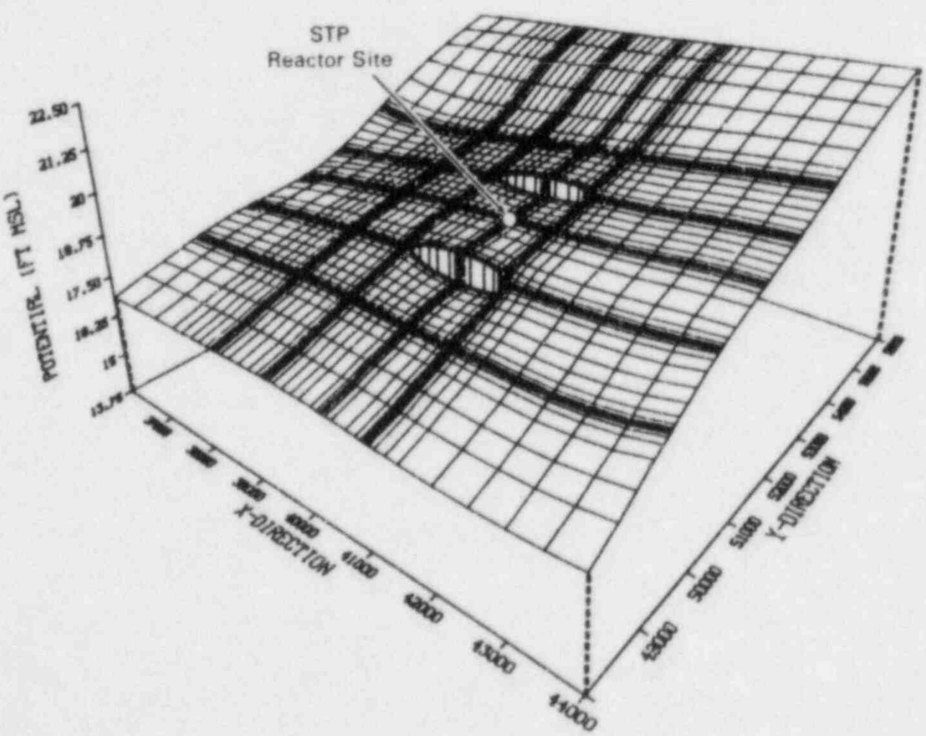


FIGURE 7.5.2-7. Simulated Potential Surface, Alt. #22 (Combination Design).

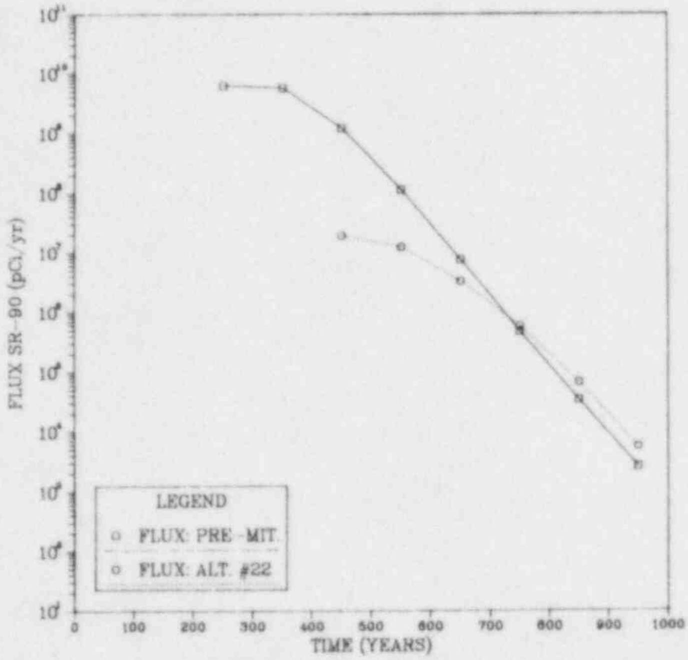


FIGURE 7.5.2-8. Simulated Flux Rates: Alt. #22 (Combination Design).

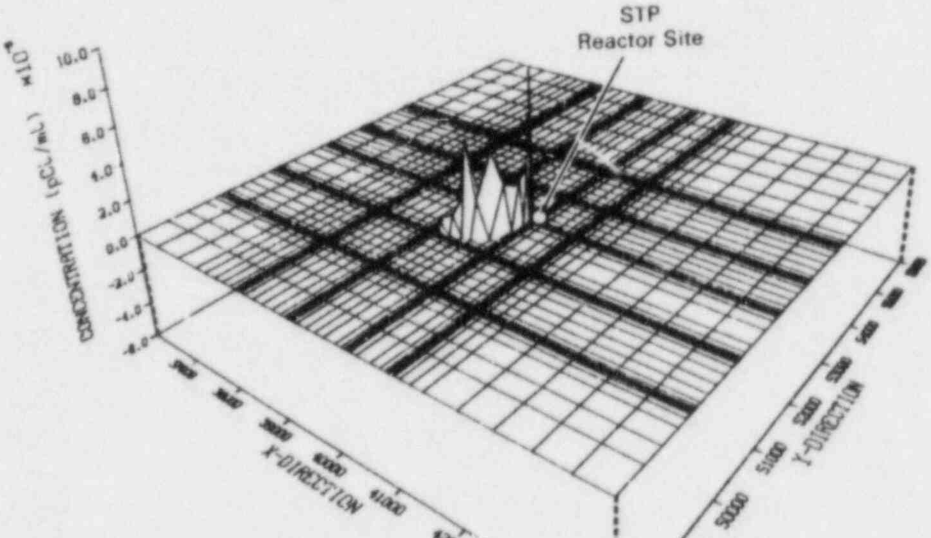


FIGURE 7.5.2-9. Simulated Strontium-90 Concentrations at 1000 years: Alt. #22 (Combination Design).

TABLE 7.5.2-1. Summary: Upgradient Design Parameters and Performance

Alternative	Type/Shape	Distance from STP Reactor (ft)	Length (ft)	Cutoff Permeability (gpd/ft^2)	Maximum Heat Drop (ft)	Potential[a] Gradient at the STP Reactor (ft/ft)	Approximate Strontium-90[b] First Arrival Time (yr)	Maximum Flux Rate[b] of Strontium-90 (pCi/yr)	Total Flux[c] of Strontium-90 (pCi)
#17	Cutoff Linear (Centered)	800	500	0.0	0.3	4.5×10^{-4}	150	8.3×10^{9}	1.7×10^{12}
#18	"	800	1000	0.0	0.4	4.3×10^{-4}	250	5.5×10^{9}	1.2×10^{12}
#19	"	800	2000	0.0	0.9	3.8×10^{-4}	250	1.9×10^{9}	3.4×10^{11}
#20	"	800	3000	0.0	1.4	3.3×10^{-4}	350	2.3×10^{8}	5.1×10^{10}
#21	"	800	4000	0.0	1.9	2.7×10^{-4}	350	3.6×10^{7}	6.8×10^{8}
#22	Cutoff/Combination (Centered)	800	3000	0.0	0.9	1.5×10^{-4}	450	1.9×10^{7}	3.5×10^{9}
#23	Cutoff/Linear (Offset-East)	800	3000	0.0	1.8	3.7×10^{4}	250	1.2×10^{9}	2.1×10^{11}
#24	Cutoff/Linear (Offset-West)	800	3000	0.0	1.5	3.7×10^{-4}	250	8.8×10^{8}	1.5×10^{11}
#25	Cutoff/Linear (Centered)	800	3000	0.0	2.2	2.3×10^{-4}	250	5.0×10^{9}	1.1×10^{12}
#26	"	800	3000	0.001	1.7	3.5×10^{-4}	250	5.5×10^{8}	1.3×10^{11}
#27	"	800	3000	0.01	1.7	3.5×10^{-4}	250	7.7×10^{8}	1.3×10^{11}
#28	"	800	3000	0.1	1.7	3.6×10^{-4}	250	7.7×10^{8}	1.3×10^{11}

(a) Average potential gradient 1000 ft downgradient of the STP reactor site.
(b) At the breakthrough section 800 ft downgradient of the reactor site.
(c) During the 1000 year simulation period.

of a cutoff should be constructed? This question must be answered based on specific performance criteria. As a hypothetical example, it might be determined that site restoration is planned 300 years following a severe accident at the STP; consequently, specific mitigation performance objectives might be to extend the first arrival time at the breakthrough section to greater than 300 years. Figure 7.5.3-1 is a plot of first arrival time as a function of cutoff length for linear, downgradient cutoffs located 1000 ft from the reactor and linear, upgradient cutoffs located at a distance of 800 ft. The plot clearly shows the relationship between cutoff length and travel time. For the pre-mitigation case (cutoff length equal to zero) the first arrival of strontium-90 occurs at approximately 250 years. For both downgradient and upgradient designs, for lengths less than 1000 ft, the arrival time is actually shortened. For downgradient cutoffs with lengths greater than 1000-ft, there is an approximate increase in first arrival time of 150 years for a 1000-ft increase in length. The upgradient designs, which are less effective for the STP, produce an average increase of less than 50 years for each 1000-ft increment over 1000 ft. For a first arrival of 300 years, the minimum acceptable linear cutoff is approximately 1500 ft for an downgradient design and 2500 ft for an upgradient alternative.

In addition to concerns about first arrival of contaminant, there could also be interest in the total flux from the site. For example, site restoration might involve removal and disposal of all contaminated soils. To minimize this future effort, the objective of mitigation would be to contain the contaminant at the site (i.e, reduce flux from the site). A performance measure for this objective is total flux from the site. Figure 7.5.3-2 shows the simulated total flux for the 1000-year simulation period for the same downgradient and upgradient designs discussed above. These curves show the increasing marginal gains in flux reduction for increased cutoff length. Such curves, if properly extended, could be used to determine the design length necessary to totally eliminate flux or to limit flux to a set level.

7.5.3.2 Cutoff Distance from the Reactor

Selection of the exact location for construction of mitigative measures will be determined by a number of factors such as the nature and location of plant facilities, the level and extent of both surface and subsurface contamination, prevailing wind conditions and the type of design. Therefore, in selecting a final design it's important to understand how distance from the contaminant source will effect the performance of designs in question. To begin to address this question, Figure 7.5.3-3 contains plots of simulated flux against time for a 3000-ft cutoff at four different locations: 1000 ft downgradient, 2000 ft downgradient, 800 ft upgradient, and 1800 ft upgradient. For both the downgradient and upgradient designs an increase in distance from the source significantly impacts performance, both in terms of flux rate and first arrival time. For the downgradient designs, an increase in distance from 1000 to 2000 ft reduces the first arrival time by about 200 years while increasing the initial flux rate by approximately three orders of magnitude. The same increase in distance for the upgradient designs produces a 100-year increase in arrival time and an order of magnitude increase in flux rate. Though limited in scope and extent, this brief analysis points out the

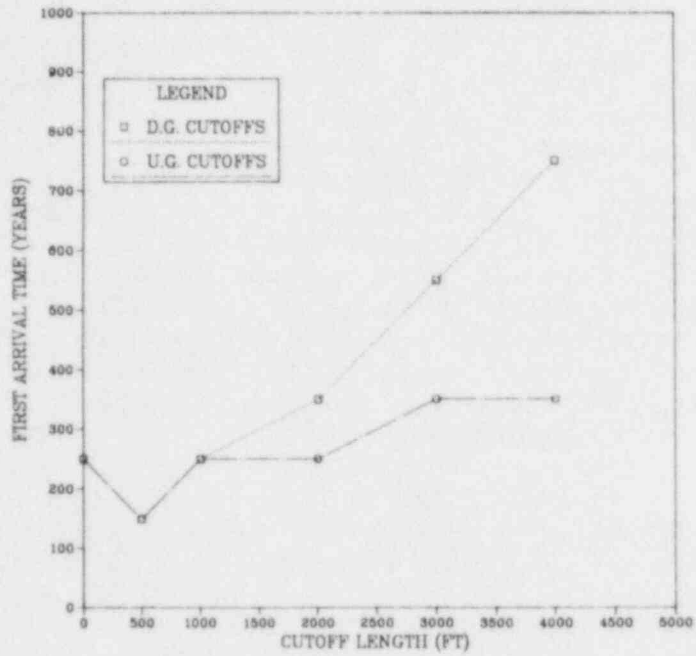


FIGURE 7.5.3-1. First Arrival Times as a Function of Cutoff Length.

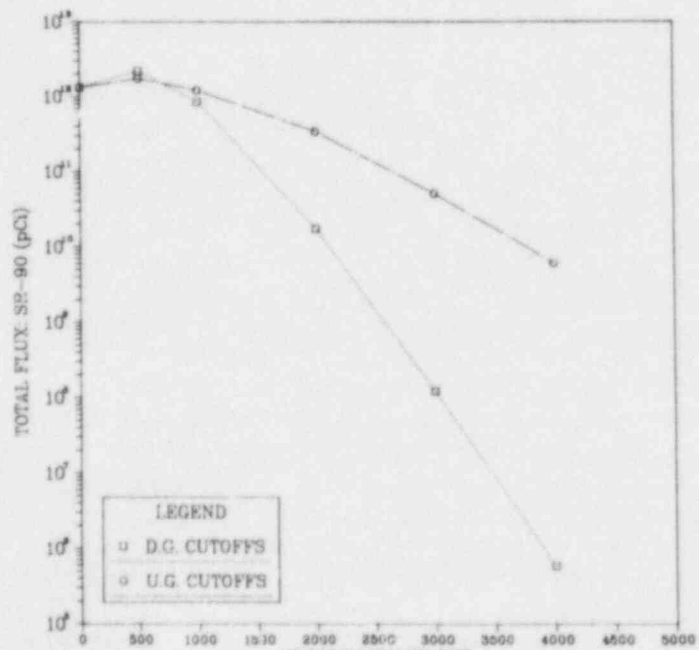


FIGURE 7.5.3-2. Simulated Total Flux as a Function of Cutoff Length.

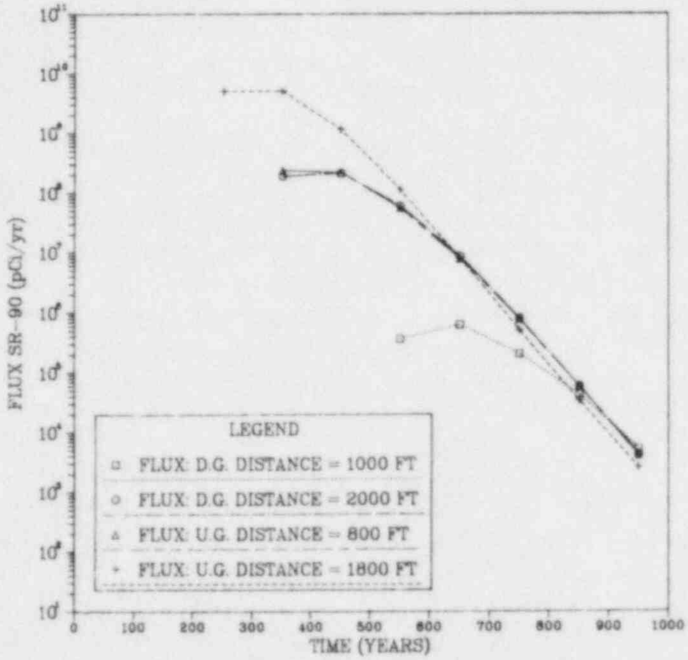


FIGURE 7.5.3-3. Comparison of Flux Rates for Downgradient and Upgradient Designs.

importance of distance from the site to mitigation performance and the potential trade-offs that could be made between distance and cutoff length.

7.5.3.3 Cutoff Permeability

An important aspect of grout cutoff performance as a barrier to groundwater contaminant migration is the grout permeability, in terms of both the "as constructed" condition and the change in permeability with time. For simplicity, all of the grout cutoff flow and transport simulations discussed above assume cutoff permeabilities equal to 0.0 gpd/sq ft. Under actual conditions it's not realistic to expect total permeability reduction. Laboratory tests with silicate-based grouts achieved permeability values averaging approximately 4.8×10^{-7} cm/sec or about 0.01 gpd/sq ft. According to Baker (1982), chemically grouted sands exhibit permeability reductions relative to the host media of three to six orders of magnitude. For the STP site this would indicate possible values on the order of about 0.001 to 0.1 gpd/sq ft. Also, less than ideal performance can be introduced during construction. Littlejohn (1982) identifies three causes of variability in grout curtain properties which affect permeability:

1. inadequate/improper mixing of grout,
2. variations in quantity and quality of grout material, and
3. apparent variations from testing procedures.

Avoiding these types of problems and assuring acceptable quality construction requires rigid engineering supervision of all grouting operations (see Section 4.3.1.6 for further discussion of grout curtain construction considerations).

Another issue for consideration is grout durability. In place, grouts are subject to deterioration due to wet-dry cycles, weathering, exposure to pH extremes, etc. Silicate-based grouts with silicate concentrations greater than 35% are resistant to these effects. Nonetheless, mitigation utilizing grout cutoffs will require continual monitoring to ensure maintenance of permeability reduction. Undoubtedly, with time, repair and/or replacement will be necessary to meet long-term mitigation objectives that span hundreds of years.

To gain insight into how variable or deteriorating cutoff permeability may affect performance, four simulations were conducted varying the cutoff permeability. The analysis is based on the 3000-ft design located 1000 ft downgradient of the reactor with permeability values of 0.001, 0.01, 0.1, and 0.1 gpd/sq ft. The simulated flux rates are presented in Figure 7.5.3-4 and compared to the zero permeability case and the pre-mitigation case (permeability equal to that of the host media). The 0.001 permeability curve, not shown on the plot is practically identical to that for 0.0 case. Similarly, the 0.01 and 0.1 permeability cases show only minor increases in flux. The increase to 1.0, however, results in a dramatic change, increasing the maximum flux rate from about 1×10^6 to almost 6×10^7 pCi/yr. The time of first arrival is also reduced about 200 years. Nonetheless, the simulation results show that even with appreciable deterioration of the grout's permeability reduction properties (i.e., over a couple orders of magnitude) significant reduction in contaminant flux is achieved. Also, over the ranges of achievable permeability reduction (i.e., 0.001 to 0.1 gpd/sq ft) performance is essentially unchanged.

Cutoff Shape

From the discussion of the different-shaped cutoff designs in Section 7.5.1.2, it's quite evident that perhaps the single most important design parameter with regard to reduction of flux is design shape. For the purpose of evaluating relative performance, several designs are compared including a linear or straight line, U-shape, L-shape, and an upgradient/downgradient combination. All four designs consist of a total cutoff length of 3000 ft and are 1000 ft downgradient from the reactor. The L-shape design has the leg section attached to the west end (high conductivity region). The combination design consists of two 1500-ft sections, one located 1000 ft downgradient and the other 800 ft upgradient. The comparison of the 1000-year flux rates for the four designs in Figure 7.5.3-5 shows the dramatic differences in performance. Clearly, the L-shape and U-shape designs are the best evaluated. This is attributable to the leg sections obstructing lateral flow through the high conductivity region of the contaminant flow path. The other two designs obstruct the flow and decrease the potential gradient but do not have the same beneficial effect. Also, the combination design, per unit length, is not as effective as the single linear design. This comparison points out the advantages to accurately determining the hydraulic and

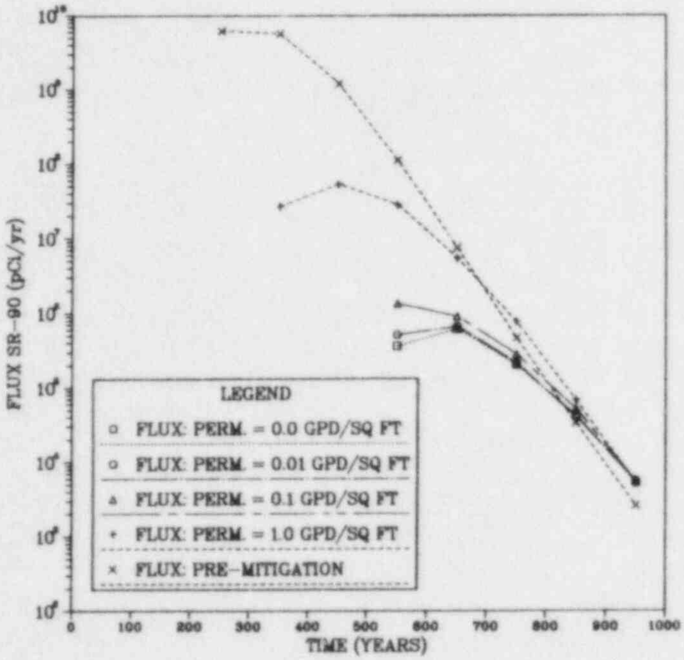


FIGURE 7.5.3-4. Comparison of Flux Rates for Various Cutoff Permeabilities (L=3000 ft, 1000 ft Downgradient).

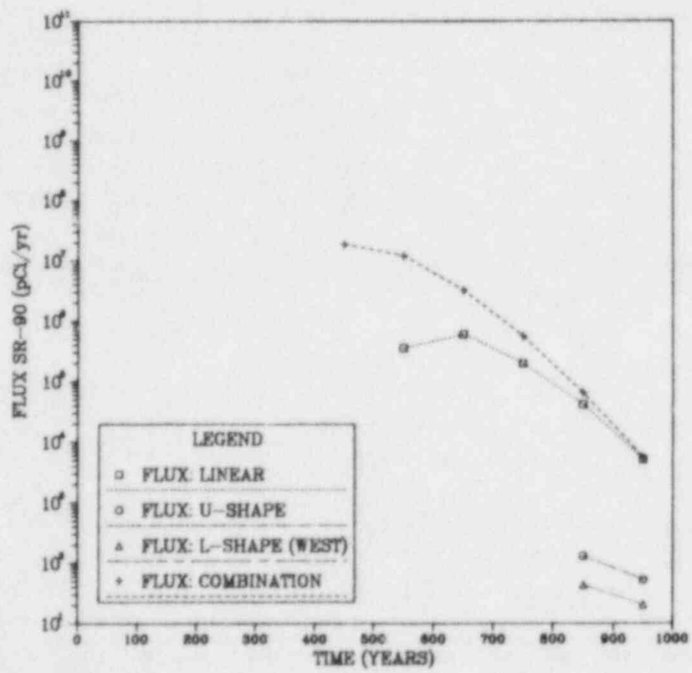


FIGURE 7.5.3-5. Comparison of Performance as a Function of Design Shape (Total Length=3000 ft, 1000 ft Downgradient).

geohydrologic conditions of a site and tailoring the mitigation design to those conditions. Though the L-shape and U-shape designs would most likely be more difficult to construct it appears the added difficulty would be compensated for by reduced total cutoff length for a desired level of performance.

7.5.4 Sensitivity Analysis: Hydrogeologic/Transport Parameters

Perhaps the most important activity in the design of appropriate mitigation measures is the site characterization which describes the pertinent soils, geology, and hydrology. Because mitigation selection, evaluation, design, and implementation are based on understanding gained from this characterization, it follows that improving the reliability of the hydrogeologic data for the site will improve the selection of the appropriate strategy. Likewise, uncertainty in determining key site parameters can result in overestimation of mitigation performance. Current research is attempting to address ways to quantify such uncertainty and integrate it directly into the evaluation process. Given the importance and the scale of mitigation that would be required following a severe nuclear reactor accident, such methods might justifiably be applied. An alternative, consistent with the demonstrative nature of this study, is to conduct sensitivity analyses whereby individual parameters are changed and, using a numerical model, the resulting effects on flow and transport determined. This approach can be employed comprehensively as is done in Monte Carlo analysis or to a more limited extent by simple adjustments to key parameters within the model. In this study, as a limited demonstration of what should and can be done using numerical models, three parameters are considered: hydraulic conductivity, retardation, and dispersivity. Hydraulic conductivity is an aquifer property which directly affects flow velocities and thus, indirectly, contaminant transport. Retardation and dispersivity on the other hand, are transport parameters. Dispersivity coefficients provide a measure of the hydrodynamic dispersion which produces mixing and spreading of transported contaminants with respect to the ground-water flow direction. Retardation represents the reduction in contaminant travel velocity relative to the ground-water flow velocity due to reversible equilibrium controlled adsorption. The base case for the sensitivity studies is the 3000-ft linear cutoff located 1000 ft downgradient from the reactor.

Hydraulic Conductivity

Hydraulic conductivity is a measure of the capacity for flow through a unit area of aquifer. Estimates of hydraulic conductivity at a site are determined by testing core samples in the laboratory or by field pump tests. Typically hydraulic conductivity values are highly variable because of heterogeneities in the geologic materials of an aquifer, ranging over several orders of magnitude. In most cases the available number of pump tests is inadequate to fully characterize the distribution of hydraulic conductivities in an aquifer. As discussed in Section 6.4, the lack of fully adequate data is compensated for through model calibration, the process of adjusting model parameters, based on understanding of the ground-water flow system, until simulated results compare favorably to observed results. Conducting the model calibration provides an appreciation for how the ground-water flow model responds to changes in hydraulic parameters. However, recognizing that

calibration is an inexact process, it's just as important to gain an appreciation for the sensitivity of the transport processes to hydraulic properties of the model.

As discussed above, extensive parameter sensitivity studies may be warranted based on the importance of the results and uncertainty associated with the available data. Here, a simple analysis was done to gain some understanding of how mitigation performance might be affected by hydraulic conductivities different from those assumed in the initial performance evaluations. In addition to the base case, two simulations were made, one assuming all hydraulic conductivities are 50% greater and the second assuming all conductivities are 100% greater. The strontium-90 flux rates for the three cases are presented in Figure 7.5.4-1. As expected the adjusted hydraulic conductivities result in reduced mitigation effectiveness, increasing flow velocities, thus reducing travel times, and increasing flux rates. The increase of 50% produced a decrease in first arrival time of approximately 200 years and increased the maximum flux rate by approximately two orders of magnitude. The incremental effect of increasing conductivities an additional 50% is markedly less. The first arrival time is reduced only an additional 100 years (300 years overall) and the maximum flux rate by just over one order of magnitude (three orders of magnitude overall). The results of this brief analysis show that indeed uncertainties in the aquifer hydraulic characteristics could result in overestimation of mitigation performance and should be quantified and factored in to the design process.

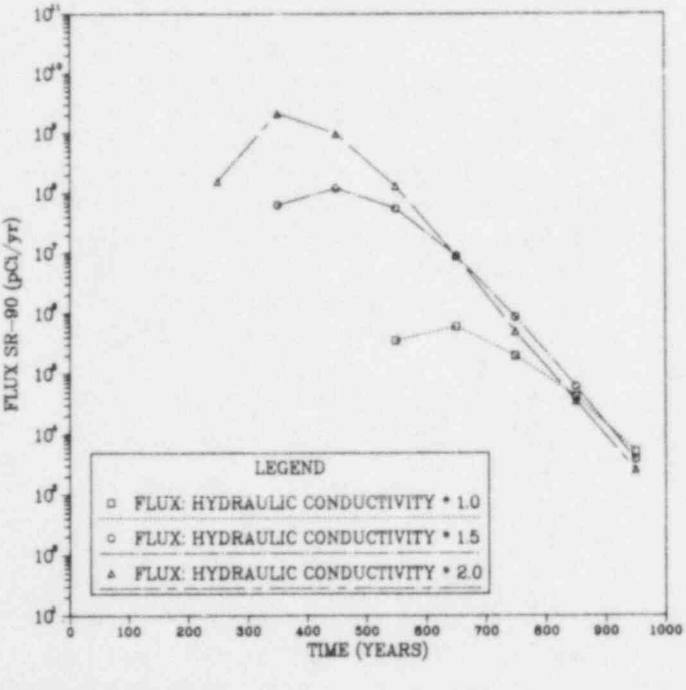


FIGURE 7.5.4-1. Effects of Varying Aquifer Hydraulic Conductivity on Flux Rates (L=3000 ft, 1000 ft Downgradient).

Retardation

Under ideal conditions determination of transport parameters parallels that of hydraulic parameters whereby initial parameter values are estimated from available data and are subsequently calibrated based on comparisons of field-measured and model-predicted contaminant transport. In reality, the necessary field data related to radionuclide migration are not likely to exist. Therefore, parameter estimates are based entirely on available information. In the case of retardation coefficients, their value is related directly to the equilibrium distribution coefficient (K_d) which is determined empirically in the laboratory and is a function of both the contaminant properties and the aquifer geologic material.

As noted in Table 3.3.2-2, a wide variation in K_d's have been determined for individual radionuclides in particular geologic materials. Representative values suggested for strontium-90 in porous silicate such as exists at the STP site are 10 to 50 ml/g. For the sake of conservatism, the mitigation evaluations conducted in this case study are based on the lower value which translates into a retardation coefficient of approximately 46.0. However, as discussed in Section 3.3.2.3 wide variations in K_d's have been determined in the laboratory: the range of values reported for unconsolidated, porous silicates not containing clay and silt is 1 to 30 ml/g (Table 3.3.2-2). In light of the potential uncertainty associated with the estimated retardation coefficient, it's imperative that a sensitivity study be conducted to quantify the potential impact of this uncertainty.

For this study the base case mitigation results (retardation equal to 46.0) are compared to mitigation results assuming three different retardation factors: 35.0, 23.0 and as a worst-case, 1.0. The resulting flux rates are shown in Figure 7.5.4-2. Reduction of the retardation coefficient by 50%, in effect, doubles the convective portion of the transport velocity. The impact of this is evident in the increased flux rates for each of the runs relative to the base case. For each 25% decrease in retardation, there is about a 100-year decrease in first arrival time and a two order of magnitude increase in the maximum flux rate. The curve for retardation equal to 1.0 (i.e., no retardation) closely resembles the source release curve, indicating that practically all of the contaminant would reach the breakthrough section within the 100-year time steps used in the transport simulations. Clearly, retardation effects are a very important consideration in mitigation design and values should be estimated conservatively.

Dispersivity

Like retardation coefficients, changes in dispersivities directly effect the rate and extent of contaminant transport. As discussed in Section 6.6.6.1, there are significant problems in considering spatial variability of aquifer hydraulic properties and their effects on field-scale dispersion processes. Given the total lack of transport data available at the STP site, the longitudinal (D_L) and transverse dispersivity (D_T) coefficients (164.0 and 8.0, respectively) were estimated based on information in the literature (Yeh 1981;

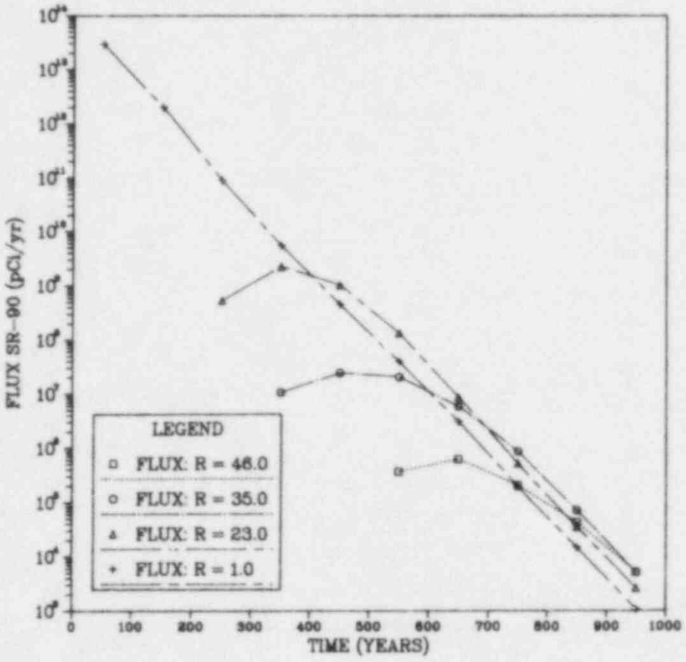


FIGURE 7.5.4-2. Effect of Varying Retardation (R) on Flux Rates (L=3000 ft, 1000 ft Downgradient)

Gelhar and Axness 1981). There is a wide range of values reported in the literature. The field observations reported by Gelhar and Axness (1981) in particular illustrate the variability of dispersivity as a function of geologic material and travel distance. To evaluate the importance of dispersivity to mitigation performance at the STP, strontium-90 transport was simulated using dispersivity coefficient values 1.5 and 2.0 times that of the base case. The results of the simulations are shown in Figure 7.5.4-3. In the uppermost curve (D_L equal to 328.0 ft and D_T equal to 16.0), the maximum flux rate is increased just over one order of magnitude and the first arrival time is reduced about 100 years. As noted with the previous parameters evaluated, the initial increment in change produces the greatest change in transport while subsequent value changes have incrementally less effect on simulation results. Overall, the simulations results are less sensitive to incremental variations in dispersivity than to comparable changes in retardation and hydraulic conductivity.

7.6 MITIGATION COSTS

In the event of a severe nuclear reactor accident, the immediate concerns related to the ground-water pathway will be prevention of releases to accessible environmental, such as wells or surface waters. Once these concerns are alleviated, either by mitigation or determination that an immediate problem does not exist, the focus will likely be toward site restoration. Whichever is the case, if mitigation is deemed warranted, the selection of an appropriate strategy will be based upon engineering feasibility, effectiveness, and cost.

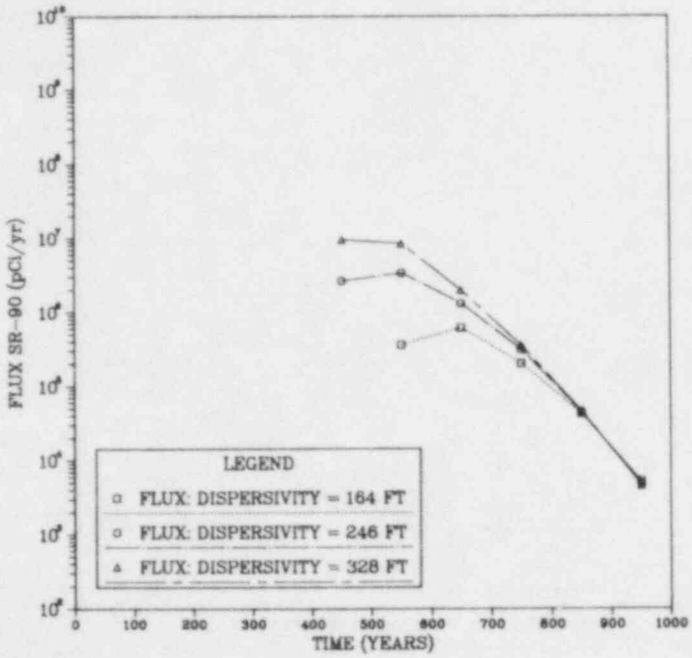


FIGURE 7.5.4-3. Effect of Varying Dispersivity on Flux Rates (L=3000 ft, 1000 ft Downgradient)

First and foremost, one or more feasible alternatives will be identified that meet pre-determined performance objectives. From these alternatives then, the least cost strategy will be implemented.

Costs for a typical grouting operation, for example, are a function of the following (U.S. Army Office of the Chief of Engineers 1973):

- Initial cost of materials,
- Location of job site,
- Quantities and types of grout to be used,
- Volume of material to be placed,
- Labor,
- Overhead,
- Equipment rental, and
- Drilling cost.

From this list it's readily seen that actual costs for mitigation at a specific site are highly dependant on site-specific factors. Example unit costs for construction of mitigation measures such as grout curtains, slurry walls, etc., are provided in Section 4.0.

Because of the type of contaminants involved and the long-term nature of the mitigation requirements related to a severe reactor accident, determination of true costs needs to consider a number of additional factors. One of the primary concerns will be worker safety. The possibility of worker exposure to atmospheric releases of radioactive material, dependant on site-specific

accident and meteorological conditions, may preclude the implementation of certain options entirely. As noted from the mitigation evaluations discussed above, the closer a scheme is to the contaminant source, the more effective it will be. Because of the presence of radiation near the site, some schemes may either cost too much because of necessary safety precautions or have to be placed so far away that they become ineffective.

Another important factor in determination of costs for selected mitigation alternatives is durability. Because of the high levels and large amounts of radioactivity associated with a severe reactor accident, mitigation strategies may have to function for extremely long periods of time, perhaps on the order of hundreds of years. There is no experience base for the design, construction and operation of mitigation schemes such as grout cutoffs or injection wells for even a fraction of this period of time. Therefore, a key consideration in selection will be the design life, maintenance costs, replacement costs, and reliability. Grout cutoffs, while requiring heavy front end capital expense, may be advantageous because of their easy repairability. An injection scheme for the development of a hydraulic barrier, on the other hand, would be maintenance intensive and would require redundant capability to sustain continuous, reliable operation.

In summary, the incorporation of costs into the selection of appropriate mitigation measures must be based on a site-specific, detailed investigation of ground-water flow and contaminant transport in conjunction with an accurate assessment of the levels and extent of both surface and subsurface contamination at the time of construction. While cost considerations may be secondary to meeting the ultimate objective of minimizing risk to man and the environment, they may be a deciding factor in the selection of a "best" alternative.

7.7 MITIGATION SCHEME SELECTION: SUMMARY AND DISCUSSION

Selection of appropriate mitigation techniques for ground-water contamination associated with a severe reactor accident is highly site specific and requires thorough evaluation of the nature and extent of the contaminant release, site characteristics, and feasible mitigative alternatives. Additionally, a myriad of other factors are integral to the selection process including the nature of accessible environments; worker safety during mitigation design, construction and operation activities; costs; etc. At present there is no known way to directly integrate all of these factors and quantitatively determine an "optimal" mitigation strategy. The alternative is to address the problem systematically and methodically, using a psuedo-decision tree approach based on detailed site characterization and modeling studies. The desired result is sufficient information to initiate detailed engineering design studies of one or more recommended strategies. The key elements of the selection process can be addressed in a hierarchical fashion at four levels:

Level 1: Is mitigation required? If yes,
Level 2: Is mitigation feasible? If yes,
Level 3: Select and evaluate performance of feasible strategies.

Level 4: Rank feasible alternatives on the basis of engineering feasibility, performance, reliability, costs and other factors deemed appropriate. As discussed below, within each of the four levels a number of issues must be addressed.

Level 1: Is Mitigation Required?

The need for mitigation is dependant on a number of factors related to the nature of the reactor accident, the general hydrological characteristics of the site, presence of accessible environments, etc. First, it must be determined if a significa source of contamination to ground water has been created (e.g., core melt breach of the reactor basemat). If this is the case, referring to the description of core melt accidents in Chapter 2.0, the time required for the core melt debris to cool sufficiently for ground water to come in contact with it is on the order of six months to a year. During this time, activities related to the ground-water pathway should focus on compiling data and information about the local and regional ground-water system underlying the site. Based on the available information, a number of preliminary assessments must be made:

- What are the general characteristics (i.e., direction) of ground-water flow?
- Are there accessible environments downgradient from the source such as lakes, streams, estuaries or water wells?
- What is the "severity" of the contamination (i.e., source term)?
- Is there significant potential for near-term or long-term contaminant migration to accessible environments?

At this stage, preliminary initial assessments are based on available data, simplified analyses, and conservative assumptions.

The next phase of investigation requires initiation of detailed data collection and site characterization studies. Utilizing the best available technology, a detailed consequence analysis is necessary to confirm preliminary conclusions. If the assessment indicates mitigation is necessary, the additional information will greatly enhance the selection and evaluation of alternative mitigation strategies. The steps involved in the consequence analysis are delineated in the upper half of the flow chart in Figure 7.7-1. An initial conceptual model is constructed based on preliminary assumptions of study area size, boundary conditions, stratigraphy, ground-water flow direction, etc. Using the conceptual model, an appropriate computer code is selected and development of the numerical model can begin. The model is first used to synthesize the available data and to test the validity of the conceptual model. As required, refinement of the conceptual model and calibration of the numerical model continue iteratively until the the two models are

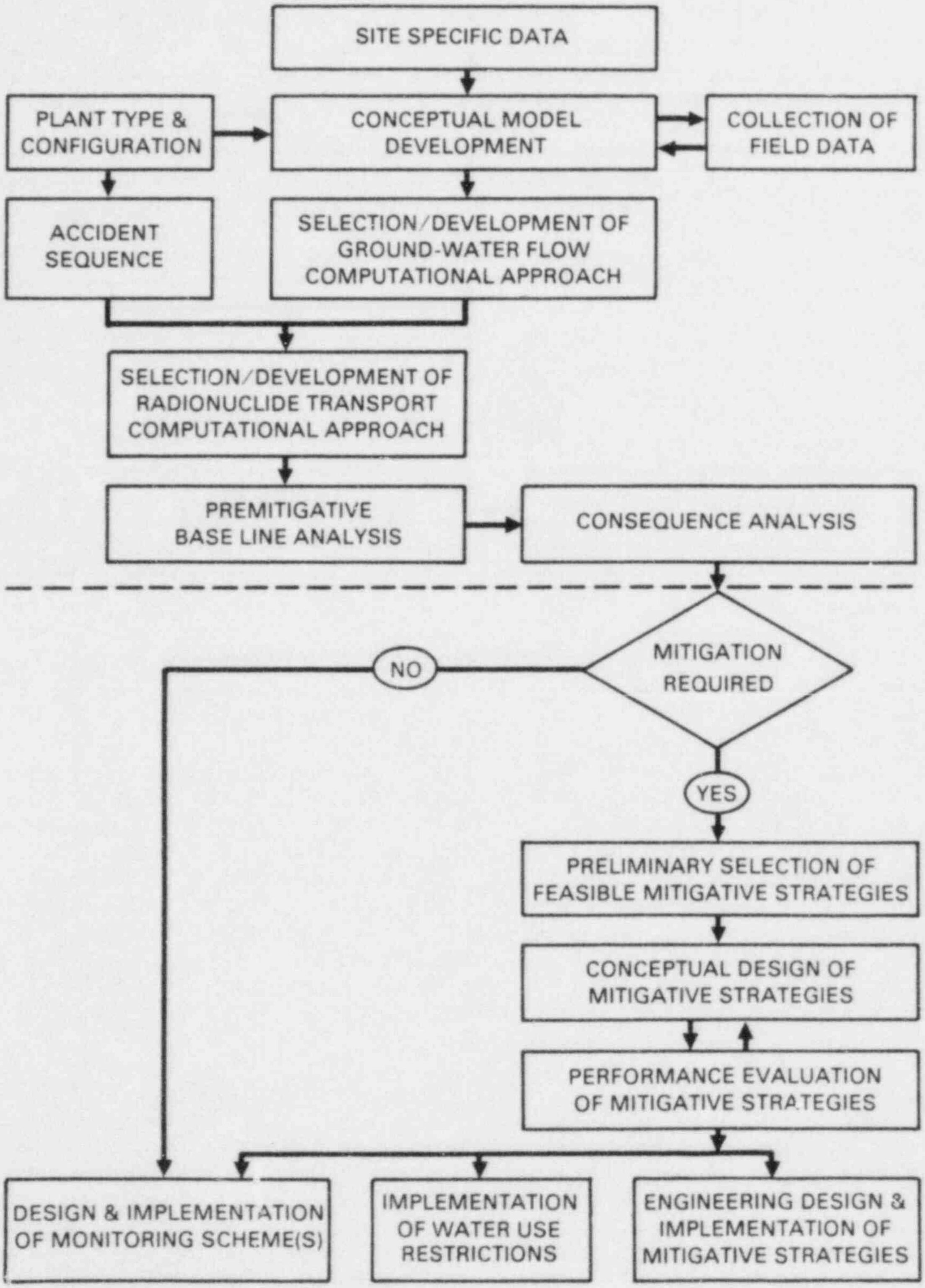


FIGURE 7.7-1. Flow Chart of the Site-Specific Mitigation Design Process.

consistent with each other and the numerical model reproduces observed data. The calibrated model is then used to predict pre-mitigated contaminant transport to determine the environmental risks and the need for mitigation.

Level 2: Is Mitigation Feasible?

If the results of the Level 1 analysis support the need for mitigation, the next critical step is to determine what the objectives of mitigation are to be. Four general possibilities exist:

1. mitigate at the greatest achievable level given site and accident specific constraints,

2. mitigate to reduce the environmental consequences of surface exposure to an acceptable risk level,

3. interdict to provide long term contaminant isolation in a portion of the ground-water system, or

4. perform interim mitigation to minimize contaminant migration pending site restoration or further analysis. Once the mitigation performance requirements are generally spelled out, it is possible to evaluate first the possibility then the feasibility of achieving them.

A good understanding of the expected contaminant transport is gained from the detailed consequence analysis. This understanding provides the basis for determining the feasibility of successful mitigation in terms of available time for implementation of an initial strategy (based on travel time estimates), suitability of geologic conditions for mitigation construction (based on preliminary screening of feasible mitigative strategies) and accessibility of construction sites sufficiently close to the source to be effective. Knowledge of the expected areal extent of the contaminant plume at the time of construction based on monitoring and pre-mitigation model results is necessary to minimize worker exposure to contamination during installation activities and to ensure containment of all contaminants. It's also important to know what the depth of the geologic media is along the expected path of the plume, how the media can best be reached for mitigation (e.g., excavation, or drilling), and whether the terrain and surface conditions are conducive to construction activities over time.

Level 3: Select and Evaluate Performance of Feasible Strategies

Provided adequate time and sites are available for implementation of feasible mitigation measures, the next level in the selection process is to iteratively develop conceptual designs for promising mitigation schemes and evaluate mitigation design. This is achieved by exploiting the simulation capability of the numerical flow and transport model, as was demonstrated in Section 7.5, to exhaustively investigate a large number of designs and combinations of designs. The simulated performance of individual schemes are compared against the pre-mitigated results and against one another.

Sensitivity studies of key design parameters are conducted for the more effective schemes until the interactions between the site hydrogeologic characteristics (e.g., the spatial distribution of hydraulic conductivities), specific design parameters, and mitigation performance are thoroughly understood. From this analysis a ranking of mitigation designs, purely on the basis of simulated performance, can be developed.

Level 4: Rank Feasible Alternatives

The final step in the selection and preliminary design stage is to integrate mitigation performance with other important factors to determine the most appropriate mitigation scheme(s) to achieve the desired objectives. At this level all designs considered meet the minimum performance criteria. In addition, the final selection considers duration of performance, reliability and cost. Reliability is assessed in part on the basis of the need for quality control during construction but also the sensitivity of performance to hydrogeologic characteristics such as hydraulic conductivity, dispersivity, retardation, etc. Inherent in the analysis of cost is the need to consider worker safety, special considerations due to site conditions, installation costs, and operation, maintenance and replacement.

7.8 CONCLUSIONS

The South Texas Plant Case Study No. 2, using the conceptual and numerical models developed in Case Study No. 1, presents a detailed, though not exhaustive review of mitigation design alternatives. The purpose was to gain an increased understanding of how mitigation performance is related to design parameters (e.g., size, shape, permeability, location) and hydrogeologic characteristics. The numerical model proved to be extremely useful in performing the necessary flow and transport computations and facilitated evaluation of numerous alternatives within the confines of limited time and cost constraints. The model also was quite flexible in representing a range of mitigation types, sizes, and shapes (28 different designs were evaluated). General conclusions developed in the process of conducting the case study are listed below. These are followed by conclusions specific to the performance of mitigative alternatives at the STP.

1. Selection of appropriate mitigation techniques is highly site specific and requires thorough evaluation of the nature and extent of the contaminant release, site characteristics and feasible alternatives.

2. Barrier performance (cutoffs or slurry walls) is closely tied to the hydraulic characteristics of the aquifer in question. Thus, a very important aspect of mitigation design is accurate, detailed characterization of aquifer properties. Barriers improperly placed may in fact modify local ground-water velocities such that contaminant migration is increased.

3. An important consideration in mitigation design is to exploit the occurrence of natural decay as an in situ treatment process by containing contaminant releases close to the plant.

4. Downgradient designs decrease hydraulic gradients, reduce flow velocities and increase the contaminant path length. Upgradient designs serve to just reduce the gradient and velocity.

5. In general, downgradient designs produce greater lateral spreading than do upgradient designs.

6. Cutoffs constructed in low hydraulic conductivity areas create greater backwater effects than cutoffs constructed in areas having relatively higher conductivity.

7. Cutoff effectiveness decreases with increasing distance from the contaminant source.

8. Barriers which obstruct flow in both the x- and y-directions (L- and U-shaped) appear to significantly out perform linear barriers.

9. Within the normal range of achievable permeability reduction (i.e., 0.001 to 0.1 gpd/sq ft) performance does not vary significantly. Absolute barrier permeability is not as important as contrast with the natural system.

10. Understanding the sensitivity of a given system to the assumed retardation coefficient is very important because of the impact it has on transport results. Thus, one should do as much as possible to either reduce the uncertainty associated with this parameter or be careful to properly bond it.

11. Incorporation of costs into the selection of appropriate mitigation measures must be based on a site-specific, detailed investigation of ground-water flow and contaminant transport in conjunction with an accurate assessment of the surface and subsurface contamination at the time of construction.

12. Pumping may be more flexible and less costly than construction of an engineered barrier, but will require considerable more upkeep and maintenance.

Conclusions specific to the design and performance of mitigation at the STP include the following:

1. Based on the pre-mitigation transport results, approximately 200 years will be available to implement mitigation at a distance of 500 ft or greater downgradient from the reactor.

2. Results indicate downgradient cutoffs, if constructed outside the cooling reservoir, provide no benefit and actually increase transport

of radionuclides from the STP site. If downgradient cutoffs are to be constructed in the downgradient direction it will be necessary to locate them within the reservoir having a more centered orientation relative to the reactor site.

3. Barriers placed in the low hydraulic conductivity area in the eastern portion of the study area create greater backwater effects; however, they also induce greater east to west lateral velocities which transports contaminant around the western end of the barrier.

4. The "best" performing alternative evaluated for the STP is a L-shaped design which has the leg in the y-direction placed on the western end.

5. Downgradient injection schemes proved to be effective in creating hydraulic barriers to contaminant migration for the STP site.

5. Given the spatial distribution of hydraulic conductivity at the STP, in general downgradient barriers were more effective than upgradient barriers of the same length.

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8.0 MARBLE HILL, INDIANA NUCLEAR GENERATING STATION CASE STUDY NUMBER THREE

8.1 INTRODUCTION TO MARBLE HILL CASE STUDY

8.1.1 Objectives

The third case study considers the problem of contaminant mitigation in a consolidated carbonate hydrologic unit. This case study is designed to complement the South Texas Plant case study, which examines an unconsolidated silicic geologic formation. The major objectives of this case study are as follows:

- determine the appropriate mitigative techniques and special considerations of a site characterized by consolidated and fractured hydrologic units,
- examine the influence of surface structures (buildings, utilities, etc.) and other practical aspects of construction of a mitigative technique, and
- examine the consequences of a severe accident in a carbonate medium.

8.1.2 Site Selection

The Marble Hill Indiana site was chosen for case study analysis by concurrence of NRC and PNL. The selection process was based on four conditions, that the site must have:

1. an average sized reactor,
2. a generic classification of fractured bedrock.
3. the reactor basemat near the saturated ground water zone and,
4. an adequate data base available in the Marble Hill Final Safety Analysis Report (FSAR 1982) or other published sources.

Marble Hill Generating Station near Madison, Indiana, was selected because it met the above conditions and one additional reason. PNL had previously considered the consequences of a severe accident at the site in conjunction with a NRC Hydrologic Engineering Case Review. The hydrogeologic data base for Marble Hill was known to the staff, and a previous inspection of the site in July 1983 had been conducted to examine the hydrogeologic units, drill cores, and discharge locations involved with a postulated core melt accident. These factors gave PNL valuable insight into the "real world" site conditions and important considerations that would influence the characterization of a site following an actual severe accident.

The data for this study were compiled from various sections of the Marble Hill Generating Station Final Safety Analysis Report; the Indiana District

office of the U.S. Geological Survey, Water Resources Division; and the Indiana Department of Natural Resources. No additional field tests were conducted.

8.1.3 Geographic Location

The Marble Hill Generating Station is located in southern Jefferson County, Indiana. The site is 10 miles south of the city of Madison, Indiana and 30 miles north-west of Louisville, Kentucky. Figure 8.1.3-1 illustrates the location of the power plant and its position adjacent to the Ohio River. The climate at the site is humid with hot summers and freezing temperatures in the winter. Average annual precipitation ranges from 40 to 50 in./yr.

8.1.4 Approach to Site Characterization

The analysis of the site was conducted such that all available geologic and hydrologic information were use. to the maximum extent possible. This approach provided the most precise definition of pathways for contaminant migration at the site without collecting additional field data. This approach also provided the level of site characterization and accident simulation that would be conducted in a first round of comprehensive hydrologic modeling following a severe accident. The simulation of contaminant migration is designed to be as realistic as possible and not as a worst case or bounding conservative analysis. However, lack of data necessitated that some estimates be made (i.e., Leach Rate). In those instances conservative yet realistic values that were used in the characterization.

The Marble Hill site exhibits a greater degree of hydrologic diversity than the South Texas Plant. The ground-water flow system is anisotropic and heterogeneous with highly variable hydraulic properties and multiple hydrologic units. The ground-water flow direction, geologic units through which the contaminants are transported and predominant discharge location are not apparent from an inspection of the data. Several conceptualizations of contaminant release and flow at the site were formed under these circumstances and were examined by mathematical models. Multiple conceptualizations might occur in any first round of characterization, especially at a site where existing hydrogeologic information is sparse or the hydrogeology is complex. The rationale for selection and characterization of accident scenarios is supported by a rather extensive technical description of the site hydrology and conceptual model developement.

8.1.5 Relation to Generic Analysis

The Marble Hill Generating Station is generically classified as a consolidated, fractured carbonate. This type of site has the hydrologic and rock chemistry characteristics which combine to produce the most severe environmental consequences. Many sites in this classification have relatively high values of hydraulic conductivity and low values of effective porosity. These factors tend to produce rapid transport rates resulting in short transport times and large radiological discharge fluxes to the surface environment. Section 5.3 contains the generic analysis of this type of site.

FIGURE 8.1.3-1. Location of Marble Hill Nuclear Generating Station (Source: Marble Hill FSAR 1982)

Mitigative techniques for a generic fractured carbonate unit are listed in Table 8.1.5-1. The techniques are listed in the order of preference based on the generic analysis.

Other mitigative methods such as ground-water freezing and air injection were not considered for this site because of the existence of fractures and some 12-ft-diameter voids (FSAR 1982). Slurry walls were rejected as a mitigative technique because the hydrologic units are consolidated and blasting would be required to trench into the contaminated layers 20 to 50 ft below land surface.

TABLE 8.1.5-1. Feasible Mitigative Techniques for a Fractured Carbonate Hydrologic Unit.

Mitigative Technique	Comments
Hydraulic Removal of Contaminant Plume	Monitoring of waste stream allows close examination of effectiveness; site problem solved in shortest period of time.
Hydraulic Stabilization of Contaminant Plume	Downgradient monitoring of plume necessary to judge effectiveness; mitigative controls may require long term maintenance; and are energy intensive.
Grout Stabilization of Contaminant Plume	Grout may not seal all primary fractures and solution channels; pressure grouting may induce fracturing; mitigative controls may require long term maintenance; long term energy requirements are low.

8.1.6 Parameter Units

English units of measure were used to describe the Marble Hill site. A non-metric format was selected so that this section of the report is compatible with the Marble Hill Nuclear Generating Station, Final Safety Analysis Report (FSAR 1982). The FSAR represents the most comprehensive document for the site and is the prime reference source for this study.

8.2 PLANT DESCRIPTION

8.2.1 Reactor Type

The Marble Hill Generating Station contains two pressurized water reactors (PWRs), each designed to produce 3411 megawatts thermal and 1130 megawatts electrical power. A core melt accident would be initiated by the failure to remove sufficient heat from the reactor vessel. The core materials could increase in temperature to the point of liquification and melt a path into the basemat of the containment structure.

The basemat of the Marble Hill plant consists of reinforced concrete 14.6 ft thick. Decay heat of the core debris would decompose and liquify the concrete and allow penetration of the containment structure. This could occur by fracturing of the basemat or by the core debris melting through the basemat and continuing down into the underlying stratum. A complete melt through of

the containment structure is not an automatic consequence of a core melt accident. The accident sequence, post-accident containment cooling procedures and thickness of the basemat would determime if a melt-through of the containment structure would occur. For the purposes of this study a complete melt penetration of the basemat is assumed.

8.2.2 Radionuclide Release

8.2.2.1 Quantity and Type of Radionuclides

A suite of radionuclides would be released in a severe accident at a nuclear power plant. The environmental consequences resulting from the release of each nuclide is dependent on the initial quantity released, half life, toxicity, and bioaccumulation factors. The relative consequences of a core melt accident is analized by determining the discharge quantity of the more hazardous radionuclides. The generic analysis as detailed in Section 2.2 considered the radionuclides strontium-90 and cesium-137 as indicators of relative contamination.

In carbonate geologic media, cesium-137 is more strongly sorbed and has a retarded velocity about 10 times slower than strontium-90. The slower velocity of cesium with respect to strontium provides additional time for radioactive decay of cesium-137 along the flow path. Although cesium-137 has an initial inventory of activity greater than strontium-90, and a similar half life, when the radionuclide travel times are greater than 90 days strontium-90 activity will be higher than cesium-137 at the discharge point. Scoping calculations based on hydrologic characteristics described in Section 8.3 indicate that the unretarded ground-water travel time to the environment is greater than 90 days. Therefore, strontium-90 is used to determine the relative environmental severity of an accident at this site.

Radioactive contaminants could enter the ground-water flow system through two major mechanisms: a liquid release of contaminated sump water and/or a leach relaese of radionuclides from the solidified core debris. Neither of these release mechanisms would be instantaneous. Liquid sump water releases would be limited by hydraulic flow restrictions and availability of sump water. Leach releases would be limited by the rate of radionuclide transfer from the solid to aqueous phase and are much slower and of a longer duration than sump water releases. The maximum quantity of strontium-90 that could be released in a severe accident is scaled to the Marble Hill plant in Table 8.2.2-1 (Niemczyk et al. 1981).

TABLE 8.2.2-1. Initial Amount of Indicator Radionuclide

Radionuclide	Half-Life (days)	Reference Reactor pCi (NRC 1975)	Marble Hill pCi (Single Unit)
Strontium-90	10519	3.71×10^{18}	4.19×10^{18}

The inventory of radionuclides would be partitioned between the core melt debris and the sump water. The fraction of the inventory that would be released in each category is presented in Table 8.2.2-2. A more detailed description of these processes is given in Section 2.0.

TABLE 8.2.2-2. Release Fractions for the Indicator Radionuclide (Source: Niemczyk et al. 1981)

Radionuclide	Sump Water Release	Core Melt Debris Leach Release
Strontium-90	0.11	0.89

8.2.2.2 Sump Water Releases

The release rate of contaminated sump water is governed by specific conditions associated with the accident and the hydrogeology beneath the reactor basemat. The sump water would enter the geologic formations below the plant at a rate determined by the size of the melt opening, hydraulic conductivity of the zone adjacent to the melt, and hydraulic driving force. The driving force would be a function of the height of the standing water in the reactor sump and the amount of pressurization in the containment building. The geology of the Marble Hill site allows two feasible contaminant pathways which depends on the hydraulic head in the containment structure. The site is therefore evaluated under two sump water assumptions: 1) a low hydraulic head release that allows contaminants to seep into the hydrologic units adjacent the core melt debris and 2) a high hydraulic head release that forces contaminants upward into a more permeable zone near land surface. These two release scenarios are illustrated in Figure 8.2.2-1.

In a low head release, the contaminant would be transported through the geologic strata that received the core debris. The radionuclides would migrate downward through the debris driven by overlying sump water and an existing downward ground-water gradient into the Soluda Formation. Continued downward perculation below the Saluda Formation is effectively limited by an underlying shale unit.

The assumption of a high hydraulic head release of sump water implies that there is either a column of standing water over the core debris or that the containment structure is pressurized at the time of melt penetration. This condition is capable of forcing water into strata above the core debris. Water inside the containment structure would need to be at an elevation of over 765 ft to force a pathway to the geologic unit of greatest permeability at the site. Under this assumption the Laural Member of the Salamonie Dolomite would receive the sump water.

Two conditions make a high head flow event less likely than a low head release. First, the sump water would have to travel upward along a path not created by the core melt mass. That is, sump water would have to be forced upward through undisturbed rock and backfill along the containment structure to

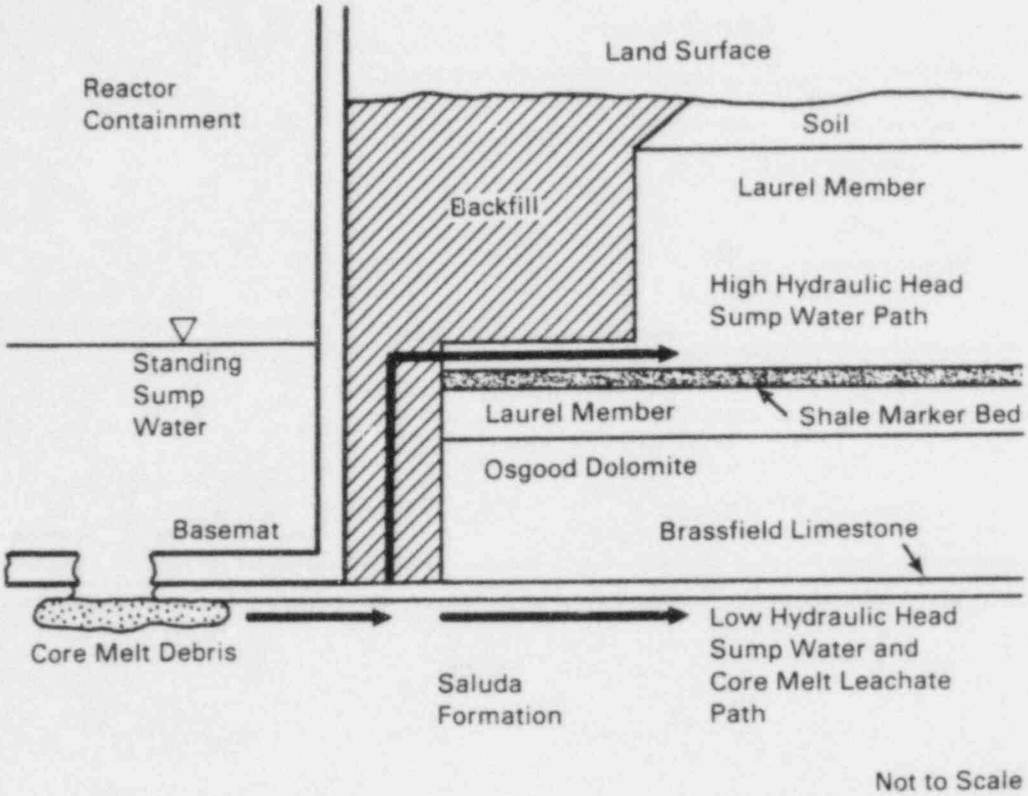

FIGURE 8.2.2-1. Hydrologic Units for High and Low Hydraulic Head Release of Sump Water

reach the Laural Member. Second, waterproofing applied to rock surfaces adjacent to the the containment structure has formed a barrier to water movement into the upper strata. A violent vaporization of ground water under the plant would be required to fracture a pathway up into the overlying unit. A high hydraulic head release is considered because it provides a pathway to the most permeable hydrologic unit and therefore has the potential to create the most severe or "worst case" environmental consequences.

The estimated release rates of radionuclides under low and high head conditions are determined by assuming that sump water is standing in the containment structure at two elevations and allowing it to drain into the geologic formation. The hydraulic properties of the geologic unit adjacent to the core debris are conservatively assumed to be unaltered by the accident and the flow rate is determined by the Theis equation. Hydraulic characteristics used in this analysis are detailed in Section 8.3. The transmissivity of the hydrologic units is taken as the median point of the cummulative probability distributions presented in Section 8.3.3. Figures 8.2.2-2 and 8.2.2-3 illustrate the release of strontium-90 in a low and high hydraulic head release respectively. The high head release would drain a nominal volume of water of 40,000 cu ft in about 5.5 years. This amount of liquid release assumes that the remaining sump water would not be pumped out of the reactor as an initial

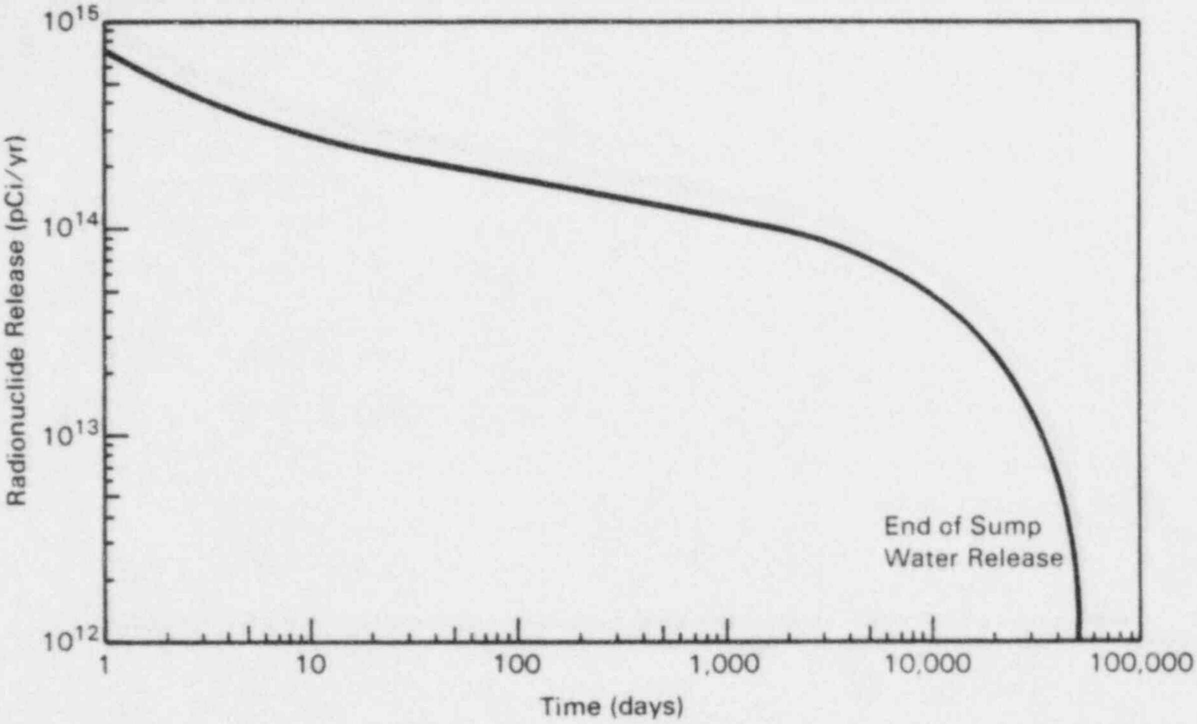

FIGURE 8.2.2-2. Strontium-90 Release Rate For A Low Hydraulic Head Sump Water Release

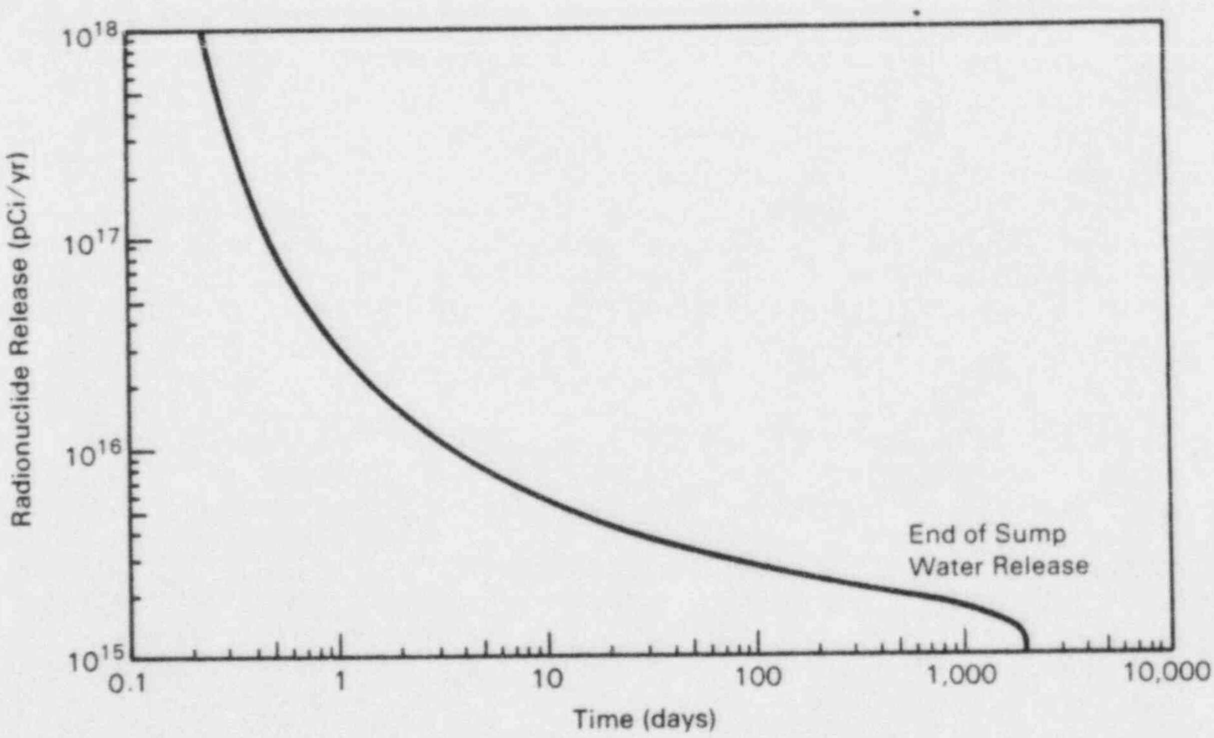

FIGURE 8.2.2-3. Strontium-90 Release Rate For A High Hydraulic Head Sump Water Release

step to contaminant interdiction. A low head release to the lower unit would require over 100 years to fully drain all of the sump water. The less permeable lower unit would release radionuclides 40 times slower the first day of release and 17 times slower at 1000 days than the upper unit.

8.2.2.3 Core Melt Release

The liquified core materials would melt a pathway about 10 ft below the basemat and solidify in the Saluda Formation. The contaminant leached from the core debris would enter the ground-water flow system over a period of time extending into decades. The basic process for release of radionuclides from the calcine melt debris would be diffusion through the solid material matrix and into the ground water contacting the melt materials. The leach rate depends on several phenomonological factors (e.g., particle size and diffusion coefficient) detailed in Section 2.4.3. The leach rate of the core debris is scaled to the size of a single reactor at Marble Hill in Figure 8.2.2-4. The contaminant would enter the Saluda Formation which is the lower hydrologic unit for this study.

8.2.3 Plant Configuration

The physical layout of the power plant has an influence of the mitigative scheme. Ideal mitigation designs may not be feasible due either to restrictions in the placement of control structures or locations of the stie itself. For example, an arc of injection wells may need to curve around a building or the optimal spacing of wells may need to be altered to accommodate vital roads, underground utilities, or operational overhead power lines. Construction may also be limited by natural site features such as surface water bodies and hill slopes. This section of the case study examines the restrictions that the plant configuration at Marble Hill would have on implementing the feasible mitigative techniques. Emphasis is placed on the practical considerations of a mitigative scheme.

8.2.3.1 Topography

The topography of the Marble Hill site consists of a broad and gently sloping upland dissected by steep stream and river valleys. The site is situated on a upland peninsula about 300 ft above the Ohio River. The normal pool elevation of the Ohio River is 340 ft above mean sea level (MSL) and is regulated by dams upstream and downstream of the plant. The land surface elevation in undisturbed areas of the plant range between 750 and 800 ft above MSL. The natural slope of the upland at the site is about 40 ft/mi northward. Site drainage of surface water is to the west and north into Little Saluda Creek and easterly into the Ohio River.

The topography of the site is illustrated in Figure 8.2.3-1. The area adjacent to the plant is relatively flat and poses no special problem to the feasibility of constructing an engineered barrier to contaminent movement. The area around the plant is generally available for construction and transportation of materials. A natural obstacle to access are the steep slopes and cliffs found along surface water drainages. The steeply sloped areas near the

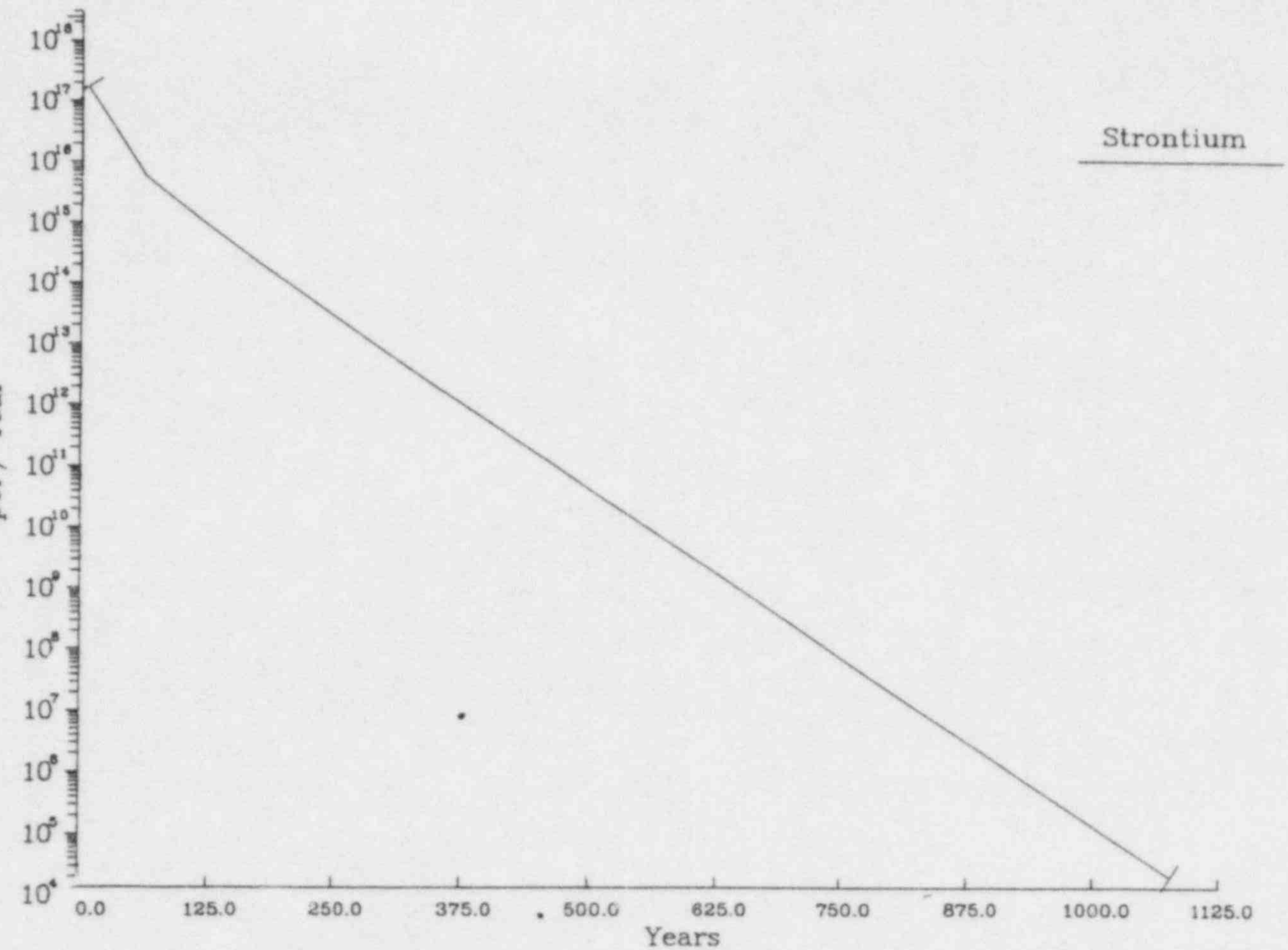

FIGURE 8.2.2-4. Leach Release Rate For Core Melt Debris

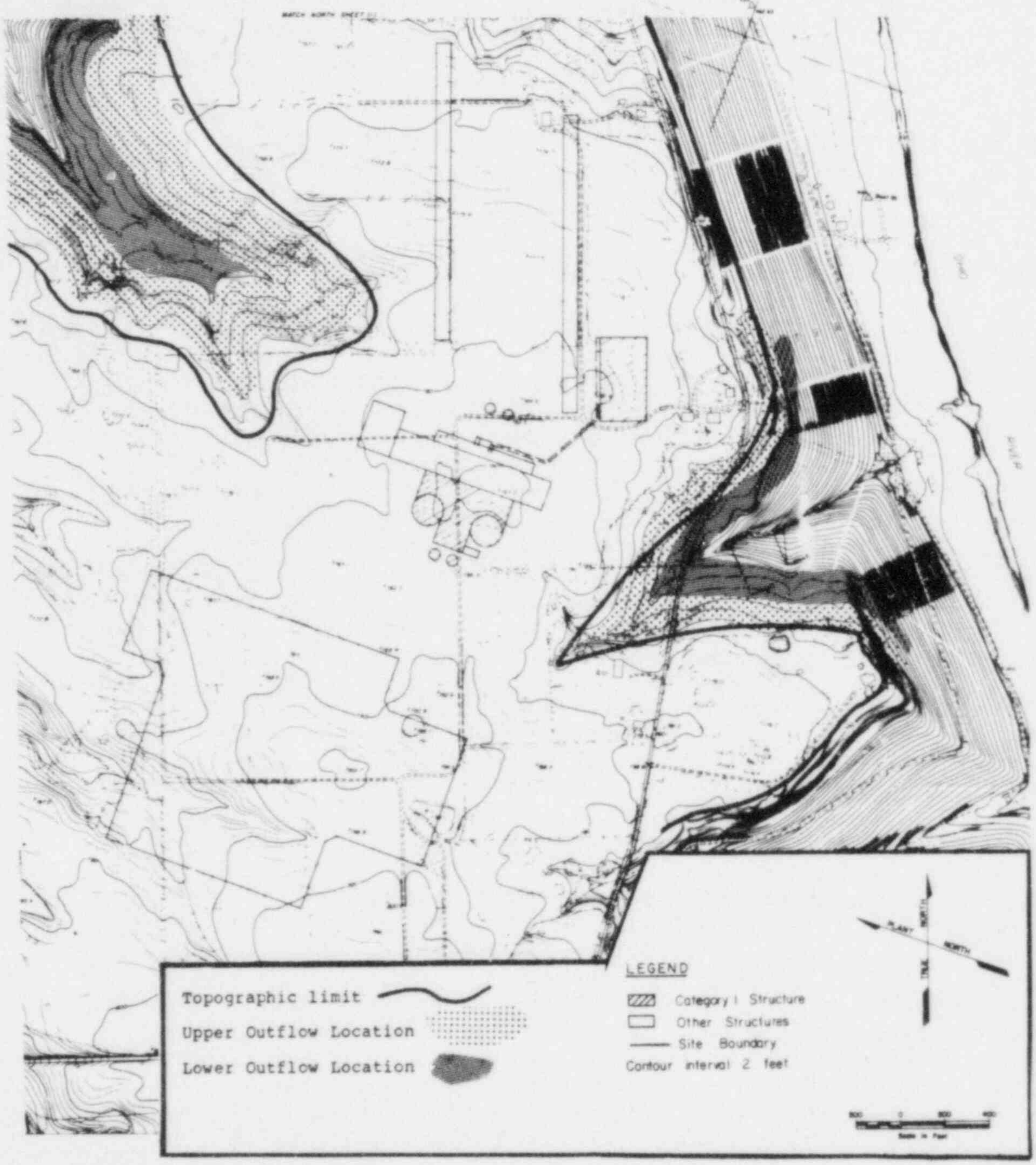

FIGURE 8.2.3-1. Topography of Marble Hill Site (After Source: Marble Hill FSAR 1982)

plant are coincident with the forested areas commonly found below the 750 ft elevation contour. The heavily forested areas begin along the river at bluffs and continue to the bottom of the stream valleys. Although the trees could be removed for construction activities, the steep slopes and cliffs in these areas would restrict heavy equipment access. Under special circumstances topographic restrictions to mitigation could be eliminated through extensive site preparation. An example of this type of construction already existing at the site is the makeup water pipeline from the plant to the Ohio River. Following a severe accident, the time delays and added costs of such a major undertaking would have to be: 1) justified based on any special advantages of that location or 2) necessited by the proximity of contaminant to the accessible environment.

The topographic limit to construction at the Marble Hill site is defined generally by the 750- to 760-ft elevation contour and is highlighted in Figure 8.2.3-1. The largest areas available for construction are found to the north and south of the plant. At these locations the peninsular upland extends along a gentle northward slope and there are no major topographic restrictions. The eastern topographic restriction to construction is the Ohio River bluff which approaches within 400 ft of the plant. To the northwest and southeast of the plant, stream valleys deeply incised into the bedrock limit the access for construction of a mitigative barrier. The width of the construction zone is 400 ft to the southeast and 800 ft to the northwest. This restriction is expressed nearest the plant in an unnamed drainage basin to the southeast of the reactors. The construction limitation to the northwest borders a small drainage basin that forms a tributary to Little Saluda Creek. These two locations are noteworthy because the probable contaminant pathway, as defined by the local fracture pattern and hydraulic gradients described in Section 8.3, would reach land surface in these areas. The probable discharge or outflow locations for an upper and lower hydrologic unit are indicated on Figure 8.2.3-1 as stippled areas.

8.2.3.2 Plant Structures

The locations of all major buildings and structures at the site were taken from the FSAR (1982). Detailed plans of the site are presented in Figures 8.2.3-2 to 8.2.3-5. Of primary importance are the reactor containment domes, ultimate heat sink, and the fuel handling building. These are classified by the NRC as Category I structures and are constructed of heavily reinforced steel and concrete. All mitigative schemes must accomplish contaminant interdiction outside of these structures. Other large and massive structures such as the cooling towers and primary water storage tanks would preclude construction at those locations. Noncritical structures may be removed or modified to accommodate mitigative construction on a case-by-case basis. Surface buildings are discussed in relation to the primary pathways of contaminant travel as detailed in Section 8.3.

The major plant structures are highlighted in Figure 8.2.3-2. The buildings adjacent to the reactors would prevent construction of a barrier northward for a distance of 300 to 400 ft. On the south, east and west sides of the reactor buildings, a barrier could be constructed close to the contaminant source. Contaminant migrating to the northwest would pass under the turbine

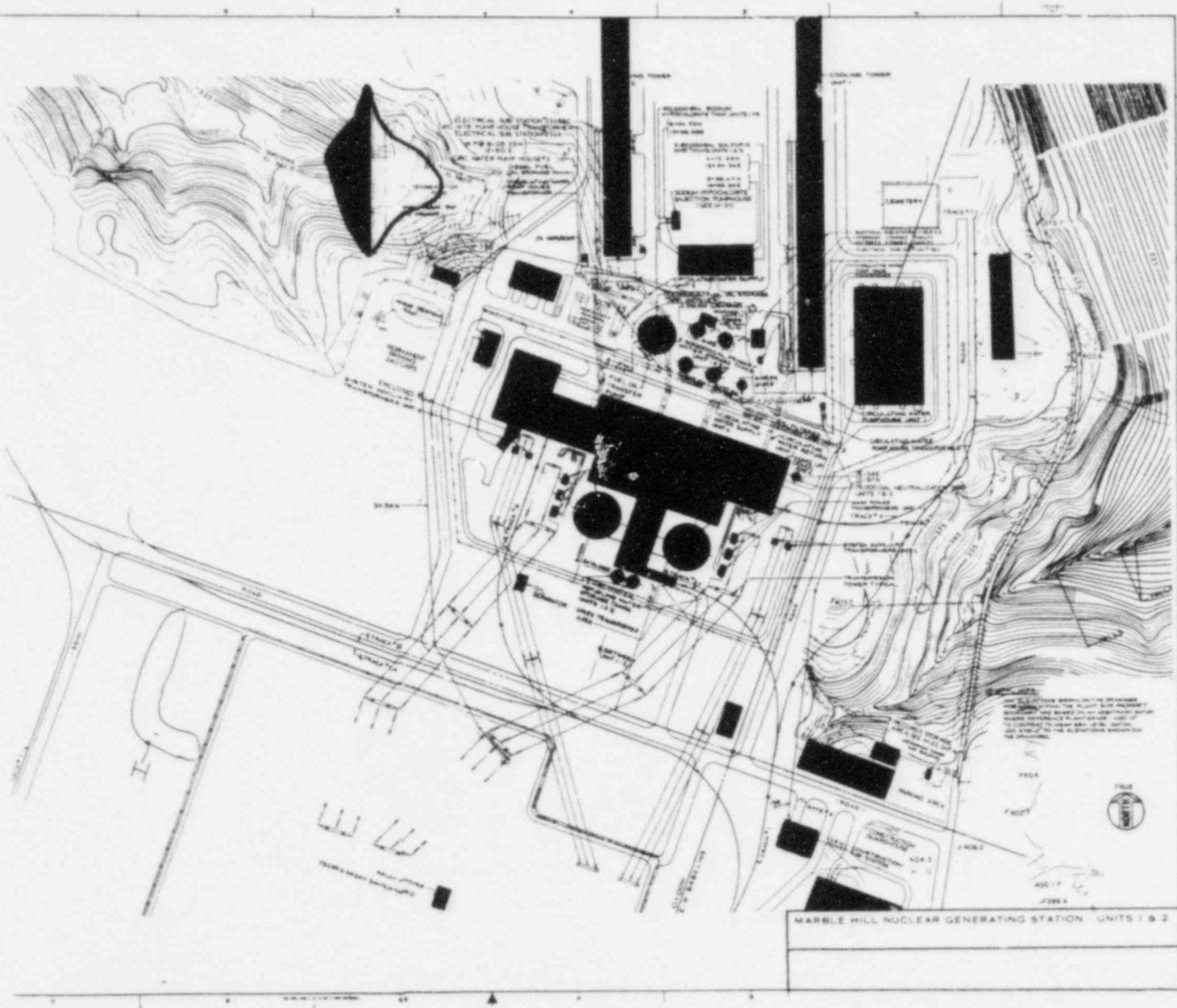

FIGURE 8.2.3-2. Plot Plan of Major Plant Structures (Source: Marble Hill FSAR 1982).

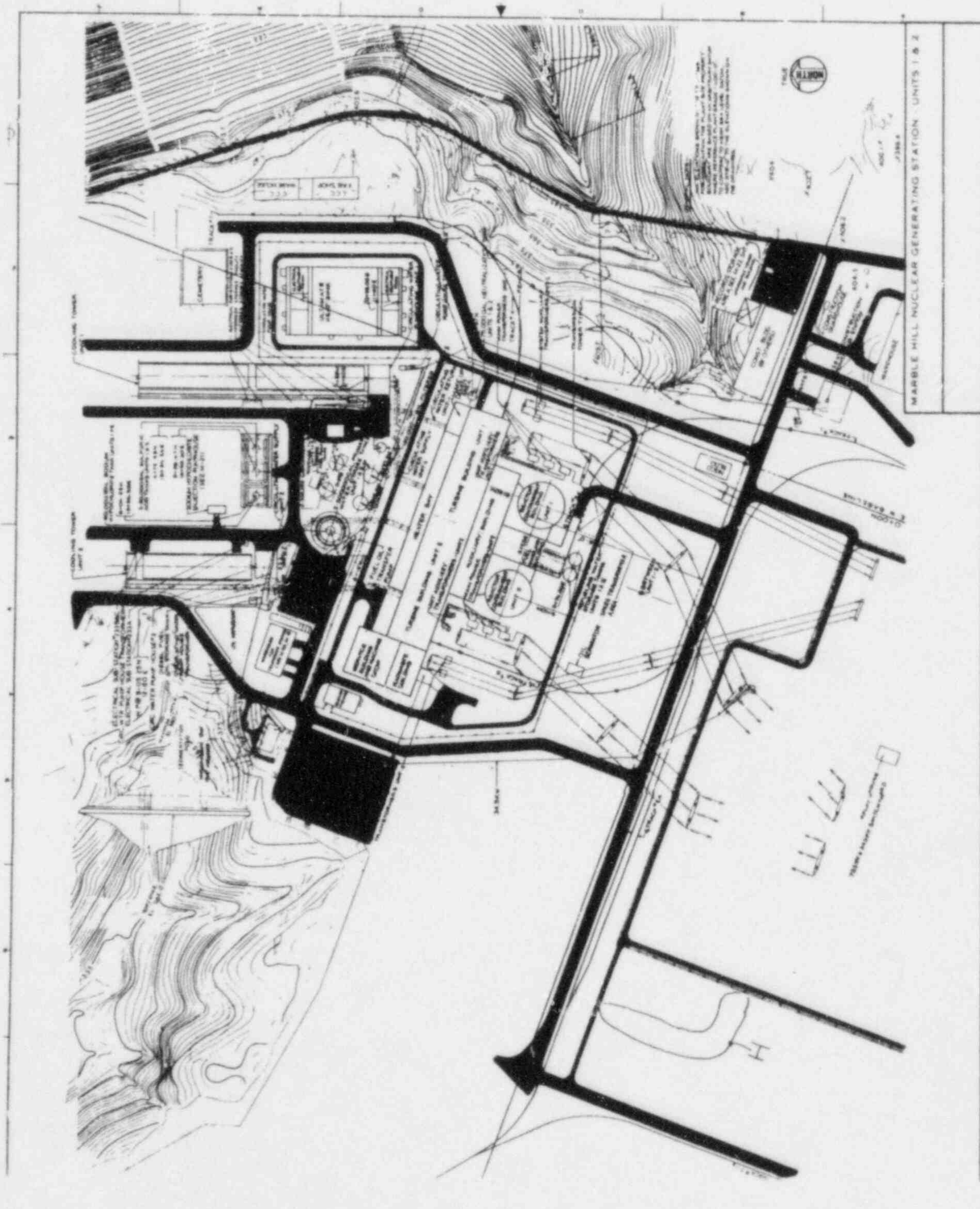

FIGURE 8.2.3-3. Plant Road System (After: Marble Hill FSAR 1982)

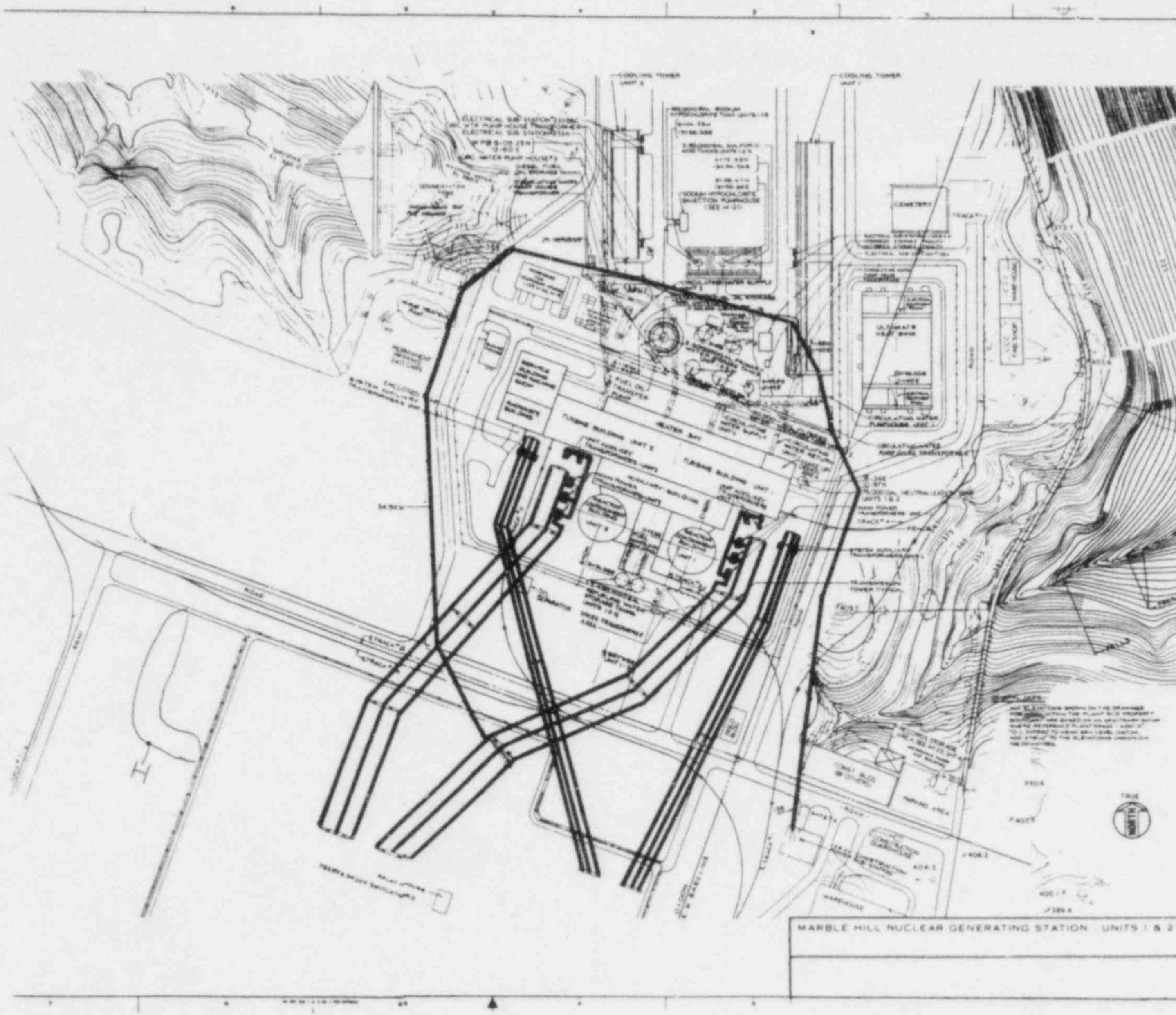

FIGURE 8.2.3-4. Plant Electrical Utilities (After: Marble Hill FSAR 1982)

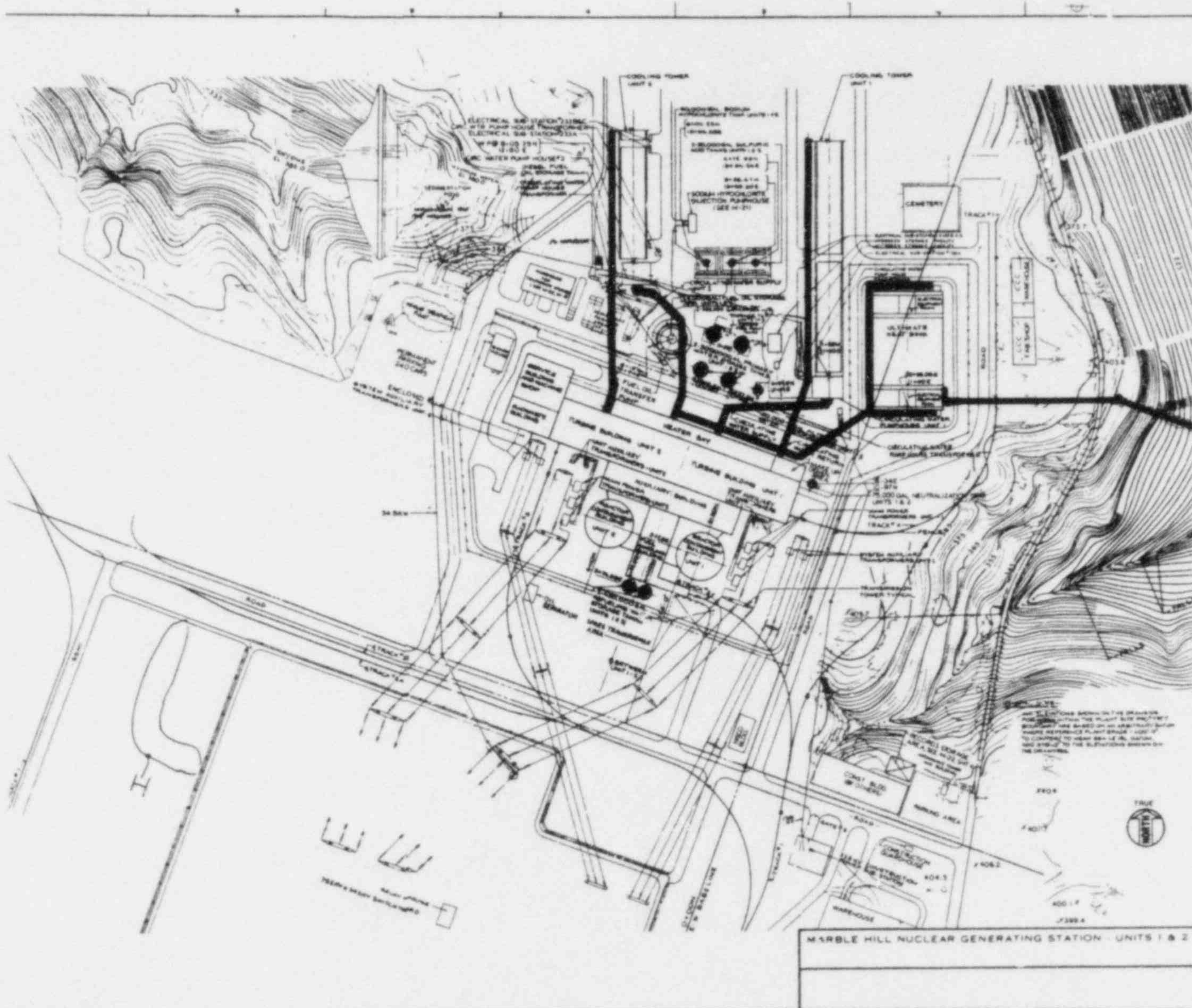

FIGURE 8.2.3-5. Plant Cooling Water Pipelines and Storage Locations (After: Marble Hill FSAR 1982)

building, radwaste building, service building, guard house, and sanitary waste water lagoon. Interdictive techniques implemented down the hydraulic gradient, which is to the northwest, could be constructed between the guard house and the radwaste building. Continuing the barrier to the east would require that it follow the security fence on the north side of the Category I structures. Contaminant migrating to the southeast would not pass under buildings. An exception to this situation would be contaminant from reactor unit No. 2 migrating southeastward and passing under the fuel handling building and reactor unit No. 1. In general, interdictive techniques constructed on the eastern side of the plant would not have to be designed around builings.

8.2.3.3 Utilities

Primary roads, power lines, electrical junction yards, and cooling water pipelines at the plant would have to remain in service following a severe accident to service the other reactor unit and the contaminant control systems in operation at the accident site. Construction activities must be designed to accommodate reactor systems operating to prevent further radionuclide releases. Restoration of essential services caused by the accident may be the first step to accident mitigation.

The road system would be especially important for the transportation of materials and as a readily available location for construction of mitigative barriers. Roads can remain in service at most locations when the areas adjacent to the road are used for the actual construction of mitigative barriers. Larger roads and parking areas are especially well suited for construction (i.e., well drilling). The road system at Marble Hill is presented in Figure 8.2.3-3. On the east side of the plant area there are two roads along the river bluff. The larger road adjacent of the plant and the area eastward to the topographical limit would be a suitable location for a line of injection or withdrawal wells.

The western side of the plant contains two parking lots and a major north-south road providing access along Little Saluda Creek, a probable discharge location. A small road inside the southern perimeter of the security fencing around the containment domes and the main entrance road provide access to a large area south of the plant. In general, existing roads provide adequate site access and feasible construction areas between the plant and the probable discharge locations. The areas north of the plant are much less accessible for construction. The cooling towers extending 1300 ft northward from the plant effectively block construction of a continuous barrier.

Railroads can be disrupted for construction; however, if the mitigative effort requires large-scale transportation of machinery or materials (i.e., massive ion exchange colunms or large quantities of cement), rail service should remain open through the materials aquisition stage. Rail service would be interrupted by any large mitigative construction project on the eastern and southern sides of the plant.

Electrical power must be maintained throughout the accident and into the post-accident period to fulfill the basic requirements of the site. The source of this power could be the contingency diesel generators or an offsite supplier. At the Marble Hill site, primary and secondary power lines and transformers flank the reactor units on both the east and west sides on the plant as seen in Figure 8.2.3-4. A much smaller power line encircles the turbine building and provides power to plant facilities. Drilling machine masts must maintain a distance of 5 to 10 ft from live electrical lines. In some instances this requirement may create gaps in grout barriers, necessitate angle drilling, or require temporary switching to an alternate power source.

The power line for site facilities would be a consideration in any construction at this site. The facility power line would interfere with drilling activities on the east and west sides of the plant. This line could be periodically interrupted or relocated to speed construction. The main power lines exiting the turbine building could not as easily be interrupted or relocated. Construction along the south side of the plant would have to accommodate the large electric towers and any live overhead lines.

Water systems used to cool either the damaged or undamaged reactor must not be disturbed by any facet of the mitigative scheme. This would preclude trenching, exavating, drilling, and pressure grouting adjacent to cooling water pipes. Under normal operation, cooling water for the reactors is provided by the cooling towers in the northern portion of the site. A secondary method of reactor cooling is the ultimate heat sink located on the eastern edge of the site. Make up water for plant operation is pumped from the Ohio River into an aboveground pipeline and into the ultimate heat sink. These features of the cooling system are noted on Figure 8.2.3-5. North of the turbine building is a tank farm containing:

- two 500,000 gallon primary water storage tanks,
- two 500,000 gallon condensate storage tanks,
- one 125,000 gallon fuel oil tank,
- one 150,000 gallon filtered water tank,
- one 75,000 gallon neutralization tank,
- two 50,000 gallon sulfuric acid tanks, and
- one sodium hypochlorate tank.

The tank farm area contains an extensive pipe network that should be avoided in construction of a mitigative barrier. Access for large equipment is limited at this location and adjacent areas are available for construction.

8.2.3.4 Feasible Locations for Mitigative Barriers

The plant site is partitioned into areas available and unavailable for construction based on the requirements of:

- access of heavy equipment,
- underground utilities, and
- maintenance of priority corridors into plant areas.

Figure 8.2.3-6 illustrates the feasible construction zones superimposed on the plant base map. These areas are generalized in that each location must undergo an onsite inspection for obstructions and a detailed records search of the "as built" diagrams of site maps before construction began. Plant structures are excluded from the construction area in the generalized figure. Major road surfaces are also excluded to provide continuous access to primary plant areas. The areas adjacent to structures and roads are included in the feasible construction area where sufficient space exists for construction. Minor roads can be expected to be blocked for indeterminant periods of time while construction is under way. These roads can also be altered or rerouted to accommodate the construction.

The road surface along the Ohio River bluff on the east side of the plant is an exception to this concept. The road at that location is ideally situated in the eastern discharge area of both the upper and lower hydrologic units. However, the steep hillsides offer poor access for equipment adjacent to the road. In this instance, the superior location of mitigative techniques such a line of injection or withdrawal wells, becomes more important than the continued access provided by the road. Wells for mitigation or monitoring could be drilled as nearly as possible to the upslope side of the road to allow construction and maintenance traffic.

The narrowest feasible construction zones shown on Figure 8.2.3-6 are where mitigation would be most advantageous, that is, along the probable contaminant pathways from the reactors. On the eastern side of the plant the situation is somewhat alleviated by the presence of the road along the river bluff. The western side of the plant is restricted by the sewage treatment plant, the associated waste water lagoon, and the steep hill sides along Little Saluda Creek. These spatial restrictions would tend to compress any mitigative scheme that relied on multible barriers (i.e., rows of injection wells or grout barriers behind withdrawal wells). The space restrictions would also require that montioring systems would be close to engineered barrier(s) and the discharge area(s) giving a short response time to a failure of the mitigative scheme.

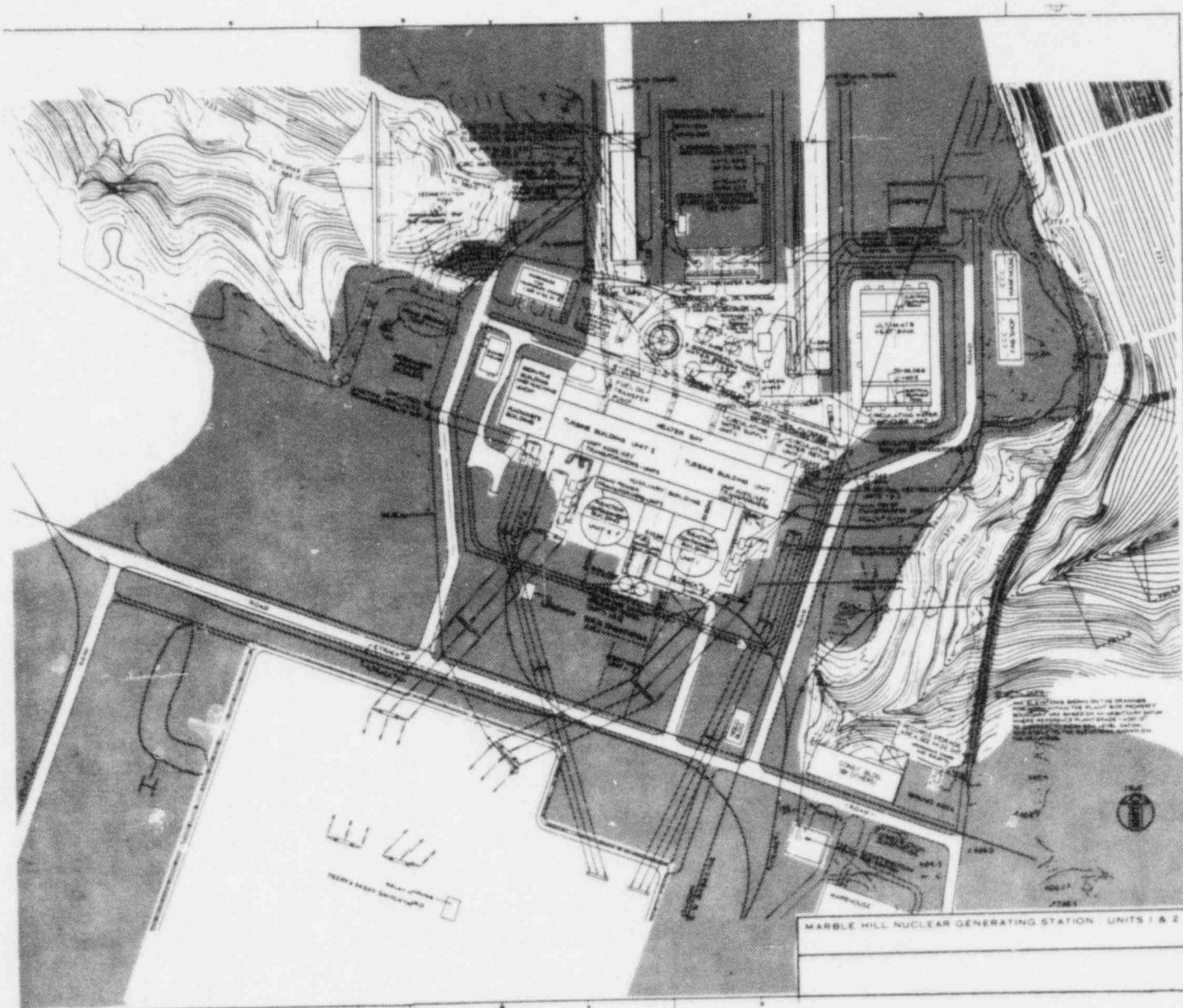

FIGURE 8.2.3-6. Feasibile Construction Areas at Site (Source: Marble Hill FSAR 1982)

8.3 HYDROGEOLOGY

8.3.1 Geology

8.3.1.1 Physiography

Marble Hill is situated on the eastern border of the Muscatuck Regional Slope of the Central Lowlands Physiographic Provience (FSAR 1982). The physiographic regions of the area are presented in Figure 8.3.1-1. The region was formed as a result of uplift to a Paleozoic depositional basin. Limestone, dolomite, siltstone, and shale comprise the major sedimentary units. Bedrock structures are gently sloping at less than 1 degree to the west and southwest. The bedrock geology is illustrated in Figure 8.3.1-2 and demonstrates the horizontal character of the geologic units. Glacial advances in the Pleistocene Epoch have smoothed the uplands and deposited a mantle of till on an older erosional surface (Gray 1982). The uplands have little topographic relief except where they are dissected by steep sided stream valleys. Stream bottoms generally have a low gradient. Outcroppings of bedrock are common along major streams and rivers. The bedrock forms resistant bluffs that can produce vertical cliffs. These features are found at the Marble Hill site and other nearby locations (Gray 1972).

8.3.1.2 Stratigraphy

The stratigraphy at the site consists of a deep Precambrian basement complex overlain by Paleozoic and recent formations. The Precambrian rocks are at depths of over 5500 ft and are not included in this study. The Paleozoic sequence of Ordovician through Silurian rocks are exposed at the surface on the Marble Hill site. For the purposes of this study only the geologic units involved in a severe accident are described in detail. Figure 8.3.1-3 gives the generalized stratigraphic column for the site. The description for each geologic unit includes a brief summary of the unit's hydraulic properties.

Paleozoic Era Bedrock Units

Ordovician Period

Dillsboro Formation

This unit is composed of shales interbedded with limestone. The elevation of the top of the formation is 645 ft and has an estimated thickness of 450 ft. The shale layers are thin to medium bedded and form a barrier to vertical ground-water flow. Ground-water production from this unit is minimal and it is classified as a aquitard. The horizontal hydraulic conductivity is estimated at 1 ft/yr. The Dillsboro is the basal unit considered in this study. The thick sequence of shale and limestone would prevent significant downward percolation of contaminants into any underlying units.

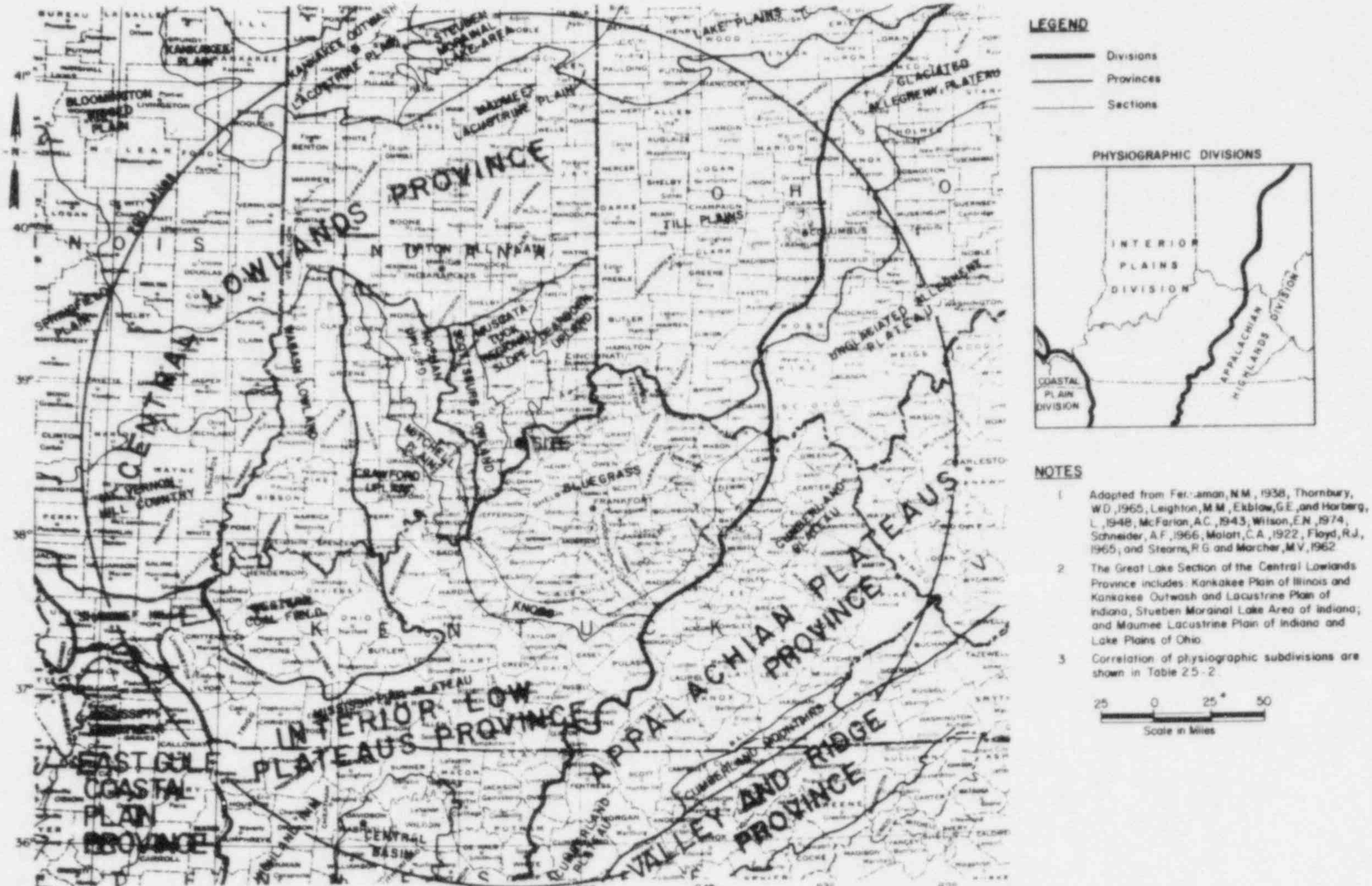

FIGURE 8.3.1-1. Physiographic Provence (Source: Marble Hill FSAR 1982)

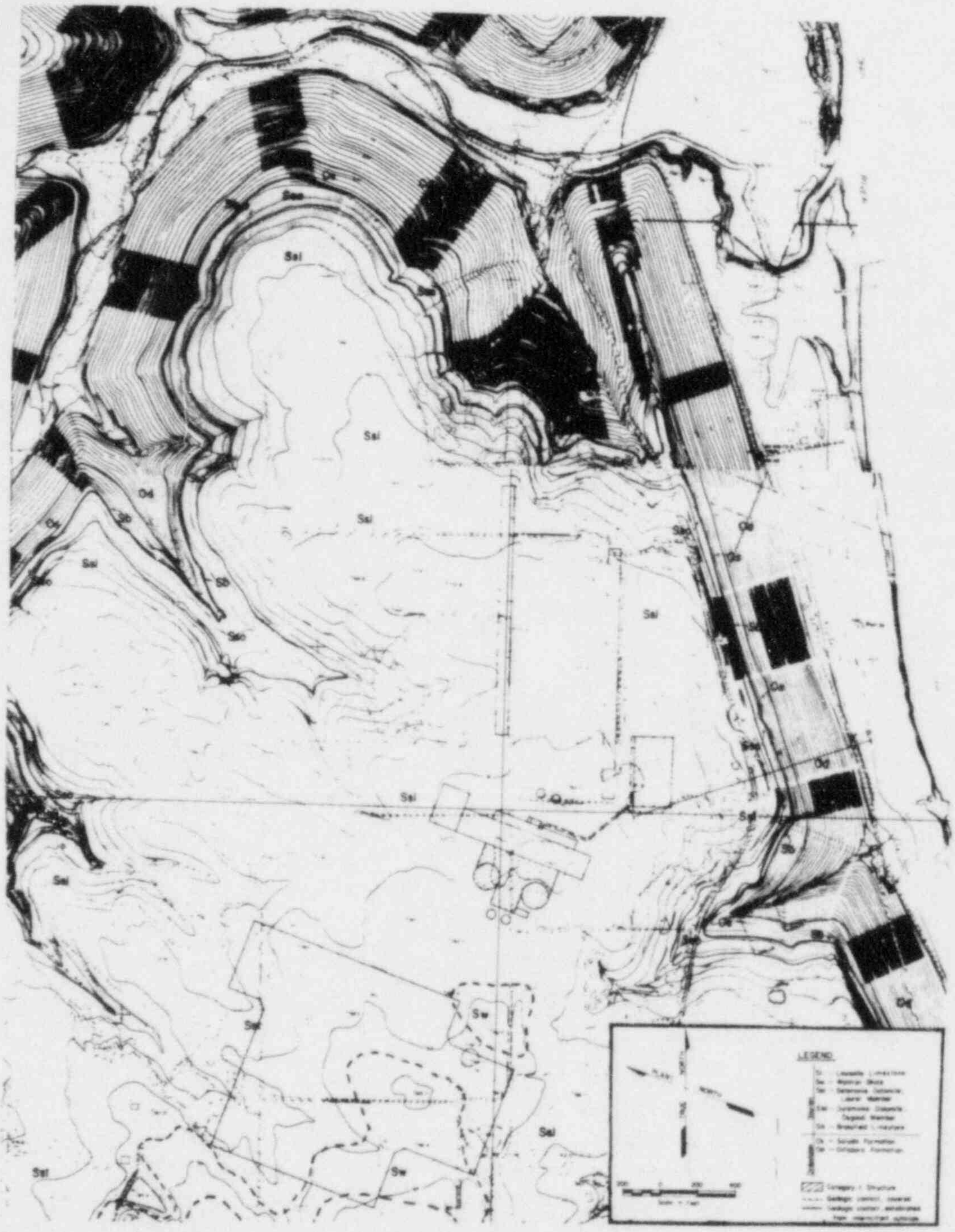

FIGURE 8.3.1-2. Bedrock Geology of Marble Hill Site (Source: Marble Hill FSAR 1982)

Era	System	Series	Stage	Formation and Member	Lithologic Symbol	Thickness (feet)[1]	Predominant Soil or Rock Type
Cenozoic	Quaternary	Pleistocene	Wisconsinan	Atherton Formation		0-3	Loess: silty clay
Cenozoic	Quaternary	Pleistocene	Illinoian to Kansan	Jessup Formation		0-20	Till: clayey silt to silty clay, with interspersed sand and gravel
Cenozoic	Early Pleistocene to Tertiary					0-32	Residuum: silty clay with chert and carbonate fragments
Paleozoic	Silurian	Niagaran		Louisville Limestone		0-18	Limestone
Paleozoic	Silurian	Niagaran		Waldron Shale		0-7	Shale
Paleozoic	Silurian	Niagaran		Salamonie Dolomite: Laurel Member		18-60	Dolomite with a shale marker bed two to three feet thick near the base
Paleozoic	Silurian	Alexandrian		Salamonie Dolomite: Osgood Member		11-18	Interbedded shale and dolomite
Paleozoic	Silurian	Alexandrian		Brassfield Limestone		1-4	Limestone
Paleozoic	Ordovician	Cincinnatian	Richmondian	Saluda Formation		61-67	Dolomite
Paleozoic	Ordovician	Cincinnatian	Richmondian ? Maysvillian	Dillsboro Formation		480	Interbedded shale and limestone

FIGURE 8.3.1-3. Stratigraphy of Bedrock Units (Source: Marble Hill FSAR 1982)

Saluda Formation

The Saluda Formation contains fine to medium crystalline dolomite with thin to medium bedding. The unit is somewhat argillaceous and contains minor shale partings. Solution features are widespread and are normally expressed as pencil-point-sized vugs. The elevation of the top of the formation at the location of the reactors is 711 ft. The thickness of the unit ranges from 61 to 67 ft. Ground water occurs in joints and is perched on the underlying Dillsboro Formation. The hydraulic conductivity is variable normally ranging between 1 and 20 ft/yr.

Sillurian Period

Brassfield Limestone

The Brassfield Limestone is a thin unit 2 ft thick beneath the site. The limestone is massively bedded and contains glouconite grains. The unit lies on an erosional surface at the contact with the Saluda Formation. Hydrologic properties of this individual unit are difficult to determine because of its limited thickness. Hydraulic testing of the brass field was conducted in composite with the Saluda Formation. The top of the Brassfield Limestone is at an elevation of 713 ft. The Saluda and Brassfield units are considered very limited in production of ground water.

Salomonie Dolomite

Osgood Member

The Osgood Member consists of interbedded shale and dolomite and has a maximum thickness of 18 ft. The shale portions are calcareous and contain minor pyrite. The dolomite is fine to medium crystalline, thinly bedded, and argillaceous. The top of the Osgood Member is at 729 ft elevation beneath the reactor area. The Osgood member has a hydraulic conductivity between 1 and 5 ft/yr and is classified as a aquitard. Insignificant amounts of water are found in fractures.

Laurel Member

The Laurel Member of the Salomonie Dolomite contains two units of dolomite and a single unit of shale. The basal unit is a 2- to 8-ft section of dolomite. The unit is fine to medium crystalline, thinly bedded, and argillaceous. Ground water in the upper dolomite is found under water table or unconfined conditions. In places, the lower dolomite is confined between the underlying and overlying shale units.

The middle unit of the Laurel Member is a 2- to 3-ft layer of shale. This unit is traceable over a wide extent and is referred to in the FSAR (1982) as the shale marker bed. The shale is calcareous, variably fissile and contains pyrite seams and nodules. Dolomite lenses up to 2 ft thick and 10 ft long were found in excavations. The shale marker bed is an aquitard that restricts downward movement of ground water. Peizometric water levels can be as much as

16 ft below the shale marker unit. This unit serves as the division between the upper and lower hydrostratigraphic units defined by this study. The elevation of the shale marker bed is at 734 ft beneath the reactor location.

The surface of the upper dolomite in the Laurel Member forms the erosional bedrock surface at the plant site. The upper dolomite has three major sections and has a maximum thickness of 60 ft. The lower most section is 5 to 7 ft thick, fine to medium crystalline dolomite, and contains thin shale partings. Solution features are limited to scattered pinpoint vugs. Most solution enlarged joints stop or become tight above this unit.

The middle section of the upper dolomite is 17 to 22 ft thick and has numerous vugs ranging from pinpoint in size to 3 in. Major solution features typical of karstic bedrock are found in this section. Joints are widened by ground-water solutioning, voids of up to 12 ft in diameter were encountered during drilling, and closed topographic contours and swallow holes are observed where this section forms the erosional bedrock surface. Solution features, of the Laurel dolomite beneath the site are summarized as follows:

- Most of the solution activity is found in the middle section of the Laurel Member and is stratigraphically limited by the underlying setion of dolomite and especially by the shale marker bed.

- The solution enlargement is greatest at the top of bedrock and along bedding plains. Exposed joints ranged from 12 ft to less than 1 inch wide at the top of bedrock. Joints become progressively narrower with dep.h and at tens of feet are tight.

- At a depth of 20 to 25 ft below the bedrock surface solution activity is found as elliptical voids about 1 ft wide and 2 ft long.

- The joints enlarged by solution activity contained clay and soil migrating from the surface.

The middle section of the Laurel Member receives vertical recharge from the overlying unconsolidated glacial deposits and direct infiltration where fractures extend to the land surface (e.g., areas of closed topographic contours and swallow holes). The upper 10 to 20 ft of the member is fine to medium crystalline, thin to medium bedded, and contains stylolites. Erosion has removed this section at some areas of the site.

The Laurel Member is the primary source of ground water for most domestic wells around the site. Ground-water production is variable from 5 to 50 gal/min depending on the fracture density adjacent to the well (McKay 1976). Hydraulic conductivity ranges from less than 1 ft/yr from in unfractured and nonsolutioned portions to over 1000 ft/yr.

Cenozoic Era Soils

Tertiary Period to Pleistocene Epoch

Overlying the Paleozoic units are Pleistocene glacial deposits of the Illinoisan glacial period. The glacial deposits consist of stratified loess and till and commonly overlay older Tertiary to Pleistocene residuum. The residuum is clay rich and forms a barrier to percolation of water into the underlying bedrock. The rate of ground-water recharge at the site is limited by the residuum. A generalized soil profile is given in Figure 8.3.1-4. A typical soil profile at the site consists of 0.5 to 2 ft of loess (a wind blown silt) overlying 2 to 7.5 ft of till (silty clay with minor sand and gravel) overlying 2 to 24 ft of residuum (silty pebbly clay with major rock fragments). Not all of these components are found at every location. The thickness of soil ranges from 0 to 48 ft and averages 16 ft. Along the steep stream valley sides the soil consists of loess and colluvium weathered from bedrock outcroppings.

Soil consisting of glacial silts and clay permit overland travel by heavy machinery for much of the year. Laboratory penetration tests classify till and residuum as stiff to very stiff (FSAR 1982). Early spring rains may require aggregrate placed over primitive access roads or the assistance of bulldozers to reach some locations. Soil characteristics are not judged a hindrance to construction activities at the site.

8.3.2 Regional Fractures

The formation of fractures, also referred to as joints, have a large influence on the hydrogeology at the Marble Hill site. Ground-water flow rate and direction are a function of joint geometry and joint orientation. The characterization of joint patterns is of primary importance for understanding of the site-specific aspects of radionuclide transport in a fractured geologic medium.

Fracturing of the sedimentary rocks in the Marble Hill area occurred in response to regional compressive and tensional forces. Lateral forces are basically structural in origin although the area experienced vertical stress in previous depositional periods and as a result of the advance and retreat of the glaciers. The scale of observable fractures range from less than 1 in., as seen in hand samples, to fracture traces tens of miles long noted in aerial photographs. Fracture traces are quite apparent in the common orientation and linear appearance of creeks and streams in the area. Little Saluda Creek, an unnamed creek south of the plant, and the Ohio River demonstrate a linear trend along primary and secondary joint orientations.

8.3.2.1 Azimuthal Orientation

Joint orientations are recorded for the Marble Hill site in the FSAR (1982). However, soil cover and limited exposure in plant excavations prevents determining the length of most of these joints. Larger joint sets, or fracture traces, are discernable at the regional scale in topographical lineations.

STRATIGRAPHIC NOMENCLATURE					LITHOLOGIC SYMBOL	THICKNESS (feet)	PREDOMINANT SOIL OR ROCK TYPE
ERA	SYSTEM	SERIES	STAGE	FORMATION			
CENOZOIC	QUATERNARY	PLEISTOCENE	RECENT to LATE WISCONSINAN	MARTINSVILLE FORMATION		32[3]	Alluvium: clayey, sandy silt underlain by interbedded silt and fine sand, some gravel.
			WISCONSINAN to KANSAN	ATHERTON FORMATION		48[3]	Glaciofluvial deposits: sand and gravel.
PALEOZOIC	ORDOVICIAN	CINCINNATIAN	RICHMONDIAN ? MAYSVILLIAN	DILLSBORO FORMATION		160	Interbedded shale and limestone.

Notes

1. Not drawn to scale.
2. The stratigraphic units below the Dillsboro Formation are the same as shown on Fig. 2.5-39,SH.1
3. Thicknesses of the Martinsville and Atherton Formations are based on data from 24 borings and 19 borings, respectively, in the flood plain. The estimated thickness of the Dillsboro Fm. is that thickness under the Martinsville and Atherton Formations.

Reference

Wayne, W. J., 1963, Pleistocene Formations in Indiana, Indiana Geological Survey Bulletin 25, 85 p.

FIGURE 8.3.1-4. Stratigraphy of Soil Units (Source: Marble Hill FSAR 1982)

These fractures are observable at a large scale in topographic maps and aerial photographs. Marble Hill site data do not contain sufficient information to determine the distribution of fracture lengths. Geophysical studies conducted at the site were used to locate fractured areas but were not able to resolve fracture sets.

Information on fracture orientation and length is available at the regional scale. Fracture-induced lineaments are mapped adjacent to the plant site in northern Jefferson County (Greeman 1981). The lineaments were determined from air photographs and a minimum fracture length of 500 ft was observable at map scale. These lineament data were statistically analyzed to determine: 1) the distribution of fracture orientations, and 2) the distribution of fracture lengths.

The fractures mapped by Greenman cover an area of 467 sq mi. The stastistical analysis of fractures was limited to a 32-sq mi area located nearest to the Marble Hill site, and consisting of the same surficial geologic units as at the Marble Hill site. In the 32-sq mi sub-area 339 fracture orientations and 622 fracture lengths were examined. The distribution of joint set lengths is expected to be similar to that beneath the Marble Hill site.

The distribution of regional fracture orientations is illustrated in Figure 8.3.2-1. Azimuthal orientation of these fractures demonstrates a strong primary trend at N40E (read as north 40 degrees east; by convention, linear features are described by their orientation from north, the southward component of S40W is apparent and not included). A moderate secondary trend of N35W is also noted. The regional distribution of fractures also shows the common association of primary and secondary orientations at approximately 60 to 90 degrees to each other.

8.3.2.2 Fracture Length

The distribution of regional fracture lengths is presented in Figure 8.3.2-2 on a logrithimic scale. The figure demonstrates two linear relationships in the data set. Fractures less than 3150 ft follow a different distribution than those greater than 3150 ft. This may be due to the visual interpretation methods used to identify fractures rather than a data association. It is possible that at the map scale nearly coinciding fractures are visually perceived as a single fracture. This would tend to produce a few sets of very long fractures. The maximum fracture length of interest for the Marble Hill site is shorter than the change in the distribution illustrated in Figure 8.3.2-2. The data trend defined by a logarithmic transformation results in a linear fit to the data with a correlation coefficient of 0.993.

8.3.3 Hydrology

8.3.3.1 Hydrostratigraphic Units

The choice of hydrostratigraphic units for characterization is governed by the conceptual models of the site hydrology and core melt processes. The

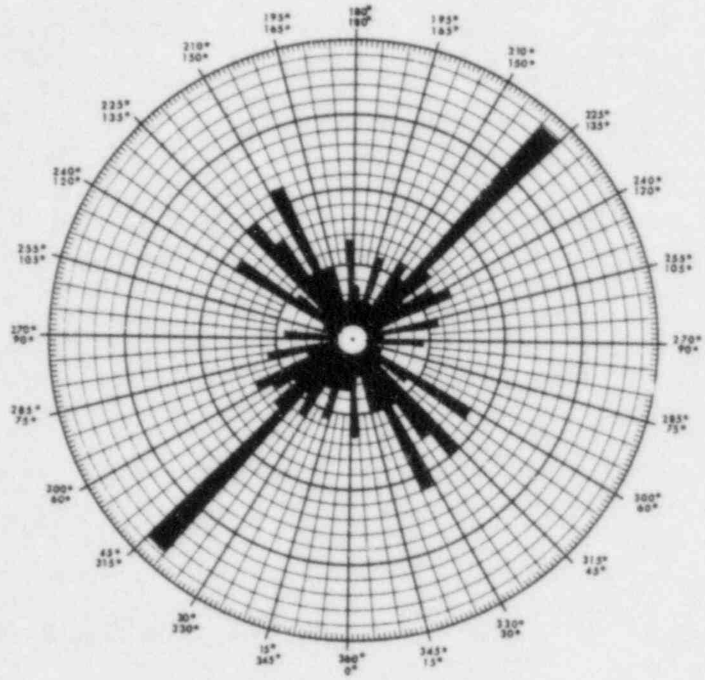

FIGURE 8.3.2-1. Regional Orientation of Fractures

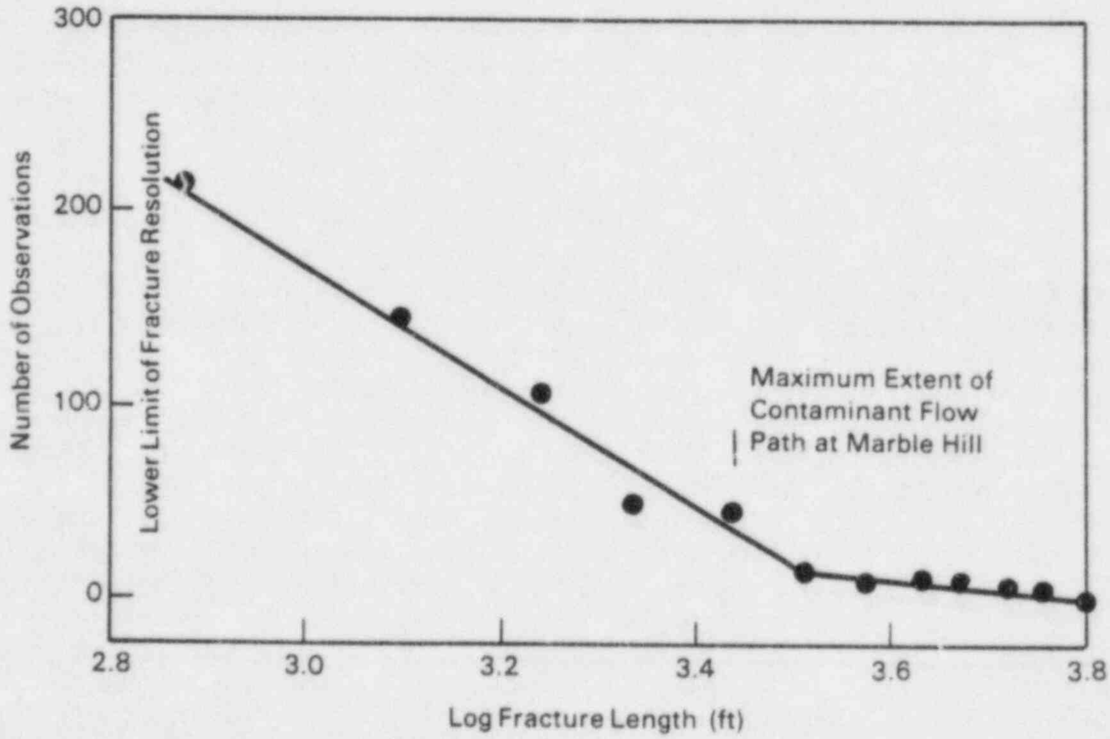

FIGURE 8.3.2-2. Distribution of Regional Fracture Lengths

Marble Hill site is complex in that the geologic unit(s) into which the contaminants would flow is uncertain. Separate components of an accidental release of radionuclides could enter into the same or different geologic units. The rationale for selection of two hydrogeological units is given below.

A core melt accident at the Marble Hill site could consist of two contaminated components, core melt debris and sump water. The core debris would mainly contaminate the geologic units at or below the melt zone. The sump water would seep into the geologic units at the level of the melt zone and under high hydraulic head could be forced up into the more permeable units near land surface. These two conditions of radionuclide release are discussed in Section 8.2.2. The geologic units that could be involved in a sump water release have different fracture orientations, hydraulic conductivities, and ground-water gradients. Therefore, the direction and flow rate of contaminated ground water would be determined by accident specific conditions that at the Marble Hill site cannot be determined prior to a core melt accident.

8.3.3.2 Lower Hydrologic Unit

Fractures

Joint orientations at the Marble Hill site are presented for two geologic classifications: 1) the Saluda Formation through the Osgood Member of the Salomonie Dolomite and 2) the units above the shale marker bed of the Laurel Member of the Salomonie Dolomite. The joints are observed to be vertical or near vertical at all locations.

The generalized orientation for 400 fractures measured at outcrops is given in Figure 8.3.3-1. The pattern of orientation is similar to the regional fractures given in Figure 8.3.2-1. The primary orientation is N5W and the secondry orientation is at N55W. This represents a counterclockwise rotation of about 45 degrees of the primary orientation from that of the regional data and probably represents a change in the major direction of stress of northern and southern Jefferson County at the time the fractures were formed. The fractures in the primary direction are more slighty numerous in the site data at 27 as compared to 22 precent for the regional data. The secondary orientations are correlated in position and frequency. The secondary fracture orientation provides numerous interconnections between parallel fractures in the primary direction. These data are the best represention of the geologic units below the shale marker bed.

Fractures at outcroppings were observed by PNL staff to the east of the plant in the Saluda Formation. A 30 ft vertical section of the dolomite is exposed in cliffs along the Ohio River bluff. At this location joints are spaced horizontally at 5- to 15-ft intervals. The joints extend vertically through most bedding planes although offsets of up to 1 ft are found at thin shale layers. Fracture faces show minor solutioning, normally expressed as fluting of the rock face. The joints are tight with no measurable openings. Seepage of ground water was observed as a damp zone along the fracture.

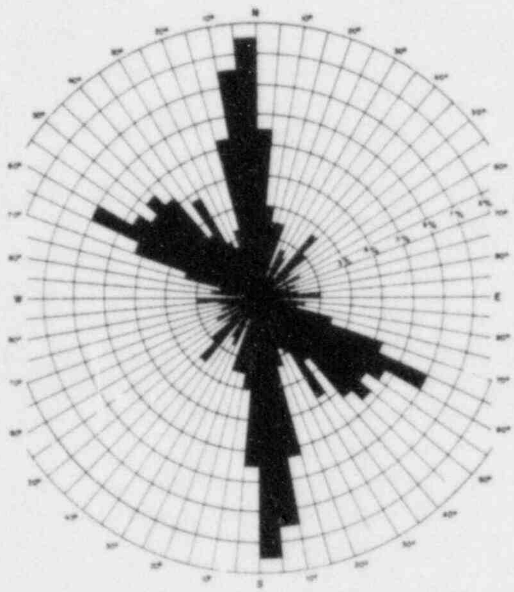

FIGURE 8.3.3-1. Orientation of Joints in Lower Hydrologic Unit (Source: Marble Hill FSAR 1982)

Water Levels Observed in Lower Unit

In a low hydraulic head release, the geologic unit of interest would be the Saluda Formation. Water level measurements in this formation have been summarized in the FSAR (1982) as: 1) very slow to stablize after drilling (up to 3 years), and 2) about 20 ft lower than in the overlying Laurel member and 3) not reflecting a common potentiometric surface. Water level measurements for the lower unit are plotted in Figure 8.3.3-2. The dashed lines indicate the tentative location of lines of equal water level. Interpretation of water-level is complicated by the hydraulic effects of irregular fracture openings and the three-dimensional nature of a solutioned geologic unit.

The plant site is clearly situated on a ground-water divide. Assuming that the direction of flow will be down the hydraulic gradient and along fracture avenues, the contaminant pathways are to the northwest into a tributary of Little Saluda Creek and into an unnamed creek to the southeast. It is conceivable that radionuclides from the two reactors would travel in separate directions or that the plume could bifurcate and enter both surface water drainages.

Recharge

The lower hydrologic unit receives recharge through downward perculation from overlying shale and dolomite. The water levels in the Saluda Formation are 10 to 20 ft lower than the upper unit. Vertical fractures do not extend to land surface and increased precipitatation in the spring time produces a gentle

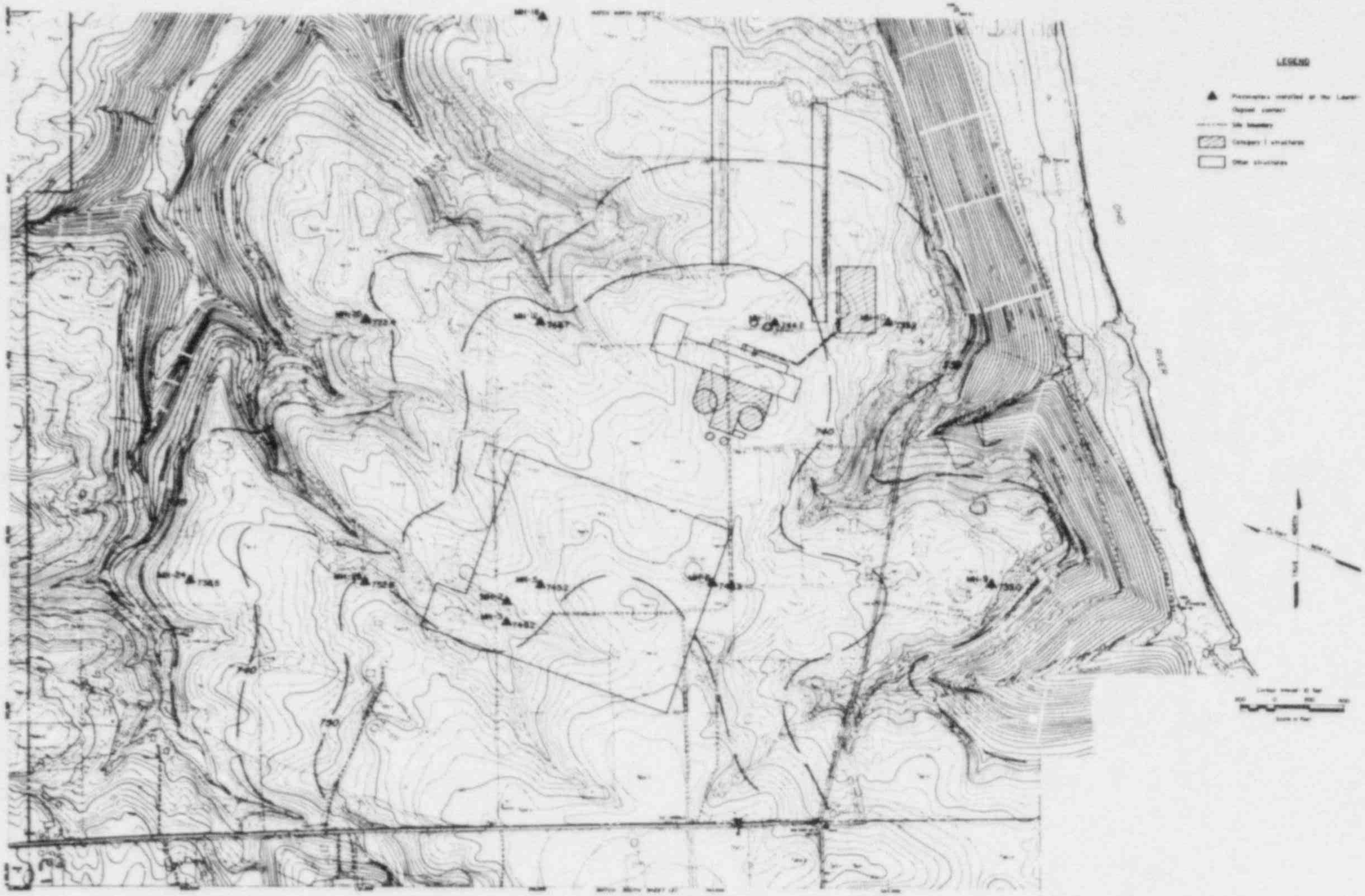

FIGURE 8.3.3-2. Water Levels Observed for Lower Hydrologic Unit
(Source: Marble Hill FSAR 1982)

rise in observed water levels of 1 to 3 ft. The lower unit is buffered from rapid pulses of percolation by the overlying shale and is insensitive to extreme precipitation events.

Transmissivity

The transmissivity of the Saluda Formation was determined from hydraulic conductivity tests at 444 intervals located in 70 boreholes. Hydraulic conductivities determined at 10 ft intervals were combined to determine a single transmissivity for each bore. The tests show that highly permeable zones in a bore are normally limited to a short vertical distance and that the remaining length of the bore has a very low or negligible permeability. These observations support the conceptual model where the bulk of ground-water flow is in the fractured portions of each geologic unit.

The permeability of porous media often are lognormally distributed about an average value (Freeze and Cherry 1979). The reported transmissivities range over four orders of magnitude and do not group about an average or median value. The large spread of values requires that the characterization of permeability be more detailed than a simple mean or range of extreme values. The data for the Saluda Formation are presented in Figure 8.3.3-3 as a cumulative density function.

Porosity

The effective porosity in the Saluda Formation was judged to be very low. Geophysical measurements in boreholes at Marble Hill include a relative measure of total porosity. Effective porosities are not included in site documents. Visual inspection of drill corings, exposed rock faces, and the reported values of the rock quality designation indicated that fractures are the main contributor of effective porosity and that the value is quite low. The outcrops of Saluda Formation examined along the Ohio River bluff exhibited tight fractures and no significant solutioning or seepage. The effective porosity of the Saluda Formation is estimated by the author through hydrological judgment to be less than 1×10^{-3} and possibly as low as 1×10^{-4}. A conservative yet realistic value of 5×10^{-4} is used to characterize this unit.

8.3.3.3 Upper Unit

Fractures

Fractures measured in plant excavations are given in Figure 8.3.3-4. The fracture orientations are from 187 measurements mostly in the Laurel Member of the Salomonie Dolomite. The fracture pattern contains a strong primary direction at N30E and no clear secondary orientation. The lack of a secondary fracture orientation increases the anisotropy of the upper unit and reduces flow between parallel fractures along the primary orientation. These fractures are above the shale marker bed and solutioning may cause them to be more prominent in the direction of the ground-water gradient. Fractures along the major flow direction would tend to become preferentially enhanced. An alternate explanation for the predominance of a single orientation is that it

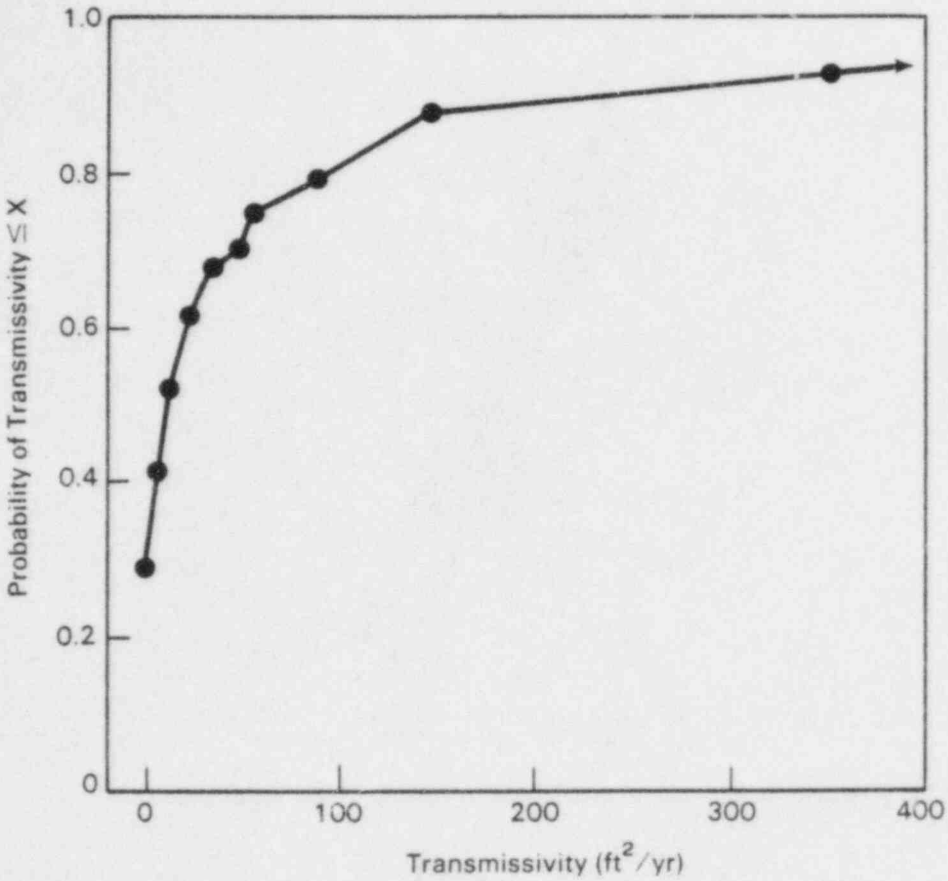

FIGURE 8.3.3-3. Cumulative Density Distribution of Transmissivities for Lower Hydrologic Unit

is a local anomaly. These data represent joint orientations in the rock immediately adjacent to or under the reactors and indicate the preferred direction of ground-water flow.

Spatial distribution of joints or fractures for this unit is known through geophysical studies. Seismic reflection investigation at the plant site identified zones of low bedrock velocity that have been interpreted as heavily fractured areas. These features are in agreement with the primary orientation of the joints in the Laurel Member. However, these data are too few to deterministically describe the preferential ground-water pathways that contaminant would follow.

Potentiometric Surface

The upper hydrologic unit is comprised of the sections of the Laurel Member above the shale marker bed. Water level measurements for this unit are

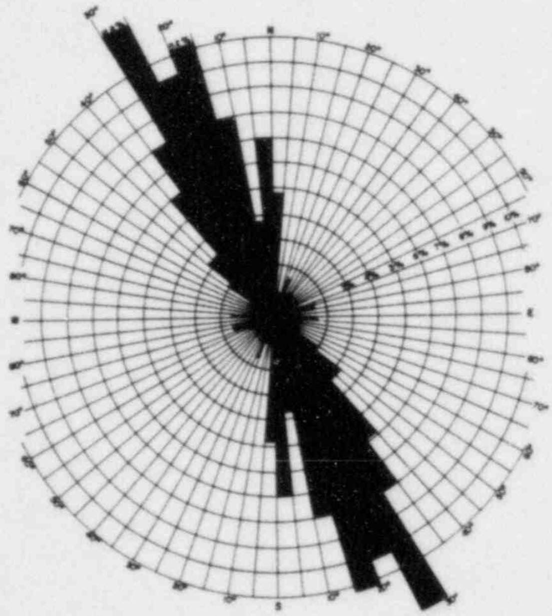

FIGURE 8.3.3-4. Orientation of Joints in Upper Hydrologic Unit (Source: Marble Hill FSAR 1982)

plotted in Figure 8.3.3-5. The potentiometric surface for the upper unit is based on a greater number of data points and is better spatially defined than the lower unit. The ground-water divide observed in the lower unit is also present in the upper unit. The apparent crest of the divide passes between the two reactors, and the direction of contaminant migration is not obvious. A situation similar to the lower hydrologic unit exists, and contaminated ground water could flow to the east or northwest depending on which reactor is under consideration and the exact nature of the hydrology under the plant. As may be possible at a severe accident site, uncertainty exists as to the preferential contaminant pathway. A bifurcated contaminant plume emanating from either of the reactors is also possible in this hydrologic unit.

Lateral ground-water flow is from the topographic high to the south and discharges along outcroppings located at the river bluff. Downward migration of water is limited by the shale beds of the Salomonie Dolomite. The upper hydrologic unit receives recharge through vertical infiltration through the overlying soil cover and where solution features reach land surface. Soil composed of residuum is relatively impermeable, and rapid increase of water levels following precipitation is not noted on piezometer hydrographs. Discharge was observed by the author mainly where the unit is dissected by intermittent surface water streams. Fractures exposed in this unit during construction did not show seepage associated with rainfall events.

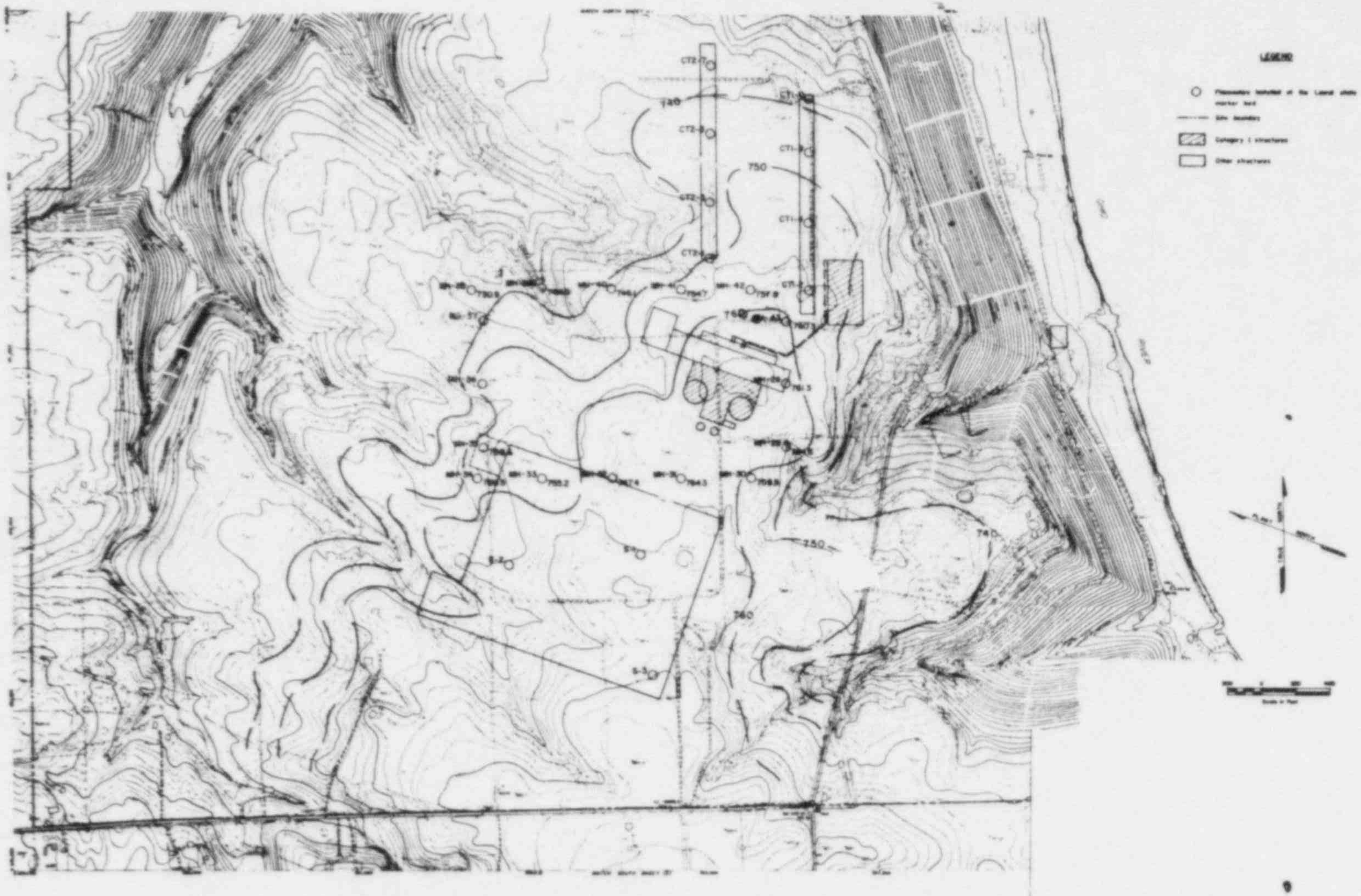

FIGURE 8.3.3-5. Potentiometric Surface for Upper Hydrologic Unit (Source: Marble Hill FSAR 1982)

Transmissivities

The permeability of the upper hydrologic unit also varies widely with location similarly to the lower hydrologic unit. The transmissivity at each bore penetrating the lower unit was determined by summing the hydraulic conductivity data and multiplying by the thickness of the test zones. Spatial trends to the value of transmissivity are not observed in the data. Commonly the permeability is very low for most of the bore with small discrete zones providing most of the transmissivity. Bores that did not intercept a fracture had no measurable permeability. The transmissivity for this unit is calculated from 245 hydraulic conductivity measurements in 105 boreholes.

The tranmissivity data range over three orders of magnitude and do not group about an average or median value. Transmissivity values in porous media often are log normally distributed (Freeze and Cherry 1979). These data for fracture controlled transmissivity do not form a lognormal distribution. The data are presented in Figure 8.3.3-6 as a cumulative density function.

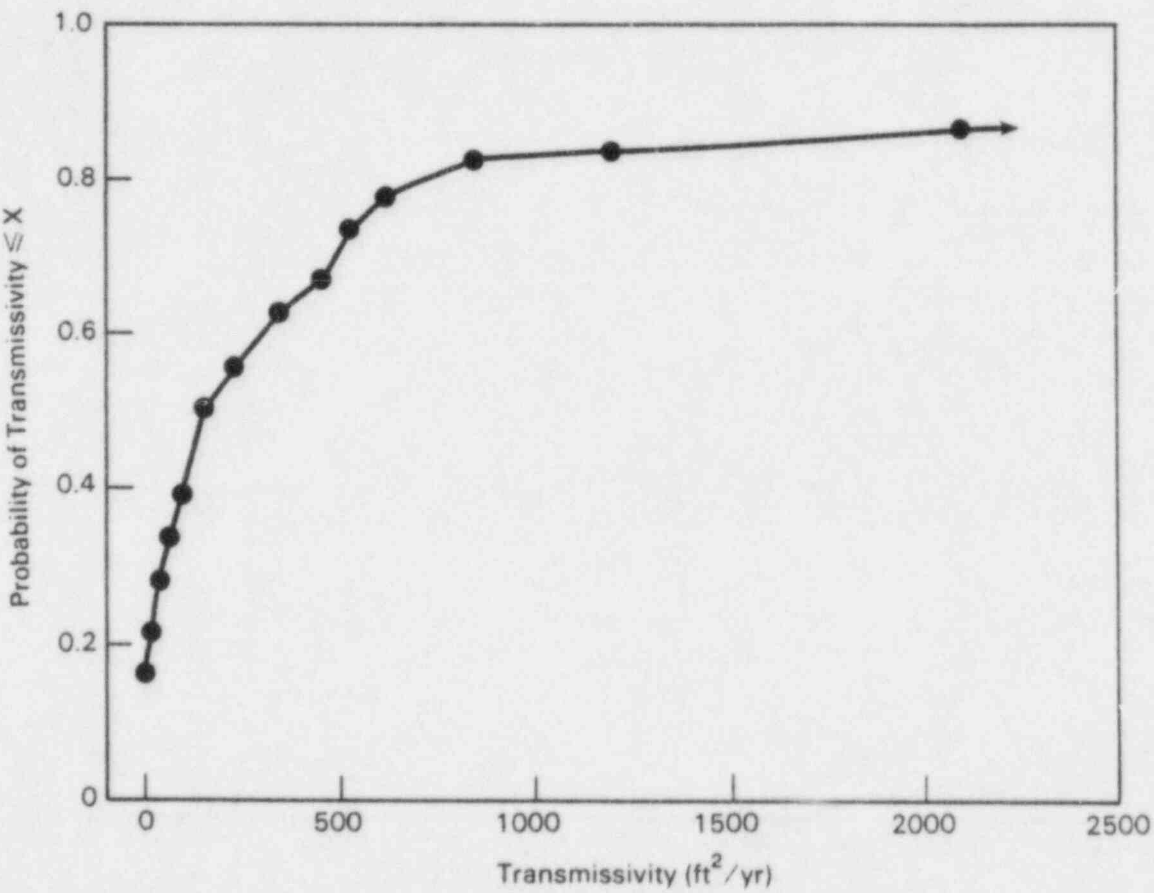

FIGURE 8.3.3-6. Cummulative Density Distribution of Transmissivities for Upper Hydrologic Unit

Porosity

The effective porosity in the Laurel Member is judged to be low. In highly solutioned areas the effective porosity may reach as high as 1×10^{-2}. Visual inspection of drill corings, exposed rock faces and the reported values of the rock quality designation indicate that fractures and solution openings are the main contributors of effective porosity and that the value is quite low. Effective porosities are not included in site documents. The Laurel Member is estimated by hydrological judgment to have an average effective porosity less than 1×10^{-2} and possibly as low as 1×10^{-4}. A conservative yet realistic value of 1×10^{-3} is used to characterize this unit.

8.4 PREMITIGATIVE FLOW ANALYSIS

8.4.1 Selection of Accident Scenarios

The Marble Hill site is inductive to the conceptualization of a suite of feasible contaminant pathways and initial radionuclide quantities. As discussed in Section 8.2.2, there are three possible release classifications: core melt leachate, high hydraulic head sump water, and low hydraulic head sump water. The hydrogeology of the site yields two distinct hydrologic units with dissimilar characteristics, an upper and a lower hydrologic unit. In addition, the direction of flow could be to the northwest and/or southeast depending on the accident conditions and the precise post-accident flow field. These situations are presented in Figure 8.4.1-1 demonstrating the various combinations of accident conditions. The prime accident scenarios for analysis are those that are most probable and those that would be the most severe to the environment. The scenarios selected for premitigative analysis are labeled as Number 1,2,3 and 4 on the figure. The selection allows:

- examination of the most severe accident (i.e., case No. 1),
- comparison of westerly and easterly ground-water flow paths,
- comparison of sump water and core debris leachate releases,
- determination of the severity of an accident in the more probable hydrologic unit to be contaminated (i.e., lower unit) and,
- determination of the necessity of mitigative actions in both the upper and lower hydrologic units.

8.4.2 Considerations for Selection of a Modeling Approach

8.4.2.1 Modeling Objectives

The intricate and variable nature of the hydrogeology at the Marble Hill site complicates any examination of contaminant migration. The fracture network would introduce major differences to flow and transport as compared to porous media. The movement of water and transport of radionuclides in a fractured system would be mainly along the preferential flow channels created

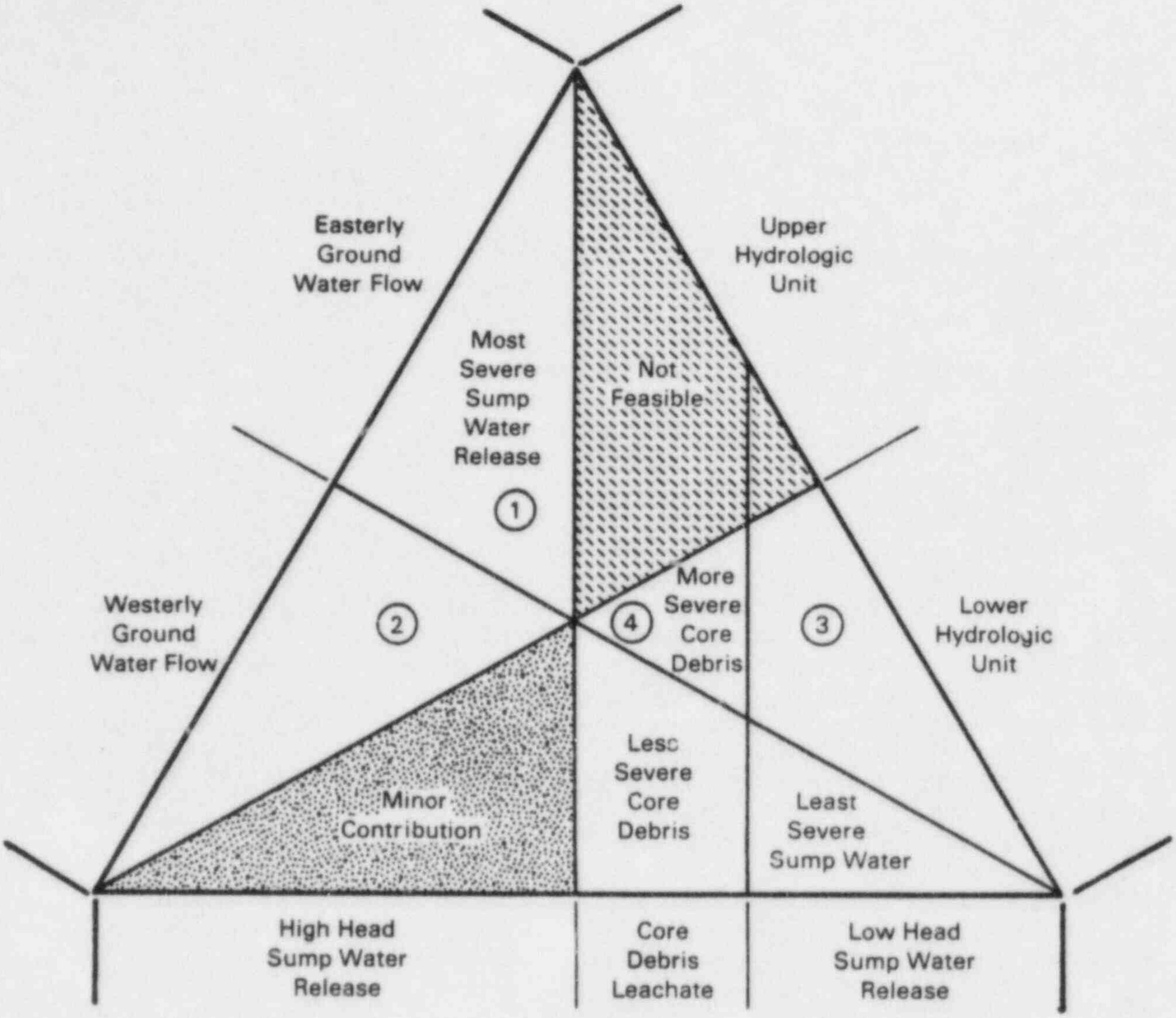

FIGURE 8.4.1-1. Selection of Accident Scenarios

by fracturing and solutioning. The characterization of these fracture flow paths is of prime importance to the determination of contaminant transport rates, flow direction, and design of a mitigative scheme. Detailed examination of mitigative factors unique to fractured systems is a prime objective of this case study.

8.4.2.2 Discrete versus Continuum Analysis

Simulation of ground-water flow in fractured media, such as found at Marble Hill, is fundamentially different than for porous or extensively fractured media. In porous formations the degree of interconnection of the pore spaces allows the medium to be considered as a continuum. From a

megascopic viewpoint the rock and fluid properties are considered to be continuous throughout the given volume of space. The continuum approach computes the flow through an elemental control volume of porous material without analyzing microscopic flow. The determination of the microscopic flow field within the control volume is avoided in the continuum approach because only the macroscopic properties of the porous medium are evaluated. The macroscopic flow rate is taken as the Darcian average through a given volume (Endo 1984). When the fracturing of the rock is extensive, a hydrologic unit can be considered an equivalent of porous media. This assumption can also be made when the scale of examination of the fractured units is very large, as in the case of large-scale regional analysis.

The site data for the Marble Hill site demonstrates that the fracture network results in a highly variable hydraulic system at the scale of mitigative barriers. Fractures at this site are preferentially orientated and an anisotropic flow field is clearly indicated. However, no hydraulic determination of the degree of anisotropy has been made at the site. The wide range in transmissivity indicates that an average or extreme value will not properly describe the flow system. Assuming an average transmissivity value would severly underestimate the contaminant migration along some preferential flow paths. In this case, mitigative techniques could be designed based on an improper response time and safe distance from the reactor for construction of a mitigative scheme. Assuming an upper-extreme value of transmissivity in more simulations would over estimate the quantity of contaminant migrating at a high rate. Also, the necessity and design basis of a mitigative technique would be severely overestimated.

Both problems of determining the proper scale of the control volume to preserve the fractured character of the flow system and determining the degree of anisotropy are related to the continuum approach. Therefore, a discrete approach which simulates flow and transport in a representive number of individual fractures is indicated. The discrete approach requires more detailed information of a flow network and analyzes the internal structure and activities within the control volume to determine the system output (Endo 1984).

8.4.2.3 Data Base Limitations

The spatial limitations in the data also influence the approach taken to characterize the site. The FSAR (1982) contains a large amount of hydrologic data for areas near the reactors. However, other locations along the probable pathway(s) are not hydrologically characterized. Specifically, there are few measurements of transmissivity, ground water levels, and fracture orientations outside the reactor area. The lack of spatial data for a significant portion of the contaminant pathway requires that additional hydrologic data be estimated or interpolated. Transmissivity values are dependent on location in relation to fractures and fracture zones. The spatial dependence of transmissivity is not described as broad areas of gradual change, but rather as highly linear transmissive zones through a relatively impermeable matrix. The use of estimation techniques such as the inverse method to map relative changes in permeability is prevented by: 1) a lack of sufficient water level and

hydraulic conductivity data along the flow path, and 2) the reported water levels for the lower hydrologic unit were judged in the FSAR as not representitive of a common potentiometric surface. Finally, the fracture network at Marble Hill is mapped in insufficient detail to deterministically model individual fracture zones between the plant area and the discharge points.

The hydrogeologic data previously collected at the site are sufficient to statistically characterize the major elements of a discrete model. Specifically, the data base for this study contains 689 measures of hydraulic conductivity, 587 fracture orientations, and 622 measures of fracture length. A statistical approach to the flow modeling was suggested by the large size but low or unrecorded spatial distribution of the data base.

8.4.2.4 Approach to Modeling the Marble Hill Flow System

The modeling approach for ground-water flow at the Marble Hill site is based on three major considerations:

1. the desirability of conducting a discrete analysis of flow through fractures,

2. the necessity of retaining the parameter variability of the fracture flow system, and

3. the spatial limitations of the existing data base.

The approach taken to this ground water flow problem was to statistically retain the variability of transmissivity (expressed as an aperture width), fracture orientation, and fracture length in a stochastic realization of the flow field. It must be recognized that this type of flow simulation does not represent the actual positions, lengths, apertures, or orientations of specific site fractures. Knowledge of the system at that level of detail is not realistically achievable. Stochastic modeling of discrete fractures is a developing methodology in hydrology, which will certainly undergo further refinement. The model simulations provided here are not designed to answer many outstanding questions concerning fracture flow (e.g., fracture density, aperture scaling, diffusion into the rock matrix, and retardation). This case study is a demonstration of some of the transport characteristics of an idealized fractured hydrologic unit. The approach results in a simulation which is statistically representive of the site.

8.4.2.5 Code Selection

The code used to simulate radionuclide transport at this site is FRACTURE by L. W. Vail and T. J. McKeon (personal communication, 1984). The code stochastically generates a series of statistical estimates of aperture width, joint orientation, and joint length to create a network of interconnected flow pathways. There are five key assumptions that are made in the course of applying this code:

1. the rock matrix is conservativly assumed to be impermeable and contaminant does not enter the space between fractures,

2. dead-end fracture segments do not contribute to the flow process in this realization of a fractured system. These segments are truncated to produce a network of fractures that outwardly resembles interconnected pipes,

3. boundary conditions are Dirichlet consisting of constant head along the exterior boundaries,

4. the flow field is under steady state conditions, and

5. retardation mechanisms are described by equiliberium distribution processes.

The code uses an iterative head solver which computes the head at each fracture intersection. Simulation of discrete pathways for solution of fractured flow networks is a recent development (Endo 1984). The advantages of this methodology are that it uses a relatively simple code and a rapid solution of the equations is possible. The transport of radionuclides is simulated by releasing a large number of particles into the flow stream. The number of particles released over time is proportional to the radionuclide release rate of sump water or core debris. Mixing of waters at fracture intersections is acheived by stochastically distributing the contaminant based on relative flow quantities.

8.4.2.6 Model Limitations

This model does not represent any specific fracture or fracture zone known to exist at the Marble Hill site. As with all models, this simulation does not represent all aspects of the phenomena being studied. Specifically, the assumption of no flow or transport into the rock matrix and the idealized treatment of retardation introduce an unknown level of uncertainty. Other major factors such as the fact that fracture apertures are not a single width along their entire length, but rather vary in width and fluid volume over short distances limits the model. Fractures are also three dimensional in nature, not just the two dimensions considered herein, and are subject to abrupt offsets.

Modeling of fractured systems by stochastic-discrete codes is an emerging technology severely limited by the difficulty in characterization of fracture geometry and the physics of flow in an irregular and narrow pathway (Endo 1984). The flow and transport code used in this case study represents the basic character of the fracture flow with the intent of highlighting the more important features (i.e., preferential flow direction, large range of flow velocities, etc). As further development of this methodology takes place, improvements in our understanding of fracture processes can be expected.

8.4.3 Stochastic Characterization

8.4.3.1 Hydrologic Data Requirements

The data required for a stochastic generation of the flow networks are the probability density functions of fracture length, fracture orientation and fracture aperture. The first of these data requirements are fulfilled by values contained in Section 8.3.3. The transmissivity values derived from the FSAR 1982 are used to compute an equivalent aperture width through a series of data manipulations described below. Effective porosity is not used to determine pore velocity in this methodology. The hydraulic conductivity of a hydrologic unit is defined as:

$$K = \frac{T}{b} \tag{8.1}$$

where
K = hydraulic conductivity (L/t)
T = transmissivity (L^2/t)
b = hydrologic unit thickness (L)
L = unit of length
t = unit of time.

The volumetric flux in porous media, sometimes referred to as a Darcy flux or a Darcy velocity, is defined as:

$$v = KI = \frac{\rho g k I}{u} \tag{8.2}$$

where
v = volumetric flux (L/t)
ρ = fluid mass density (M/L3)
g = gravitational acceleration (L/t2)
k = intrinsic permeability (L2)
I = hydraulic gradient (L/L)
u = dynamic viscosity (M/tL)
M = unit of mass

The flux in fractures is defined by Witherspoon (1979) as:

$$v = \frac{\rho g B^2 I}{12\mu} \tag{8.3}$$

where

B = fracture aperture (L) and other terms as previously defined.

Relating these equations results in:

$$B = [k12]^{0.5} \tag{8.4}$$

8.4.3.2 Contaminant Source and Discharge Locations

The source of contamination is conservatively assumed to be the reactor nearest the discharge location. That is, the contaminant migrating to the east is assumed to emanate from the eastern reactor (Unit No. 1) and the contaminant from the western reactor (Unit No. 2) migrates to the west. The selection of the accident location as being the reactor nearer the discharge area provides the shortest flow path, the steepest hydraulic gradient and is consistent with flow directions estimated from the generalized potentiometric surface. At many other nuclear power plant sites, including the South Texas Plant, the long travel path to the accessible environment makes selection of the reactor unit immaterial.

The discharge areas are judged to be located at the incised stream heads located down the hydraulic gradients from the plant. Precise location of the discharge areas is also difficult to determine because anisotropic fracture networks do not allow contaminant to travel directly down the hydraulic gradient except when the gradient and the fractures are aligned along a common orientation. The plant is located on a ground-water divide for both the lower and upper hydrologic units, and flow direction is uncertain. Bifurcation of the contaminant plume is possible and radionuclides would be transported to both the eastern and western discharge areas. The probable extent of the discharge areas are indicated on Figure 8.2.3-1. The areas shown on the figure were determined through hydrologic judgment based on fracture orientation, hydraulic gradient, topographic position and stratigraphic position.

8.4.3.3 Basis of Comparison for Selected Accident Scenarios

The prime aspects of contaminant transport through the flow system are summarized as the amount of contaminant discharged to the accessible environment over time. For this case study the accessible environment is taken as the point of groundwater discharge from the carbonate units into the unconsolidated glacial-fluvial deposits. This format allows direct comparison among the simulations of first arrival times, peak discharge rates, and the length of time hazardous discharges would occur. The graphical representation of the amount of contaminant passing through a reference plane is commonly referred to as a contaminant breakthrough curve. The shape of the break through curve in fractured systems is more irregular and of a longer period than would be produced in a continuum system.

8.4.3.4 Premitigated Discharge from Lower Hydrologic Unit

Fracture Network

A fracture network representive of the lower hydrologic unit is presented in Figure 8.4.3-1. The fracture lengths and orientations are simulated from statistical distributions of site data. Fracture apertures are also variable in the fracture network, which is not shown in the figure. The location of the

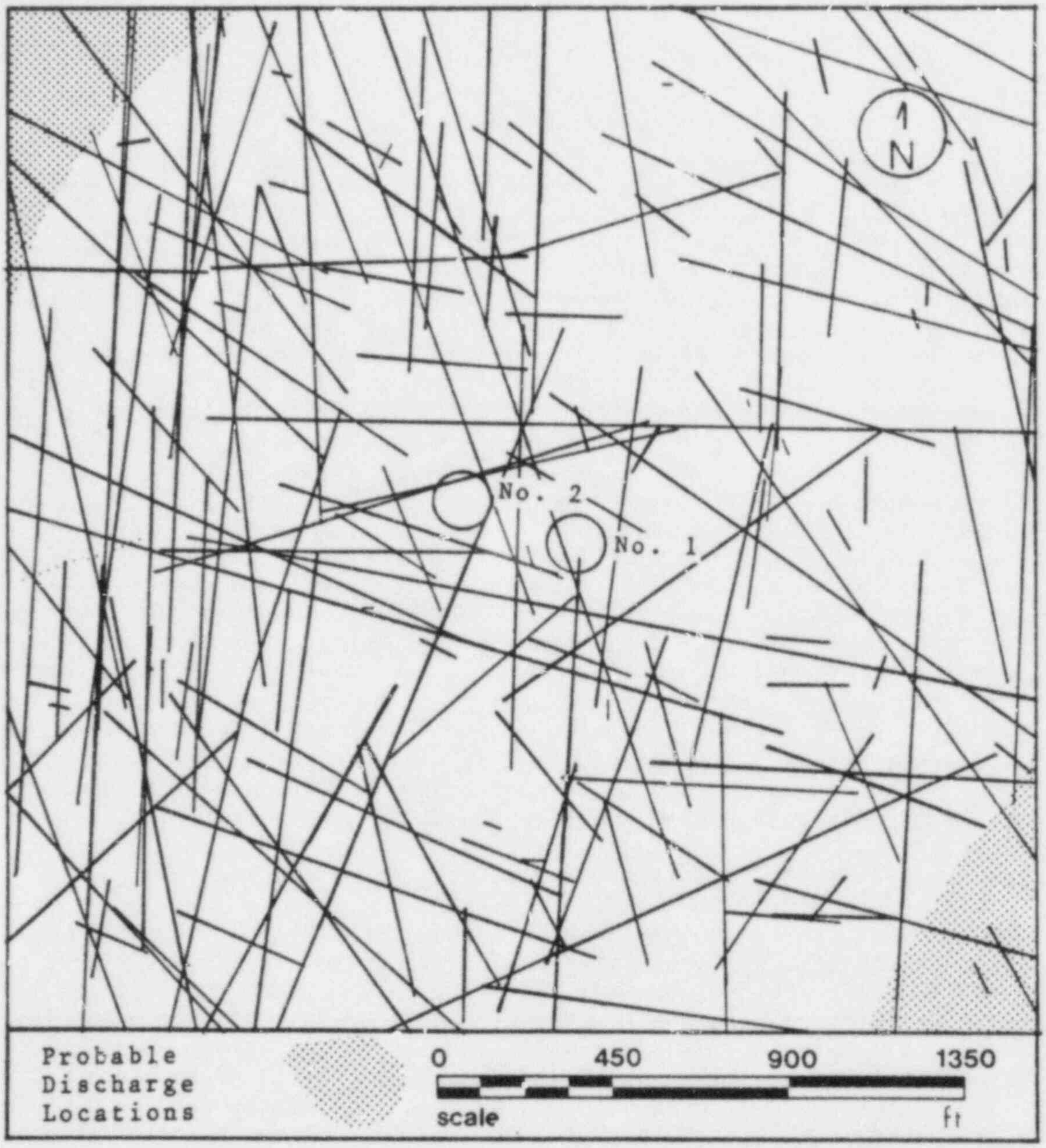

FIGURE 8.4.3-1. Stochastic Fracture Network for Lower Hydrologic Unit

reactors and probable discharge areas are indicated with the fracture network. The primary and secondary orientations of the fractures are seen in the figure.

Anisotropy

The degree of anisotropy of the flow system is determined by applying a hydraulic gradient at various angles to the fracture network and observing the relative flow quantities at the discharge boundary. The relative permeability of the system to the direction of stress is presented in Figure 8.4.3-2. As the predominant fracture orientation becomes normal to the hydraulic gradient, flow is reduced in the system. The ratio of permeability differences with network orientation defines the degree of anisotropy. The lower hydrologic unit has a maximum degree of anisothopy of 27:1 along an orientation of N15E. The fracture network produces a maximum permeability positioned between the primary and secondary fracture orientations.

Contaminant Discharge for Sump Water Flowing to the East, Case No. 3

The contaminant breakthrough curve for the lower unit is presented in Figure 8.4.3-3. The irregular discharge flux of strontium is due to separate fractures transporting varying quantity of radionuclides at varying rates. Some short duration and high activity flux events have been averaged into

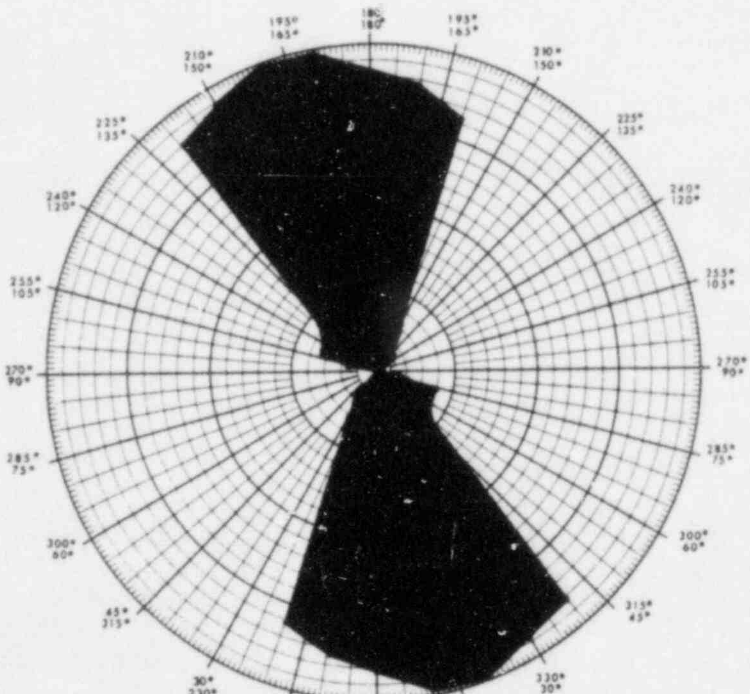

FIGURE 8.4.3-2. Anisotropy of Lower Hydrologic Unit (Permeability/Highest Permeability)

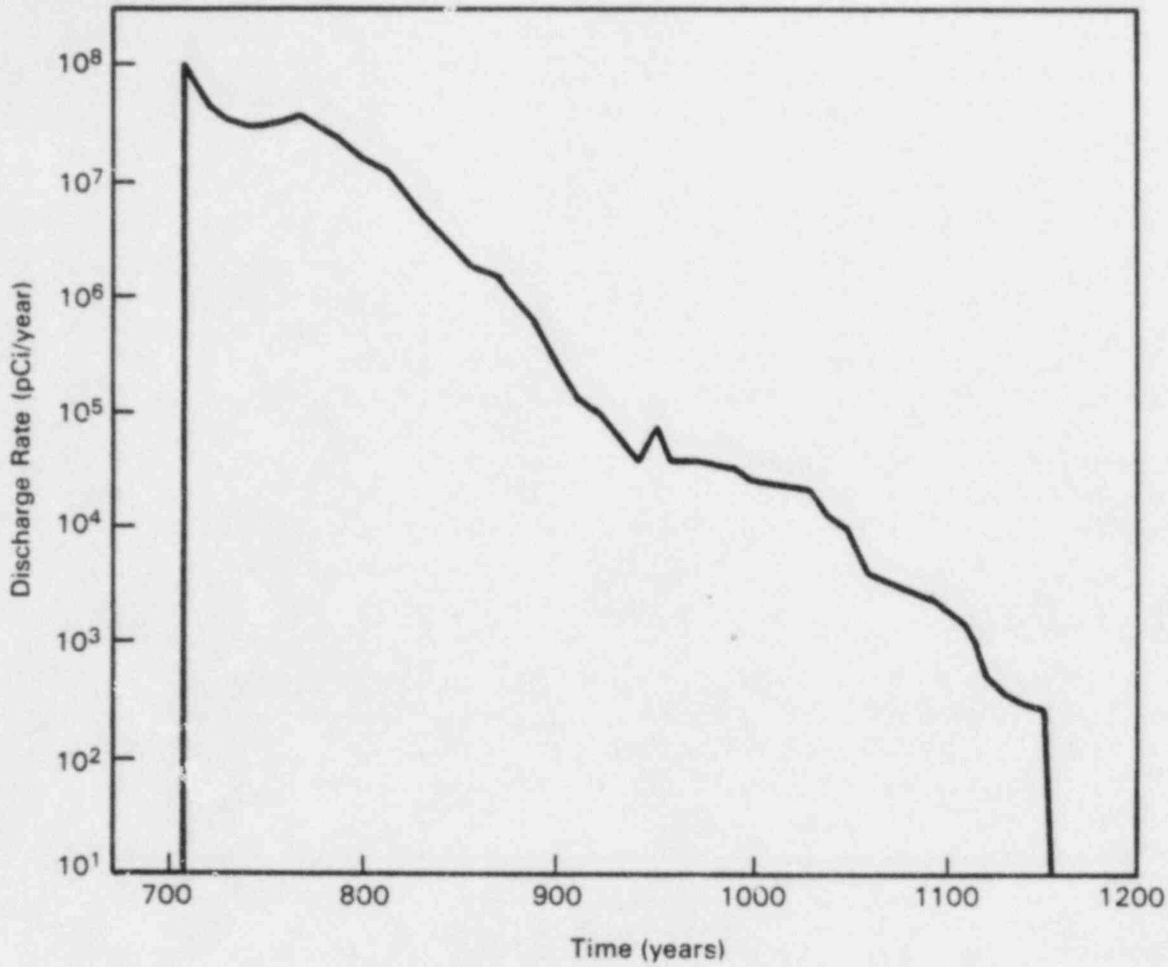

FIGURE 8.4.3-3. Contaminant Discharge for Case No. 3

cumulative amounts and are not seen in the figure. The first arrival of strontium-90 at the surface environment would be at 705 years and the peak strontium flux would be 1.5 x 10^8 picocuries/year. The majority of contaminant has reached the accessible environment within 1150 years after the accident. Minor quantities of strontium-90 continue to arrive at the discharge location at fluxes of 1 x 10^{-5} to 1 x 10^{-20} picocurries per year for an additional 2500 years.

The long period of contaminant discharge is a result of two factors: 1) contaminant being delayed in low permeability-low velocity fractures, and 2) a slow sump water release into the bedrock. When a low permeability fracture lies along the primary orientation, most contaminant bypasses that particular flow channel in favor of the more permeable channels. Examination of Figure 8.4.3-3 shows that no single fracture connects the contaminant source area with the discharge area. Therefore, the fractures transmit water in a series of subparallel flow channels of limited extent. The cross connections produced by fractures not along the primary orientation provide the vital interconnections among the subparallel fractures for water and contaminant transport.

Contaminant Discharge for Core Melt Leachate Flowing to the West, Case No. 4

The contaminant discharge for strontium-90 at the accessible environment is presented in Figure 8.4.3-4. The leach release rate is described in Section 8.2.2. The first arrival of strontium-90 at the surface would be at 2245 years at 3×10^{-9} pCi/yr. The peak strontium flux would be 1.45×10^{-6} pCi/yr. A secondary surge in discharge flux occurs at 2405 years and reaches 1×10^{-10} pCi/yr. The long period of core debris leaching provides for a continuous source of contaminant and the discharge flux continues to decline at the rate indicated on Figure 8.4.3-4. This case produced the lowest environmental fluxes of this case study. The intent of this case is to demonstrate the severity of the most likely core melt scenario (i.e., core debris in the lower unit).

8.4.3.5 Premitigated Discharge from Upper Hydrologic Unit

A fracture network representive of the upper hydrologic unit is presented in Figure 8.4.3-5. Fracture apertures are also variable in the fracture network, which is not shown in the figure. The preferential orientation of fractures to the northeast is clearly seen in the figure. No single fracture connects the contaminant source area and the discharge location. Hence, the

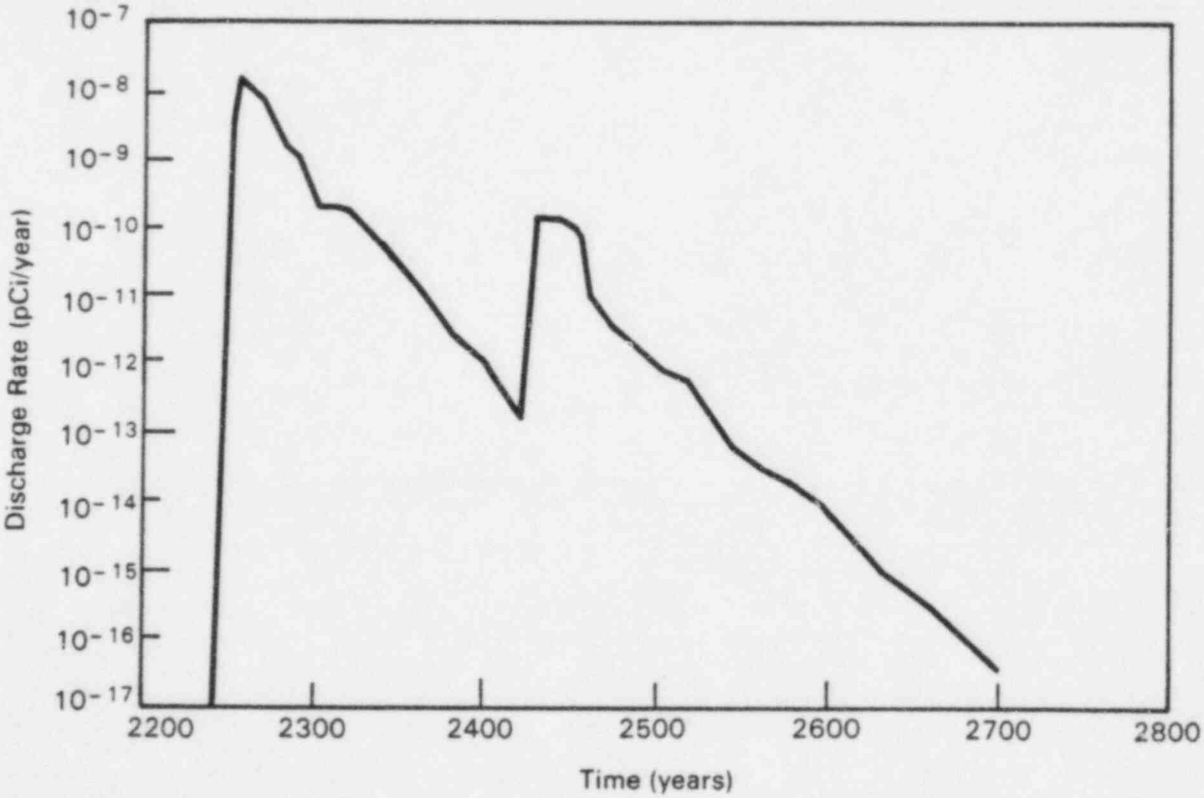

FIGURE 8.4.3-4. Contaminant Discharge for Case No. 4

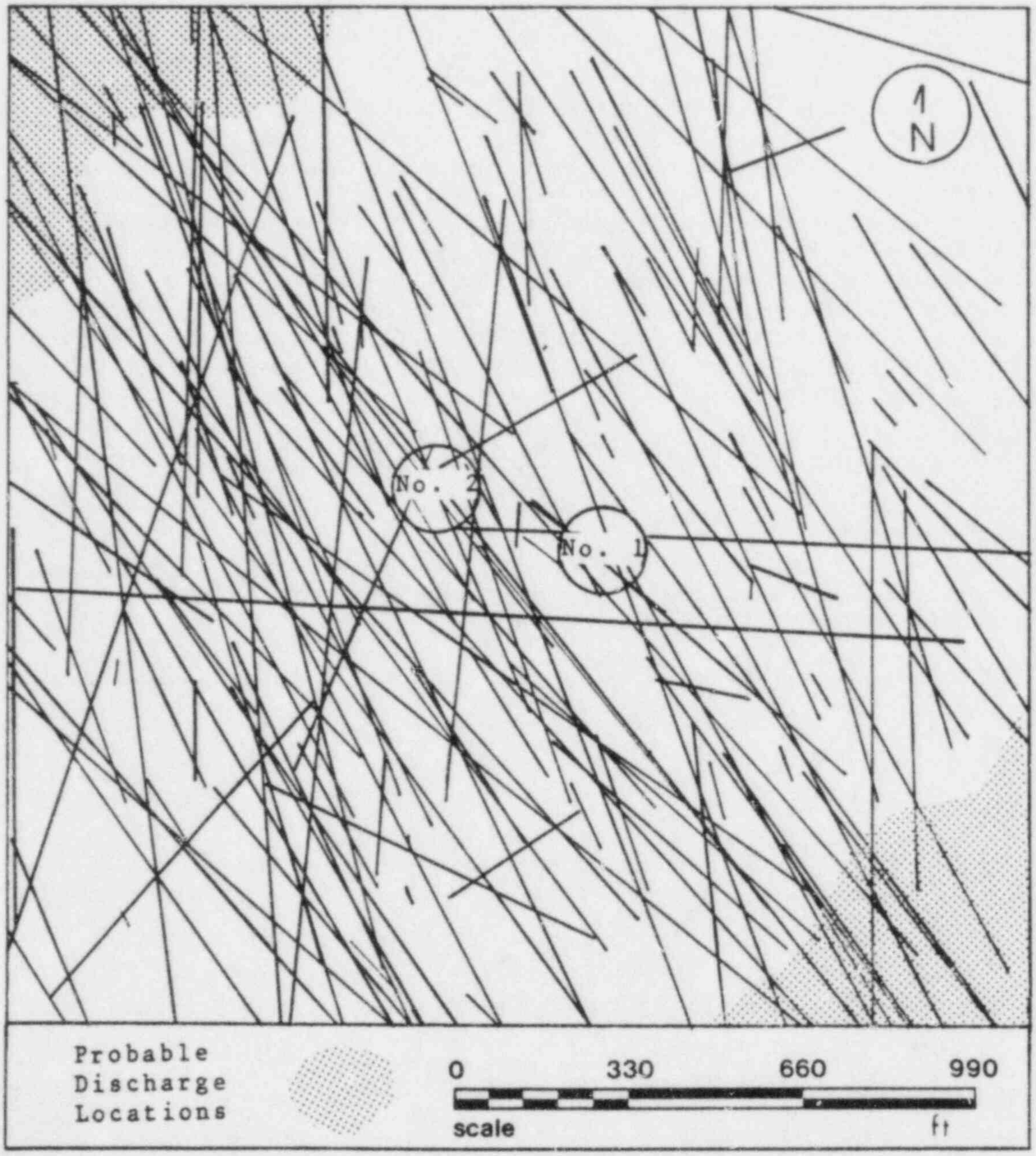

FIGURE 8.4.3-5. Stochastic Fracture Flow Network for Upper Hydrologioc Unit

flow of ground water is in a system of interconnected fractures individually of limited extent. The fractures not along the primary orientation provide key cross connections between subparallel fractures. Contaminant source at the reactor containments and the probable discharge locations are also illustrated in Figure 8.4.3-5.

Anisotropy

The degree of anisotropy of the flow system is determined by applying a hydraulic gradient at various angles to the fracture network. Figure 8.4.3-6 presents the relative permeability of the upper unit to various orientations of stress. The permeability plot follows the same preferential orientation as the fracture pattern. The interconnection of fractures at various angles produces a composite permeability that is more regular with orientation than fracture orientations. The lack of a strong secondary fracture orientation makes the upper hydraulic unit more anisotropic than the lower unit. The upper hydrologic unit has a maximum anisothopy ratio of 48:1 along an orientation of N30E.

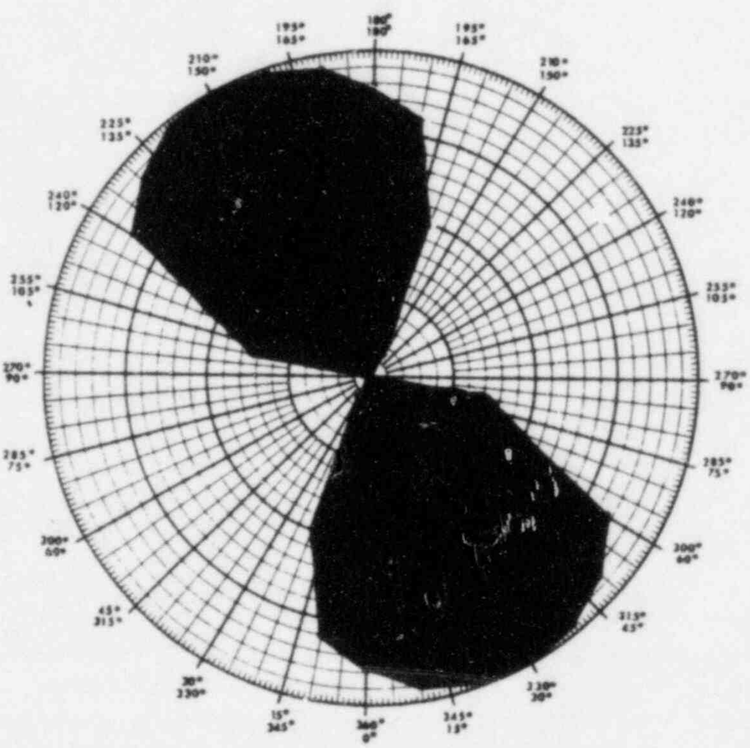

FIGURE 8.4.3-6. Anisotropy of Upper Hyrdologic Unit (Permeability/Highest Permeability)

Contaminant Discharge for Sump Water Flowing to the East, Case No. 1

The contaminant breakthrough curve for the upper unit is presented in Figure 8.4.3-7 and demonstrates the characteristics of a fractured unit. In this realization of the flow field, the strontium-90 activity flux drops to

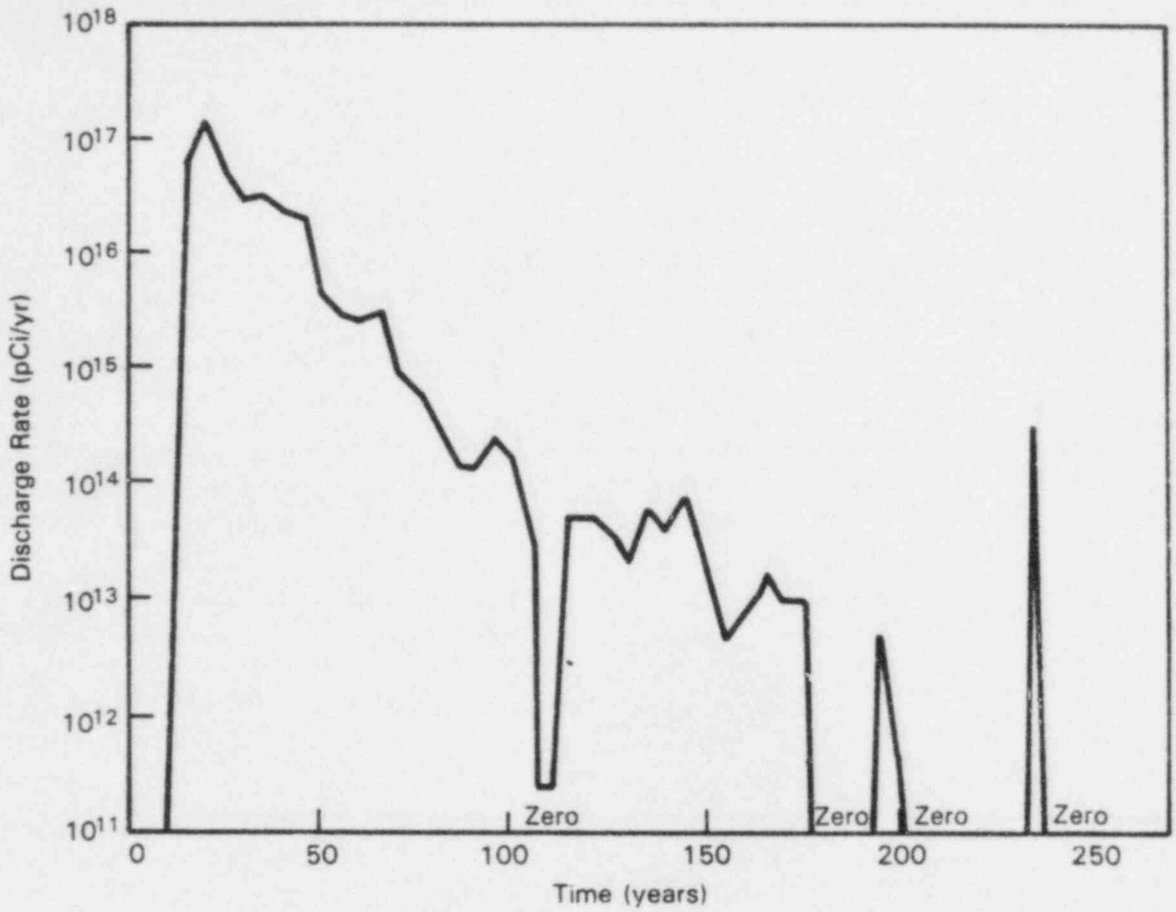

FIGURE 8.4.3-7. Contaminant Discharge for Case No. 1

zero three times during the discharge period. The first arrival of strontium-90 at the surface would be 9 years after the accident. The peak strontium flux would be 2.2 x 10^{17} picocuries/year at 12 years. The figure indicates an irregular trend toward lower activity fluxes. The last of the contaminant would be discharged at 235 years following a short-term activity spike. The contaminant dropouts (e.g., contaminant discharges going to zero or near zero) is representive of probable events in this fractured flow system. The release rate of contaminants into the geologic units beneath the reactor is the same for cases No. 1 and No. 2 and occurs over a 5 year period.

Contaminant Discharge for Sump Water Flowing to the West, Case No. 2

The contaminant discharge of strontium-90 to the west in the upper unit is presented in Figure 8.4.3-8. The first arrival of contaminant is at 13 years and reaches a peak activity flux of 1.9 x 10^{17} picocuries/yr at 14 years. The shape of the break through curve follows a generally decreasing trend over a period of 275 years. There is a sharp decrease in the activity discharging to the surface environment which reverses at 65 years and, reaches a rate one order of magnitude less than the peak flux. This is likely caused by a highly

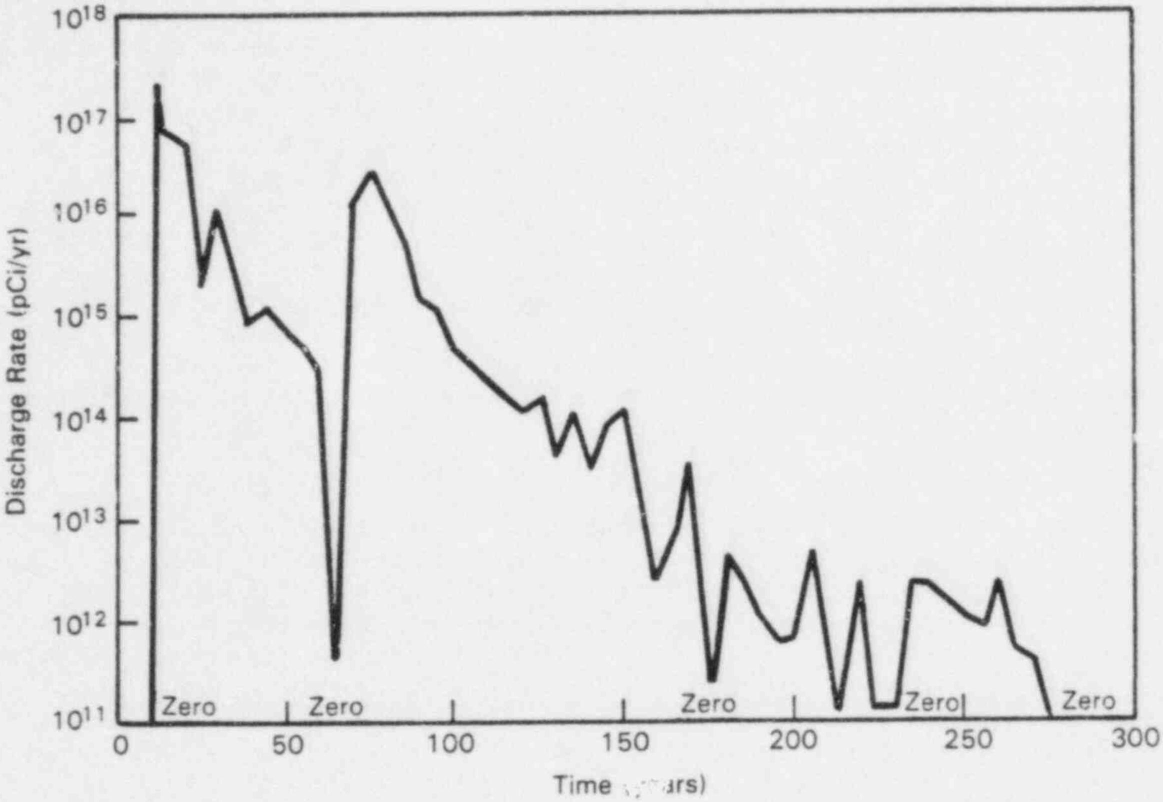

FIGURE 8.4.3-8. Contaminant Discharge for Case No. 2

permeable flow channel being swept clean and then receiving additional contaminant from a low permeable fracture. If diffusion of contaminant into the rock matrix had been considered in this case study the contaminant dropouts would not go to zero. The slope of the activity decrease is less following the first drop out, indicating that the contaminant in the low permeability channel continues to slowly enter the system.

8.4.4 Comparison of Contaminant Discharges

8.4.4.1 Summary of all Cases

A summary of the four cases is presented in Table 8.4.4-1.

TABLE 8.4.4-1. Summary of Premitigative Transport Results

Peak Flux Case No.	First (pCi/yr)	Last Arrival (yr)	Arrival (yr)
1	2.2×10^{17}	9	235
2	1.9×10^{17}	13	275
3	1.5×10^{8}	705	1150
4	1.4×10^{-6}	2245	5000

8.4.4.2 Upper and Lower Hydrologic Units

The basis of this comparison are Cases No. 1 and No. 3 where sump water migrates to the east. The slower rate of contaminant transport in the lower hydrologic unit is clearly reflected in the resulting breakthrough curves in Figures 8.4.3-3 and 8.4.3-7. Strontium-90 in the lower unit requires 700 years longer for the first contaminant to be discharged than in the upper unit. The longer transport time allows for radioactive decay to reduce the peak radionuclide flux in the lower unit by 9 orders of magnitude as compared to the upper unit. The much slower hydraulic release rate into the lower unit is also in part responsible for the lower level of radionuclide discharge. These results are not unexpected given the characteristics of the site. The upper unit has the capacity to produce the greater environmental risk, but the lower unit is more likely to receive the sump water and/or core debris contaminant.

8.4.4.3 Easterly and Westerly Flow Directions

The comparison of environmental consequences of contaminant flowing to the east and west in the upper unit is made through examination of Cases No. 1 and No. 2. In both cases a sump water release is assumed to occur in the upper hydrologic unit. Figures 8.4.3-7 and 8.4.3-8 present the eastern and western contaminant discharges respectively. The somewhat shorter flow path and higher hydraulic gradient to the east forms a slightly more severe radionuclide release. Contaminant arrives at the eastern discharge location about 4 years before contaminant reaches the western area. Peak radionuclide discharge rates are slightly less at 1.9×10^{17} pCi/yr for the western flow direction as compared to 2.2×10^{17} pCi/yr for eastern flow.

The shape and magnitude of the breakthrough curves are similar with the western contaminant discharge continuing for an additional 40 years. There are no significant differences in contaminant flowing east or west in the upper unit. Both flow directions are capable of producing activity dropouts, reversals, and spikes. Monitoring in the upper unit would yield noisy data and short-term trends not representative of the system as a whole. The small-scale differences between the two flow directions would not affect the design requirements for a mitigative system. In the upper unit only, changes in contaminant interdiction for radionuclides migrating east or west would be due to plant configuration and topography. Much of the difference in magnitude of the discharges to the environment is due to a longer flow path and lower hydraulic gradient to the west. These factors result in a travel time about three times longer to the west which lowers the flux rate by decay 17 orders of magnitude.

8.4.4.4 Core Melt Leachate and Sump Water

Core melt leachate and sump water releases are compared by examining Cases No. 3 and No. 4. The release rates of radionuclides in sump water and core debris can be compared in Figures 8.2.2-2 and 8.2.2-4. In the lower hydrologic unit, the sump water release rate is about one half an order of magnitude less than the core debris leach rate. The shape of the breakthrough curves in

Figures 8.4.3-3 and 8.4.3-4 demonstrates that the core debris release drops off more rapidly but remains a source of contaminant over a longer period of time.

8.4.4.5 Anisotropic Stochastic Discrete Fracture and Isotropic Homogeneous Equivalent Porous Media Conditions

An example calculation of a sump water release and transport in a isotropic homogeneous flow field is presented in Figure 8.4.4-1. The purpose of this example is to demonstrate the key features of a fractured flow system

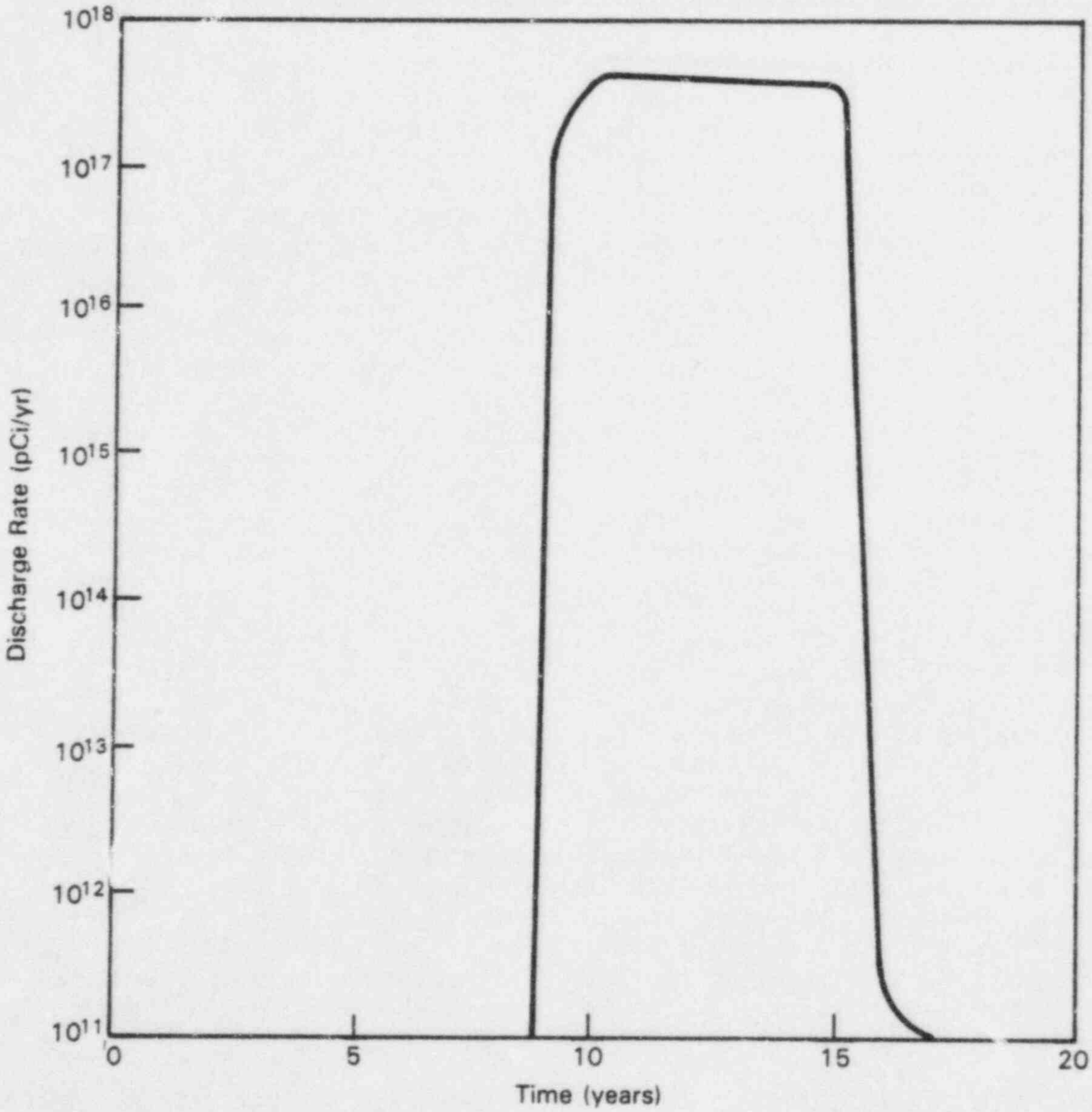

FIGURE 8.4.4-1. Example of Radionuclide Discharge from a Isotropic Homogeneous Hydraulic Unit

that are not seen in idealized isotropic homogeneous simulations. This example specifically addresses the type of bias that is introduced when an anisotropic fractured hydraulic unit is considered to be an equivalent porous media.

The example case is for sump water in the upper unit flowing to the east. The formation porosity was assumed to be as 0.001 and the transmissivity as 136 ft^2/yr determined at the 50-percent level of the cumulative probability density distribution. The hydraulic gradient is determined from water level data and stratigraphic elevation of the hydrologic units at the discharge areas. The isotropic model allows the water injected at the source to hydraulically spread in the flow unit (Oberlander and Nelson 1984).

The equivalent porous media example case is contrasted to the fracture flow calculation given in Figure 8.4.3-7. The time of first contaminant arrivals are similar for both simulations. The porous media case yields a first arrival of 8 years as compared to 9 years for a fractured unit. Although the fractured unit contains flow paths that have permeabilities two orders of magnitude higher than the mean value, these high permeability flow paths were limited in their capability to transport radionuclides by being connected to the less permeable fractures. As seen in the fracture network figures, no single fracture extends from the source to the discharge area. Therefore, at the transport distances of 800 ft and greater, this system functions as a composite of average conditions with respect to first arrival times. At lesser distances, particulary the short distances at which a mitigative barrier may be constructed, this correspondence may not exist. Certainly some of the fractures encountered in interdictive construction could be carrying contaminants at higher than average velocities. The similarity in first contaminant arrival times for the fracture model and the isotropic example demonstrates: 1) scaling of equivalent aperture by relating volumetric flux in porous media and flow in an individual fractures is appropriate for this flow system and, 2) the effective porosity estimated by hydrologic judgment yields reasonable results.

The peak strontium-90 flux to the surface environment is higher in the equivalent porous media case at 2.8×10^{17} pCi/yr as compared to 2.2×10^{17} pCi/yr for the fractured case. The porous media breakthrough curve also demonstrates a much shorter period of contaminant discharge of 8 years in contrast to 226 years in the fractured simulation. Both of these characteristic differences can be explained by the presence of low permeability fractures in the discrete analysis. In the fractured unit some of the contamination is held in low velocity flow paths and is not released at early times. This effectively reduces the peak discharge flux as compared to the homogeneous example. With time, the contaminant in the low velocity zones is released to more rapid flow paths and is discharged to the surface environment. The time required for a quantity of strontium-90 to be passed through the ground-water flow system is, therefore, significantly longer when the low permeability and low velocity fractures are considered.

In summary five statements can be made concerning a comparison of discrete fracture and equivalent porous media simulations:

1. With respect to first contaminant arrival times, at a distance of 800 ft or more, the upper unit of the Marble Hill site behaves similarly to equivalent porous media.

2. Low permeability-low velocity fractures retard a portion of the contaminant producing a slightly lower peak contaminant discharge flux.

3. The importance of high velocity flow channels is not observed at distances of 800 ft in this system, but could be an important consideration at the distance where mitigative construction takes place (e.g., 200 ft or less).

4. The time period over which contaminants would be discharged to the surface environment is underestimated by the homogeneous model compared to results from discrete fracture modeling. The design and longevity of the mitigative system may be inadequate unless the longer transport times for the slow moving portion of the contaminant is considered.

5. Hydraulic test interpretations based on porous media theory may be improper unless a large volume of rock is stressed representing an effectively homogeneous control volume. At this site the control volume appears to be less than 800 ft in diameter.

8.4.5 Necessity of Mitigation

The various contaminant discharge scenarios considered produce a wide range of results. Clearly, the severity of a radionuclide release and the decision to mitigate the environmental consequences must be based on what constitutes an acceptible level of contaminant discharge. This study will define that discharge rate as any amount that would result in the Ohio River having concentrations of strontium-90 above the 10 CFR Part 20 limit of 300 picocuries/l. This methodology is given not as an absolute technique for determining the total biological hazard. Rather, it is intended to provide a somewhat realistic and relative guide to the biological hazard posed by the contaminant pathways at this site. The contaminant is conservatively assumed to reach the Ohio River when it discharges from the consolidated limestone and dolomite units. The flow of the Ohio River is assumed to be at the mean average rate of 116,000 cubic feet per second (1.367×10^{14} l/yr). Low flow rates in the Ohio River can be ten times less than the average flow. Given the average dilution factor of the Ohio River, and the maximum allowable concentration for strontium-90, yields a maximum strontuim discharge rate of 3.11×10^{16} picocuries/year.

The calculated contaminant concentrations for each of the release scenarios is given in Figure 8.4.5-1. The most severe discharges are found in Case No. 1 and No. 2 at about 1000 pCi/l. Both of these case are sump water releases in the upper unit. The peak concentrations of strontium-90 for sump water and core debris in the lower unit are 1.45×10^{-6} and 1.35×10^{-22} pCi/l, respectively. Based on this analysis, mitigative actions were deemed necessary only for the contaminant in the upper hydrologic unit.

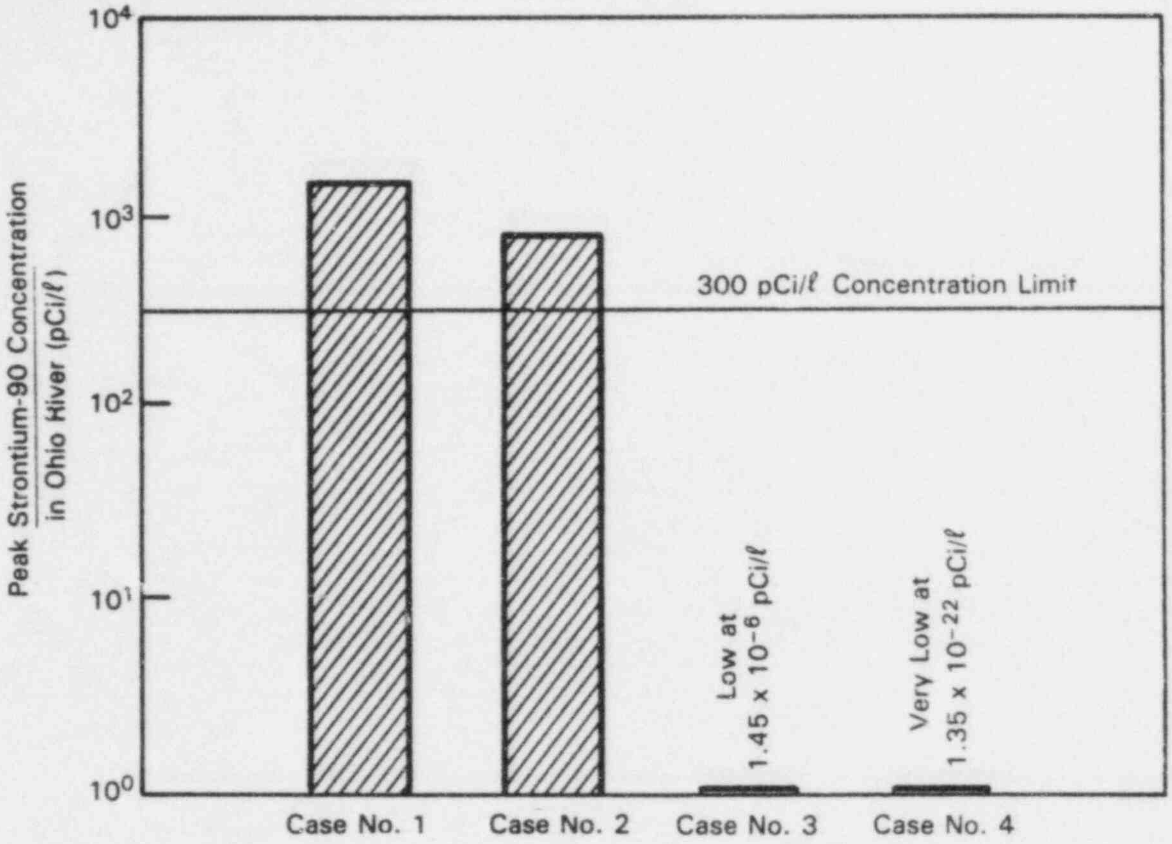

FIGURE 8.4.5-1. Comparison of Relative Environmental Severity for Contaminant Pathway Scenarios

8.5 MITIGATIVE TECHNIQUES

8.5.1 Selected Methods

The feasible mitigative techniques for fractured limestone and dolomite as determined from the generic anlaysis are listed in Section 8.1 of this case study. Through an examination of the effects of plant siting and the hydrologic characterization, three major additional factors are identified that enter into the selection process.

First, the distance to the accessible environment is not lengthy as in the case of the South Texas Plant. The plant configuration and the site's short distance to discharge areas do not allow construction of an extensive system of multiple mitigative barriers and monitoring installations down the contaminant pathway. For example, there may not be sufficient space for a system comprized of encircling grout barriers with interpositional injection wells followed by dewatering and monitoring wells. Mitigative construction lacks both implementation space and time for plume control for the interdictive technique to built piecewise, as deemed necessary, by monitoring and evaluation. In other words, the first mitigative scheme implemented must be designed to effectively control

the plume or the limited space available for construction will be wasted. Secondary and tertiary barriers and monitoring nets would be constrained by space limitations and become more costly as building removal and extensive site preparation is required.

Second, grout and/or hydrodynamic barriers used to redirect or channelize contaminant to flow along specific pathways would not change the ultimate receptor of the contaminant or significantly increase the contaminant flow path length. Contaminant diverted from the existing hydraulic pathway by a barrier would still eventually discharge to the environment along the topographic bluffs and into the Ohio River. Contaminant diversion through construction of a nominal barrier would not lengthen the contaminant travel distance by more than a factor of ten. Major changes in contaminant pathway to alternate hydrologic systems (e.g., force contaminant into another hydrologic unit) or alternate pathways (e.g., a pathway that would discharge contaminant at a great distance from the plant) to produce extremely long travel times are not feasible at this site. Removal of contaminated water from the upper unit and injecting it into the lower unit would be hampered by the lower transmissivity of the lower unit. High injection pressures used to force water into the lower unit at useful flow rates would locally create an upward vertical gradient, increase contaminant transport rates by increasing horizontal components of hydraulic gradient and possibly hydrofracture the lower unit.

The Marble Hill fracture network already channelizes the contaminant along a prefered orientation. Solutioning along fractures has, to some extent enhanced the prefered direction of flow toward the discharge areas. To attempt, alteration of the basic discharge area(s) at the site against the direction of anisotropy would require extensive energy and/or material resources. The mitigative scheme for the Marble Hill site should be designed to work with the natural flow system rather than against it. The natural channelization of ground-water flow into prefered orientations allows mitigative and monitoring methods to be concentrated in a selected portions of the hydrologic unit.

Third, the highly variable permeability of the fractured rock would require highly location-specific construction of any barrier designed to isolate the plume. For example, in constructing of static barriers (i.e., cement or chemical grout) the injection pressure must be controlled to prevent excessive hydrofracturing of the rock. Fracturing with grout material would increase the "grout take" at a bore but could also fracture adjacent ungrouted areas not directly connected to the bore. For example, injection pressures that resulted in ground uplift could produce radial and annular fractures beyond the grouted zone and vertical fractures through important confining layers.

The fractures at the site are not evenly distributed in space and the ability to seal fractures around grout injection bores would depend on fracture orientation and degree of interconnection. In areas where there are few or no fractures, the grout may not penetrate far from the injection bore. A second or third line of injection bores would be necessary to form a grout wall of substantial thickness and durability. Bores may have to be spaced at intervals

as small as 1 to 3 ft in the lateral and longitudinal directions to ensure that adjacent fractures are grouted. A static or grout barrier can function as a mitigative technique only in the fractures containing grout.

Dynamic barriers, such as injection and withdrawal systems, can generally function more effectively than static barriers for two reasons: 1) hydraulic fracturing and/or fracture expansion by overpressurization would enhance the effectiveness of a dynamic injection barrier, and 2) the response of the system can be altered by changing pumping rates or changing the purpose of the bore between injection and withdrawal. Dynamic systems could also be hydrofractured to open fractures and increase fracture interconnection prior to pumping. A withdrawal system used in conjunction with a grout or injection barrier would have to be located a sufficient distance upgradient to avoid drilling into sealed or pressurized fractures.

In comparison of static and dynamic systems, the advantages of a dynamic system are:

- flexible response to changing ground-water conditions,
- hydraulic stress can alternately be positive or negative,
- contaminant can be captured and removed from the ground water, and
- fractured hydrologic units would be less susceptible to clogging by fine-grained material in an injection system.

The advantages of a static system over a dynamic system are:

- much less energy extensive,
- much less maintenance extensive,
- less dependence on system parameters (i.e., pumping rates) for proper operation,
- greater reliability,
- monitoring wells could be placed close to the barrier without interfering with system function, and
- the barrier can be augmented at a later time (i.e., increased barrier thickness or length) without large space requirements or interference with system function.

An example of the last item in the list of advantages of a static system would be the potential problems would be augmenting a dynamic system while it is in operation. Attempting to construct a grout barrier or a secondary line of mitigation wells while the primary system maintained integrity would require careful design and construction. Sufficient distance between systems would have to be maintained to prevent drilling problems or system degradation

through loss of: drilling fluids, drill cuttings, and over/under-pressurization at the drill site and mitigative location.

Uncertainty exists in the design and construction of any spatially dependent system in a highly variable environment. Neither static or dynamic systems can positively stop contaminant migration at all locations in a fractured media. Both types of systems would need monitoring conformation of successful mitigation. However, a monitoring system would also be subject to uncertainty because of the irregular concentrations of contaminant with time and the spatial dependence of fracture interception.

The mitigative system for this case study will also be selected so that surface handling of contaminated water is unnecessary. In general, barriers to contaminant migration can be hydraulically enhanced by withdrawing a portion of the plume. This method introduces the hazards of surface contact, such as operation of contaminated equipment, collection and concentration of radionuclides, and ultimate disposal. A radionuclide collection system would need to demonstrate a positive cost-benefit ratio or be supported by policy before justifing the added exposure risks at the surface of the site.

Static and dynamic systems are feasible at the Marble Hill site and possibly either system or a combination of the two systems could be sucessfully implemented. The mitigative technique selected as a first measure for this site is grout injection to form a static barrier to ground-water flow and radionuclide transport.

8.5.2 Design of Grout Barrier

8.5.2.1 Design Objectives and Constraints

A mitigative scheme is devised based on four assumptions: 1) the eastern reactor has suffered a severe accident, 2) the upper hydrologic unit is contaminated by a sump water release, 3) site knowledge is limited to the FSAR (1982) (i.e., no post-accident analysis has been conducted), and 4) mitigation is desired before the contaminant leaves the carbonate unit and enters the glacio-fluvial sediments. The design for this mitigative system should be considered as a conceptual rendering of an actual system. It is neither the intent or purpose of this case study to present finialized or optimal designs in anticipation of the start of construction.

8.5.2.2 Placement of Mitigative Barrier

The location of a contaminant mitigation scheme is based on the probable contaminant pathways as determined by fracture orientation and hydraulic gradient. Although the discussion of a mitigative scheme is centered around the upper unit, many of the same considerations also apply to the lower unit. For example, the orientation of contaminant migration would be similar for both the upper and lower hydrologic units. In addition, the selection of a mitigative technique would be based on the same considerations (i.e., preferential flow directions, fracture hydraulics, etc.) but each hydrologic unit would have different design constraints (i.e., transmissivity, aperture size, etc). The

upper unit could produce the most severe environmental consequences; however, the lower unit is more likely to be contaminated. If both units were contaminated, then two seperate mitigative systems could be superimposed. That is, where there were space limitations, alternating grout injection bores or withdrawal wells would intercept each hydrologic unit. Care in design and construction would be necessary to prevent cross interference of any overlapping mitigative systems.

Idealized contaminant plumes are illustrated in Figure 8.5.2-1. The predominant orientation of contaminant movement in the upper and lower units is indicated by the stippled area. The plumes are assumed to be bifurcated into eastern and western flow components at each reactor location. The eastern reactor represents contaminant migrating in the lower unit and the western reactor represents the upper unit. The longitudinal extent of the contamination is shown truncated at the position of feasible mitigative construction. The figure does not attempt to show diffusion or hydrodynamic dispersion along the limited extent of the flow path depicted.

The dark solid line around the plant areas indicates all areas where a line of mitigative construction could be conveniently placed. Mitigation closer to the plant is possible; however the costs and construction delays created by circumventing obstacales (i.e., electrical transmission towers and pipe lines) may not be worthwhile unless demonstrated as being necessary. The cooling towers north of the plant prevent drilling wells continuously around the contaminant source. The flow of contaminant along the fracture orientations in the lower hydrologic unit would tend to follow a course under the cooling towers where interdictive measures would be difficult to construct. Angle drilling at that location and other design considerations could reduce the impact of the cooling towers on mitigative effectiveness.

One additional consideration was used to determine the placement and design of the grout barrier. The purpose of the mitigative simulation was to evaluate the performance of a mitigative scheme that represents a reasonable first attempt at interdictive design. Sections 6.0 and 7.0 of this report considers mitigative designs and performance and much of that information is applicable to fractured media. Only information available in the FSAR (1982) and interpreted, as in this case study, were used in the design of a mitigative system. This simulation also assumed that no post-accident information would be collected and no pre-design modeling was conducted. The goal of the mitigative scheme was to:

- seal the fractures along the major fracture orientations that carry ground water to the reactor area,

- create a system that could be constructed in a realatively short period of time, and

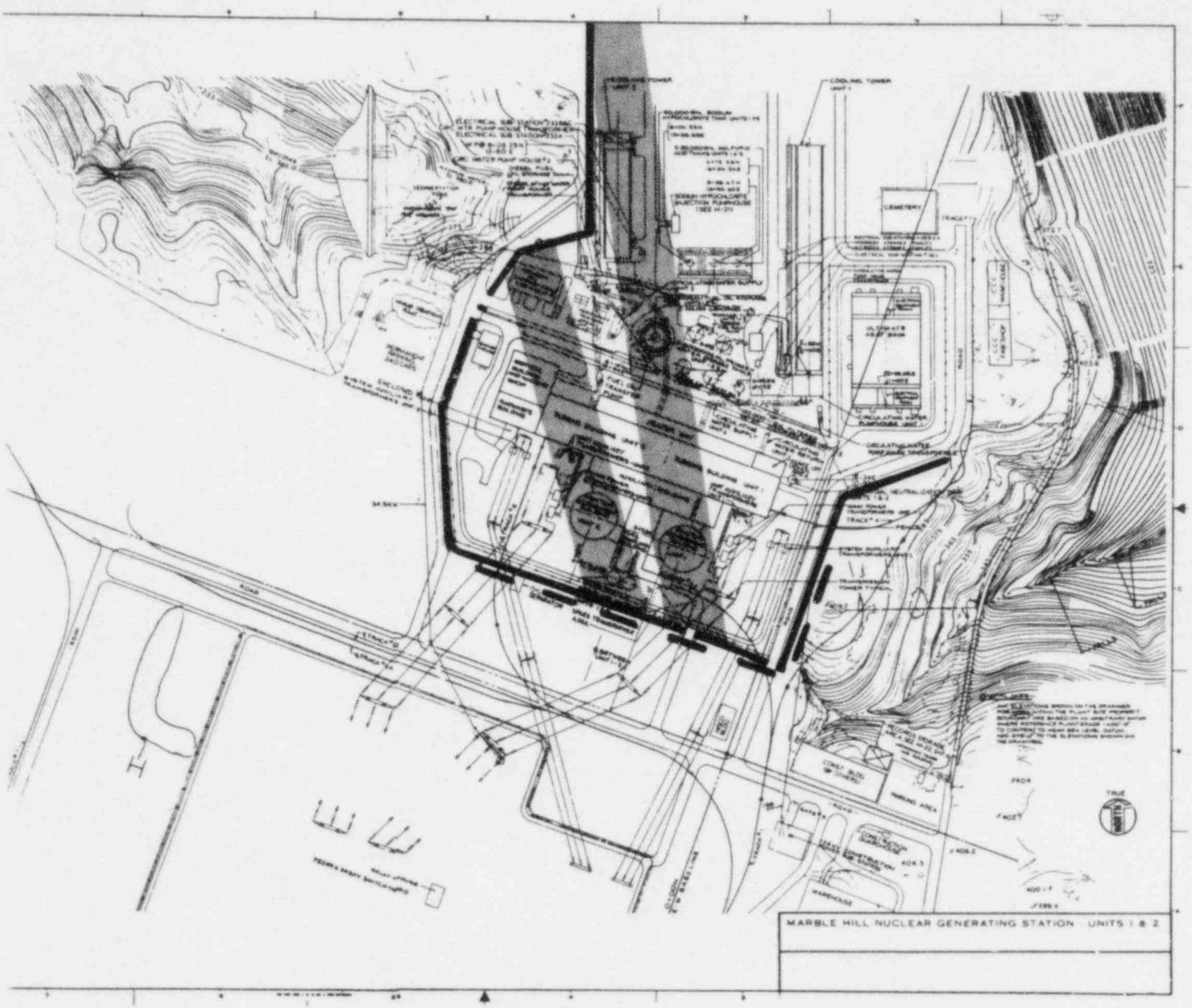

FIGURE 8.5.2-1. Location of Grout Barrier (Source: Marble Hill FSAR 1978)

- consider a somewhat limited system in arealy extent that does not attempt mitigation by construction of multible feasible techniques as space would allow (i.e., a conservative system including grouting a continuous ring around the site coupled with injection and withdrawal wells.

Based on the above assumptioms and considerations, a grout barrier as indicated by the bold dashed line in Figure 8.5.2-1 was simulated. The grout wall is clearly of limited extent. The southern limb of the grout barrier is 960 ft long and the eastern limb is 370 ft long. The grout barrier was placed down-gradient of the premitigative plume location that produced the greatest radionuclide releases (i.e., contaminant flowing to the east in the upper unit).

8.5.3 Mitigation of Contaminant Migration in Upper Unit

8.5.3.1 Modeling Technique

The mitigative system was simulated by severely reducing the aperture width of fractures at the location of mitigative implementation. This allowed an added realism to the model by assuming that not all fractures were perfectly sealed, which may be the case in actual conditions. The model was run with the same external configuration and boundary conditions as used in the premitigative simulations. Two mitigative simulations were conducted: a downgradient barrier with ground water flowing to the east, and an up gradient barrier with ground water flowing to the west. Flow through the barrier was negligible and nearly all of the ground-water flow was diverted to alternate fracture pathways around the simulated grout wall.

Graphic display of contaninant being diverted around the barrier is difficult to portray. The concentrations and quantities of radionuclides in this fracture network are very site specific depending on: 1) aperture width, 2) degree of interconnection, and 3) time. In contrast to the continuum approach for porous media, the concentrations of adjacent areas of fractured media are much more irregular. The rock matrix (i.e., the rock between fractures) is conservatively assumed to contain no contaminant which creates many large discontinuum in areal concentrations. Differences in contaminant concentration of many orders of magnitude in localized areas is an expected condition of this flow system consisting of a fractured anisotropic system with variable aperture widths. In these circumstances contaminant concentrations are not amenable to contouring as a regular and undulating surface. Although the irregular transport characteristics of a fractured system are a key factor in this analysis, the graphic portrayal of selective contaminant pathways and concentrations remains an area for further development.

The imposition of a barrier reduced the overall permeability of the site by a maximum of 15 percent. The relationship of permeability reduction with direction of flow is given in Figure 8.5.3-1. As seen in the figure, the greatest reduction in permeability is normal to the orientation of the barrier. As the direction of hydraulic stress approaches the least permeable orientation, the effect of the barrier is diminished.

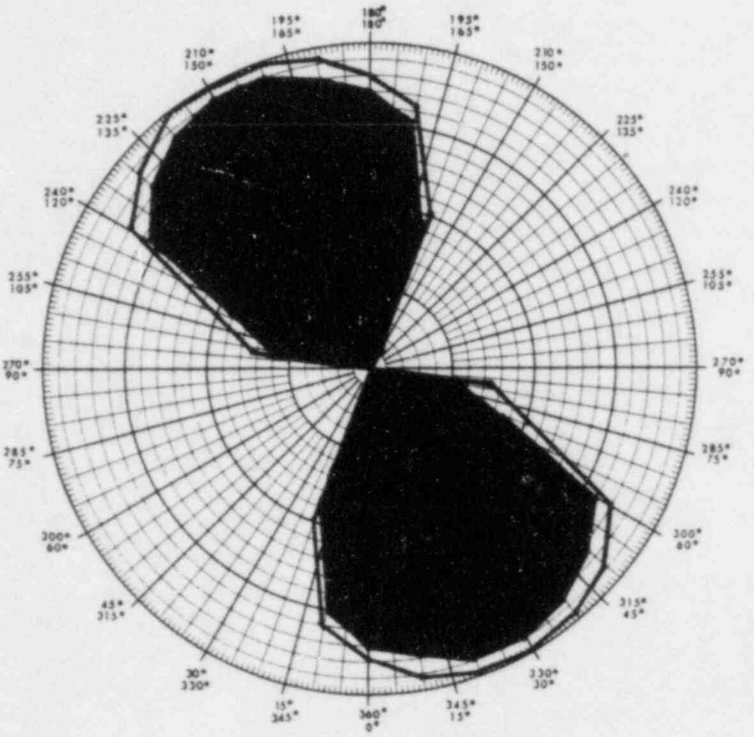

FIGURE 8.5.3-1. Permeability Reduction of Flow System by Grout Barrier

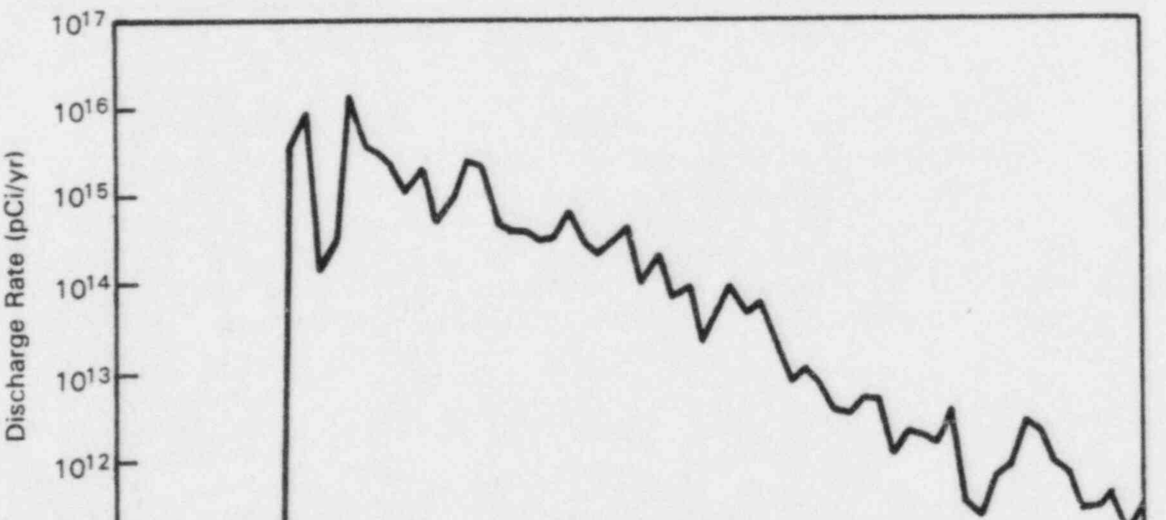

FIGURE 8.5.3-2. Mitigated Strontium-90 Discharge for Sump Water Moving to the East in Upper Hydrologic Unit

8.5.3.2 Mitigation of Sump Water Flowing East

The mitigation of contaminant moving to the east is accomplished by a down-gradient barrier at the location previously indicated in Figure 8.5.2-1. The goal of the mitigative scheme is to: 1) lengthen the contaminant pathlines around the barrier, and 2) create a zone of lower hydraulic gradient directly in front and behind the barrier. Additional time for radioactive decay is provided by the barrier and, hence, it reduces the severity of the environmental consequences. The contaminant discharge at the bluff outcropping of the Laural Formation is presented in Figure 8.5.3-2. The first arrival of strontium-90 would occur 58 years after the accident. Peak flux occurred in this simulation at 70 years after the accident. The imposition of a barrier significantly changed the characteristics of the contaminant breakthrough as compared to the unmitigated case illustrated in Figure 8.4.3-7.

Major differences in the two cases are:

- the mitigated case does not demonstrate intermittent periods of strontium-90 dropouts (i.e., when contaminant discharges go to zero),
- the first arrival of strontium-90 in the mitigated case is delayed by an additional 49 years,
- the peak strontuim-90 flux of the mitigated case is reduced by one order of magnitude and,
- the total period of contaminant release is increased from 235 to 1150 years.

The down gradient barrier is demonstrated to delay radionuclide migration. The barrier also causes a greater degree of contaminant mixing. As the flow field diverges in front of the barrier and converges behind the barrier additional lateral flow is created. The added lateral component to flow causes the contaminant to experience more fracture intersections and a blending of waters takes place. The more regular outflow of contaminant with time is a result of this process and is observed in comparison of Figures 8.4.3-7 and 8.5.3-2.

8.5.3.3 Mitigation of Sump Water Flowing West

Mitigation of contaminant migrating to the west is simulated by reversing the hydraulic gradient and allowing the barrier illustrated in Figure 8.5.2-1 to represent an upgradient scheme. In this case the goal is to produce an area of low hydraulic gradient in the reactor area that will slow water and contaminant movement. Not all ground water in the flow system is slowed by the barrier. The water that would normally flow through that area now occupied by the barrier must pass along the outer edges of the barrier. This effect increases the flow velocities in areas away from the contaminant in order to reduce them at the contaminated location. The distance of contaminant travel is not significantly changed by this method. The discharge area at the bluff outcroppings is the line of evaluation for this mitigative simulation. The

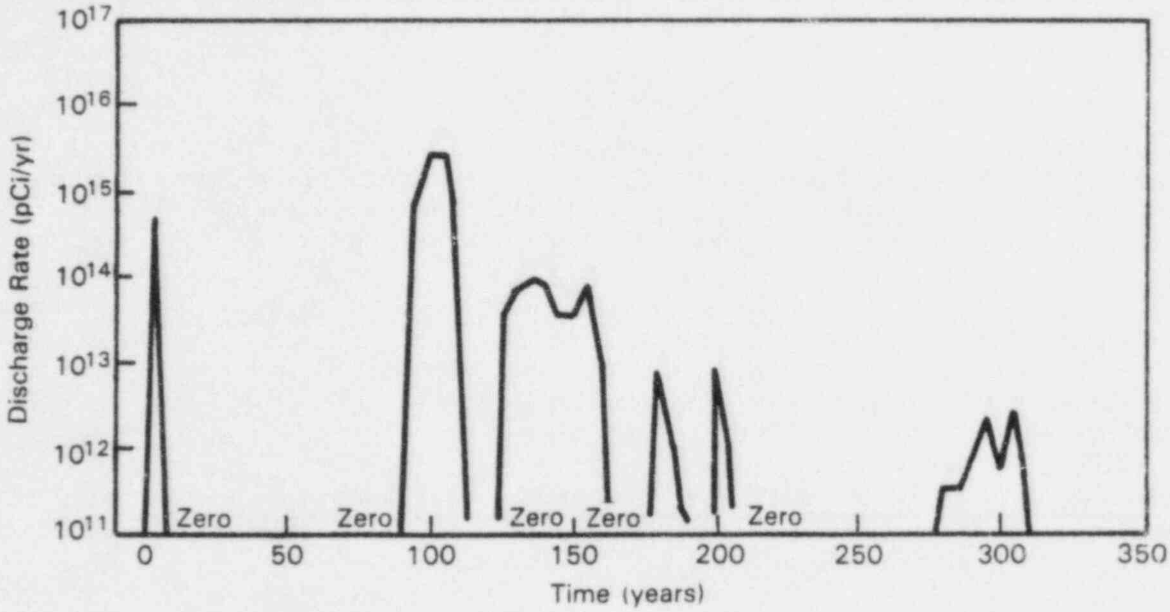

FIGURE 8.5.3-3. Mitigated Strontium-90 Discharge for Sump Water Moving to the West in the Upper Hydrologic Unit.

flux of strontium-90 leaving the limestone unit is presented in Figure 8.5.3-3. The mitigated contaminant flux is summarily compared to the unmitigated case as:

- a reduction of peak flux of about two orders of magnitude from 1.9 x 10^{17} to 3.4 x 10^{17},
- a much more sporadic contaminant discharge history with long intervals of contaminant dropout when no strontium-90 leaves the ground-water system,
- a slight decrease in first arrival time for a very small quantity of strontium-90,
- a increase in arrival time of about 80 years for the major portion of the strontium-90, and
- an increase of the total period of strontium-90 from 275 to 1790 years.

The increase of first arrival time for a minor amount of contaminant is due to some strontium-90 migrating into the higher velocity zone surrounding the down gradient stagnation area. In this realization of the flow field at least one contaminant pathway along a fracture is not effectively interdicted. The low quantity of contaminant and short travel suggest that a small aperture fracture with a high ground-water velocity is responsible for the early arrival. This effect is possible when the mitigatige technique creates an

increase in hydraulic gradient and forces small aperture fractures to carry more of the ground-water flow (Roberts 1984).

8.5.4 Effectiveness of Mitigative Systems

The necessity of mitigation in each hydrologic unit was identified in Section 8.4.5. The effectiveness of how well the mitigative systems reduced the environmental consequences is evaluated with the same methodology (i.e., estimated strontium-90 concentrations in the Ohio River).

8.5.4.1 Mitigation of Strontium-90 Flowing East in Upper Hydrologic Unit

The concentration of strontium-90 in the river is plotted in Figure 8.5.4-1 for the mitigated and premitigated flow of strontium-90 to the eastern discharge location. The unmittigated case results in concentrations above 10 CFR Part 20 limits for about a 20-year period. Mitigation delays the contaminant arrival by 49 years and reduces the concentration at in the river to about one third of the maximum permissible level. No mitigated strontium-90 discharges exceeded the 300 pCi/l level at any time.

8.5.4.2 Mitigation of Strontium-90 Flowing West in Upper Hydrologic Unit

Comparison of mitigated and unmitigated strontium-90 transport in presented in Figure 8.5.4-2. The unmitigated discharge exceeds 300 pCi/l for about a 10 year period beginning 12 years after the accident. The insertion of an upgradient barrier reduced the concentration to levels less than 10 CFR Part 20 limits for the entire discharge period. The peak mitigated strontium-90 flux at 105 years would reach about one tenth of the 300 pCi/l level. Later discharges occurring over about 1800 years would be well below this level.

8.5.4.3 Summary of Mitigative Design Effectiveness

The grout barrier, despite its limited design basis, is judged as being successful in mitigating environmental consequences to the Ohio River. In actuality, an interdictive scheme would be expected to provide, where possible, more protection than the amount that is illustrated in this example. Overdesign of a interdiction scheme would be desirable to compensate for uncertainty in transport and mitigative estimations or may be a policy goal of the mitigative system. Further development of the grout barrier would provide even fewer radionuclides discharging into the environment. Options of the designer include:

- adding length to the sides of the barrier,
- adding pressure relief wells on the upgradient side of the upgradient barrier,

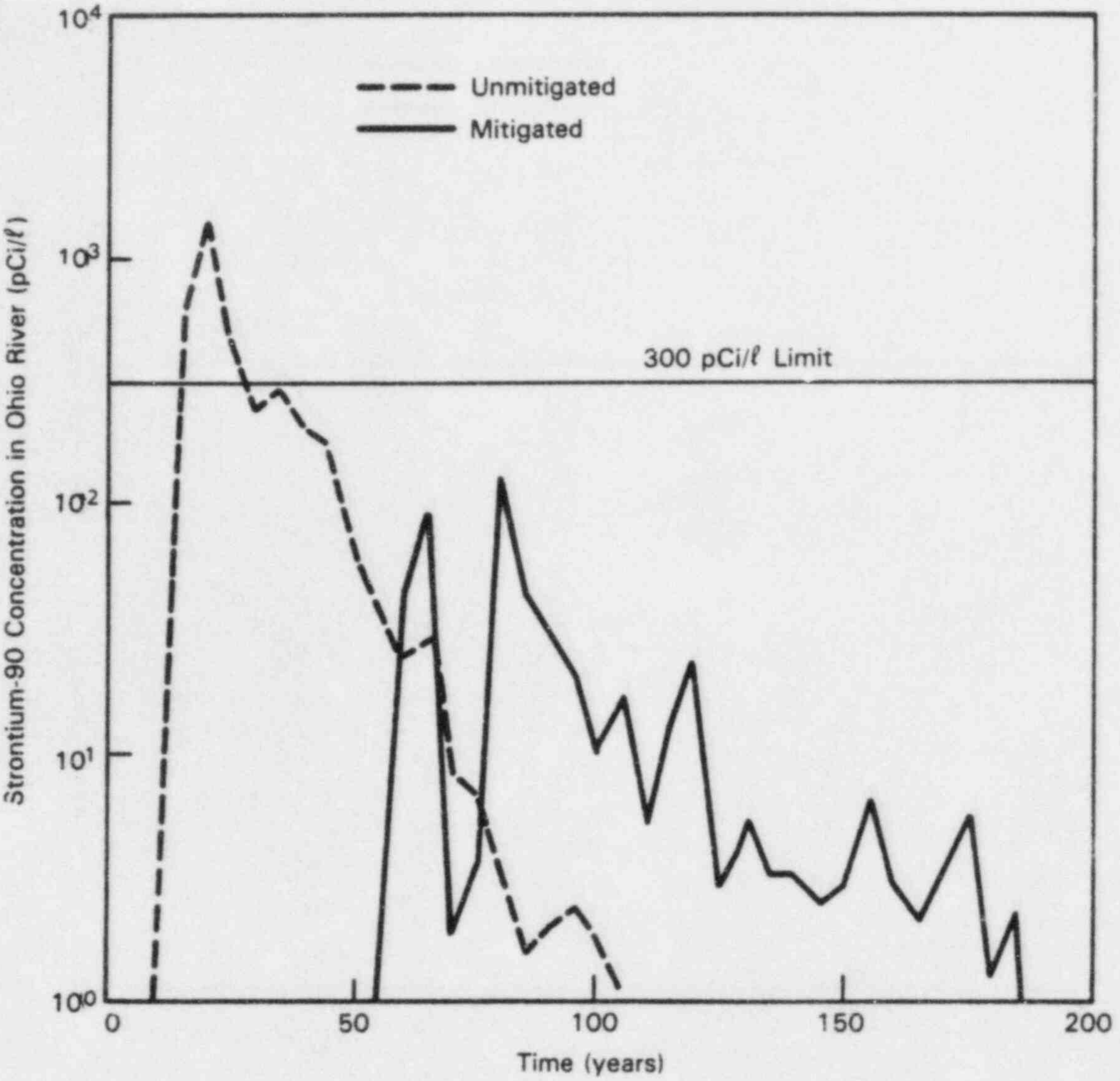

FIGURE 8.5.4-1. Premitigative and Mitigative Strontium-90 Concentrations in Ohio River for Ground-Water Flow to the East

- adding withdrawal wells in the plume, or
- surrounding the entire plant area with a grout barrier and/or installing withdrawal/dewatering wells.

Additional designs and configurations were not modeled because this topic is discussed at length in Section 7.0.

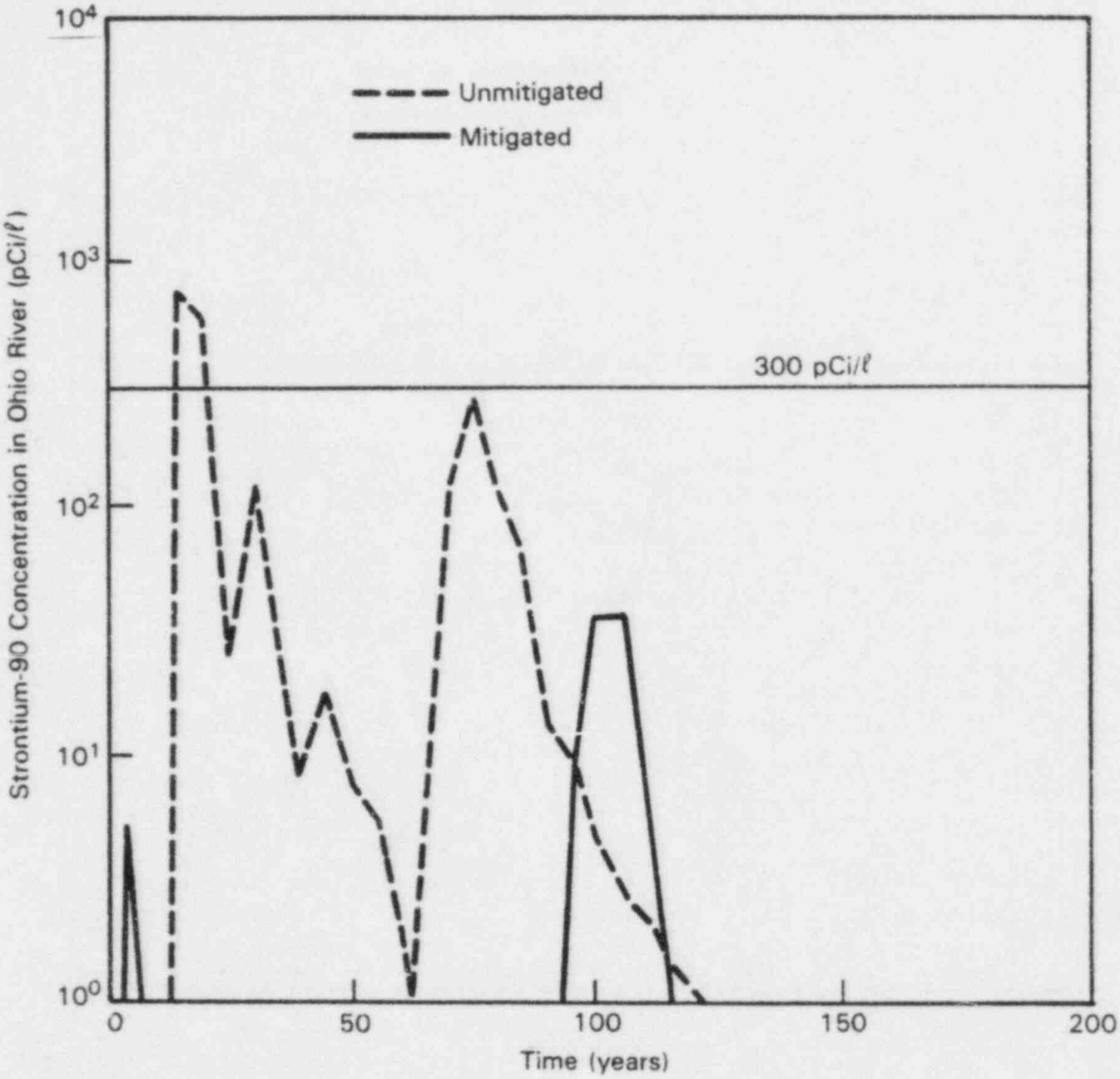

FIGURE 8.5.4-2. Premitigative and Mitigative Strontium-90 Concentrations in Ohio River for Ground-Water Flow to the West

8.5.5 Nonintrusive Collection

Mitigation outside the carbonate units is also possible. The delay period for contaminant to reach the discharge location in the upper unit would allow planning and construction before contaminant outflow. Interdiction at these location(s) would be less construction intensive. This interdictive concept is not advanced as a prefered method but rather as an alternate to the scheme previously discussed.

The contaminant in either the Saluda Formation or the Laurel Member of the Salomonie Dolomite would be prevented from downward migration by numerous shale

layers and beds. Areal spreading of the contaminant would be limited to the zones of fracture permeability. With time (tens of years for the upper unit and hundreds to thousands of years for the lower unit), the contaminant would exit either of the carbonate units along surface water drainages and enter the unconsolidated glacio-fluvial materials. The ground-water flow system at the Marble Hill site forms a natural collection system along the river and stream bluffs. The discharge locations from the carbonate stratum forms an ideal place for contaminant monitoring and interdiction.

Removal of radionuclides could be accomplished by gravity drainage collection galleries along the outcroppings of the contaminated unit(s). This system would intercept radionuclides before they enter the glacio-fluvial materials and subsequently the Ohio River. The contaminants could then be either relocated to an approved offsite repository or injected into a deep geologic zone. The injection system could possibly be passive in that the topographic elevation would be used to supply the energy to force the contaminants into a low permeability zone at great depth. The advantages of the scheme are:

- contaminant would not be wic ly dispersed at the interdiction location,
- decay during ground-water transport would lessen exposure risk to workers,
- extensive engineering works would not have to be constructed for contaminant collection,
- contaminant interdiction and possibly disposal would occur within the site boundary,
- the system could be designed to be passive in that safe handling of the contaminant would require little or no outside energy input, and
- the system could be maintain d and operated by a low-level technology.

The disadvantages of the scheme are:

- some contaminant may travel to the discharge location before construction works were completed,
- the system would need to be mantained for a long period of time · o 100 years, and
- extensive monitoring would be required to ensure that contaminants were not bypassing the collection system.

8.6 CONCLUSIONS TO THE MARBLE HILL CASE STUDY

8.6.1 Review of Case Study Assumptions and Limitations

This case study demonstrates some of the more important features of site characterization, code selection, and contaminant interdiction in a fractured anisotropic geologic environment. It must be realized that these results, as are all model simulations, are based on an idealized characterization of a complex real world situation. Although this case study utilizes a large amount of site data and incorporates a state-of-the-art flow model, it should be noted that simulation of flow and transport in fractured media introduces a greater level of uncertainty than comparable porous media studies. Therefore the reader is reminded:

- there is uncertainty in the application of regional fracture data (i.e., fracture lengths) to a small localized area,
- fracture characterization data and hydrologic characterization data commonly have a greater amount of spatial variability than data collected in porous media,
- the discrete approach for fracture network simulation is a developing technique that has not been substantiated by a long history of field verification as are the more common continuum models for porous media,
- the theoretical basis for flow in individual fractures is not as well understood as flow in porous media and the basic equations for flow and transport in discrete fracture networks are applied to this case study while in a state of continuing development,
- the performance of a mitigative technique in fractured media is less certain and therefore simulated in a more idealized manner than in porous media because of the greater spatial variability of hydrologic parameters and construction limitations in fractured environments.

8.6.2 Conclusions

1. Two hydrologic units are identified at the Marble Hill site that could feasibly receive and transport significant quantities of core melt contaminants. Core debris leachate would enter and contaminant the lower hydrologic unit. Sump water would enter the lower unit and possibly the upper hydrologic unit. The hydraulic conditions during a sump water release would determine which hydrologic unit(s) the contaminant would enter. Collecting this information may not be feasible during the severe accident and the contaminated unit would then have to be determined through post-accident monitoring.

2. The upper hydrologic unit is less likely to be the transport medium following a severe accident because it would lie 40 ft above the core debris. To force sump water into the upper unit, the containment structure would have to be pressurized and/or have standing water in

the reactor sump. The upper hydrologic unit has a permeability about 100 times greater than the lower unit and would transport the greater strontium-90 flux (2.2 X 10^{17} pCi/yr) to the accessible environment.

3. The lower hydrologic unit was predicated to transport radicnuclides at a slower rate than the upper unit and allow much of the contaminant to decay before reaching the surface environment. The predicted peak of strontium-90 entering the surface environment from the lower unit would be 5 X 10^{8} pCi/yr for a sump water release migrating to the east.

4. The direction of contaminant travel is uncertain at this site despite an extensive hydrologic data base. Plant location astride a groundwater divide results in uncertainty in the ultimate direction of contaminant migration. A bifurcated plume is feasible at this site in the upper and lower units with contaminant moving easterly and westerly toward outcroppings along the Ohio River.

5. Topography and plant structures limit the available construction space for a mitigative technique at this site. If cost is not a concern of the construction project, the available space for interdiction can be extended through site preparation. However, the short distance to the recieving water body and the limited space for construction suggest that the contaminant mitigation scheme be designed to accomplished by the performance objectives by the first system installed.

6. The flow system at Marble Hill, Indiana, consists of fractured limestone and dolomite. The fractures lie along preferential orientations producing an anisotropic flow field. The commonly applied continuum approach is not feasible in this fractured media because: 1) the orientation of anisotropy, and 2) the degree of anisotropy were not determined by onsite testing. The model results from the discrete approach indicates that the maximum ratio of anisotropy is 1:48 along N30E for the upper unit and 1:27 along N15E for the lower unit.

7. The permeabilities of the hydrologic units are quite variable and demonstrate little spatial correlation. The permeabilities range over 3 and 4 orders of magnitude in the lower and upper units, respectively. Average values of permeability in small scale simulations (i.e., less than 800 ft) would not serve to represent this system because the characteristics of both low and high permeability fractures are fundamental to site description. Estimates of contaminant transport using an average value would underestimate the contaminant velocity along preferential fracture pathways.

8. A stochastic representation of the flow fields based on cumulative distributions of site parameters (i.e., aperture width, fracture length and fracture orientation) can preserve the variability of

permeability and anisotropy in the system and demonstrate the key factors of transport in fractured hydraulic units.

9. Low permeability fractures comprised of small apertures are of great importance to overall system function for two reasons. First, the small aperture fractures are an intergral part of the fracture system interconnection. These fractures can provide critical interconnections among the larger fractures. When small apertures are the only interconnections among the larger aperture fractures, they form impediments to flow and transport. This effect is observed in the comparison of fractured versus equivalent porous media first arrival times. In composite, the upper fractured system has first contaminant arrival times similar to equivalent porous media. This indicates that although some large aperture fractures have high ground-water velocities, the interconnection of large fractures to small fractures creates the primal flow pathways with average velocities approximately that of an porous media equivalent. This situation was most evident in the upper unit where estimates of effective porosity required for a porous media calculation were considered to be the most accurate. Second, restricted flow pathways and low velocity fractures delay contaminant migration and release radionuclides to higher velocity pathways over long periods of time.

10. Predicted contaminant breakthrough curves for this fractured system are characteristically different than the results from a porous media model. The major items that distinguish the fractured flow system at this site are: 1) the breakthrough curves are irregular and contain time periods when all fractures discharging to the surface are swept nearly clean of contaminants (the consideration of matrix diffusion would lessen this effect), 2) the peak flux is less than predicted by an isotropic-homogenous model because a portion of contaminant being delayed in low velocity pathways, and 3) the total period of a contaminant release to the environment is extended by the late arrival of radionuclides from low velocity pathways that require long time periods to reach the discharge location.

11. The precise location of fractures transporting contaminant to the discharge points cannot be determined by a stochastic model. Insufficient data exist for a deterministic model of each fracture at the Marble Hill site. Indeed, except for very small areas, knowlege of characteristics for each individual fracture is beyond current technology. Monitoring of the contaminant plume would be the best indicator of contaminant pathway. However, monitoring data would be subject to the same characteristics as observed in the contaminant outflow fluxes (i.e., irregular concentrations and arrival times at a single point as a function of fracture geometry).

12. The prediction of strontium-90 discharging to the surface environment indicates that contaminant interdiction in the lower unit to protect the adjacent Ohio River may not be necessary. The upper hydrologic unit is capable of transporting sump water contaminant to the

discharge area(s) at activity levels of concern. The peak strontium-90 flux discharging the upper carbonate unit was predicted to be 2.2 x 10^{17} pCi/yr for flow to the east and 1.9 x 10^{17} pCi/yr for flow to the west.

13. Mitigation at this site could be accomplished by several means. The method selected for analysis was a grout barrier to retard groundwater flow and radionuclide transport. The location and configuration of the barrier is based on what is considered as minimal post-accident characterization and design. The placement of an idealized grout barrier reduced predicted strontium-90 concentrations in the Ohio River to less than the 10 CFR Part 20 limit of 300 pCi/l. This level of mitigation is within the stated performance objective. Mitigation could possibly be improved, if desired, by extention of the grout barrier or coupling the grout barrier with other mitigative techniques (i.e., contaminant collection wells).

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9.0 LESSONS LEARNED AND SUGGESTIONS FOR FUTURE RESEARCH

9.1 INTRODUCTION

This section of the report presents the broadly based conclusions and observations of the research effort. Specifically, what are the prime findings of this study and how can our understanding of the problems resulting from a core melt accident be most effectively enhanced? The observations are intended to address key core melt issues and information needs identified by the authors in the course of of conducting this study.

9.2 LESSONS LEARNED

1. Most of the limited number of plant sites considered for case study analysis were not selected because there was insufficient hydrogeologic data to simulate ground water flow with an acceptable degree of accuracy. Following a severe accident, the need to define the transport characteristics of the ground-water pathway would be vital to an evaluation of environmental consequences and the decision to implement mitigative techniques. Sites not sufficiently characterized before the accident would need further site characterization, possibly before a determination of mitigative alternatives and design basis could be made. Hydrogeologic testing and sampling may have to be conducted at these sites under hazardous post-accident conditions and severe time constraints. Data collection such as static water levels, hydraulic stress testing, and ground-water sample collection require quiescent initial conditions to achieve representive values.

2. The source term for core debris leaching is subject to large uncertainties. The phenomenology is one where very complex and somewhat ill-defined physical mechanisms function in a multivariable stochastic environment. The chemical and mechanical processes and interactions that control radionuclide leach rates in admixtures of glass and calcine materials are not precisely defined. These uncertainties are likely to remain a part of a core melt evaluation since the core debris will be an uncontrolled mixture of various materials (i.e., silica, calcite, steel, etc.) at each location. The thermal history of the accident may also be important because fracturing and granulation of the debris affects leach rates. The evaluation of the necessity and the design basis of a mitigative scheme could be overestimated by several orders of magnitude if a typically conservative analysis were conducted. Contaminant concentrations determined by in situ measurements of the plume may be the only method to determine leach rates within an order of magnitude.

3. Sump water release rates for pressurized water reactors are also subject to large uncertainties and are strongly site and accident specific. Data gathered during the accident (i.e., containment pressure, standing water level, volume of water lost with time, and

activity of remaining water) would provide the primary information on the release rate. Collection and interpretation of this information following an accident would be the first step in evaluation of the sump water source term. If such detailed information is unavailable, monitoring of the contaminant plume could provide the best information of the contaminant release.

4. The hydrologic unit contaminated by sump water releases at some sites would be a function of the accident processes. Accident specifics such as hydraulic driving head could determine where in the stratigraphic sequence the contaminant was placed. In these cases the contaminated unit and possibly the direction of travel would be difficult to determine prior to the accident. Mitigative actions would either be delayed for post-accident characterization or proceed under the initial assumption that all feasible units were contaminated. This would complicate mitigation efforts and possibly cause cross interference between various methods in separate hydrologic units.

5. The site- and accident-specific uncertainties discussed in topics 1, 3, and 4 above could be reduced through a program of site testing, monitoring, and evaluation. Site-specific uncertainties (e.g., direction and values of hydraulic gradient, effective porosities, hydraulic conductivities, etc.) could be reduced before an accident occurred. Accident-specific questions such as which units are contaminated and what is the release rate of radionuclides must be answered through a post-accident review of: severe accident records, sampling, and monitoring. Monitoring data collection and integration into the design of mitigative schemes would be an important element of any post-accident study, as illustrated in Section 1.5 of this report. The topic of monitoring schemes and incorporation of post-accident data into mitigative designs is explicitly outside the statement of work for this study.

6. While any severe accident would represent a potential health hazard, immediate contaminant interdiction is conservativly estimated to be unnecessary at most of the sites. The generic analysis based on an equivalent porous media approach indicates that fractured sites may be twice as likely as non-fractured sites to need implementation of a mitigative scheme. The fractured media case study demonstrates that the assumption that a fractured site responds similar to porous media can overestimate the predicted radionuclide discharges.

7. Based on limited site-specific data of all sites, sufficient time exists for mitigative techniques to be implemented at plant locations before radionuclides reach the accessible environment via the ground water pathway. However, the minimum first arrival times of contaminant at surface water bodies are estimated to be on the order of months in which case site-specific factors not addressed at those locations may be of prime importance. Both passive (i.e., grout

curtain) and active (i.e., hydraulic injection) can be used either singularly or combined into composite systems to reduce contaminant migration rates.

8. Contaminant interdiction techniques and hydrogeologic characterization methods are sufficiently developed to select and design an effective barrier to imminent environmental consequences caused by ground-water contamination resulting from a core melt accident. The generic site characterization provides a screening tool for this process. Design of mitigative schemes can only be made after consideration of site-specific factors including 1) source term, 2) ground-water flow directions and rates, 3) location of recharge and discharge areas, 4) material properties of contaminated unit(s), and 5) plant configuration.

9. The design basis of a mitigative scheme would fall into one of four classifications:

   - mitigation at the greatest level achievable given the site- and accident-specific constraints,
   - mitigation to reduce the environmental consequences of surface and ground water contact to an acceptable risk level,
   - interim mitigation to isolate the contaminant from further transport pending further analysis and evaluation or,
   - interdiction to provide long-term isolation in a portion of the ground-water system.

   The decision as to which of these options to follow would be based on site-specific considerations and include governmental, scientific and public input. Mitigative measures are not the final response to a core melt accident, but rather are part of an iterative process involving characterization, monitoring, numerical simulation, evaluation, and decision making. The level of effort and specific types of information required for this process are difficult, if not impossible, to delineate a priori. Each site and each accident would be unique, requiring a characterization and mitigative plan specifically tailored to that event. Ideally, the maximum amount of information that is feasible to collect or statistically required would be used to evaluate a core melt accident.

10. This study is predominantly concerned with mitigative actions to prevent imminent environmental consequences of surface water and ground-water contact. However, applying mitigative techniques to limit exposure risk at a surface water body may result in a long-term and/or short-term exposure risk elsewhere. By design, these risks are significantly less than not mitigating contaminant transport at the site. Mitigative strategies that contain contaminant concentrate radionuclides in space limit the area of contamination. Unless the

interdictive scheme has an element of contaminant collection, this action elevates ground water concentrations inside the mitigative barriers. Inadvertant contact with this fluid could be hazardous for a long time period. In the case of core debris, radionuclides would be leach released for hundreds of years at levels that are predicted to produce ground-water concentrations at the plant site above present 10 CFR Part 20 limits.

Mitigative strategies that use contaminant collection systems would require some exposure to workers. These exposures could be tightly controlled to limit the risk to any single individual. No mitigative system presently available can indefinitely isolate the contaminant without periodic maintanence and refurbishing. Failure of a passive barrier or dewatering system would remobilize radionuclides and possibly create an exposure risk at a later time. Continued and vigilant monitoring and reevaluation of the site would be required to prevent hazardous contaminant breakouts. Each subsequent mitigative effort would use additional construction space between the contaminant source and the surface or ground-water body the mitigation is protecting. For long-term isolation of the contaminant, the site and the highly contaminated portion of the travel path must either remain under institutional control or the contaminant must be removed and placed in a disposal facility.

11. The methodologies demonstrated in this study to characterize ground-water flow systems and the selection and design of near-surface interdictive schemes are generally applicable to other near-surface sites where disposal of low-level nuclear waste and non-nuclear contaminants has taken place.

9.3 SUGGESTIONS FOR FURTHER RESEARCH

1. Hydrologic data bases for all operating power plants should be suitable to establish, as possible, the contaminant flow pathways; this would include the hydraulic characterization of sites to sufficient detail that preliminary simulations of contaminant migration and mitigation are feasible without additional data collection.

   Research topics for further consideration include: establishment of hydrogeologic data requirements to provide initial selection of a mitigative technique(s) and preliminary construction designs as demonstrated by this report, and a review of all operational power plant sites for identification of locations that lack an adequate data base as defined above and have characteristics that would require a quick mitigative response to a severe accident. Selected plants would undergo further hydrogeologic data gathering and/or interpretation.

2. Very large uncertainties remain in the core-melt-debris leach release functions. A better descriptor for this process will greatly improve

the understanding of which radionuclides and at what quantities the mitigative system would be expected to control. Without an accurate source term, the only method to determine the magnitude of contaminant concentrations is through direct sampling of the plume. Additional information on contaminant source terms should be gathered through experimentation and incorporation of data currently being collected for low-level radionuclide leaching experiments at Savannah River Laboratory, under saturated conditions, and Pacific Northwest Laboratory, under partially saturated conditions. This new information should be examined and applied where relevant to leach rate estimation techniques for core melt debris.

Research topics for consideration include: short- and long-term leach rates and processes, effect of mixed debris composed of silicic and calcine materials, possible differences in leach rates of simulated core materials and manmade isolation materials (i.e., grout and glass).

3. A review of site restoration issues, processes, and feasibility should be conducted. The techniques to remove core debris and reclamation of sump water contaminants should be identified or developed to the conceptual stage. Research topics for consideration include: feasibility of core debris recovery, methodology of debris collection, identification of technology that would result in total insitu isolation of radionuclides, ultimate disposal of core debris removed from the site, worker saftey, and cost effectiveness.

10.0 CONCLUSIONS OF GENERIC AND SITE ANALYSES

10.1 INTRODUCTION TO A CORE MELT ACCIDENT

The release and transport of radionuclides following a core melt accident is a complex process which is dependent on many accident- and site-specific parameters and events. To the degree possible, these factors have been generalized to determine the salient features of a severe accident. The conclusions presented in this section are limited in scope to aspects of mitigating the more imminent effects of ground-water contamination. Atmospheric releases, long-term, low-level contamination of surface water and ground-water bodies, core debris removal and site restoration issues are not part of this study.

The conclusions of this study fall into two classifications: the conclusions of generic analyses of a core melt accident and the site specific conclusions of conducting case studies. The major conclusions are presented by topic in the following sections.

10.2 CONCLUSIONS OF THE GENERIC RELEASE AND TRANSPORT ANALYSIS

A severe or core melt accident would release contaminants to the geologic environment through two mechanisms. First, and applicable to both boiling water reactors (BWRs) and pressurized water reactors (PWRs), is a core melt debris leachate release. This would occur when molten reactor materials penetrate the containment basemat and cool in the rock or soil beneath the plant. As ground water contacted the debris, radionuclides would leach from the melt zone and enter the subsurface flow system. Secondly, and pertaining only to PWRs is a penetration of the containment basemat by contaminated reactor sump water. The conclusions concerning these two types of releases of radionuclides are presented below.

10.2.1 Core Melt Debris Leaching

The chemical composition of the aggregate in the concrete basemat and the underlying geologic materials has a large influence on the rate of solid material leach release rate. Calcine debris derived from concrete and carbonate rock would be:

- relatively porous with a high surface area,
- contain a high density of radionuclides per unit volume,
- melt to a depth of about 3 meters below the basemat, and
- release radionuclides to the ground-water flow system through a diffusion process.

Silicic debris produced by the melting of sand or igneous material would:

- be more glass-like with a porosity and permeability determined by the density of the fracture network,

- have a relatively lower surface area and porosity,

- involve a larger melt zone extending to 10 meters below the basemat, and

- release radionuclides through a dissolution process.

As a result of these fundamentally different characteristics and leach mechanisms, calcine debris would release radionuclides at a rate at least two orders of magnitude greater than silicic debris. The difference in leach rates would increase if less conservative assumptions of material properties are assumed. These two characterizations of the chemical composition and leach release of core melt debris are representive of the range of conditions that would be found at an actual site.

Both calcine and silicic debris would continue to leach release radionuclides for migration as long as the debris was in contact with the ground-water flow system. The quantity of radionuclides released by leaching would eventually reach insigificant levels caused by radioactive decay and a decreasing leach release rate. Assuming no mitigation or restoration, the ground water adjacent to the melt zone is estimated to have concentrations of strontium-90 above 10 CFR Part 20 limits for a period of between 700 and 1200 years, depending on debris composition and ground-water flow rate.

10.2.2 Sump Water Release Rate From Containment

Sump water drainage rates through the containment basemat and core melt debris would be highly site and accident specific. Feasible rates based on the hydraulic properties of each site indicate that sump water drainage rates can produce a radionuclide release from containment greater that core melt leach rates. However, the actual drainage rate could be somewhat greater to much less than predicted. The time over which an actual sump water release would occur is a period of days to months. Very slow drainage rates could allow removal of liquid contaminant from the containment structure before it entered the ground water flow system. A release of sump water driven by a pressurized containment dome (pressurized water reactors only) could produce a rapid hydraulic spreading of contaminant and decrease the travel time to the surface environment. This would result in the largest possible radiological flux to the surface environment and the greatest need for contaminant interdiction.

10.2.3 Generic Hydrogeological Classification of Nuclear Power Plant Sites

A hydrogeologic classification system for contaminant interdiction at nuclear power plants must consider the geologic factors of a core melt accident from the creation of the melt debris to the eventual contaminant arrival at land surface. The major hydrogeologic factors are: 1) the rock chemistry at the location of the accident for the determination of the basic leach rate factors, 2) feasibility of contaminant mitigation technique(s) in different geologic environments and, 3) the ground-water transport parameters to land surface. Application of this system to existing and proposed nuclear power sites in the United States resulted in six generic classification:

1. Fractured Consolidated Crystalline Silicates,
2. Fractured and Solutioned Consolidated Carbonates,
3. Porous Consolidated Silicates,
4. Porous Consolidated Carbonates,
5. Porous Unconsolidated Silicates, and
6. Fractured Consolidated Silicates - Shale.

Assigning of "average" hydraulic parameters to a generic classification and generating "average" radionuclide discharges to a surface water body is undesirable and was not attemped because:

- there is a wide range in hydraulic values among geologically similar sites,
- such "average" conditions may not occur at any real site,
- averaged parameters that are inversely related (e.g., hydraulic gradients and permeabilities) may not produce an average result, and
- the variability of transport within a given classification would be lost.

Simulation of indivdual sites and analyzing the results by generic groups is applicable to the analysis and demonstrates the large differences in contaminant release and transport among and within the generic classifications. There are two major findings of the analysis of the generic hydrogeologic classification system.

First, the discharge of radionuclides to the surface water environment is more a function of site hydrogeology than the type of accident sequence (e.g., PWR 1-7 and BWR 1-4). The range of contaminant quantities available for transport because of a less probable accident is small (several tenths of the total amount) in comparison to the large range of values (up to 6 orders of magnitude) for hydrologic transport parameters. The different accident sequences would alter the quantity of contaminant by a linear function (a percentage of the total inventory) while changes in hydrologic parameters allow for longer transport times which exponentially decreases the total quantity discharged to the environment.

Secondly, the hydrogeologic aspects of a site that determine the environmental sensitivity to an accident can be ranked by relative importance as:

1. chemical composition of basemat and bedrock,
2. type of flow system being either porous or fractured media,
3. sorbtion of contaminant, and
4. hydraulic gradient and conductivity.

10.2.4 Indicator Radionuclides

Three radionuclide indicators of contamination are used in this study because of their initial quantity, longevity, and mobility. The analysis is conducted for strontium-90, cesium-137 and ruthenium-106. Ruthenium-106 is found to be sorbed and retarded under core melt conditions. Previous studies assumed that 50 percent of the ruthenium was complexed by nitrate and was a water coincident contaminant. This assumption was based on the migration of ruthenium-106 in high-nitrate nuclear processing wastes at the Hanford site. Nitrate concentrations found in natural ground water is not sufficient to mobilize ruthenium and it would be sorbed following an accident. The short half life of ruthenium of one year allows decay to insignificant levels proir to reaching surface water bodies at most power plant sites.

Cesium-137 would be released in the sump water from accidents at pressurized water reactors. Empirical testing indicates that cesium-137 is more strongly sorbed than strontium-90, but the retardation mechanism is phenomologically complex and not fully described by present geochemical models.

Strontium-90 is the preferred radionuclide for use as a singular indicator of the realtive sensitivity to a core melt accident. It is more mobile than cesium-137, is present in sump water and core debris, and arrives at the discharge location at relatively early times. The activity rates of strontium-90 is found to be commonly within an order of magnitude of cesium-137.

10.2.5 Premitigative Contaminant Discharges

The generic discharges of contaminant are evaluated at two basic levels. The first level determines whether the contaminant will arrive at an adjacent surface water body at an early time and at a high flux rate, or at a long time in the future at an insignificant level. A conservative definition of a significance is based on a 40 half-life travel time to surface water. At this long time, discharges of radionuclides are decayed to very low levels or fall into the catagory of a non-imminent situation that would require immediate interdiction. The analysis based on a conservative definition of significance and for the selected indicator radionuclides indicates that:

- ruthenium-106 would produce a significant discharge to adjacent surface water at 7 percent of the power plant sites,
- cesium-137 would produce a significant discharge to surface water at 37 percent of the sites,
- strontium-90 would result in a singificant discharge at 56 percent of the sites,

- 43 percent of all sites do not produce a significant discharge to surface water bodies that would require immediate contaminant interdiction to prevent severe environmental consequences,

- interdiction would be desirable at 85 percent of the fractured geologic sites, and

- interdiction of contaminant would be desirable at 42 percent of the nonfractured sites.

Secondly, the generic sites can be ranked as to their relative environmental sensitivity to a core melt accident by comparison of the percentages of sites that would result in a significant discharge and those that would produce a minor radionuclide discharge. The values are presented in Table 10.2.5-1.

TABLE 10.2.5-1. Generic Sensitivity to a Severe Nuclear Accident

Rank	Generic Classification	Percent of Sites with Significant Surface Water Discharges*
1	Fractured Consolidated Crystalline Silicates	94
2	Fractured and Solutioned Consolidated Carbonates	83
3	Fractured Shale	60
4	Porous Unconsolidated Silicates	49
5	Porous Consolidated Silicates	38
6	Porous Consolidated Carbonates	20

*All three indicator radionuclides considered.

When generic trends in arrival times and discharge fluxes are observed in the premitigative analysis they indicate that:

- the earliest time of contaminant arrival of individual sites are in the fractured media classifications at 6 months for carbonates and 8 months for silicates,

- contaminant arrival at a surface water body at 90 percent of the sites would be greater than 5 years, allowing time for evaluation and assessment proir to mitigative actions,

- the generically grouped arrival times at a surface water body ranged from 5 years in fractured and solutioned carbonates to over 200 years for porous consolidated silicates,

- the greatest radionuclide flux entering a surface water body is produced by a sump water release of cesuim-137 in a fractured and solutioned carbonate at 2.5 X 10 17 pCi/yr,

- peak flux rates of cesium-137 and strontium-90 in sump water discharges are similar to an order of magnitude at contaminant arrival times of less than 30 years,

- silicic media has a peak radionuclide flux at a surface water body about 100 times less than carbonate media because of the difference in leach rates, and

- although the core debris contains 10 times more strontium-90 than the sump water, when coupled with a rapid release rate, sump water can produce a higher radionuclide flux to the environment.

When there are no trends within a classification of first arrival times or the quantity of radionuclides reaching the surface environment, site-specific hydraulic parameters are more important than generic classification. This situation occurs for porous consolidated carbonates and fractured shale. These sites are best evaluated for environmental sensitivity by observing the percentage of sites that produce a significant discharge (prior to 40 half-lives of decay).

10.3 MITIGATIVE TECHNIQUES FOR CONTAMINANT INTERDICTION

There are two general classes of ground-water contaminant interdiction techniques that may be used to mitigate the environmental effects of a severe nuclear accident. These two classes are: 1) static or passive techniques, and 2) dynamic or active strategies. The individual techniques or schemes that comprise each class are designed to interact directly with ground-water flow, and consequently the contaminant being transported, to achieve an acceptable level of contaminant mitigation.

10.3.1 Static Barriers

Static or passive mitigation techniques are typically engineered/constructed barriers to ground-water flow containing contaminant. The primary objective of a constructed barrier is to redirect the ground-water flow away from potentially accessible surface environments. Achievement of this objective usually results in ground water being forced to follow more circuitous routes with longer travel times. Constructed barriers are considered static ground-water contaminant mitigation techniques because once in place they are not readily adaptable to changing conditions of ground-water contamination. Engineered/constructed barriers do not normally require a significant amount of maintenance or energy. Three basic types of constructed barriers were analyzed for their feasibility and suitability as mitigation measures for ground-water

contamination resulting from a severe power plant accident: grout curtain cut-off walls, slurry trench cutoff walls, and steel sheet piling.

10.3.2 Dynamic Barriers

Dynamic or active ground-water contaminant mitigation techniques are primarily conceptual strategies for actively influencing the state of ground-water contamination. Active influence is accomplished by either changing the ground-water flow regime by pumping and/or injection, directly treating the contaminated ground water or combinations of both approaches. Active ground-water contaminant mitigation schemes are generally better able to respond to changes in the state of ground-water contamination than static barriers. However, typically associated with dynamic schemes are relatively high energy and maintenance costs. Also extensive monitoring feedback is usually recommended to ensure adequate performance. The dynamic ground-water contaminant mitigation schemes analyzed for their feasibility and applicability are:

1. Ground-water withdrawal for potentiometric surface adjustment,
   - 1a. prevent discharge at receiving surface water body
   - 1b. prevent saturated contact with core melt debris
   - 1c. prevent contamination through leaky aquifers

2. Ground-water withdrawal and/or injection to control contaminant plume,
   - 2a. withdrawal and injection
   - 2b. withdrawal without injection
   - 2c. withdrawal with surface treatment and recharge
   - 2d. injection only

3. Subsurface drains,

4. Selective filtration via permeable treatment beds,

5. Ground water freezing,

6. Air injection to form a permeability barrier.

In summary, the implementation considerations for ground-water contamination mitigation schemes are extremely important in the overall assessmint of the applicability of each measure. However, these issues are also highly sensitive to specific and individual site characteristics ranging from the physical plant configuration, to local meteorological condition as the time of the accident. Therefore it is difficult, if not impossible, to detail the absolute effect of these issues in a generic manner. For this reason the general limitations are presented in the generic analysis and the performance of a mitigative scheme is examined through case studies.

The generic examination of mitigative techniques is summarized at its most basic level in Table 10.3. The table presents the feasibility of each major mitigative technique for the 6 generic hydrogeological classifications.

TABLE 10.3. Feasibility of Mitigative Techniques for each Generic Hydrogeologic Classification.

Mitigative Technique	Generic Classification* A	B	C	D
	Feasibility**			
1. Grouting with Particulate and Chemicals	Y	Y	Y	Y
2. Slurry Trenches	N	N	Y	N
3. Steel Sheet Pilings	N	N	Y	N
4. Ground-Water Withdrawal for Potentiometric Surface Adjustment	M	Y	Y	N
5. Ground-Water Withdrawal and/or Injection for Contaminant Plume Control	Y	Y	Y	M
6. Interceptor Trenches	N	N	Y	Y
7. Permeable Treatment Beds	N	N	Y	N
8. Ground-Water Freezing	M	Y	Y	Y
9. Air Injection	M	M	M	M

* Generic Hydrogeologic Classification: A = Fractured Consolidated Silicates and Fractured and Solutioned Carbonates, B = Porous Consolidated Carbonates and Porous Consolidated Silicates, C = Porous Unconsolidated Silicates, D = Fractured Shale.

**Feasibility of Mitigative Techniques: Y = yes, N = no, M = marginal

10.4 CASE STUDY CONCLUSIONS

10.4.1 Introduction

The components of a case study are designed to start with the information gained from the generic anlaysis and follow an iterative process of collecting more information and developing more sophisticated conceptual and numerical models. This process is outlined in Figure 1.5-2, Section 1.5 of this report. In the event of a severe accident, the process would be continued until either the analysis indicated that no contaminant interdiction was necessary, or that the mitigative scheme in place would be an effective safeguard of environmental concerns.

10.4.2 Case Study Number One South Texas Plant

The primary objective of the STP case study is to develop and demonstrate general methodology for evaluating the desirability and feasibility of implementing ground-water contaminant mitigation strategies following a severe nuclear power plant accident. The study was conducted with readily available data sources including the STP Final Safety Analysis Report, regional hydrology reports, and the open literature. The level of technical detail attained in the case study results is commensurate with a reconnaissance or better level of analysis. The STP case study results include:

1. a detailed hydrogeologic characterization of a Texas Gulf Coastal Plain aquifer,
2. a complete discussion of data requirements, sources and procedures for the hydrogeologic characterization,
3. a two-dimensional ground-water flow and contaminant transport numerical model development based on the hydrogeologic characterization,
4. a baseline pre-mitigative analysis of radionuclide transport, and
5. a limited evaluation of the effect of selected engineered barriers and hydraulic barriers on radionuclide transport.

Major conclusions from the study results are the following:

1. flow and transport model simulation results show that following a severe accident at the STP ground-water radionuclide concentrations would be well below maximum permissible concentrations, therefore, mitigative action would not be necessary,
2. for the STP, all mitigation techniques evaluated significantly increased ground-water and contaminant travel times, and
3. model evaluations indicate that hydraulic and constructed barriers could prove to be effective in mitigating radionuclide discharges at the STP.

10.4.3 Case Study Number Two South Texas Plant

The South Texas Plant Case Study No. 2, using the conceptual and numerical models developed in Case Study No. 1, presents a detailed, though not exhaustive review of mitigation design alternatives. The purpose is to gain an increased understanding of how mitigation performance is related to design parameters (e.g., size, shape, permeability, location) and hydrogeologic characteristics. The numerical model proved to be extremely useful in performing the necessary flow and transport computations and facilitates evaluation of numerous alternatives within the confines of limited time and costs. The model also is quite flexible in representing a range of mitigation types, sizes, and shapes (28 different designs are evaluated in this case study).

Based on the analyses conducted, it is concluded that selection of appropriate mitigation techniques is highly site specific and requires thorough evaluation of the nature and extent of the contaminant release, site characteristics, and feasible alternatives. Simulation results indicate that barrier performance (cutoffs or slurry walls) is closely tied to the hydraulic characteristics of the aquifer in question. Thus, a very important aspect of mitigation design is accurate, detailed characterization of aquifer properties. Barriers improperly placed may in fact modify local ground-water velocities such that contaminant migration is increased. Another important consideration in mitigation design is to exploit the occurrence of natural decay as an in-situ treatment process by containing contaminant releases in proximity of the plant.

Some of the general insights gained from the mitigation analyses are the following:

- downgradient designs produce greater lateral spreading than do upgradient designs.
- cutoffs constructed in low hydraulic conductivity areas create greater backwater effects than cutoffs constructed in areas having relatively higher conductivity.
- cutoff effectiveness decreases with increasing distance from the contaminant source.
- barriers which obstruct flow in both the x- and y-directions (L- and U-shaped) appear to significantly out perform linear barriers.
- in the normal range of achievable barrier permeability reduction (i.e., 0.001 to 0.1 gpd/sq ft) performance is relatively unchanged.
- understanding the sensitivity of a given system to the assumed retardation coefficient is very important given the uncertainty associated with determining its value and its direct impact on transport results.

- incorporation of costs into the selection of appropriate mitigation measures must be based on a site-specific, detailed investigation of ground-water flow and contaminant transport in conjunction with an accurate assessment of the surface and subsurface contamination at the time of construction.

- pumping may be more flexible and less costly than construction of an engineered barrier; however, pumping will require considerable more upkeep and maintenance over long time periods.

Conclusions specific to the design and performance of mitigation at the STP are as follows:

- based on the pre-mitigation transport results, approximately 200 years will be available to implement mitigation at a distance of 800 ft or greater downgradient from the reactor.

- results indicate downgradient cutoffs, if constructed outside the cooling reservoir, provide no benefit and actually increase transport of radionuclides from the STP site. If downgradient cutoffs are to be constructed in the downgradient direction it will be necessary to locate them within the reservoir having a more centered orientation relative to the reactor site.

- barriers placed in the low hydraulic conductivity area in the eastern portion of the study area create greater backwater effects; however, they also induce greater east-to-west lateral velocities which transport contaminant around the western end of the barrier.

- the "best" performing alternative evaluated for the STP is a L-shaped design which has the leg in the y-direction placed on the western end of the barrier.

- downgradient injection schemes proved to be effective in creating hydraulic barriers to contaminant migration for the STP site.

- given the spatial distribution of hydraulic conductivity at the STP, downgradient barriers are generally more effective than upgradient barriers of the same length.

10.4.4 Case Study Number Three Marble Hill Indiana Nuclear Generating Station

10.4.4.1 Plant Location

The Marble Hill case study examines plant siting and hydrogeologic characteristics of a fractured carbonate location. The plant is located in south central Indiana on a peninsular bluff along the Ohio River. The flow system at Marble Hill, Indiana, consists of fractured limestone and dolomite interbedded with shale units. The fractures in the limestone and dolomite lie along strong preferential orientations producing an anisotropic flow field. The direction of contaminant travel is uncertain at this site despite an

extensive hydrologic data base. Plant location is astride a ground-water divide which results in uncertainty as to the ultimate direction of contaminant migration. A bifurcated plume is feasible at this site with contaminant moving basically to the east and/or west toward outcroppings along the Ohio River.

10.4.4.2 Contaminant Pathways

Two dolomitic units are identified that could feasibly receive and transport singificant quantities of core melt contaminants. Core debris leachate would enter and contaminate the lower hydrologic unit. Sump water would enter the lower unit and possibly the upper hydrologic unit. The upper hydrologic unit is less likely to be the transport medium following a severe accident because it lies 40 feet above the core debris. To force sump water into the upper unit, the containment structure would have to be pressurized and/or have standing water in the reactor sump. The upper hydrologic unit has a permeability about 100 times greater than the lower unit.

10.4.4.3 Modeling Approach

The modeling approaches commonly applied to porous media is not used for this fractured site because:

- the orientation of anisotropry is undetermined,
- the degree of anisotrophy has not been determined by onsite testing,
- the permeability of the hydrologic units range over three orders of magnitude and demonstrate little spatial correlation.

Insufficient data exist for a deterministic model of each fracture at the Marble Hill site. Therefore, a stochastic representation of the flow fields based on cumulative data distributions of site parameters (i.e., aperture width, fracture length and fracture orientation) is used to preserve the flow system's variability of permeability and anisotrophy. The modeling results from the stochastic-discrete approach indicates that the maximum ratio of anisotropy is 1:48 for the upper unit and 1:27 for the lower unit.

10.4.4.4 Characteristics of Fractured Flow Units

Low permeability fractures comprised of small apertures are of great importance to overall system function for two reasons. First, the small aperture fractures are an integral part of the fracture system interconnection. These fractures can provide critical interconnections among the larger fractures. When small apertures are the only interconnections among the larger aperture fractures they form impediments to flow and transport. This effect is observed in the comparison of fractured versus equivalent porous media first arrival times. In composite, the upper fractured system has first contaminant arrival times similar to equivalent porous media. This indicates that although some large aperture fractures have high ground-water velocities, the interconnection of large fractures to small fractures creates the primal flow pathways with average velocities approximately that of a porous media

equivalent. This situation was most evident in the upper unit where data estimates of effective porosity required for a porous media calculation were considered to be the most accurate.

The second reason that low permeability fractures are important is that they restrict flow pathways and delay contaminant migration and release radionuclides to higher velocity pathways over long periods of time. Contaminant breakthrough curves for this fractured system are characteristically different than the results from a porous media model. The major items that distinguish the fractured flow system at this site are:

- the breakthrough curves are irregular and contain time periods when all fractures discharging to the surface are swept clean of contaminants,
- the peak flux is less than predicted by an isotropic-homogenous model caused by a portion of contaminant being delayed in low velocity pathways, and
- the total period of a contaminant release to the environment is extended by the late arrival of radionuclides from low velocity pathways that require long time periods to reach the discharge location.

10.4.4.5 Strontium-90 At the Surface Environment

The evaluation of strontium-90 flux discharging to the surface environment indicates that contaminant interdiction in the lower unit to protect the adjacent Ohio River would not be necessary. The maximun rate of strontium-90 entering the Ohio River from the lower unit would be 1.5×10^8 pCi/yr for a sump water release migrating to the east. The upper hydrologic unit is capable of transporting sump water contaminant to the discharge area(s) at activity levels of concern. The peak strontium-90 flux discharging the upper carbonate unit would be 2.2×10^{17} pCi/yr for flow to the east and 1.9×10^{17} pCi/yr for flow to the west. These levels of strontium-90 discharge would result in concentrations above 300 pCi/l in the Ohio River.

10.4.4.6 Mitigation of Contaminant Dicharge

Mitigation at this site could be accomplished by several techniques. The method selected is a grout barrier to retard ground-water flow and radionuclide transport. The location and configuration of the barrier is based on what is considered as minimal post-accident characterization and design. Upgradient and downgradient barriers of limited extent proved effective in delaying contaminant arrival at the discharge areas. The grout barrier reduced strontium-90 concentration in the Ohio River to less than the 10 CFR Part 20 limit 300 pCi/l. This level of mitigation is within the stated performance objective and could be improved if desired by extention of the grout barrier or coupling the grout barrier with other mitigative techniques (i.e., contaminant collection wells).

Topography and plant structures limit the available construction space for a mitigative technique at this site. The short distance to the receiving water body and the limited space for construction suggest that the contaminant mitigation scheme be designed to be accomplished by the performance objectives by the first system installed.

APPENDIX A

GLOSSARY OF GEOTECHNICAL TERMS

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GLOSSARY OF GEOTECHNICAL TERMS

ACCESSIBLE ENVIRONMENT - (1) the atmosphere, (2) land surface, (3) surface water, (4) oceans, and (5) the portion of the lithosphere that is outside of the controlled area [1]. As used in this study the term is synonymous with (3 and 4) above.

ADDITIVE - Any material added to the basic components of grout [3].

ADSORPTION - The attachment of water molecules or ions to the surfaces of soil or rock particles [2].

AGGEGRATE - Relatively inert granulate mineral material, such as sand, gravel, slag, crushed stone, etc. that is mixed with a cementing agent to form a grout material [2].

ALLUVIAL - Clay, silt, sand, gravel, or other rock materials that have been transported by flowing water and deposited in comparatively recent geologic time as sorted or semisorted sediments [2].

ANISOTROPIC - A hydrologic unit having different hydraulic properties in different directions at any given point [2].

BASEMAT - The reinforced concrete floor of the reactor containment structure, commonly 5 to 15 feet in thickness.

BEDDING PLANE - Plane of stratification. The surface marking the boundary between a bed and the bed above or below it [3].

BENTONITE - A colloidal clay composed largely of the minerial montmorillonite, characterized by high adsorption and a very large volume change with setting or drying [2].

CATOGORY I STRUCTURE - A highly reinforced structure designed to withstand severe environmental stress and continue to provide necessary isolation of radionuclides.

CEMENT - The powdered dry cement prior to the addition of mixing water [3].

CONCRETE - A mixture of cement and coarse aggregate used to form work pads and bulk grouting of large openings.

CONFINING BED - A body of impermeable material stratigraphically adjacent to one or more aquifers [3].

CONSOLIDATED ROCK - Geologic units that are firm and cannot be worked by earth-moving equipment.

CONTAINMENT STRUCTURE - The building designed to house the reactor and contain nuclear fuel and materials.

CORE DEBRIS - The nuclear fuel, steel, and concrete that would solidify beneath the plant.

CORE MELT ACCIDENT - A highly unlikely nuclear accident where the reactor fuel and components overheat to the point of liquidfication.

CURE TIME - The interval between combining all grout ingredients and the substantial development of its final physical properties. Sometimes referred to as "set time" [2].

DYNAMIC TECHNIQUE - A mitigative method that is functional through the input of additional energy to the flow system. Examples include pumping and injection wells.

FRACTURE - A break or open crack in a geologic unit.

FRACTURING - A break in the rock caused by shear or tensile stress. This condition can be caused by overpressurization during grout injection.

GENERIC - Relating to a distinctive group or class.

GROUND WATER - all water which occurs below land surface [1].

GROUND WATER DIVIDE - A mounding of the water table or potentiometric surface separating flow into different directions.

GROUT - A material injected into the soil or rock to change the hydraulics and other physical characteristics of the formation [2].

HYDRAULIC - Relating to the properties of a liquid in motion.

HYDRAULIC CONDUCTIVITY - The proportionality factor in Darcy's law as applied to the flow of water in soil and rock. The flux of water per unit gradient of hydraulic potential [4].

HYDRAULIC GRADIENT - The change in static head per unit of distance in a given direction [4].

HYDROLOGIC - Relating to the properties of a system in which water is moving.

HYDROLOGIC UNIT - A single geologic formation designated based on its hydologic properties.

HYDROSTRATIGRAPHIC UNIT - Bodies of rock with considerable lateral extent that compose a geologic framework for a reasonably distinct hydrologic system [5].

INTERSTITIAL - An opening or space between adjacent particles that is not occupied by solid material [4].

INTERDICTION - To prohibit further advance by interception and disruption of an activity or process.

LEACH RELEASE - The loss of material in a immobile matrix to a flowing liquid.

MITIGATION - Actions to reduce the danger or hazard associated with a situation.

MITIGATIVE SCHEME - A combination of mitigative techniques distributed in time and/or space to reduce the environmental hazard of a contaminated release.

PERCHED ZONE - A water-saturated zone maintained above the normal free water elevation by the presence of an intervening relatively impervious confining stratum [2].

POROSITY - The ratio of the volume of voids is a material to the total volume of the material including the voids, usually expressed as a percentage [2].

POROSITY, EFFECTIVE - The amount of interconnected pore space available for fluid transmission. It is expressed as a percentage of the total volume occupied by the interconnected interstices to the total volume of the mass [4].

SEVERE ACCIDENT - A large accidental release of radoinuclides. As used in this report, it is synonymous with a core melt accident.

SLURRY - A fluid mixture of solids such as sand or clay in water [2].

STATIC TECHNIQUE - A mitigative technique that functions without the addition of outside energy to the flow system. An example would be a grout curtain.

STOCHASTIC - A process involving random events.

SUMP WATER - The contaminated water that could be released from a pressurized water reactor following a core melt accident.

SURFACE WATER - Any lake, river, stream, reservoir, ocean where water is free standing at land surface.

[1] Code of Federal Regulations 10 Part 60.

[2] "Preliminary Glossary of Terms Relating to Grouting". 1980. American Society of Civil Engineers, Journal of Geotechnical Engineering Division 106(7):803-815.

[3] Cambell, M. D., and J. H. Lehr. 1973. Water Well Technology, McGraw-Hill, New York,

[4] Johnson, A. I. 1981. "Glossary." In Permeability and Groundwater Contaminant Transport, ASTM STP 746 (3-17). T. F. Zimmie and C. O. Riggs, Eds.

[5] Maxey, George B. 1964. Hydrostratigraphic Units: Journal of Hydrology, Vol. 2, p. 124-129.

APPENDIX B

SITE CHARACTERIZATION AND CODE SELECTION

APPENDIX B

SITE CHARACTERIZATION AND CODE SELECTION

INTRODUCTION

Site characterization and code selection are primary to any modeling effort. Often decisions as to which kinds of data and how much data should be collected are based on professional experience. Each power plant site is hydrologically individualistic and any severe accident would have unique features. A "cool book" approach to severe accidents characterization is likely to be inappropriate in any circumstances. At the present time successful modeling requires both skill and judgment on the part of the hydrologist. An extensive guide to code selection was prepared by Simmons and Cole 1985.

Site characterization should be based on first developing a relevant conceptual model for the specific accident at the plant site and associated ground-water system. A conceptual model is essentially a picture of the flow system developed from the available site characterization data. The complexity of such a picture should be consistent with study objectives, which are the purposes for performing a modeling exercise. The technical details that enter into a conceptual model will depend on both objective and subjective scientific judgments of the modeling professionals involved. The final conceptual model developed will depend on how the various transport modeling technical issues are addressed. Site characterization and code selection is based on the descriptive requirements of the physical and chemical processes identified in the conceptual model as acting at a particular plant site.

This viewpoint for developing a successful site-specific simulation model was broken down into nine key steps, which form the operational approach of this appendix. Completion of those steps will result in the development of all interrelated components of a systems simulation model.

Site characterization is based on the following five steps:

1. Identify specific questions and study objectives.
2. Establish costs and schedules for achieving answers.
3. Enlist the aid of professional model applications group.
4. Decide on approach with applications group and guide code selection.
5. Facilitate the availability of site-specific data.

These five steps are discussed in detail following an explanation of the nine systems model development steps, which are presented first to clarify what site characterization entails.

MODELING NEEDS

Computer simulation models are needed to organize and analyze site characterization information in order to make decisions about the necessity and design basis of any mitigative actions. As scientific tools, the needed simulation models must be reliable and credible representations of the site and its response to a severe accident. Simulation models are needed to assess every pathway for possible escape of radionuclides from a plant site. Ground water is identified as a major environmental pathway for contamination following an accident and this appendix is devoted specifically to ground-water transport modeling.

Computer codes (programs) are needed to build the systems simulation models required to represent a complicated flow and transport systems. A systems model is usually composed of many computer codes representing various subsystems and their associated physical and chemical processes. No single computer code can presently meet all modeling needs. The interfacing of different codes is usually necessary to describe the various interacting subsystem. Unified and simplified generic systems models have been used in Volume 1 to compare the relative merits of plant sites. On the other hand, detailed and mechanistic systems models are necessary to predict contaminant concentrations under specific ground-water flow conditions in order to assess actual environmental impact, as demonstrated in Volume 2.

Ground-water scientists are confronted with a seemingly vast variety of codes, which are potentially useful for performing a ground-water contaminant transport study. There are many publications (e.g., Bachmat et al. 1980; and Kincaid et al. 1984a, 1984b) that provide an inventory of available codes. Such code inventories present a confusing array of possible choices. Nevertheless, an appropriate code selection(s) must be made to identify the various key aspects to determine environmental consequences and construction safety.

PURPOSE

This appendix provides guidance for the selection and evaluation of ground-water transport models and site characterization. The guidance given here is primarily directed toward and applications-oriented user of a computer simulation model. But the information presented here is also important to a site operator or manager who will have the responsibility of coordinating the steps involved in accomplishing a successful modeling exercise, which will ultimately require a great deal of scientific credibility.

In view of the diversity in typical modeling needs and objectives connected with severe accident sites, these guidelines are formulated as a general plan for selecting relevant ground-water transport codes. They are not intended to serve as an absolute set of regulations for accepting or rejecting codes for possible use in evaluating a contamination problem. Instead, the guidelines deal in general terms with ground-water transport modeling methodology; they do not give specific advice on what constitutes the "best" codes for a particular study.

These guidelines deal only with the selection of existing codes, not with the development of numerical algorithms for constructing new codes. This latter mathematical subject is beyond the scope of this report. Moreover, this appendix will identify certain technological weaknesses in ground-water transport theory, but it does not recommend specific future research directions.

THE CODE SELECTION AND CHARACTERIZATION APPROACH

To build a systems model of a severe accident, appropriate computer codes must be selected. Code selection for purposes of modeling subsurface contaminant migration is actually a problem of developing a relevant systems model to represent the particular plant site and ground-water system. Code selection, however, is just one aspect of developing a systems model as outlined in the following ideal development steps:

1. Define site study objectives.
2. Collect and analyze site characterizing data.
3. Formulate the conceptual model.
4. Identify process descriptive equations.
5. Select the computer codes.
6. Couple/interface the selected codes.
7. Evaluate code performance.
8. Run site-specific simulations.
9. Compare results with study objectives.

The above nine steps will form the basis of these guidelines for code selection and evaluation. Code selection cannot be successfully accomplished without regard for the overall simulation model that will achieve the study objectives (step 1), and an active evaluation of code simulation capabilities (step 7) is necessary to ensure a proper selection. As shown in Figure 1, these steps are involved in the development of each component of a systems model for a specific burial site. A conceptual model based on the site characterization data and consistent with study objectives is the hub of a systems model. Other system model components are arranged as a wheel on that hub. Clockwise progress around the wheel, following the nine steps, is required to complete the systems model. During the development of a systems simulation model, the hub may require repeated modifications and revisions to produce a well-rounded and balanced wheel. These nine steps are each explained briefly below. The steps and their relationship to ground-water transport modeling are discussed in greater detail in Simmons and Cole (1985).

STEP 1. DEFINE SITE STUDY OBJECTIVES

The study objectives are the purpose for performing a simulation of a burial system. Some common study objectives for site characterization are:

- assessment of actual environmental impact: prediction of contaminant migration and dose modeling
- optimal control of contaminant migration plume in a ground-water system: design of a mitigation strategy
- site monitoring and surveillance network design.

These study objectives constitute some probable concerns of site operators and managers who would use modeling simulation results as a basis for making decisions.

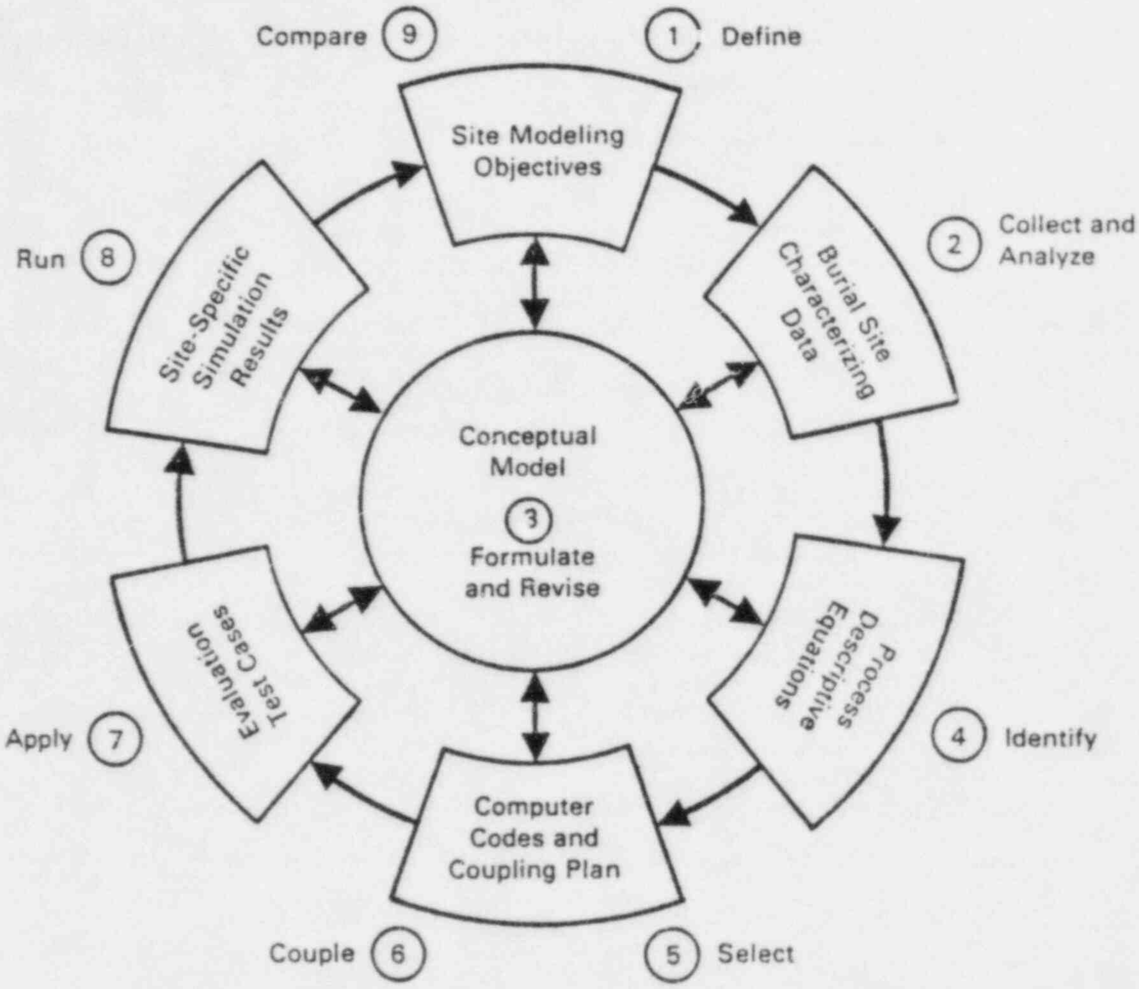

FIGURE B.1. Systems Model Components. Arrows show direction for completing the systems model development steps (numbers).

The site modeling objectives indicated in Figure B.1 are in a sense only a subset of the overall study objectives, because some objectives might not require examination by means of a simulation. Specific questions to be addressed by numerical simulation of a site have to be deduced from the study objectives. For instance, a modeling objective might be to estimate the concentration of a particular contaminant at a specific aquifer location, as observed through a sample well over some future period. A related modeling objective might then be to project the cumulative biological dose associated with water drawn from that sample well. The original study objective might have been to provide an environmental impact assessment. Thus, modeling objectives are just more explicit and detailed questions, originating in the study objectives.

The complexity of a particular study objective determines the degree of modeling sophistication required to attain relevant answers to the questions posed by a transport assessment problem. A study objective may call for either

a near- or far-field transport analysis or, perhaps, both. The appropriate codes will depend on the kind of transport analysis required.

STEP 2: COLLECT AND ANALYZE SITE CHARACTERIZATION DATA

After establishing study objectives, a modeler should proceed with assembling all information necessary for forming the conceptual model and gaining a preliminary view of how the ground-water system may function. These data include all measurements that describe the plant site, details of severe accident events and all post-accident measurements. The data should also include the following: regional geologic and hydrologic maps, climatological records, hydrologic property measurements, and an inventory of nuclear materials known to exit containment. These data must be complete enough for a modeler to formulate a technical representation (i.e., initial and boundary conditions) for the severe accident and for those mechanistic processes that contribute to contaminant migration. The guidelines include a more detailed description of typical data requirements.

A report by Lutton et al. (1982) describes the typical parameters needed to characterize a low-level waste disposal site. Table B.1 provides a list of those parameters. Jones and Gee (1984) discuss the specific parameters that would be required to model a shallow-land burial system at an arid site. The general group of processes that must be described at a shallow contaminant sites are shown in Figure B.2. A complete systems model for a site would incorporate al of those process models in order to account for an accurate water balance.

TABLE B.1. Common Parameters for Characterizing Plant Sites

General	Geochemical
Core Debris - geologic interface	Ion exchange capacity
Time history of liquid release	Soil pH
Material zone boundaries	Soil solubles
Geologic characteristics	Surface water chemistry
	Ground-water chemistry
Hydrological	**Geotechnical**
Hydraulic conductivity	Classification
Anisotropy	Compaction relation
Porosity	Grain-size distribution
Hydraulic potential	Density
Flow direction	Strength
Hydrodynamic dispersion	
Water-holding parameters	
Water content	
Precipitation	

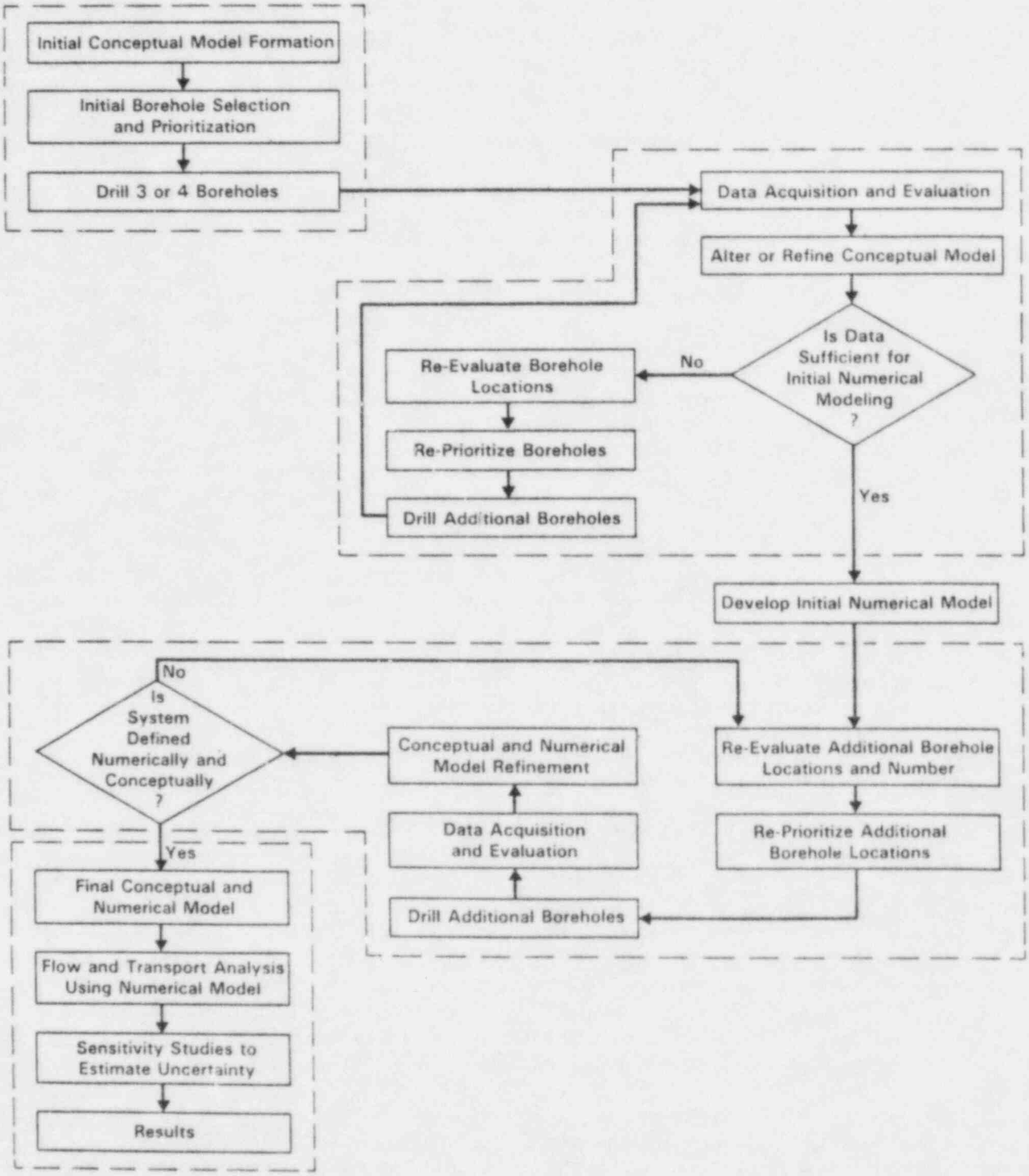

FIGURE B.2. Flow Diagram for the Site Characterization Process (Myers Personal Communication 1985)

The collection of site characterization data does not have to be absolutely complete before proceeding with further steps in the model development plan. In fact, the data base may need to be continually supplemented as the model development steps are applied. Formation of the conceptual model, identification of process descriptive equations, and the selection of computed codes will usually point out specific data deficiencies that must be filled in to accomplish simulation runs consistent with the study objectives.

STEP 3. FORMULATE THE CONCEPTUAL MODEL

The conceptual model is a mental idealization of a severe accident and does not remain static. Basic site characterization data in conjunction with the study objectives (step 1) are needed to form a preliminary conceptual model, which is then progressively modified as the other planning steps of Figure 1 are applied. For instance, the conceptual model may have to be simplified if site data are inherently limited or if available code technology is not adequate to simulate the initially perceived system's complexity. On the other hand, the study objectives and the conceptual model believed most appropriate may dictate the further collection of site characterization data or even the development of improved computer codes.

For purposes of systems simulation, the conceptual model is a simplified, yet rigorously technical, picture of the burial system. That picture must be technical enough in terms of fundamental processes, initial and boundary conditions, external hydrologic and climatic influences, and contaminant sources and sinks to determine unique predictions for a specific site. This is to say, a unique solution to the mathematical problem embodied in the appropriate process descriptive equations (step 4) must be achieved. The detail that enters into a conceptual model should represent the site characterization data base that is actually used in the final computer simulation model.

The conceptual model, no matter how technically complex, will always be a simplified picture of the real ground-water system. Current computer technology and data-gathering capabilities simply do not allow a real ground-water system and core debris to be described in every detail. To form a sufficiently accurate simplified picture, certain ground-water transport modeling technical issues must be considered. The technical issues are simply questions as to what constitutes the correct way to describe the modeled system. The issues stem from limitations on current physical and chemical theories and computer modeling capabilities. In many cases the technical issues do not have absolute resolutions (i.e., answers).

Many of these technical issues are discussed in the guidelines, because their treatment will determine the modeling outcome and predictions. For instance, an issue associated with the modeling described by Ahlstrom et al. (1977) and Arnett et al. (1977) might be the question: "Is a two-dimensional areal description of transport adequate?" In a general context, the answer clearly depends on the simulation study objectives and whether or not one believes that a three-dimensional process is ever reasonably represented by a low spatial dimensionality. In the case of the study by Ahlstrom et al. (1977) and Arnett et al. (1977), the answer seems affirmative, in view of the large

areal extent, when compared with the aquifer thickness involved. In this example, issues about field-scale dispersion, however, are probably unresolved.

STEP 4. IDENTIFY PROCESS DESCRIPTIVE EQUATIONS

Process descriptive equations are the fundamental mathematical equations required to represent those physical and chemical processes appearing in the conceptual model. The appropriate equations need not be expressed in any greater generality than will be necessary to implement the conceptual model.

A common practice is to begin with the most general form of applicable mathematical theory, and then, by assuming various simplifications that are compatible with the conceptual model, to reduce the complexity of the general equations. This is a deductive logical approach; an inductive approach, however, is just as valid. This means that sufficiently general equations can just as well be derived, while limited in context to the conceptual model. Moreover, it is possible in some cases that processes might be described only in terms of numerical algorithms, not explicit equations.

Site characterization data for very detailed characterization must be sufficient to define all necessary parameters appearing in the appropriate descriptive equations or algorithms. For simulating contaminant migration, these equations must describe ground-water flow, solute transport, and chemical behavior in the particular medium. However, many equations involving other system aspects such as runoff, evapotranspiration, biological processes, core debris material processes, and dose calculations may also be required to complete a systems model.

Commonly, the subprograms that appear in a computer code are concerned with solving each of the various process descriptive equations. The linking of such subprograms often represents the coupling of basic subsystems of a total systems model.

A user who is not an expert in ground-water transport theory may have to rely on a code developer's documentation report and user's guide when identifying the relevant basic descriptive equations. For such a user, the matching of fundamental processes appearing in the conceptual model with reported code capabilities will be necessary; this is the next step. A user should at least be able to identify the basic processes acting at the specific site.

SITE 5. SELECT THE COMPUTER CODES

Codes are simply the computer language algorithms for obtaining numerical solutions to the process descriptive equations, when site characterization data have been converted into the required input parameters.

Having identified all appropriate process descriptive equations, or at least having identified the basic processes believed to be involved, the kinds of codes required are nearly determined. In principle, a search through code summary reports (e.g., Bachmat et al. 1980) and specific code documentation

(see guidelines for example codes) will help identify those codes that are potentially applicable. The potentially useful codes need only include the relevant processes. In some cases a relevant code may be so general that it needs only to be restricted to solve the special case of interest. For instance, a three-dimensional ground-water flow code should be able to solve a restricted two-dimensional problem. But an application of the more general code may be rather inefficient or even present difficulties in obtaining simulation control, as a consequence of insufficient data.

In some cases, a user may unfortunately misuse this code selection step by attempting to force fit the conceptual model or even the study objectives into the mold of a pre-chosen code. This may be successful provided the selected codes are general and flexible enough, but an unnecessary amount of model preparation effort may result. A user should avoid such modeling overkill as much as possible, especially when site information does not justify a complicated analysis. Application of a complicated code may demand further collection of site data and refinement of the conceptual model. The study objectives, or time constraints, may not warrant the extra effort.

The key aspect that a user should keep in mind when selecting a code is whether relevant evaluations of code capabilities have already been performed. Evaluation test cases should be used to prove every capability to be applied. Quite often, for various technical reasons, a code may fail to operate as claimed in a documentation report. Evaluation test cases discussed in step 7 are special example simulations of the basic processes. They are often used to verify or validate modeling capabilities. Such test cases establish how much confidence a user has in a code's ability to achieve its intended purpose. A more advanced code, which has not been sufficiently tested, can actually place a greater burden on a user who will have to test run the selected code himself, instead of relying on a developer's test cases.

Proper code selection, therefore, depends critically on a careful evaluation of needed capabilities. An evaluation of the unified systems model being developed for treating a particular problem, however, cannot be accomplished without having a plan for code coupling or interfacing. When more than one code is involved, the code coupling plan (step 6) needs to be considered in conjunction with this step. This is why steps 5 and 6 are shown together as a single component in Figure B.1.

STEP 6. COUPLE/INTERFACE THE SELECTED CODES

When more than one code is required, the selection step 5 must actually take into account a plan for how the needed codes will be joined together (coupled) to solve the entire systems simulation problem. Codes that pass numerical information as control data are said to be coupled. Codes that require coupling to form an entire systems model generally represent groups of processes that influence each other directly in some mechanistic way. Coupled codes may represent the relationship between parts of a systems model at either the fundamental process level or at the level of environmental pathways connecting subsystems.

As an example of process coupling, a solute transport code must often be coupled with a ground-water flow code to perform a transport simulation. The prior computed ground-water flow is passed on to a solute transport code, which then calculates concentration and migration pattern.

A user may be able to find codes that already have the required coupling, but separation (decoupling) of the component codes that comprise the systems model can be helpful for testing each code independently. Then, if a particular code fails to meet the necessary capabilities, it can be replaced without having to rebuild the entire systems model. How strongly codes must be coupled depends on the interdependence of the involved processes. Codes describing processes that are linked in a reciprocal way may not allow decoupling. For instance, a strongly coupled relationship may be required to model spatial and temporal variation of chemical reactions occurring in conjunction with flowing ground water. In this case, a code that computes transport for each chemical species in a unified way (i.e., decay chains) may be needed, and decoupling may not be possible.

To reach an objective of assessing health effects, the generic systems model described by Hung et al. (1983) seems to allow a user a way of circumventing the rigors of these first six modeling steps. However, without a careful evaluation as discussed under step 7, there is no assurance that model predictions would be relevant or accurate for a plant site.

STEP 7. EVALUATE CODE PERFORMANCE

This is the critical step of systems model development, during which presumed code simulation capabilities are tested. Code capabilities are the processes that a simulation model can describe. The purpose of this step is to confirm that selected codes will actually work as intended. Moreover, code capabilities must be evaluated for their relevance to the system's conceptual model.

In step 5, a user should have considered codes that have already been tested as much as possible. In any case, all capabilities should be test run by the user and results compared with standard test cases. Evaluation test cases may take the form of analytical solutions obtained for special conditions or experimental data sets obtained for validation purposes. Test cases might also take the form of special benchmark cases (e.g., Ross et al. 1982), which are used to qualify codes for making certain performance assessments. Test cases usually represent the ideal behavior of the fundamental processes acting in the modeled system. A selected code that cannot reproduce the expected behavior of the basic, identifiable processes known to act in a system cannot provide accurate or credible predictions when incorporated into the complete systems model. Code evaluations must take into account the various ground-water technical issues discussed in the guidelines, as are relevant to the conceptual model.

As a recommendation, an inexperienced user is advised to select codes that have a long-standing history of successful applications as found in reports. Usually, this means that more evaluations have been accomplished successfully by others.

STEP 8. RUN SITE-SPECIFIC SIMULATIONS

The final component in the development of a site-specific systems model is the running of the selected codes, while using the site characterization data. At this stage the coupling plan is implemented. The codes will have been evaluated (step 7) to make certain that all the required capabilities work. The auxiliary software and methods for preparing data as input and analyzing program output are also important parts of running the simulations. Conclusions will depend greatly on how numerical output is displayed and analyzed. That aspect involving the display software and supporting analysis should be considered during the code selection (step 5) as well.

It is not always straightforward to run site-specific simulations. Adjustments and calibrations of a simulation model are usually required to make it match the known information. This is a necessary part of making a simulation model give relevant predictions about a specific system.

STEP 9. COMPARE RESULTS WITH STUDY OBJECTIVES

This is the last step that completes Figure B.1. A model exercise is not finished until is is certain that original simulation study objectives are achieved. This may require a return to any previous step for modifications or adjustments to achieve site-specific modeling objectives. On the other hand, a certain study objective might be found to fall beyond the capabilities of currently available code technology or even fundamental science. In this way a modeling effort may point the direction to needed future research. The systems modeling effort then becomes a logical justification for further research, as well as a way of obtaining answers to specific questions.

This completes the explanation of the nine steps seen in Figure B.1. The guidelines provide a more detailed discussion of these steps and a discussion of how code documentation reports should contribute to completing these steps.

CONCLUSIONS AND RECOMMENDATIONS

Some general conclusions related to the use of these guidelines are:

- No single code covers all problems. This means that no single code currently available includes all processes and subsystems required to describe a low-level waste burial site. A number of codes need to be joined (coupled) to form a systems model for a accident site. An adequate generic code to analyze any core melt location is not currently feasible.

- Code selection and site characterization is site specific. These guidelines emphasize that the characterization process must be site specific to meet varying study objectives and to make appropriate use of available site characterizing data.

- Code use and site characterization is modeler dependent. This means that predictions will be modeler dependent, even when the same codes are used. Different uses will apply the same codes in different ways, depending on how a simulation problem is conceptualized. Conceptual models of a system will differ depending on how users address the technical issues. Technical issues will arise at each decision point out of questions as to what modeling approach is appropriate. Technical issues stem from every attempt to simplify the conceptual picture of core melt conditions; therefore, modeling results depend on how the technical issues are addressed by a particular user.

- Code selection may be iterative. This means that code selection may have to be repeated for each new, changing study objective or to accommodate modifications of the conceptual model as additional site characterizing data are incorporated. The best way to implement an iterative selection may be to modify or extend available and familiar codes, rather than reselect entirely new and unprovided ones.

- Site characterization demands technical evaluations of model results. A successful model use demands that presumed simulation capabilities (modeled processes) be proven to work properly. At the least every capability to be applied in a particular study should be tested. Technical evaluation involves rigorous testing of codes. Testing is accomplished through comparison with analytical solutions of the governing equations and actual experimental data. Testing is necessary because codes often do not operate as claimed.

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APPENDIX C

TRANS CODE DESCRIPTION

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TRANS CODE DESCRIPTION

The simulations discussed in Chapters 6 and 7 were conducted using TRANS--the random walk solute-transport model of the Illinois State Water Survey. A detailed and complete description of the code and instructions for its use are presented by the authors (Prickett, Naymik and Lonnquist 1981). A brief summary of this information regarding the code and its capabilities are provided here for the convenience of interested readers.

TRANS is a generalized computer code that can simulate a large class of solute transport problems in ground water. The effects of convection, dispersion, and chemical reactions are included. The solutions for ground-water flow include a finite difference formulation. The solute transport portion of the code is based on a particle-in-a-cell technique for the convective mechanisms, and a random walk technique for the dispersion effects. The code can simulate one- or two-dimensional nonsteady/steady flow problems in heterogeneous aquifers under water table and/or artesian or leaky artesian conditions. The code covers time-varying pumpage or injection by wells, natural or artificial recharge, the flow relationships of water exchange between surface waters and ground water, the process of ground-water evapotranspiration, the mechanism of possible conversions of storage coefficients from artesian to water table conditions, and the flow from springs. It also allows specification of chemical constituent concentrations in any segment of the model. Further features of the program include variable finite difference grid sizes and printouts of input data, time series of heads, sequential plots of solute concentration distribution, concentration of water flowing into sinks, and the effects of dispersion and dilution or mixing of waters having various solute concentrations.

The calculation of flow by the TRANS code is accomplished by a finite difference solution to the partial differential equation governing the nonsteady-state, two-dimensional flow of ground water in artesian, nonhomogeneous, and isotropic aquifer expressed as

$$\partial/\partial_x(T\partial h/\partial x) + \partial/\partial y(T\partial h/\partial y) = S\ \partial h/\partial t + Q$$

where T = aquifer transmissivity
h = head
t = time
S = aquifer storage coefficient
Q = net ground water withdrawal rate per unit area
x,y = rectangular coordinates.

The finite difference approach involves replacing the continuous aquifer system parameters with an equivalent set of discrete elements written in finite difference form. The resultant finite difference equations are then solved numerically using the modified iterative alternating direction implicit method. This method solves, for a given time step, the equations for each row

of the model while holding the terms in the equations of the two adjacent rows constant. After all rows are processed row by row, the columns are solved in the same manner. After all columns and rows have been solved, an iteration has been completed. The process is continued until the solution converges to a pre-determined error level. The results from one time step serve as the initial conditions for the successive time steps until the simulation is complete.

The basis for computing contaminant transport in TRANS is the assumption that the distribution of contaminant concentrations in ground water can be represented by a finite number of discrete particles. Each particle represents a fraction of the total mass of contaminant involved and is assumed to move with ground-water flow. The random walk technique used by TRANS is founded on the concept that dispersion in porous media is a random process whereby particles move through an aquifer with two types of motion. One motion is that of the mean flow along computed streamlines. The other type is random motion governed by scaled probabilities related to flow length and the longitudinal and transverse dispersion coefficients. The density function that relates the number of particles found in particular cells within the finite difference model to concentrations of contaminant is derived from the expression

$$C(x,t) = \frac{1}{(4\pi d_L Vt)^{1/2}} \exp \left[\frac{-(x-Vt)^2}{4d_L Vt}\right]$$

where C = concentration
d = longitudinal dispersivity
V = interstitial velocity
t = time
x = distance along the x axis

This equation describes the one-dimensional movement of a slug release of fluid in a porous medium with steady flow. Based on the realization that dispersion in a porous medium can be considered a random process and assuming the process tends toward a normal distribution, this expression is converted into a density function that relates the concentration of contaminant to the concentration of particles found within the cells of the finite difference model.

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APPENDIX D

SUPPLEMENTAL REFERENCES FOR CONTAMINANT MITIGATION TECHNIQUES IN GROUND-WATER SYSTEMS

APPENDIX D

SUPPLEMENTAL REFERENCES FOR CONTAMINANT MITIGATION TECHNIQUES IN GROUND WATER SYSTEMS

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13. ABSTRACT (200 words or less)

Pacific Northwest Laboratory evaluated the feasibility of using ground-water contaminant mitigation techniques to control radionuclide migration following a severe commercial nuclear power reactor accident. The two types of severe commercial reactor accidents investigated are 1) containment basemat penetration of core melt debris, which slowly cools and leaches radionuclides to the subsurface environment; and 2) containment basemat penetration of sump water without full penetration of the core mass. Six generic hydrogeologic site classifications were developed from an evaluation of reported data pertaining to the hydrogeologic properties of all existing and proposed commercial reactor sites. One-dimensional radionuclide transport analyses were conducted on each of the individual reactor sites to determine the generic characteristics of a radionuclide discharge to an accessible environment. Ground-water contaminant mitigation techniques that may be suitable for severe power plant accidents, depending on specific site and accident conditions, were identified and evaluated. Feasible mitigative techniques and associated constraints on feasibility were determined for each of the six hydrogeologic site classifications. Three case studies were conducted at power plant sites located along the Texas Gulf Coast and the Ohio River. Mitigative strategies were evaluated for their impact on contaminant transport. Results show that the techniques evaluated significantly increased ground-water travel times and reduced contaminant migration rates.

14. DOCUMENT ANALYSIS - a. KEYWORDS/DESCRIPTORS
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13. ABSTRACT (200 words or less)

Pacific Northwest Laboratory evaluated the feasibility of using ground-water contaminant mitigation techniques to control radionuclide migration following a severe commercial nuclear power reactor accident. The two types of severe commercial reactor accidents investigated are 1) containment basemat penetration of core melt debris, which slowly cools and leaches radionuclides to the subsurface environment; and 2) containment basemat penetration of sump water without full penetration of the core mass. Six generic hydrogeologic site classifications were developed from an evaluation of reported data pertaining to the hydrogeologic properties of all existing and proposed commercial reactor sites. One-dimensional radionuclide transport analyses were conducted on each of the individual reactor sites to determine the generic characteristics of a radionuclide discharge to an accessible environment. Ground-water contaminant mitigation techniques that may be suitable for severe power plant accidents, depending on specific site and accident conditions, were identified and evaluated. Feasible mitigative techniques and associated constraints on feasibility were determined for each of the six hydrogeologic site classifications. Three case studies were conducted at power plant sites located along the Texas Gulf Coast and the Ohio River. Mitigative strategies were evaluated for their impact on contaminant transport. Results show that the techniques evaluated significantly increased ground-water travel times and reduced contaminant migration rates.

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