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Assessment and Remediation of Contaminated Sediments (ARCS) Program

ESTIMATING CONTAMINANT LOSSES FROM COMPONENTS OF REMEDIATION ALTERNATIVES FOR CONTAMINATED SEDIMENTS

◉ United States Areas of Concern

● ARCS Priority Areas of Concern

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ASSESSMENT AND REMEDIATION OF CONTAMINATED SEDIMENTS
(ARCS) PROGRAM

ESTIMATING CONTAMINANT LOSSES FROM COMPONENTS OF REMEDIATION ALTERNATIVES FOR CONTAMINATED SEDIMENTS

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
feet	0.3048	meters
inches	2.54	centimeters
pounds (mass)	0.4535924	kilograms
square feet	0.09290304	square meters

Preface

This study was conducted as part of the Assessment and Remediation of Contaminated Sediments (ARCS) Program developed by the U.S. Environmental Protection Agency (USEPA), Great Lakes National Program Office (GLNPO), pursuant to Section 118(c) (3) of the Water Quality Act of 1987. The report was prepared by the U.S. Army Engineer Waterways Experiment Station (WES) in cooperation with the USEPA Environmental Research Laboratory-Athens (ERL-A) and the U.S. Army Engineer Division, North Central, under interagency agreements between the USEPA and the U.S. Army Corps of Engineers.

The study was conducted between March 1991 and April 1994 as a three-phase study. Phase I of this effort was completed in September 1991 under ARCS Work Element E.17. Funding for Phases II and III was provided under ARCS Work Element E.29.

Project Manager for the GLNPO was Mr. David C. Cowgill. Mr. Jan A. Miller was the ARCS Program Manager for North Central Division. The study was conducted under technical guidance from the ARCS Program's Engineering/Technology Work Group, chaired by Dr. Steven M. Yaksich, U.S. Army Engineer District, Buffalo, Buffalo, NY.

The report was prepared by Messrs. Tommy E. Myers and Daniel E. Averett, Environmental Restoration Branch (ERB), Environmental Engineering Division (EED), Environmental Laboratory (EL), WES; Ms. Trudy J. Olin, Environmental Applications Branch (EAB), EED; Dr. Michael R. Palermo, Research Projects Group (RPG), EED; Dr. Danny D. Reible, Department of Chemical Engineering, Louisiana State University; Dr. James L. Martin, AScI Corporation, Athens GA; and Dr. Steven C. McCutcheon, USEPA ERL-A, Athens, GA. Mses. Melody Currie and Martha Huie, EED, assisted with tabular and graphical data presentations.

The study was conducted under the general supervision of Dr. Raymond L. Montgomery, Chief, EED, and Dr. John Harrison, Director, EL.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

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1 Introduction

Background

Assessment and Remediation of Contaminated Sediments (ARCS) Program

Industrial and municipal point-source discharges and nonpoint source pollution from agricultural and urban areas over many years have contaminated bottom sediments in the rivers, harbors, and nearshore areas of the Great Lakes. Improved controls for discharges have reduced pollutant loads to the Great Lakes. However, toxic substances in bottom sediments continue to impair sediment and water quality and may contribute to toxic effects in aquatic biota and, potentially, in humans. Areas in the Great Lakes that remain seriously impaired have been designated as "areas of concern" (AOCs) under the Great Lakes Water Quality Agreement (U.S. Environmental Protection Agency (USEPA) 1988). Public support for control of pollution in these AOCs has prompted increased attention by Government agencies and environmental organizations toward development of plans for remediation.

The Water Quality Act of 1987, which amended the Federal Water Pollution Control Act, authorized a program specifically aimed at the contaminated sediment problems in the Great Lakes AOCs. Section 118, paragraph (c) (3), directed the USEPA Great Lakes National Program Office (GLNPO) to study and demonstrate remediation of contaminated sediments in the Great Lakes. The Act specified that priority AOCs for implementation of demonstration projects were Saginaw Bay, Michigan; Sheboygan Harbor, Wisconsin; Grand Calumet River, Indiana; Ashtabula River, Ohio; and Buffalo River, New York.

The GLNPO program authorized by Section 118 has been named "Assessment and Remediation of Contaminated Sediments (ARCS) Program." The following objectives were developed for the ARCS program:

a. Assess the nature and extent of bottom sediment contamination at Great Lakes AOCs.

b. Evaluate and demonstrate remedial options including removal, immobilization, and advanced treatment technologies, as well as the no-action alternative.

c. Provide guidance to the various levels of government in the United States and Canada in the implementation of remedial action plans for the AOCs in their jurisdictions, as well as direction for future evaluations in other areas, including how to assess the need for action, options available, selection of appropriate remedial measures.

U.S. Army Corps of Engineers involvement

The U.S. Army Corps of Engineers (USACE) in fulfilling its mission to maintain, improve, and extend navigable waters in the United States dredges, relocates, and disposes 191 to 229 million cubic meters of sediment annually (Engler, Patin, and Theriot 1990). In addition, the USACE regulates the discharge of dredged and fill material in the waters of the United States involving 115 to 153 million cubic meters annually (Engler, Patin, and Theriot 1990). Most of the material dredged each year is suitable for a wide variety of beneficial uses and open-water disposal (Francingues et al. 1985). The presence of heavy metals and organic chemicals in about 10 percent of the materials dredged requires special handling and site-specific restrictions on disposal operations.

Although the USACE is responsible for and regulates dredge and fill activities in the waters of the United States, the lead responsibility for the development of environmental guidelines and criteria for regulating the discharge of dredged and fill material to the waters of the United States was legislatively assigned to the USEPA. The USEPA develops regulations for dredge and fill activities in consultation or conjunction with the USACE. In addition, the USEPA has an oversight role in the USACE regulatory program.

The need to evaluate pollutant potential and disposal alternatives has prompted the development and continued improvement of procedures and supporting laboratory tests for predicting environmental impacts of dredging and dredged material disposal by the USACE. USACE and USEPA concerns over the possibility of adverse environmental effects of dredged material disposal were evident as early as 1966 when an investigation of water quality problems in the Great Lakes was conducted by the U.S. Army Engineer District, Buffalo, in cooperation with the Federal Water Pollution Control Administration (now the USEPA) (U.S. Army Engineer District, Buffalo 1969). This work identified alternatives to open-water disposal of contaminated dredged material in the Great Lakes.

Between 1973 and 1978, a USACE laboratory, the U.S. Army Engineer Waterways Experiment Station (WES), conducted a national program of laboratory and field investigations on the environmental effects of dredged material disposal (Dredged Material Research Program (DMRP)). The DMRP

produced first-generation procedures for preproject evaluation of the environmental consequences of dredging and dredged material disposal. Following the DMRP effort, research and technology transfer programs, such as the Long-Term Effects of Dredging Operations (LEDO) and Dredging Operations Technical Support (DOTS) programs, were implemented by the USACE at WES. LEDO focuses on development, refinement, and field application of procedures for estimating the environmental effects of dredging operations, and DOTS is a direct field assistance and technology transfer vehicle to assist USACE Districts.

Between 1981 and 1987, a cooperative field verification program (FVP) among the U.S. Army Engineer Division, New England, WES, and the USEPA Environmental Research Laboratory, Narragansett, RI (ERLN), was conducted using contaminated dredged material from Black Rock Harbor at Bridgeport Harbor, Connecticut. FVP results showed that laboratory methods for predicting effluent and runoff water quality and plant toxicity in upland disposal sites compared well with field results (Peddicord 1988). WES has also been involved in extensive dredging and disposal alternative assessments for Indiana Harbor, Indiana (Environmental Laboratory 1987), Everett Bay, Washington (Palermo et al. 1989), and New Bedford Harbor Superfund Site, Massachusetts (Averett and Otis 1990).

Because of the experience, institutional knowledge, and technical expertise of the USACE in dealing with contaminated sediment and the history of interagency coordination and collaboration between the USEPA and the USACE, GLNPO tasked various USACE elements for support to the ARCS Program through interagency agreements. The USACE elements involved in the ARCS Program included the U.S. Army Engineer Division, North Central, the U.S. Army Engineer District, Buffalo, the U.S. Army Engineer District, Chicago, the U.S. Army Engineer District, Detroit, and WES. The USACE primary involvement was through ARCS Program technical groups, such as the Engineering/Technology Work Group (ETWG).

Engineering/Technology Work Group

The ETWG was one of three technical work groups within the ARCS Program that identified and prioritized tasks to be accomplished in support of overall program objectives. The ETWG was responsible for design, demonstration, and evaluation of remedial options for removing, treating, and disposing contaminated sediment. Selecting a remedial alternative requires evaluation of pollutant releases so that alternatives can be compared and the resulting ecological and human health risks can be evaluated. The best alternative is the alternative that minimizes contaminant losses and risks while maximizing treatment and/or containment effectiveness, but no alternative presents zero losses or zero risks.

In recognition of the need for estimating and evaluating contaminant losses associated with various remedial options, ETWG tasked WES and the USEPA

Environmental Research Laboratory, Athens, GA (ERL-A), to develop generic procedures for estimating contaminant losses from components of remediation alternatives. These procedures were needed for preproject evaluation of the performance characteristics of remedial alternatives and in other ARCS studies for the purpose of estimating their associated risks.

Objectives

The overall objective of this study was to develop procedures for making comparative estimates of contaminant losses from components of remedial alternatives for contaminated sediments based on existing predictive techniques and reported case studies. Supporting objectives were as follows:

a. Identify migration pathways associated with contaminant release.

b. Identify generic predictive techniques for contaminant release during stages of remediation for various alternatives, including the no-action alternative.

c. Evaluate the applicability and reliability of predictive techniques for contaminant releases associated with remediation of contaminated sediment.

d. Develop example contaminant release calculations for remedial alternatives at a selected AOC.

Application of a sediment remediation technology at any site will require a series of steps or components. For most sediment treatment alternatives, these components have been identified as follows (Averett et al. 1990):

a. Removal (Dredging).

b. Transport.

c. Pretreatment.

d. Treatment.

e. Disposal.

f. Effluent/Leachate Treatment.

The ability to quantify losses varies from component to component and within remediation components among migration pathways. Some alternatives, such as in situ capping, do not involve these components. For such alternatives, special contaminant loss estimation procedures are required.

Scope

This study was conducted as a desktop review and analysis of available predictive techniques for contaminant losses from components of remedial alternatives. The predictive techniques identified in this study include laboratory tests, simple focused vignette models, and existing contaminant transport models. No laboratory or field data collection was performed in this study. Predictive techniques for losses to air, surface water, and groundwater are identified and described in this report. Transport in air and water outside the physical boundaries of a remediation component was not modeled. In addition, plant and animal uptake were not evaluated. This report does not provide estimates of risks. The focus is on identifying and applying quantitative predictive techniques for developing the numerical information needed to evaluate risks. The Risk Assessment and Modeling Work Group of the ARCS Program addressed the issues of comparative risk calculations and their use in making remediation decisions. The reader is referred to the report "ARCS Risk Assessment and Modeling Overview Document" (USEPA 1993a) for more details.

This report includes evaluation of the research and development behind the available predictive techniques for various components and, hence, the relative reliability of these techniques. This report, however, does not include statistical analysis of the uncertainty associated with using the predictive techniques.

Report contents

Following this introduction are 10 parts. Contaminant Losses During Dredging describes procedures for estimating losses during dredging. Contaminant Losses During Dredged Material Transport deals with losses during dredged material transportation. Contaminant Losses During Pretreatment describes procedures for estimating losses during pretreatment. The procedures in Contaminant Losses During Pretreatment are also applicable to confined disposal facilities. Losses From Confined Disposal Facilities describes the performance characteristics of confined disposal facilities. Contaminant Losses for In Situ Capping and Capped Disposal describes procedures for estimating losses associated with the in situ capping alternatives. Contaminant Losses During Effluent and Leachate Treatment describes procedures for estimating losses associated with treatment of effluent and leachate from pretreatment and confined disposal facilities. Contaminant Losses for the No-Action Alternative provides an overview of the models applicable to no-action assessments. Dredged Material Treatment describes assessment techniques for treatment alternatives. Example Application to Contaminated Sediments in the Buffalo River presents example calculations of contaminant losses. Summary and Recommendations provides concluding remarks about the procedures described in Contaminant Losses During Dredging through Dredged Material Treatment and the results obtained in Example Application To Contaminated Sediments in the Buffalo River. There are three appendices: Appendix A: Notation; Appendix B: A Priori Estimation of Distribution

Coefficients; and Appendix C: Input Parameters for Open-Water Disposal Models.

2 Contaminant Losses During Dredging

Background

All remedial options for contaminated sediments, other than containment or treatment in place, require dredging. During dredging, sediment is resuspended in the water column when dislodged sediment is not completely captured by the dredging equipment. Contaminants are released to the water column in particulate form by resuspension of solids and in dissolved form by desorption from resuspended solids and dispersal of interstitial water. Chemicals that remain adsorbed to sediment particles may be transported and redeposited at locations some distance from the dredge. Volatile contaminants not bound to suspended particulate are available for transport into the atmosphere.

In this section, dredging equipment and techniques for estimating contaminant mass release rates at the point of dredging are reviewed. Estimating contaminant release at the point of dredging primarily involves estimation of the rate of resuspension of sediments during dredging. In this report, resuspended particles and their associated contaminants are assumed lost even though they may eventually settle back to the sediment bed. This is an overly conservative assumption if material settles into the path of the dredge. However, in the context of sediment remediation, resuspended contaminated sediment that is not eventually captured by the dredge remains in the waterway and in a sense is lost by the dredge. Predictive techniques for chemical release from resuspended sediments and volatilization of dissolved chemicals to air are also discussed as inputs needed for contaminant transport models used to assess impacts and risks.

Field data on contaminant releases during dredging are scarce. Most of the available data are for sediment resuspension, not contaminant release. The predictive techniques discussed in this report are, therefore, based on resuspension data. Predictive techniques for sediment resuspension are available for hydraulic cutterhead and bucket dredges. Predictive techniques have not been developed for other types of dredges. Collins (1989) developed predictive correlations for sediment resuspension by hydraulic cutterhead and bucket dredges using the field data from studies by Hayes (1986); Hayes, McLellan,

and Truit 1988; McLellan et al. 1989. Although the data are insufficient for full validation, the equations developed by Collins (1989) model the qualitative effect of variations in key cutterhead and bucket operational parameters and, therefore, provide a good starting point in the effort to estimate contaminant release during dredging. Field studies have identified a range of resuspended sediment concentrations that can be expected in the vicinity of many dredges. This information can be used with the available predictive techniques for cutterhead and bucket dredges to estimate losses from other types of dredges. Herbich and Brahme (1991) reviewed the literature on sediment resuspension during dredging and tabulated information according to project site, type of dredge, and sampling location for suspended solids data (Table 1). Table 1 indicates a wide range in particulate concentrations around various dredges. Since contaminant release rates vary relative to the resuspended sediment concentration, an indication of contaminant release rates can be obtained by comparing the resuspended sediment concentrations.

Dredging Equipment

Several types of dredging equipment are available for removing contaminated sediments (Cullinane et al. 1986; Palermo and Pankow 1988; Herbich and Brahme 1991). Dredges can be classified according to the basic means for entraining sediments (hydraulic or mechanical), the method of dredged material transport (pipeline, scow, or hopper), the equipment used for excavating sediments (cutterhead, dustpan, or plain suction), and the type of pump used (centrifugal, pneumatic, or airlift). Conventional dredges were not specifically designed for remedial dredging and, thus, not specifically designed to minimize contaminant release.

Among the conventional dredges available for contaminated sediment removal are cutterhead dredges, dustpan dredges, hopper dredges, and bucket dredges. Descriptions of the general design and operation of various dredges are briefly discussed in this section beginning with the hydraulic cutterhead dredge. Discussion of the hydraulic cutterhead dredge is followed by descriptions of other hydraulic dredges, the bucket dredge, and then other mechanical dredges. Additional information on the dredges briefly described in this report and other dredges is available elsewhere (Cullinane et al. 1986; Palermo and Pankow 1988; Herbich and Brahme 1991; USEPA 1994a).

Cutterhead hydraulic pipeline dredge

The cutterhead hydraulic pipeline dredge is a commonly used dredging plant (Figure 1). It is equipped with a rotating cutter surrounding the intake of the suction pipe. By combining the mechanical cutting action with hydraulic suction, the dredge has the capability to efficiently extract and remove materials. Although the cutterhead dredge was developed to loosen densely

Table 1
Suspended Solids Concentrations Produced by Various Dredges[1]

Type of Dredge	Suspended Solids Concentration	Remarks	Predictive Techniques
Cutterhead 10 rpm 20 rpm 30 rpm	 161 mg/ℓ (sandy clay) 52 mg/ℓ (med. clay) 187 mg/ℓ (sandy clay) 177 mg/ℓ (med. clay) 580 mg/ℓ 266 mg/ℓ	Observations in the Corpus Christi Channel (Huston and Huston 1976)	
18 rpm 18 rpm	1 to 4 g/ℓ within 3 m of cutter 2 to 31 g/ℓ within 1 m of cutter	Soft mud at Yokkaichi Harbor, Japan (Yagi et al. 1975)	Proposed by Collins (1989)
Trailing suction dredge	Several hundred milligrams per liter above background (at surface and middepth). As high as several grams per liter	San Francisco Bay (Barnard 1978)	
	2 g/ℓ at overflow 200 mg/ℓ at 200 m behind	Chesapeake Bay (Barnard 1978)	No predictive techniques available
Mudcat dredge	1.5 m from auger, 1 g/ℓ near bottom (background level 500 mg/ℓ) 1.5 to 3.5 m in front of auger, 200 mg/ℓ surface and middepth (background level 40 to 65 mg/ℓ)		
Pneuma pump	48 mg/ℓ at 1 m above bottom 4 mg/ℓ at 7 m above bottom (5 m in front of pump)	Port of Chofu, Japan	No predictive techniques available
	13 mg/ℓ at 1 m above bottom	Kita Kyushu City, Japan	No predictive techniques available
Cleanup system	1.1 mg/ℓ to 7.0 mg/ℓ at 3 m above suction 1.7 mg/ℓ to 3.5 mg/ℓ at surface	Toa Harbor, Japan	No predictive techniques available
Grab/bucket/clamshell dredges	Less than 200 mg/ℓ and average 30 to 90 mg/ℓ at 50 m downstream (background level 40 mg/ℓ)	San Francisco Bay (Barnard 1978)	
	168 mg/ℓ near bottom 68 mg/ℓ at surface	100 m downstream at lower Thames River, Connecticut (Bohlen and Tramontaro 1977)	
	150 to 300 mg/ℓ at 3.5-m depth	Japanese observations (Yagi et al. 1977)	Proposed by Collins (1989)

(Continued)

[1] From Herbich and Brahme (1991).

Table 1 (Concluded)

Type of Dredge	Suspended Solids Concentration	Remarks	Predictive Techniques
Antiturbidity overflow system	6 mg/ℓ at surface 8.2 mg/ℓ at 1 m below surface	Side of the ship (Ofuji and Naoshi 1976) Japan	
	6.5 mg/ℓ at surface 8.9 mg/ℓ at 1 m below surface	Aft of the ship	No predictive techniques available
Antiturbidity Watertight buckets	30 to 70 percent less turbidity than typical buckets	Japan (Barnard 1978)	No predictive techniques available
	500 mg/ℓ at 10 m downstream from a 4-cu m watertight bucket		

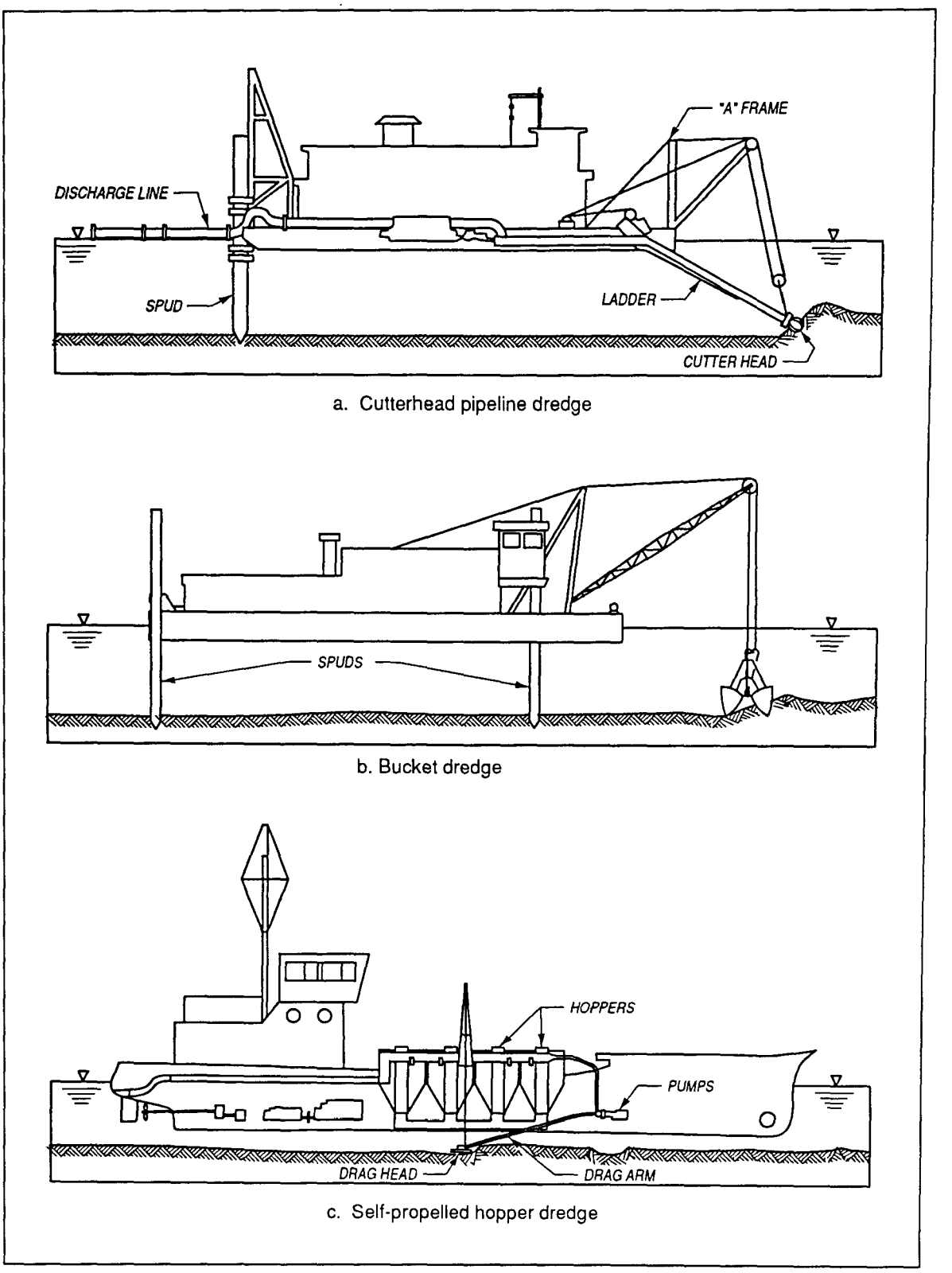


Figure 1. Hydraulic cutterhead (a), bucket (b), and hopper (c) dredges (from Palermo and Pankow 1988)

packed deposits and cut through soft rock, it can excavate a wide range of materials including clay, silt, sand, and gravel.

The cutterhead dredge is suitable for maintaining harbors, canals, and outlet channels where wave heights are not excessive, allowing it to work effectively in all types of alluvial sediments and compacted deposits. A cutterhead dredge is typically equipped with two stern spuds that alternately serve as a pivot swinging the cutterhead from side to side during operation. Resuspension of sediments during cutterhead excavation is strongly dependent on operational parameters such as thickness of cut, rate of swing, and cutter rotation rate. Proper balance of operational parameters can result in suspended sediment concentrations as low as 10 mg/ℓ in the vicinity of the cutterhead (Hayes, Raymond, and McLellan 1984).

Dustpan dredge

The dustpan dredge is a hydraulic suction dredge that uses high pressure water jets to loosen sediment for capture by suction. Dustpan dredges are used primarily for dredging sandy sediments on inland rivers. Dustpan dredges generate suspended solids plumes similar to cutterhead dredges. Plume suspended solids averaged 3.8 times background concentrations during removal of kepone-contaminated sediments from the James River, Virginia (McLellan et al. 1989).

Matchbox suction dredge

A matchbox dredge is a suction dredge that eliminates the cutterhead and water jets used in other hydraulic pipeline dredges. The dredge was originally designed to remove contaminated sediments in Rotterdam Harbor, The Netherlands (Hayes, McLellan, and Truitt 1988). The dredge head is designed to remove sediments close to in situ density and minimize resuspension. The absence of mechanical mixing associated with a cutterhead or water jet should reduce the sediment resuspension rates. However, the limited field data available indicate that particulate release rates for the matchbox and cutterhead dredges are about the same (Hayes, McLellan, and Truitt 1988; McLellan et al. 1989). Sediment resuspension with both types of dredges is highly dependent on operator skill and experience. In the studies conducted by Hayes, McLellan, and Truitt (1988), operator inexperience with the matchbox dredge contributed to poor control of matchbox position and frequent clogging of the suction line.

Hopper dredge

Hopper dredges (Figure 1) are usually self-propelled vessels equipped with dredge pumps for removing sediments and large hoppers for storing dredged material during transportation. Sediment is raised by dredge pumps through

drag arms connected to drag heads and discharged to hoppers built in the vessel. Dredging past hopper overflow is (i.e., allowing water and fine-grain sediment particles to flow over and out of hoppers) sometimes practiced to improve dredging economics by trapping coarse-grain material in the hoppers and releasing fine-grain material in the overflow. Since contaminants tend to be associated with fine-grain material, overflow is not a recommended dredging alternative for remediation. However, hopper dredging (without overflow) is an alternative that should be considered for sites requiring good maneuverability and minimum interference with navigation.

Horizontal auger dredge

A horizontal auger (HA) dredge is a cutterhead suction dredge with horizontal cutter knives and a spiral auger that cuts the material and moves it to the suction. The dredge is designed for the removal of small amounts of sediment (50 to 120 yd^3/hr (Averett et al. 1990)). Nawrocki (1974) reported resuspended sediment concentrations two to four times background within 4 m (12 ft) of the auger between the surface and the bottom. In a pilot study in New Bedford Harbor, the HA-type dredge experienced problems with positioning, anchoring, and effectiveness of the mudshield. Sediment resuspension at the dredgehead was substantially higher than for either cutterhead or matchbox dredges.

Cleanup dredge

Sato (1976a,b) describes an instrumented, covered auger dredge that is designed to clean up highly contaminated sediments. The instrumentation includes a sonar to determine the bottom elevation and an underwater television camera for monitoring of dredging operations. Resuspended sediment concentrations observed during tests of this system were essentially indistinguishable from background levels (Herbich and Brahme 1991).

Pneuma pump

The Pneuma pump uses compressed air and hydrostatic pressure rather than centrifugal motion to move dredged material through a pipeline. During the dredging process, the pump is submerged and sediment and water are forced into one of three cylinders by opening the cylinder to atmospheric air. The pump must be used at depths in excess of approximately 4 m (12 ft) to provide sufficient hydrostatic pressure for effective filling. After filling, compressed air is supplied, forcing the water and sediment through an outlet valve. Richardson et al. (1982) conducted field tests on a Pneuma pump and observed low turbidity levels in the vicinity of the pump. It was not possible to dredge sand, and the hydraulic efficiency of the dredge was consistently below 20 percent. Barnard (1978) reported suspended solids concentrations an order of magnitude above background within 1 to 2 m of a Pneuma pump.

Oozer pump

The Oozer pump operates in a manner similar to the Pneuma pump, but vacuum is applied during the filling stage to achieve more rapid filling, increase solids concentrations, and allow operation in more shallow waters. In the 11-year period from 1974 to 1984, approximately one million cubic meters of contaminated sediment were removed by the Oozer dredge (Ikalainen 1987). The Japanese Dredging and Reclamation Engineering Association conducted a field test of the Oozer dredge in Osaka Bay, Japan. The Oozer dredge removed organically contaminated fine-grained sediment in 16 m of water. Results indicated that the primary source of sediment resuspension around the Oozer dredge is the swing speed. Suspended solids concentrations were monitored at locations 50, 100, 200, and 300 m in front of the Oozer dredgehead; three sample stations were radially located at these distances. The maximum concentration observed at the three stations was 14 mg/ℓ (Zappi and Hayes 1991).

Bucket dredge

A bucket dredge is a mechanical device that utilizes a bucket to excavate sediment (Figure 1). Unlike hydraulic dredges that typically remove four times as much water as in situ sediment, the bucket dredge can remove material at close to in situ densities. It is used near surface and submerged structures due to the greater degree of control allowed during dredging. It can also be used to dredge at greater depths than many hydraulic dredges. Most of the contaminant losses during bucket dredging occur during the impact, penetration, and removal of the bucket from the sediment (Hayes, McLellan, and Truitt 1988). Significant losses also occur during hoisting through the water column and after the bucket breaks the surface due to drainage from the bucket. Palermo, Homziak, and Teeter (1990) estimated that 20 to 30 percent of the sediment excavated from a clay and silt bed was spilled from a clamshell bucket before reaching the disposal scow. These losses can be minimized through operational controls and the use of enclosed buckets (Barnard 1978). Operational controls include smooth hoisting of the bucket and use of a hoisting speed less than 2 m/sec (McLellan et al. 1989).

Other mechanical dredges include the backhoe, bucket ladder and wheel, dipper, and dragline dredges. All of these dredges are expected to increase the amount of resuspended sediment over the bucket dredge (Averett et al. 1990). The backhoe, bucket ladder and wheel, and dragline dredges are not appropriate for remediation dredging, which is the focus of this chapter.

Particulate Contaminant Releases During Dredging

The discussion below will outline procedures for estimating contaminant losses from those dredges for which predictive techniques have been

proposed, specifically cutterhead and bucket dredges. These predictive techniques are based on limited studies and, therefore, not fully developed nor verified. Because additional studies involving a wide range of dredging conditions are needed, the proposed predictive techniques for losses during dredging should be regarded as unproven techniques requiring additional research and development.

An alternative approach is use of the sediment resuspension information compiled in Table 2 (from Nakai 1978 as cited by Herbich and Brahme 1991). This information provides rough guidelines for estimating resuspension rates by cutterhead and bucket type dredges. The guidance provides insufficient information, however, to indicate the effect of operational controls or the influence of different types of sediments.

Table 2
TGU's[1] for Different Dredges and Dredging Projects (Nakai 1978)[2]

Type of Dredge	Installed Power or Bucket Volume	Dredged Material			TGU kg/m³
		(d <74μ, %)[3]	d <5μ, %	Classification[4]	
Pump	4,000 hp	99.0	40.0	Silty clay	5.3
		98.5	36.0	Silty clay	22.5
		99.0	47.5	Clay	36.4
		31.8	11.4	Sandy loam	1.4
		69.2	35.4	Clay	45.2
		74.5	50.5	Sandy loam	12.1
	2,500 hp	94.4	34.5	Silty clay	9.9
		3.0	3.0	Sand	0.2
	2,000 hp	2.5	1.5	Sand	3.0
		8.0	2.0	Sand	0.1
Trailing suction	2,400 hp	92.0	20.7	Silty clay loam	7.1
	x 2	88.1	19.4	Silty loam	12.1
	1,800 hp	83.2	33.4	Silt	25.2
Grab	8 cu m	58.0	34.6	Silty clay	89.0
	4 cu m	54.8	41.2	Clay	84.2
		45.0	3.5	Silty loam	15.8
	3 cu m	62.0	5.5	Silty loam	11.9
		87.5	6.0	Silty loam	17.1
Bucket		10.2	1.5	Sand	17.6
		27.2	12.5	Sandy loam	55.8

[1] TGU = kilograms of suspended sediment per cubic meter material dredged.
[2] Nakai (1978) as cited by Herbich and Brahme (1991).
[3] d = diameter of soil particles.
[4] Classification is according to the triangular soil classification system.

General considerations

Resuspension of particulates is a function of dredge type and operation and sediment properties. The effects of operational factors on resuspension for selected dredges will be reviewed here. Sediment properties are a site-specific concern that cannot be definitively quantified without reference to a specific dredging project. In general, finer, less cohesive sediments have the greatest potential for resuspension.

Contaminants associated with resuspended particulates are primarily metals and other elemental species and organic contaminants. Elemental species of concern may be in geochemical phases with slow release properties or in geochemical phases that readily accept and release elemental species. Organic contaminants are usually bound in the organic fraction of the sediment through reversible sorption reactions. Contaminant species may also be dissolved in the pore water adjacent to the sediment particles; but for most contaminants, the dissolved fraction is much smaller than the particulate fraction.

The mass release of a contaminant during dredging is defined by

$$m = f_r \, \rho_s \, A \, D \, C_s \tag{1}$$

where

m = contaminant mass released, g

f_r = fraction of sediment resuspended during dredging, dimensionless

ρ_s = in situ bulk density of the sediment, g/cm^3

A = dredging area available for mass transfer, cm^2

D = dredging depth, cm

C_s = contaminant concentration in sediment (dry wt), g/g

Equation 1 is useful as a definition, but it is not as a predictive equation because the fraction of sediment resuspended is difficult to estimate and mass release is more conveniently expressed on a rate basis. To obtain the rate of mass release, the dredging area, A, is replaced with A_d, the area of dredging per unit time (square centimeters per second) and m becomes R_D, the mass of contaminant released per unit time (grams per second). Alternatively, if an average water column resuspended solid concentration is known over some volume, the rate of contaminant resuspension, R_D, is given by

$$R_D = C_p \, Q_d \, C_s \tag{2}$$

where

R_D = rate of contaminant release, g/sec

C_p = suspended solids concentration averaged over a characteristic volume at point of dredging, g/cm^3

Q_d = volumetric flow of water through averaging volume, cm^3/sec

Figure 2 shows a definition sketch for Equation 2. It should be noted that the bulk sediment contaminant concentration is generally reported as mass of contaminant per mass of dry sediment and implicitly assumes that all the contaminant mass resides on the solid phase. The contaminant release rate defined in Equation 2 is based on the total contaminant concentration initially in the in situ sediment and, therefore, includes both particulate and dissolved contaminant fractions.

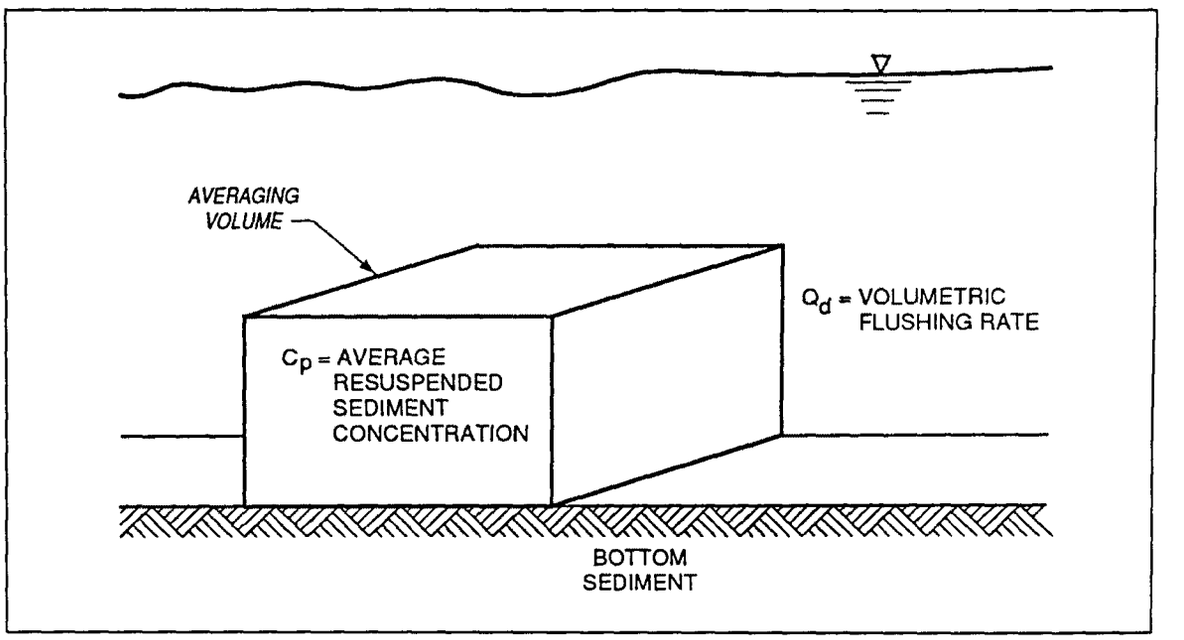


Figure 2. Definition sketch for contaminant release at point of dredging

Estimation of the total contaminant release or the release rate per unit time by resuspension of the sediment is thus reduced to estimation of the fraction of particles that are resuspended. The rate of sediment resuspension is discussed for cutterhead hydraulic and bucket dredges in the sections that follow. The dissolved fraction of the total contaminant loss will be discussed in a later section.

Cutterhead dredges

Cutterhead dredges loosen the bottom sediment by the mechanical action of the multiblade rotating cutterhead. The sediment dislodged in this manner is drawn via hydraulic suction into the suction pipe and transported to the disposal site by pipeline. Particulate contaminant release occurs during this process when the hydraulic suction is unable to completely entrain all of the dislodged sediment.

A controlled study of particulate releases by a cutterhead dredge was conducted by Hayes, McLellan, and Truitt (1988) at Calumet Harbor, Illinois. Key operational parameters that affected sediment resuspension rates were the rotation rate of the cutterhead and swing speed of the cutterhead ladder on which the cutterhead is supported. Overcutting, when the sense of rotation of the cutterhead and the ladder are the same (Figure 3), resulted in higher rates of sediment resuspension. During overcutting, the shear of the cutterhead relative to the water is greatest when the cutterhead is at the top of its rotation, resulting in more resuspension of dredged material on the cutterhead. As shown in Figure 3, undercutting (which occurs when the sense of cutterhead at the top of its rotation and ladder swing differ) reduces sediment resuspension. This explanation of cutterhead resuspension is consistent with the concept that the tangential velocity of the cutterhead relative to the essentially motionless water is the primary factor in sediment resuspension.

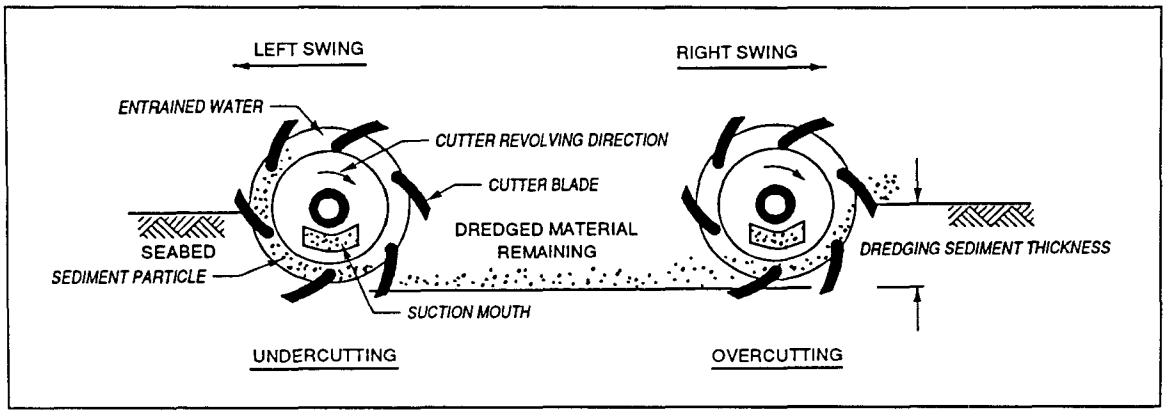


Figure 3. Cutting operation of a cutterhead dredge (front view)

During overcutting, the effective blade velocity is the sum of the tangential velocity of the cutterhead blades about their axis of rotation and the swing velocity of the dredge, that is, the velocity of the ladder with respect to the dredge. During undercutting, the effective blade velocity is the difference between the two velocities. Additional factors controlling cutterhead resuspension include the degree of head burial in the sediment and the characteristic velocity of the cutterhead intake. Increases in the intake velocity reduce the fraction of the particles that are resuspended by the cutterhead but not removed by the hydraulic suction.

Fully buried cutterheads reduce the exposure of the loosened sediments to the overlying water and therefore increase the fraction that are removed by the hydraulic suction.

Hayes (1986) correlated resuspended sediment concentrations to powers of the dimensionless ratios of swing velocity-to-characteristic intake velocity and cutterhead tangential velocity-to-characteristic intake velocity. Collins (1989) restated this correlation in the manner given in Equation 3.

$$\frac{C_p}{\rho_w} = F_F F_D \left[\frac{V_s}{V_i}\right]^a \left[\frac{V_c}{V_i}\right]^b \tag{3}$$

where

C_p = suspended solids concentration averaged over a characteristic volume at point of dredging, g/cm^3

ρ_W = density of water, g/cm^3

F_F = coefficient for all factors other than degree of burial, dimensionless

F_D = cutterhead resuspension rate factor accounting for degree of burial, dimensionless

V_s = swing velocity of dredge ladder, cm/sec

V_i = characteristic velocity of cutterhead intake, cm/sec

a = empirical swing velocity significance factor, dimensionless

V_c = effective blade velocity, cm/sec

b = empirical tangential velocity significance factor, dimensionless

F_D = 1 for full cut (fully buried cutterhead) dredging and would be greater than 1 for partially buried dredging. F_F is a site-specific factor that accounts for sediment and dredge operational variations. Based on 12 data sets with a fully buried cutterhead at Calumet Harbor, Illinois, Hayes (1986) found that a = 2.85, b = 1.02, and F_F = 0.089 with a correlation coefficient of 0.72.

Collins (1989) extended this correlation for Calumet Harbor to other sites and dredging conditions. Using data collected during cutterhead dredging operations at Calumet Harbor, Illinois, Savannah River, South Carolina, and James River, Virginia, Collins (1989) developed the following predictive equations for the factors F_F and F_D.

$$\log_{10}F_F = 10^{(10^{-4}(D_{ch}/d)/13.32)^{7.04}} - 2.05 \tag{4}$$

$$F_D = 1 + 1.9(D_F-1)^2 + 0.41(D_F-1)^7 \tag{5}$$

where

D_{ch} = diameter of cutterhead, cm

d = effective diameter of sediment grains, cm

D_F = fractional depth of cut as a function of cutterhead diameter, dimensionless

Equation 5 is a modification of Equation 30 in the report of Collins (1989) to reflect the fact that $F_D = 1$ for $D_F = 1$. Using all of the data for the three sites, the correlation coefficient for the model is 0.556.

Use of the correlation given by Equation 4 is hindered by its sensitivity to the ratio of the cutterhead diameter to the sediment grain-size diameter. Changes in the average grain size of less than a factor of two can result in a change in the factor, F_F, by more than an order of magnitude. Extreme caution is warranted in the use of Equation 4. If sediment resuspension rates estimated using Equation 4 differ by more than a factor of 10 from the approximate estimates of Nakai (1978) (Table 2), the approximate estimates given in Table 2 should be used.

Equations 3 through 5 provide an estimate of the resuspended sediment concentration, C_p, a variable in Equation 2. In addition to the resuspended sediment concentration, the volumetric flow of water through the characteristic volume over which the resuspended solids concentration is averaged is required. Collins (1989) approached obtaining the needed volumetric flow by defining the characteristic volume for averaging as equal to the tangential velocity of the cutting head relative to fixed coordinates (V_t) times the cross-sectional area to which this velocity applies (Figure 2). Taking the height of the cutting head as H_{ch} and its length as L_{ch}, the cross-sectional area is αH_{ch} βL_{ch} where α and β account for fact that the sweep volume is typically larger than the cutterhead. Collins' (1989) estimates of this volume are equivalent to the values, $\alpha = 1.75$ and $\beta = 1.25$. Additional field tests over a wide range of dredging conditions will be needed before the general applicability of the α and β values listed above can be fully evaluated.

Using the approach suggested by Collins (1989) for obtaining the volumetric flow, the contaminant mass release rate equation (Equation 2) can be written as

$$R_{D,ch} = C_p \; V_t \; \alpha H_{ch} \; \beta L_{ch} C_s \tag{6}$$

Despite its weaknesses, Equation 6 is the only equation presently available for predicting contaminant release during cutterhead hydraulic dredging.

An alternative approach is the use of the sediment resuspension rates observed by Nakai (1978) and summarized in Table 2. The tabulated resuspension rates could be used to indicate an approximate prediction and then Equations 3 and 5 used to indicate the influence of cutterhead speed, swing speed, and fractional depth of cut. This method is suggested under conditions when Equation 4 indicates extreme sensitivity to sediment grain size.

Bucket dredges

Different types of bucket dredges can fulfill various types of dredging requirements. Typical buckets include the clamshell, orange-peel, and dragline types. This discussion will focus on the clamshell type of bucket dredge. Sediment is resuspended during bucket dredging operations by impact, penetration, and withdrawal of the bucket and during hoisting of the bucket. Bucket dredges usually excavate a heaped bucket of material, but, during hoisting, a portion of the load washes away. Once the bucket clears the water surface, additional losses may occur through rapid drainage of water and slumping of the material heaped above the rim. Loss of material is also influenced by the fit and condition of the bucket, the hoisting speed, and the properties of the sediment.

A special type of bucket, the enclosed clamshell bucket, has been developed to minimize loss of dredged material. The edges seal when the enclosed clamshell bucket is closed, and the top is covered. A comparison of conventional clamshell and enclosed clamshell bucket dredging operations indicated that the enclosed clamshell generates 30 to 70 percent less turbidity in the water column than typical buckets (Barnard 1978).

The key parameters affecting total resuspension rate are bucket size, cycle time, and type of bucket. The cycle time, or the time required to drop, fill, and withdraw the bucket, is a function of the rate of each of the individual steps (impact, penetration, withdrawal, and hoisting). The speed at which these steps are accomplished significantly influences sediment resuspension rates.

A dimensionless parameter that scales with the bucket volume is defined by Collins (1989) as

$$B = \frac{1}{h_b}\left[2\,V_{cb}\right]^{1/3} = \frac{L_{bc}}{h_b} \tag{7}$$

where

B = Collins bucket parameter, dimensionless

h_b = water depth, cm

V_{cb} = volume of clamshell bucket, cm^3

L_{bc} = characteristic length of clamshell bucket, cm

The term in brackets in Equation 7 is the characteristic size of the clamshell bucket recognizing that the bucket is approximately square on two sides and triangular on the third.

Collins (1989) defines a dimensionless cycle time T_c as

$$T_c = \frac{v_3 \tau_{cb}}{h_b} \tag{8}$$

where

T_c = dimensionless cycle time

v_3 = settling velocity of a representative particle, cm/sec

τ_{cb} = bucket cycle time, sec

h_b/v_3 is the time required for a representative sediment particle to fall over the entire depth of the water column.

Unfortunately, insufficient data exist to relate resuspension or contaminant release rates with both B and T_c. Collins (1989) used the ratio of T_c to B to define a new dimensionless variable as the ratio of the bucket cycle time to the time required for particles to settle the bucket distance.

$$\frac{T_c}{B} = \frac{v_3 \tau_{cb}}{L_{bc}} \tag{9}$$

A regression analysis of the resuspended sediment concentrations for experiments at St. John River, Black Rock Harbor, and Calumet River (Collins 1989) suggested the correlation

$$C_p = 0.0023\ \rho_w \left[\frac{B}{T_C}\right]^3 \tag{10}$$

where C_p is the resuspended sediment concentration, gm/cm^3.

Collins (1989) reports that the logarithmic equivalent of Equation 10 has a correlation coefficient of about 0.98.

Estimation of the release rate requires that the concentration estimated from Equation 10 be multiplied by the exchange rate of the volume swept by the bucket. The volumetric sweep rate of the bucket should be proportional to the square of the characteristic clamshell bucket length times the effective velocity of the bucket. As with the cutterhead dredge, the area swept by the bucket during insertion and withdrawal exceeds the bucket area. Bohlen (1978) suggests that the sweep area is approximately two to three times the area of the bucket. The effective velocity of the bucket is approximately h_b/τ_{cb}. If the concentration predicted by Equation 10 applies throughout the sweep area and dredging cycle, then the particle resuspension rate is given by

$$R_{p,b} = \gamma \frac{(L_{bc})^2 h_b}{\tau_{cb}} C_p = \gamma \rho_w (0.0023) (L_{bc})^2 \frac{h_b}{\tau_{cb}} \left[\frac{B}{T_c} \right]^3 \tag{11}$$

where γ is the Bohlen sweep area correction factor (2 to 4), dimensionless.

The only equation presently available for predicting the solids resuspension rate during bucket dredging is Equation 11. Contaminant release rate is given by modifying Equation 11 to include the concentration of contaminant in the sediment, C_s, as shown in Equation 12 below.

$$R_{D,b} = \gamma \frac{(L_{bc})^2 h_b}{\tau_{cb}} C_p C_s = \gamma \rho_w (0.0023) (L_{bc})^2 \frac{h_b}{\tau_{cb}} \left[\frac{B}{T_c} \right]^3 C_s \tag{12}$$

As indicated previously, an enclosed clamshell dredge should reduce the contaminant release rate predicted by Equation 12 by 30 to 70 percent.

An alternative approach is the use of the sediment resuspension rates observed by Nakai (1978) (Table 2). The high correlation coefficient of Equation 10 suggests, however, that the approach of Equations 7-12 is the best estimate available.

Dissolved Contaminant Releases During Dredging

Resuspension of sediment solids during dredging can also impact water quality through the release of contaminants in dissolved form. Before resuspension, contaminant distribution between sediment solids and sediment pore water is probably at equilibrium. Dredging exposes sediments to major shifts in liquids/solids ratio and oxidation-reduction potential (redox). Because the

sediment solids are removed from the equilibrium conditions previously existing, there is a potential for change in the distribution of contaminant between solid and aqueous phases. Initially upon resuspension, the bulk of the contaminants are sorbed to particulate matter. As the resuspended particulate concentration is diluted by mixing with dredging site water, release of sorbed contaminants to adjacent waters results in a continuous increase in the fraction of contaminants that are dissolved.

It should be noted that the total release of contaminants at the point of dredging is estimated by the equations of the previous section. The dissolved release calculated by the methods of this section largely occurs after the mixing and dilution of the resuspended sediments with the ambient waters. The fraction of the contaminant associated with the particulate phase continues to change as dilution reduces the particle concentration.

In this section, equilibrium partitioning is discussed as a predictive technique for dissolved organic contaminants. Because equilibrium partitioning of organic contaminants is discussed in detail in Contaminant Losses During Pretreatment in the section on leachate quality, details of equilibrium partitioning theory are not presented in this section. A pseudo-equilibrium partitioning approach for estimating dissolved metals concentrations is discussed in Contaminant Losses During Pretreatment, but this approach is not recommended for application to release of dissolved metals during dredging because the rapid and pronounced change in redox and the complicated environmental chemistry of metals make equilibrium approaches highly unreliable and uncertain.

The most accurate predictive indicator of dissolved contaminant release during dredging would be a fully researched and developed laboratory test that reproduces the mixing and dilution processes that are observed in the water column after resuspension of contaminated sediments. Such a test would indicate sediment-specific effects on desorption rate and contaminant tendency to desorb. The test would be especially important for elemental species, such as heavy metals, that undergo complex reactions that are not easily predicted by mathematical models. The test would also be important for strongly sorbed hydrophobic organic species that may desorb slowly due to mass transfer resistances.

DiGiano, Miller, and Yoon (1995) proposed an adaptation of the standard elutriate test, a dredging elutriate test (DRET), for the purpose of predicting dissolved contaminant releases. The DRET is preliminary (only one sediment tested) and requires further development before a test of this type can be adopted for routine application. The standard elutriate test (SET) was developed during the DMRP to predict contaminant release during open-water disposal operations (Jones and Lee 1978). In the SET, water and sediment are mixed in a proportion of 4:1, mixed for 30 min and allowed to settle for 1 hr. The modifications suggested by DiGiano, Miller, and Yoon (1995) were designed to achieve a more realistic solids/water ratio (0.5 to 10 g/ℓ) consistent with conditions for resuspended sediment due to dredging. DiGiano,

Miller, and Yoon (1993) employed an aerated mixing time of 1 to 6 hr and a settling time of 1 hr (0.5 to 24 hr were also investigated).

The DRET was evaluated by comparison to field dredging studies conducted in New Bedford Harbor, Massachusetts. The DRET was found to be a reasonable indicator of the soluble and total (soluble plus unsettled particulate) polychlorinated biphenyl (PCB) concentrations released during cutterhead or matchbox suction dredging but underpredicted PCB concentrations when a horizontal auger dredge head was used. Additional testing of DRET at a number of sites is needed before the general applicability of the test can be evaluated. The New Bedford Harbor studies involved a highly contaminated sediment at an estuarine location. Extrapolation of the New Bedford Harbor results to freshwater sites with one to two orders of magnitude lower contamination levels is not technically defensible at this time.

In the absence of specific information to the contrary, it, therefore, seems appropriate to use equilibrium partitioning to establish an upper bound on dissolved organic concentrations at the point of dredging. However, equilibrium partitioning is usually a very conservative assumption. DiGiano, Miller, and Yoon (1990) found that an equilibrium partitioning model did a good job of predicting the soluble PCB concentrations. At low contaminant concentrations, equilibrium partitioning between sediment and water can usually be represented by a linear isotherm, that is, $C_{sorb} = K_d C_w$, where K_d is a distribution coefficient assumed independent of concentration. Here, C_w is the water phase concentration and C_{sorb} is the concentration of the contaminant sorbed to the solid phase. The sorbed concentration in the solid phase is usually assumed to be approximately equal to the bulk sediment contaminant concentration C_s, so that, $C_{sorb} \approx C_s$.

Using local equilibrium partitioning, the dissolved concentration is given by

$$C_w = \frac{C_s \, C_p}{K_d \, C_p + 1} \tag{13}$$

where

C_w = aqueous phase contaminant concentration, mg/ℓ

C_s = bulk contaminant concentration in sediment, mg/kg

C_p = suspended solids concentration averaged over a characteristic volume at point of dredging, kg/ℓ

K_d = contaminant-specific equilibrium distribution coefficient, ℓ/kg

The distribution coefficient in Equation 13 can be determined in batch equilibrium tests or estimated using empirical relationships from the literature.

Procedures for measuring or estimating the distribution coefficient are described in Appendix B.

The release rate for dissolved contaminants is the product of the dissolved contaminant concentration averaged over the volume swept by the dredge and the volumetric flow through the averaging volume. The dissolved contaminant release rate for a cutterhead dredge is thus given by

$$R_{d,ch} = C_w \; V_t \; \alpha H_{ch} \; \beta L_{ch} \tag{14}$$

Similarly, the dissolved contaminant release rate for a clamshell bucket dredge is given by

$$R_{d,b} = \gamma \rho_w \; (L_{bc})^2 \frac{h_b}{\tau_{cb}} \; C_w \tag{15}$$

Several limitations apply to Equations 14 and 15. First, there are little field data for verification of these equations. Second, Equations 14 and 15 are not applicable to estimation of dissolved metals releases. In addition, the linear partitioning used in Equations 14 and 15 assumes dissolved phase concentrations much lower than the water solubility limit. Deviations from linear partitioning might be expected when dissolved phase concentrations approach 50 percent of the solubility limit.

Further, the total contaminant release for cutterhead hydraulic and bucket dredges is provided by Equations 6 and 12, respectively. Although dissolved losses at the point of dredging represent a small fraction of the total loss for strongly sorbing chemicals, some estimation of dissolved losses, such as provided in Equations 14 and 15, may be needed for transport models used to assess impacts and risks and to compare the no-action alternative to dredging and treatment/disposal alternatives. Finally, Equations 14 and 15 predict dissolved concentrations at the point of dredging, not downstream dissolved concentrations.

Although hydrophobic organic species often partition in the simple manner discussed previously, the release of metals is much more complex. During the development of the standard elutriate test, there was little correlation observed between sediment bulk metal concentration and the dissolved metal concentration at disposal sites or in the standard elutriate. In most cases, dissolved metal concentrations in site water prior to and during disposal operations were about the same (Jones and Lee 1978). In some cases, dissolved metal concentrations were higher in site water prior to disposal operation than after disposal operations (Jones and Lee 1978). These results can often be explained in terms of the aqueous environmental chemistry of iron. Many sediments contain a large reservoir of reactive ferrous iron that readily reacts with oxygen in site water to form amorphorous iron oxyhydroxides. Iron oxyhydroxides tend to floc and scavenge metals. Thus, an adaptation of the

SET such as DRET is probably required to get reliable estimates of soluble metal releases during dredging.

Closure on Losses During Dredging

It is clear from the previous discussion of losses during dredging that a number of dredging equipment factors and interactions between sediment and water are likely to be important in predicting contaminant losses. Prediction, however, requires simplifying assumptions about the relative importance of these factors and interactions, followed by major extrapolations about the complex and transient conditions of the field environment. Field measurements of resuspension and desorption at the point of dredging supported by data on operational factors and ambient conditions are, therefore, essential to better understanding of contaminant release rates at the point of dredging. The number of such studies is rather limited. They are complex and expensive, involving major investments in equipment (dredges) and chemical analyses. It is important, therefore, that future studies be designed to provide the maximum amount of information on relevant factors and interactions.

The predictive equations presented in this section may at first glance seem straightforward and easy to apply. For many of the variables in the equations, however, there is little guidance on selection of appropriate values. Application of these equations will necessarily involve judgment that can only be applied on a case-by-case basis.

3 Contaminant Losses During Dredged Material Transport

Background

This section is concerned with contaminant losses during transportation of dredged material. Transportation methods include pipelines, scows, barges, and hoppers. Trucks and railroad cars are rarely used. Hopper dredge transport with direct pumpout is often used in the Great Lakes, but is not the most common form of dredged material transport.

Losses during transport are easier to control than to predict. Transportation losses are largely due to accidental spills and leaks, events which are very difficult, if not impossible, to predict. Controls as discussed by Cullinane et al. (1986) can significantly reduce these losses. Controls are briefly mentioned for each form of dredged material transport discussed below.

Spills and leaks account for all the particulate and dissolved contaminant losses and a portion of the volatile losses during dredged material transport. Volatile losses can be predicted and to some extent controlled. The predictive techniques discussed in this chapter are, therefore, limited to volatile losses. Prediction of particulate and dissolved contaminant losses through spills and leaks is discussed, but no predictive techniques are available.

Losses During Pipeline Transport

Pipeline operations keep dredged material in a closed system until delivered to a destination. Pipeline operations, therefore, offer the potential for zero losses during transport of dredged material. However, accidental releases through pipeline failures and leaks can occur. In addition, dredge pump outages due to damage by objects entrained in the suction (nuts, bolts, chain, cables, rocks, etc.) can result in clogged pipelines that have to be disassembled and cleaned. During disassembly and cleaning, losses can occur. Since pump outages and pipeline failures and leaks are unpredictable, there are no a priori techniques for predicting contaminant losses during pipeline

transport of dredged material. Ideally, the losses during pipeline transport should be zero, but in reality losses are never zero.

During the design stage, planners should carefully consider pipeline routes, climatic conditions expected, material's corrosion resistance, redundancy of safety devices (i.e., additional shutoff valves, loop/by-passes, pressure relief valves), coupling methods, and systems to detect leaks.

Losses During Scow, Barge, and Hopper Transport

Contaminant losses from scows, hoppers, and barges can occur via resuspension of sediment as a result of spillage, overflow, and volatilization. The manner in which dissolved and particulate contaminants are lost depends on the type of dredging operation (mechanical or hydraulic) used to fill the transport vessel. Volatile contaminant releases, which also depend on the type of dredging, are discussed in a later section.

Material condition prior to placement into a scow or barge has a great impact on what controls planners must consider. Dredged material that has a high moisture content will require less concern about possible windblown dust, but will create much more difficult loading and unloading conditions and will require a greater number of barges. In general, lower material moisture content is better for handling and control. For purposes of discussing control mechanisms in barge transport, the dredged material will be assumed to be in one of two states: freshly dredged material, having a very high water content and being transported a short distance to an unloading site, or consolidated (dewatered) dredged material to be barge transported over long distances.

Bucket operations

Since bucket dredging produces dredged material at close to in situ densities, overflow from a scow or barge can be controlled such that overflow losses are negligible during transportation. Loading and unloading probably presents the greatest potential for uncontrolled contaminant releases during bucket operations. At loading and unloading points, spillage directly in the water can occur during boom swing between the transport vessel and the delivery point. Controls can be implemented to significantly reduce or eliminate this type of loss. Techniques for predicting contaminant losses during unloading operations using buckets are not available.

When volatile or semivolatile contaminants are present in the dredge material, open-top transport vessels are a continuous source of volatile emissions until emptied. Volatilization rates from open-top vessels depend on sediment volatile chemical concentrations, wind speed, area of exposed dredged material, and physical/chemical properties of the contaminants. Predictive techniques for volatile losses from mechanically dredged sediment during transport in open-top vessels are described in a later section.

Hydraulic operations

Scows, barges, and hoppers can be loaded hydraulically as well as mechanically. In addition, mechanically loaded vessels can be hydraulically unloaded. Since hydraulic unloading involves pipelines, losses during hydraulic unloading are similar to pipeline losses, that is, uncontrolled spills and leaks are the major contaminant loss mechanisms. As previously mentioned, open-top transport vessels may be a continuous source of volatile emissions until emptied. Predictive techniques for volatile losses from hydraulically dredged sediment during transport in open-top vessels are described in a later section.

Hopper dredges are sometimes allowed to pump past overflow in order to achieve better dredging economics by trapping heavy, coarse-grained materials and releasing light, fine-grained materials. Hopper overflow is a major source of contaminant reentry into the environment because contaminants preferentially bind to fine-grain materials. Losses during hopper overflow were not considered in Chapter 2 and are not considered in this chapter because it makes little sense to remove contaminated sediment for purposes of remediation and then put the fine-grained fraction (the fraction containing most of the contaminants) right back in the water. Hopper dredging is a dredging option that should be considered for remediation, but overflow is not recommended.

Contaminant losses during direct hopper pumpout to treatment or disposal facilities are essentially the same as pipeline losses previously discussed. The major difference is that the pipeline distance for direct hopper pumpout is significantly less than the distances normally used in hydraulic dredge pipeline operations. The potential for spills and leaks during transportation is, therefore, less for hopper dredging than for pipeline dredging.

Losses During Truck and Rail Transport

Truck and rail transport, not often used to transport dredged material during navigation dredging, has a higher probability of being used in the transport of contaminated dredged material during remedial operations. Truck and/or rail transport may be needed when the destination is not accessible by water or the transportation distance is longer than the range normally used for overland pipelines. The types of losses for truck and rail transport include spillage during loading and unloading operations, spills and leaks during hauling, and volatile emissions throughout the entire cycle of loading, hauling, and unloading. Accidental spills and leaks are unpredictable losses that can be controlled by proper planning. Volatile emissions from open-top trucks and rail cars are predictable and to some extent controllable. Predictive techniques for volatile losses from open-top vessels are discussed in a later section.

Loading and unloading operations probably present the greatest potential for contaminant loss when using truck or rail transport. During loading and

unloading operations involving buckets, conveyer belts, and slides, there will be some spillage of dredged material. Loading and unloading sites will become contaminated by spilled materials unless lined. Undercarriage washing to prevent contaminated sediment from falling on roadways and railways will generate rinse water that may require treatment. Truck and railcar cleaning will also result in wastewater that may require treatment. Treatment process trains likely to be considered for these wastewaters include sedimentation, clarification, carbon adsorption, and biological treatment. Discussion of losses from treatment processes are covered later in this report.

Regardless of loading method, there will be some spillage of contaminated materials. Controls suggested for consideration are as follows (Cullinane et al. 1986):

a. Drainage of water from loading and unloading area into central sump for periodic removal.

b. Daily removal of spilled material.

c. Specially designed loading ramps to collect spilled material.

d. Use of watertight clamshells for transferring materials from barges into truck.

Volatile Losses During Dredged Material Transport

Volatilization processes differ for exposed sediment solids and sediment solids covered by water (Thibodeaux 1989). In vessels filled hydraulically (scows, barges, hoppers), dredged material solids will be covered by water. In open-top vessels filled mechanically (scows, barges, trucks, and railroad cars), the dredged material solids may be exposed. Two predictive techniques, one for exposed dredged material solids and one for dredged material solids covered by water, are discussed in this section.

Volatilization of chemicals from open-top vessels during dredged material transportation is essentially independent of vessel type. The surface area of the vessel, however, is important because volatile emission rates depend on surface area.

Mechanically dredged sediment

Contaminated sediment that is wet and exposed directly to the atmosphere is the case that results in the highest instantaneous volatile fluxes because the pathway for loss is very short (Thibodeaux 1989). The water film covering exposed solids is very thin and provides little resistance to mass transfer across the solids-air interface. Thus, most of the resistance to mass transfer resides in the air side of the solids-air interface. Assuming negligible

resistance to mass transfer on the sediment side (including water films on sediment solids), the volatilization rate from an open-top vessel containing mechanically dredged sediment is given by (Thibodeaux 1989)

$$R_{V,es} = K_{OG} A_v \rho_1 \left[\frac{p_A^* - p_A}{P} \right] \tag{16}$$

where

$R_{V,es}$ = volatile emission rate for chemical A from exposed sediment, g/cm^2 sec

K_{OG} = overall gas-side mass transfer coefficient, cm/sec

A_v = surface area of vessel, cm^2

ρ_1 = density of air, g/cm^3

p_A^* = partial pressure of chemical A in air that would be in equilibrium with dredged material, mm Hg

p_A = background partial pressure of chemical A in air, mm Hg

P = total atmospheric pressure, mm Hg

Equation 16 is Equation 15 from Thibodeaux (1989) written in terms of partial pressures. The driving force modeled by Equation 16 is the difference between the partial pressure of a chemical in the air immediately adjacent to contaminated dredged material and the partial pressure of the chemical in the background air. The driving force is maximized when the partial pressure in the air at the contaminated solids-air interface is maximized. The maximum partial pressure in the air at the contaminated solids-air interface is the equilibrium partial pressure, p^*_A. Generally, p^*_A can be determined by Henry's Law partitioning between dissolved concentrations in the dredged material pore water and air as follows (Thibodeaux 1979):

$$p_A^* = H C_w \approx H \frac{C_s}{K_d} \tag{17}$$

where H equals Henry's constant, mm Hg ℓ/mg.

If the sediment surface is approximately flat, turbulent boundary layer theory suggests that overall gas-side mass transfer coefficient is given by

$$K_{OG} = 0.036 \left[\frac{D_{Al}}{L_v}\right] \left[\frac{L_v V_x}{\nu_1}\right]^{0.8} \left[\frac{\nu_1}{D_{Al}}\right]^{0.33} \tag{18}$$

where

D_{Al} = molecular diffusivity of chemical A in air, cm^2/sec

L_v = vessel length, cm

V_x = background wind speed, cm/sec

ν_1 = kinematic viscosity of air, cm^2/sec

Uneven surfaces will tend to increase K_{OG} if the surface roughness occurs within the boundary layer. Large mounds increase surface area and shade downwind areas (decrease effective surface area), neither of which is a term in Equation 18.

For long transportation times or for long-term storage before disposal, the surface of the dredged material will lose both water and contaminant, and volatilization will slow due to the development of internal mass transfer resistances. Procedures for estimating contaminant volatilization from exposed dredged material with internal resistances is discussed in Chapter 4.

The equation for estimating volatile losses when open-top vessels are partially filled (loading and unloading) is given below (Thibodeaux 1989)

$$R_{V,es_p} = \left[\frac{2D_v - Z}{2D_v}\right] R_{V,es} \tag{19}$$

where

D_v = effective diameter of vessel, cm

Z = distance from top of vessel to exposed dredged material surface, cm

The term in parenthesis in Equation 19 accounts for the exposed surface being a distance Z below the top of a vessel with an effective diameter D_v.

Hydraulically dredged sediment

When hydraulically dredged sediment is placed in open-top vessels for transportation to a destination, the dredged material solids will tend to settle. Volatilization during transportation of hydraulically dredged sediment in open-top vessels, therefore, takes place at an air-water interface. Volatilization

from water surfaces is discussed in Contaminant Losses During Pretreatment in the section on volatile releases from ponded water.

4 Contaminant Losses During Pretreatment

Background

Pretreatment as used in this report is the processing of dredged material for additional treatment or disposal. Dredged material slurries produced by hydraulic dredging may require pretreatment to increase the solids content when treatment technologies designed for low moisture soil, such as thermal technologies, are to be used (Averett et al. 1990). Because the rate of dredged material removal and transportation is usually irregular, flow equalization may also be necessary before initiating treatment. Flow equalization facilities can also serve as a convenient point for dewatering by primary settling.

Averett et al. (1990) surveyed the applicability of the pretreatment processes shown in Table 3 to dredged material. This chapter discusses contaminant losses from primary settling and flow equalization facilities.

Table 3
Process Options for the Pretreatment Component[1]

Dewatering	Particle Classification	Slurry Injection
Belt filter press	Flotation	Chemical Clarification
Carver-Greenfield evaporation	Grizzlies	Microbe addition
Centrifugation	Heavy media separation	Nutrient addition
Chamber filtration	Hydraulic classifiers	
Evaporation	Hydrocyclones	
Gravity thickening	Impoundment basins (CDF)	
Primary settling (CDF)	Magnetic and electrostatic separation	
Solar evaporation		
Subsurface drainage (CDF)	Moving screens	
Surface drainage (CDF)	Shaking tables	
Vacuum filtration	Spiral classifier	
Wick drains (CDF)	Stationary screens	

[1] From Averett et al. (1990).

Losses During Primary Settling and Flow Equalization

Primary settling and flow equalization facilities similar in design and operation to confined disposal facilities (CDFs) (Figure 4) will probably be needed for hydraulically dredged material. Storage facilities similar to CDFs may also be needed to stockpile mechanically dredged material for subsequent treatment. Since primary settling and flow equalization at the beginning of a treatment process train for dredged material will likely be extensions of existing CDF technology, techniques that have been developed for estimating losses from CDFs should be applicable to primary settling and flow equalization facilities.

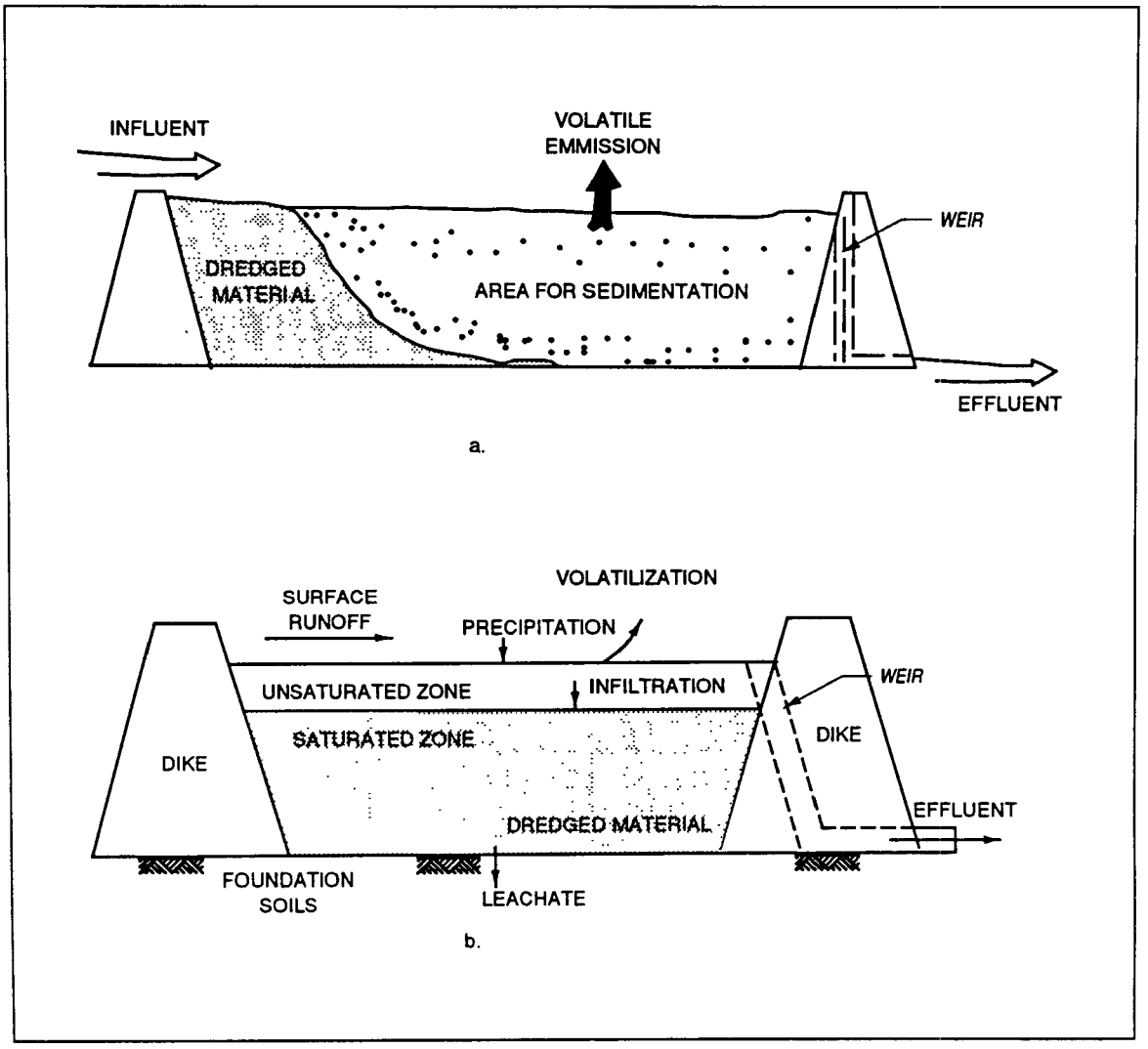


Figure 4. Pretreatment facility schematic with major contaminant migration pathways (a) during filling and (b) filled

As shown in Figure 4, the major contaminant loss pathways for pretreatment facilities are effluent, leachate, runoff, and volatilization. Predictive techniques for estimating contaminant losses along each of these migration pathways are presented in this section. Discussion of laboratory and field data for these migration pathways is presented in Losses From Confined Disposal Facilities on CDF disposal.

Effluent-hydraulic filling

In this section, procedures for estimating effluent contaminant losses during hydraulic filling of primary settling/flow equalization facilities are discussed. Treatment technologies that could be applied to the effluent, such as chemical clarification and carbon adsorption, are discussed in Contaminant Losses During Effluent and Leachate Treatment.

Data requirements for estimating effluent losses during hydraulic filling are listed in Table 4. As indicated in Table 4, information on facility design and influent flow and quality are needed in order to estimate effluent flow and quality.

Table 4
Data Requirements for Predicting Contaminant Losses During Hydraulic Filling[1]

Data Required	Source of Data
Dredge inflow	Project information, site design
Influent solids concentration	Project information
Influent total contaminant concentrations	Bulk chemical analysis of in situ sediment
Average ponding depth	Project information, site design
Hydraulic efficiency factor	Dye tracer study or theoretical retention time
Effluent suspended solids concentration	Column settling tests
Contaminant dissolved concentrations in effluent	Modified elutriate test
Fraction of contaminant in effluent suspended solids	Modified elutriate test

[1] From USACE (1987).

Influent characteristics. The initial step in any dredging activity is to estimate the in situ volume of sediment to be dredged. Sediment quantities are usually determined from channel surveys. Field sampling is required to characterize the sediment and provide material for laboratory testing. Important sediment characteristics that should be determined include water content, grain-size distribution, Atterberg limits, organic content, specific gravity, Unified Soil Classification System (USCS) classification, and bulk chemical concentrations. Although some of this information is not explicitly used to estimate contaminant losses, prediction of effluent quality is based on facility design; most of this information is needed to design a primary settling facility. Palermo, Montgomery, and Poindexter (1978) and USACE (1987) provide

guidance on designing primary settling facilities and the data required for design. Guidance on the collection of sediment samples is provided in the "ARCS Assessment Guidance Document" (USEPA 1994b).

Influent flow is based on dredge production rates. This type of information is usually available in Corps of Engineers District records of dredging activities. If no data are available, hydraulic pipeline dredge production rates can be estimated from relationships among solids output, dredge size, pipeline length, and dredging depth (Palermo, Montgomery, and Poindexter 1978; USACE 1987). Figure 5 shows solids production rates for selected pipeline dredge sizes, pipeline lengths, and dredging depths. For hopper dredges, disposal rate must be estimated from hopper or barge pump-out rate and travel time involved. Site-specific records of previous dredging activities are the best sources for this type of information.

Influent solids concentration will vary with type and size of dredge(s) and in situ sediment concentration. If data from Corps of Engineers dredging records are not available, an influent solids concentration of 145 g/ℓ (13 percent by weight) for hydraulic pipeline dredging can be used (Palermo, Montgomery, and Poindexter 1978). This number is based on a number of field investigations conducted under the DMRP.

Chemical concentrations in the influent can be estimated from bulk chemical analysis of the in situ sediment and solids concentration of the influent. Because site water quality has little effect on influent quality, influent contaminant concentrations usually reflect dilution of in situ sediment bulk chemical concentrations. It is sometimes informative to compare site water quality and effluent quality, but site water quality data are not required for prediction of effluent quality.

Effluent flow. Effluent flow is approximately equal to influent flow during hydraulic filling of sedimentation basins. Initially, there may be some storage of water in facilities with overflow weirs; however, after the head on the weir stabilizes, effluent flow is approximately equal to influent flow.

Effluent quality. Effluent quality during hydraulic filling is predicted on the basis of data from column settling and modified elutriate tests and sedimentation basin design. The modified elutriate test was developed as part of the LEDO research program to simulate the physicochemical conditions in CDFs during hydraulic disposal and involves measurement of both dissolved and total concentrations of contaminants in the elutriate (Palermo 1986). A separate column settling test is used to predict suspended solids concentration in effluent for a specific facility design and set of operational conditions. Results from the modified elutriate and settling column tests are then combined to predict total and dissolved contaminant concentrations in effluent during hydraulic disposal.

The modified elutriate and companion settling tests when used as described by Palermo (1986) account for both dissolved and particulate bound

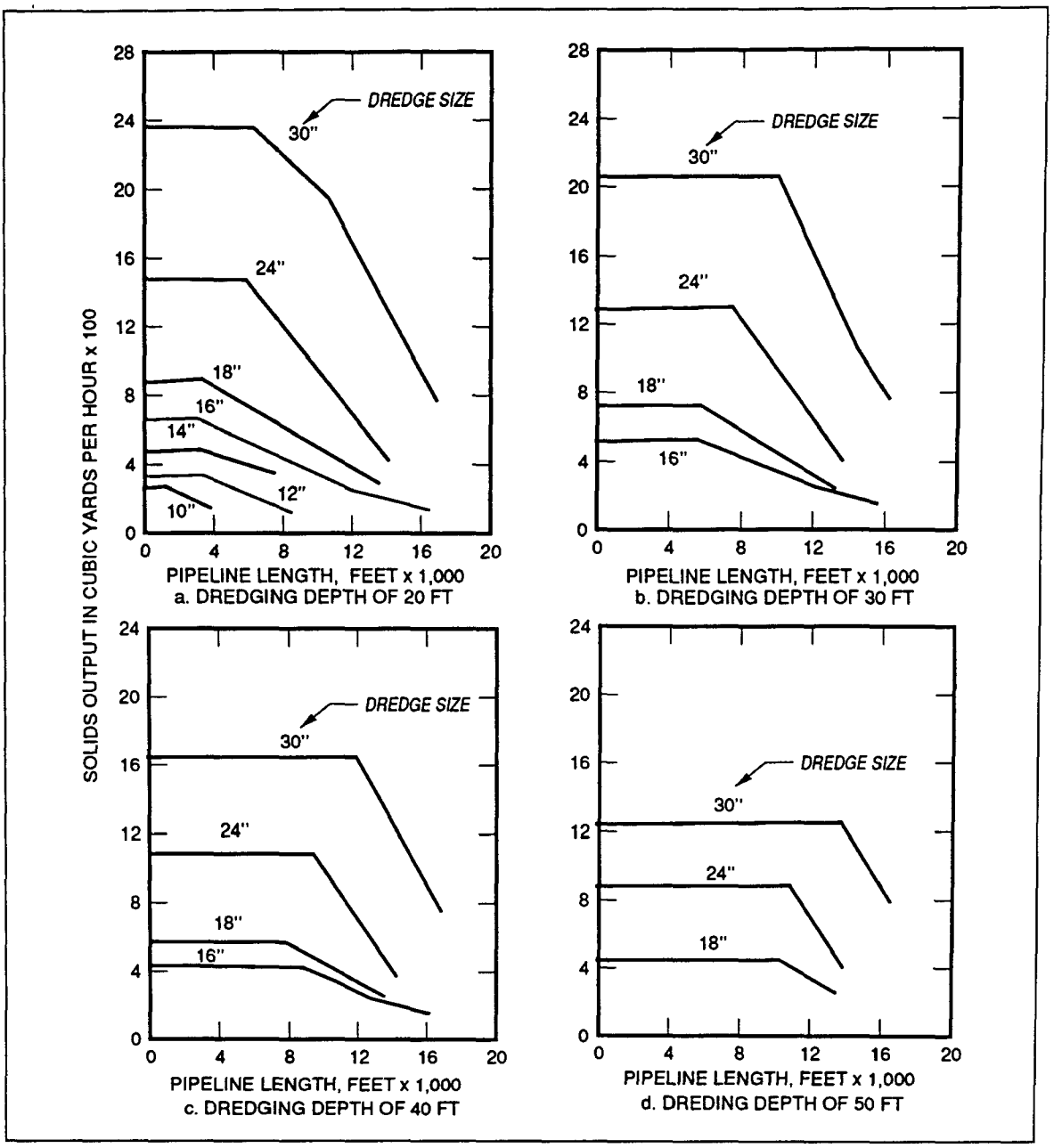


Figure 5. Solids output for selected pipeline dredge sizes, pipeline lengths, and dredging depths (from U.S. Army Corps of Engineers 1987)

contaminants and the geochemical changes affecting contaminant distribution between aqueous and dissolved phases during active disposal operations. The column settling test and facility-specific conditions (surface area, ponding depth, influent flow, and hydraulic efficiency) are essential parameters for using the modified elutriate test to predict effluent quality. A flowchart illustrating how modified elutriate and column settling tests are used to predict dissolved and particulate bound contaminant concentrations in CDF effluent is shown in Figure 6.

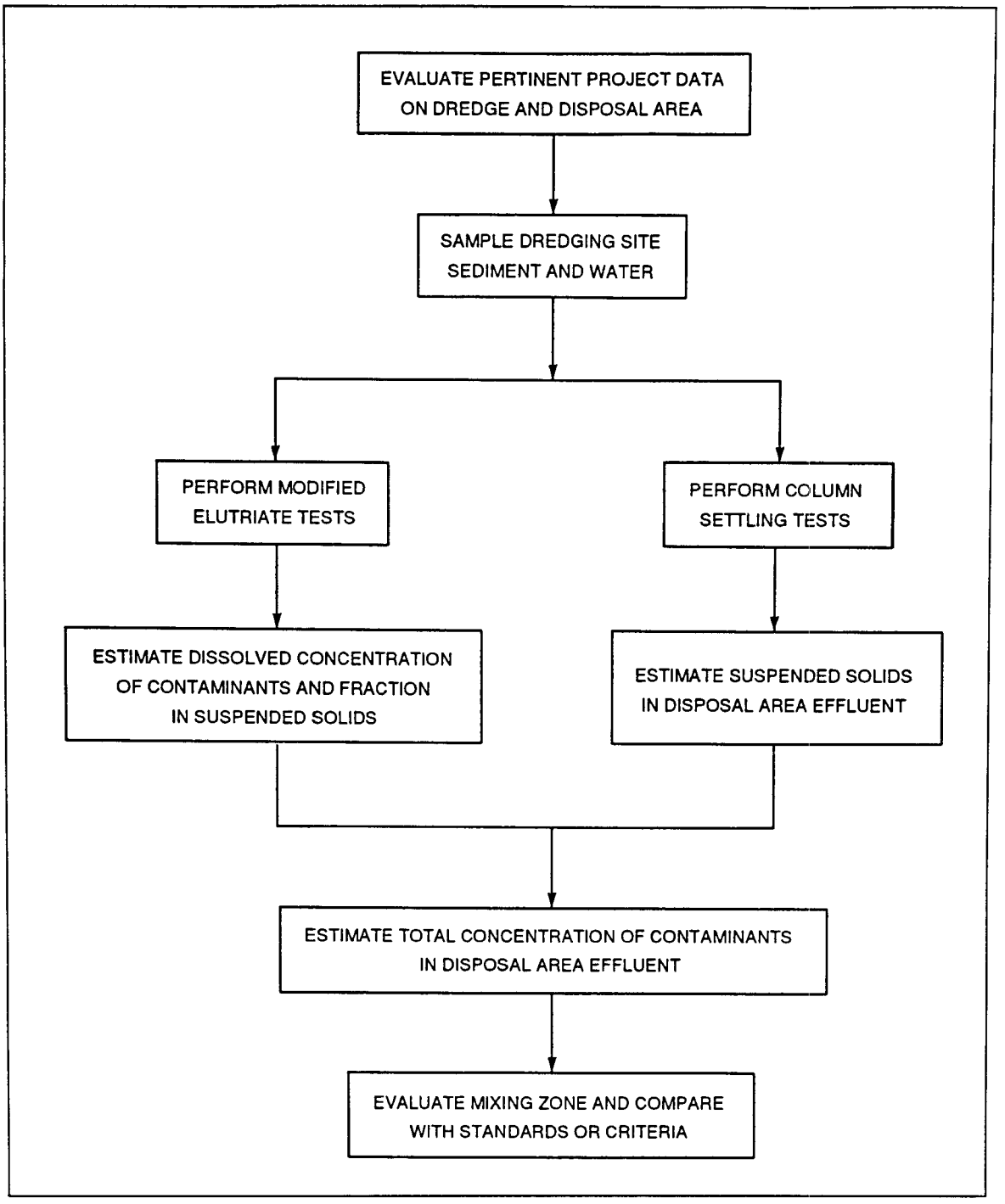


Figure 6. Steps for predicting effluent quality during hydraulic filling

The procedures shown in Figure 6 have undergone extensive research and development including field trials. Field studies on maintenance dredging projects confirmed that the procedures are reliable and usually provide conservative estimates of heavy metal concentrations in effluent (Palermo 1988; Palermo and Thackston 1988a). Field data for organic contaminants are not

as extensive as that for metals, but the available field data indicate that the procedures are also good predictors of organic contaminant concentrations in CDF effluent during hydraulic filling (Palermo 1986; Palermo 1988; Myers 1991). The modified elutriate and column settling tests are briefly described below.

The modified elutriate test consists of the following steps (Figure 7):

a. Mixing dredging site sediment and water to the solids concentration expected in the influent to the facility (discharge from the dredge).

b. Aerating the mixture for 1 hr to simulate the oxidizing conditions present in primary settling facilities.

c. Settling the mixture for a time equal to the expected or measured mean retention time of the facility, up to a maximum of 24 hr.

d. Collecting a sample of supernatant for chemical analysis of dissolved and total contaminant concentrations.

The dissolved concentrations from the test are the predicted dissolved concentrations in the effluent. The contaminant concentrations associated with suspended solids are the differences between total contaminant concentrations in whole water samples and dissolved contaminant concentrations in the filtered water samples (Equation 20).

$$C_s = \frac{C_{total} - C_w}{C_p} \tag{20}$$

where

C_s = solid phase contaminant concentration, mg/kg

C_{total} = whole water contaminant concentration, mg/ℓ

C_w = dissolved contaminant concentration, mg/ℓ

C_p = suspended solids concentration, kg/ℓ

It should be noted that C_w and C_s in Equation 20 are not necessarily equilibrium concentrations. They could be equilibrium concentrations, but equilibrium is not a necessary condition in the modified elutriate test.

The column settling test consists of the following steps:

a. Mixing the dredging site sediment and water to the slurry concentration expected in the influent to the pretreatment or confined disposal facility.

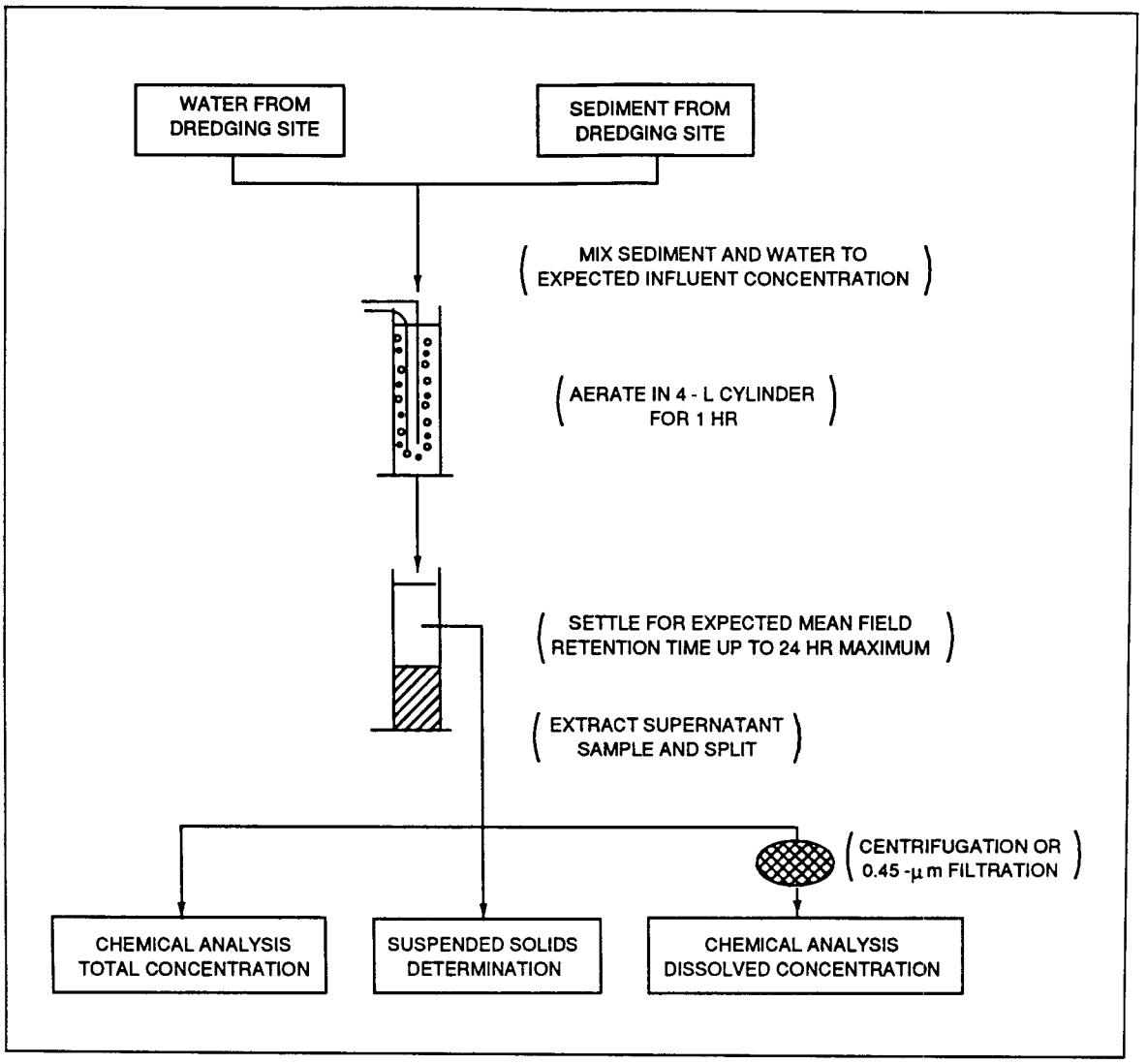


Figure 7. Modified elutriate test procedure

b. Placing the slurry into an 8-in. (20.3-cm) diameter settling column and allowing it to settle.

c. Taking samples of supernatant water above the sediment-water interface at various time intervals.

d. Analyzing the samples for suspended solids concentrations.

Effluent suspended solids concentration is predicted using the following steps:

a. Developing a relationship of column supernatant suspended solids concentration versus settling time (Figure 8).

b. Selecting a column supernatant suspended solids concentration corresponding to the expected mean field retention time.

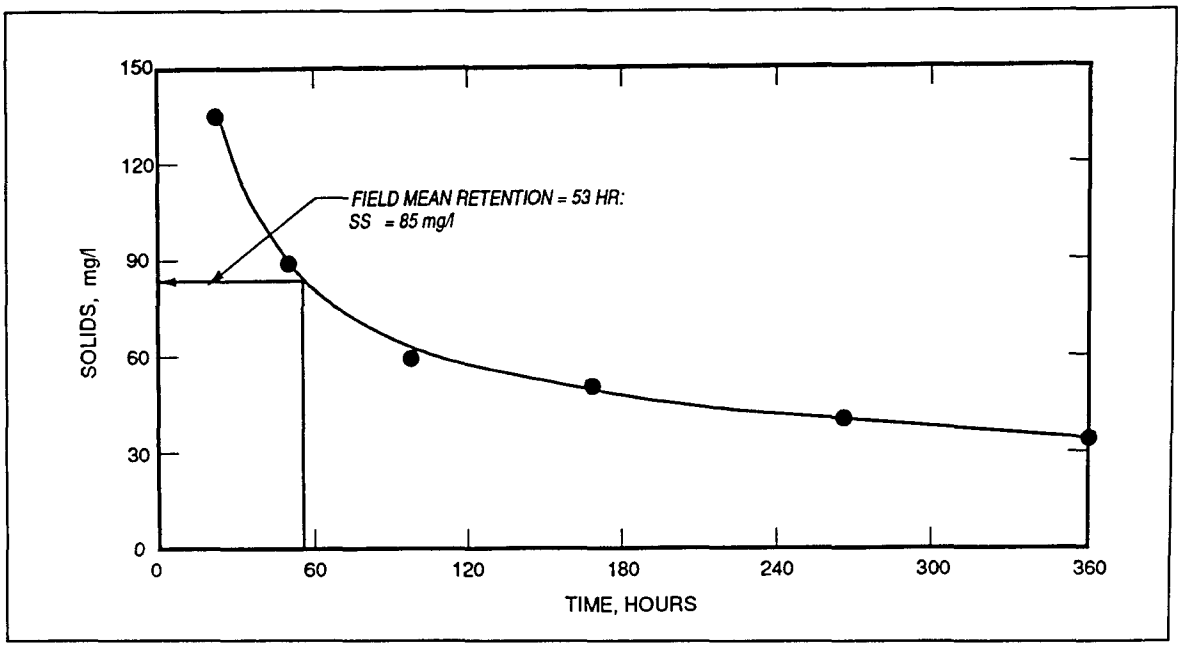


Figure 8. Typical plot of supernatant suspended solids concentration versus time for column settling test (from Palermo and Thackston 1989)

c. Estimating a predicted effluent suspended solids value by adjusting the value selected in the above step for wind and turbulence under field conditions using a settling efficiency adjustment factor.

Predicted total contaminant concentrations in effluent during hydraulic filling are estimated using the following equation:

$$C_{EEF,TOT} = C_w + C_s\, C_p \tag{21}$$

where $C_{EFF,TOT}$ is the total concentration of contaminant in effluent, mg/ℓ.

Detailed information on the development of modified elutriate and column settling tests including example calculations are provided by Montgomery (1978); Montgomery, Thackston, and Parker (1983); Palermo (1986); USACE (1987); Palermo (1988); Palermo and Thackston (1988a); Palermo and Thackston (1988b); Palermo and Thackston (1988c); Palermo and Thackston (1989); and Averett, Palermo, and Wade (1988). For a specific dredging project, hydraulic dredge, and facility design, these procedures have been shown to reliably predict effluent suspended solids, total contaminant, and dissolved contaminant concentrations.

When column settling and modified elutriate data are not available, a priori techniques for estimating effluent quality and mass releases during hydraulic placement in pretreatment facilities can be used. A priori estimation techniques, by definition, do not require site-specific data. As a result, a priori estimates are not as reliable as estimates based on site-specific test data.

Total mass concentration (particulate + dissolved) in effluent during hydraulic filling can be estimated using the following equation (Myers et al. 1993):

$$C_{EFF,TOT} = C_{INF,TOT} (1 - CEF) \tag{22}$$

where

$C_{EFF,TOT}$ = total concentration of contaminant in effluent, mg/ℓ

$C_{INF,TOT}$ = total concentration of contaminant in influent, mg/ℓ

CEF = contaminant containment efficiency factor for effluent pathway, dimensionless

The containment efficiency factor (Myers 1991; Myers et al. 1993) is a simple measure of contaminant mass retention. Palermo (1988) measured CEFs at five CDFs. The five-site average CEF for metals was 0.986 (98.6-percent retention). The one site at which PCBs were monitored showed a CEF of 0.99 (99-percent retention) for PCBs. Operation and management of pretreatment facilities for remediation of sediments in the Great Lakes will likely result in contaminant retention that is at least as good as and probably better than that measured by Palermo (1988) at CDFs designed and operated for disposal of dredged materials from maintenance dredging projects.

Dissolved organic contaminant concentrations in effluent can be estimated using equilibrium partitioning equations described previously in the section on losses during dredging (Equation 13). Equilibrium partitioning equations should not be used to estimate dissolved metal concentrations in effluent. Applications and limitations of equilibrium partitioning equations are discussed in the next section on leachate losses and in Appendix B.

Effluent-mechanical placement

Influent characteristics. For mechanical dredging and placement, the in situ water content is a good estimator of the solids content of the dredged material influent, and bulk chemical analysis of the sediment is a good estimator of influent contaminant concentrations. Influent flow can only be judged from previous operating records since many site-specific conditions affect the disposal rate when mechanical dredging and disposal methods are used. For example, during a 1986 maintenance dredging project in the Chicago River, dredging was conducted at night. Night work was necessary to minimize interference with bridge traffic on the many drawbridges that cross the Chicago River in downtown Chicago. Two barges each containing approximately 760 m^3 (1,000 cu yds) were loaded by a clamshell dredge during the night and unloaded the following day by clamshell dredge. It took approximately 3 to 4 hr to unload a barge.

Effluent quality. In an upland facility, there should be little or no effluent during mechanical dredging and placement. The small amount of water that may seep to the surface will have pore water quality. During mechanical placement of dredged material in nearshore and in-water facilities, water in the facility before disposal operations begin will be displaced by the dredged material resulting in a discharge of effluent. Predictive techniques for effluent quality during mechanical placement of dredged material in nearshore and in-water facilities that contain water prior to placement of dredged material are currently unavailable.

Leachate

When contaminated dredged material is placed in a pretreatment facility, contaminants may be mobilized and transported beyond the facility boundary by leaching. Leachate is contaminated pore water, and leaching is the combination of interphase transfer of contaminants from dredged material solids to pore water and movement of contaminated pore water. Thus, leaching is a coupling of chemistry and fluid mechanics. Techniques for estimating leachate flow and quality are discussed in this section.

Leachate flow. Leachate flow from dredged material placed in primary settling facilities and CDFs is produced by four potential water sources:

a. Interstitial water left after primary settling.

b. Rainwater and snowmelt.

c. Offsite groundwater.

d. For in-water facilities, surface water outside the facility.

The predictive technique for estimating leachate flow discussed in this section accounts for leachate generation associated with the first two water sources. Application of groundwater models to facilities with leachate generated by offsite groundwater inflow and techniques for estimating leachate generation by fluctuating water levels outside a nearshore or in-water facility are discussed in Losses From Confined Disposal Facilities on CDFs.

After filling is completed, dredged material in a primary settling facility is initially in a saturated condition (all voids are filled with water). As evaporation and seepage remove water from the voids in the dredged material, the amount of water stored in the voids and available for gravity drainage decreases. After some time, usually several years, a quasi-equilibrium is reached in which water that seeps or evaporates is replenished by infiltration through the surface. It is not likely that dredged material will be held in pretreatment facilities long enough for establishment of a quasi-equilibrium. Leachate flow from primary settling facilities will be time varying and highly dependent on local climatology, dredged material properties, and facility

design factors. To predict time-varying leachate flow, all these factors must be considered.

Preproject estimation of leachate flow, therefore, requires coupled simulation of local weather patterns and surface and subsurface processes governing leachate generation. The local groundwater regime can be important in evaluating long-term leaching trends at pretreatment facilities. Depending on local geohydrology, hydraulic conductivity of the dredged material, size of the facility, and other site-specific factors, such as liners, groundwater flow may tend to go beneath a pretreatment facility, diverge and spread around it, or even discharge into it. In most cases, however, leachate flow from a pretreatment facility is governed by the initially saturated condition of the dredged material, the amount of pore water initially available for gravity drainage, and the replenishment of water that seeps from the site by rain and snow. In short, leachate generation at pretreatment facilities is governed by the initial condition of the dredged material and surface hydrology.

Important climatic parameters include precipitation (rain and snow), temperature, and humidity. Important surface processes include snowmelt, infiltration, surface runoff, and evaporation. Important subsurface processes include evaporation from dredged material voids and flow in vadose and saturated zones in the dredged material. Important facility design factors include hydraulic properties of the foundation soils, type of liner (if any), and type of leachate collection system (if any). Due to the complexity of the interactions among climatic events, surface hydrologic processes, and subsurface hydraulics, there is no one laboratory test capable of predicting leachate flow.

There is, however, a simulation model available that couples climatic events, surface hydrologic processes, and subsurface hydraulics that is applicable to dredged material in an upland containment facility. This model is the Hydrologic Evaluation of Landfill Performance (HELP) computer program (Schroeder et al. 1988). HELP is a hydrologic water budget model that accounts for the effects of surface storage, runoff, infiltration, percolation, evapotranspiration, soil moisture storage, lateral drainage to leachate collection systems, and percolation through synthetic liners, soil liners, and composite liners. Local climatology is one of the important components of hydrologic modeling that the HELP model simulates on a daily basis.

The HELP model was developed by the USEPA to predict the amounts of seepage, drainage to leachate collection systems, at sanitary landfills. The model is used in a preproject mode by designers and permits writers to evaluate landfill designs. HELP model features that are particularly useful for estimating leachate flow are summarized in Table 5. Limitations that apply when using the HELP model to estimate leachate flow are also summarized in Table 5.

The HELP model simulates flow through as many as 12 layers with varying hydraulic properties. The first layer is usually a cap, and the bottom layer

Table 5 HELP Model Major Features
Advantages
Time varying.
Simulates site-specific climatology on a daily basis using user-supplied data, default data for 102 cities in the U.S., or a synthetic climatology generator.
Couples vegetative growth, evapotranspiration, surface runoff, unsaturated flow, saturated flow, and soil moisture storage.
Layers of differing hydraulic properties simulated, including caps, liners, and leachate collection systems.
Includes default climatological and soil property data.
Interactive and runs on desktop computers.
Documented.
Coupling of surface, vadose, and saturated flows experimentally verified (Schroeder and Peyton 1987).
Limitations
One-dimensional (vertical percolation) except for leachate collection systems.
Assumes bottom is free draining.

is usually a low-permeability barrier soil, although these are not model requirements. The model is quasi-two-dimensional in that layers can be defined as lateral drainage or vertical percolation layers. Lateral drainage layers are appropriate for designs that include a leachate collection system. Without lateral drainage layers, subsurface flow calculations in the HELP model are one-dimensional simulations of vertical percolation.

A definition sketch for application of the HELP model to recently filled primary settling facilities is shown in Figure 9. As shown in Figure 9, the dikes should be impermeable relative to the hydraulic conductivity of the dredged material. These conditions are not always met, but when they are, flow into the foundation soils is primarily vertical. In this case, the physical system closely matches the HELP model assumptions so that there are few if any limitations to application of the HELP model.

The general simulation parameters (user-supplied inputs) are listed in Table 6. The user must specify the number and thickness of each layer. There are three types of layers in the HELP model as follows: vertical percolation layers, lateral drainage layers, and barrier soil liners. Vertical percolation layers are layers without a leachate collection system. The dredged material in a primary settling facility would be classified as a vertical percolation layer. If there are dredged material layers with different properties, such as hydraulic conductivity, dredged material layers as needed could be specified as vertical percolation layers as long as the total number of layers

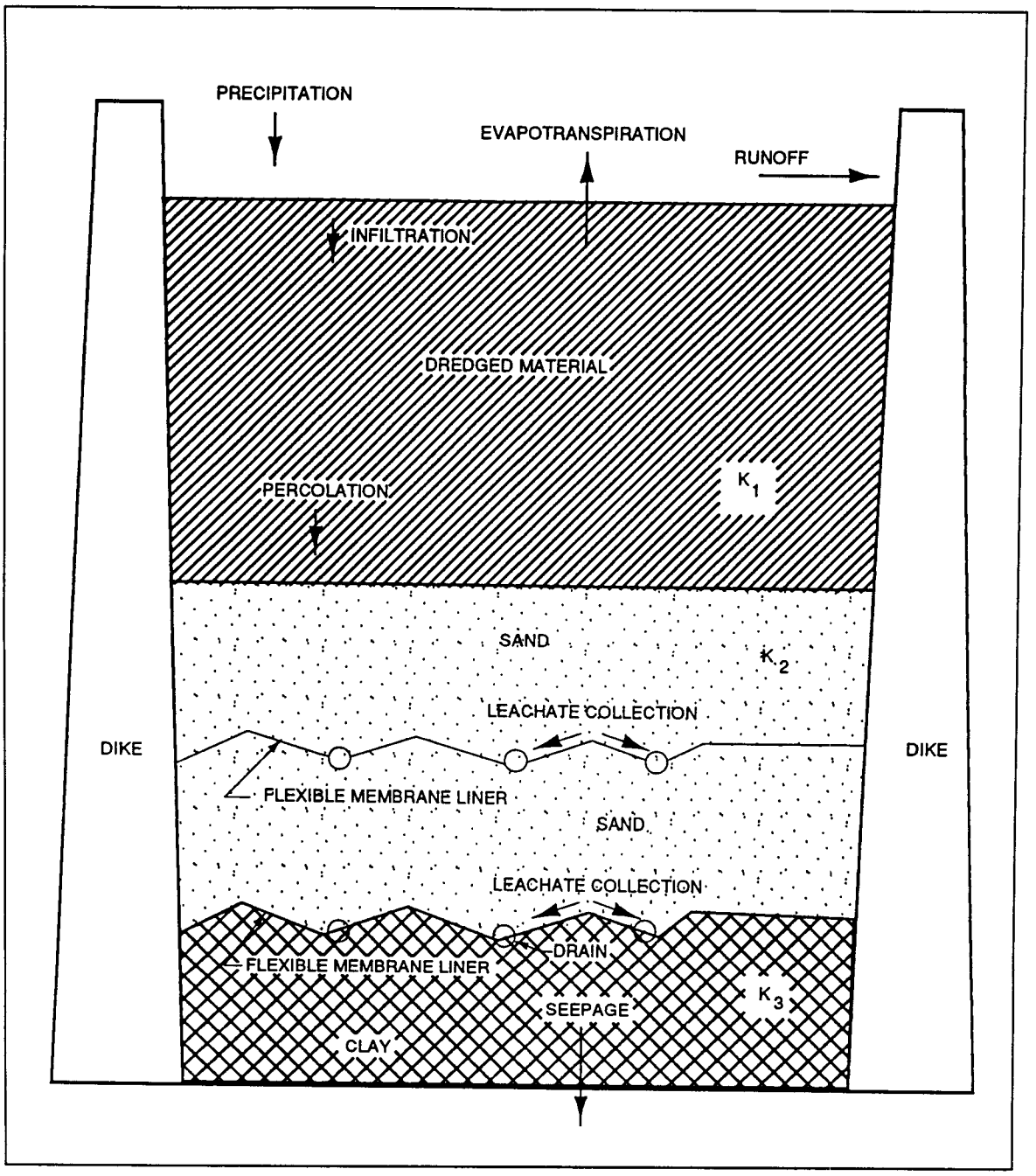


Figure 9. Definition sketch for application of HELP model to primary settling facilities for dredged material

does not exceed 12. Lateral drainage layers are layers designed to collect leachate by lateral drainage to collection pipes. Both vertical and lateral drainage are simulated by the HELP model in lateral drainage layers. A layer of material design to inhibit percolation is classified as a barrier soil liner. A layer covered by a flexible membrane liner (FML) is classified as barrier soil liner with an FML. In addition, the user can select the "active" or "closed" options. The "active" option will not allow runoff to occur. Excess

Table 6 General Simulation Parameters for the HELP Model
Facility Design Parameters
Number of layers (1 to 12)
Layer classification as vertical percolation, lateral drainage, or barrier soil liner
Thickness of each layer
Liner presence (yes/no) for barrier soil liners
Open or closed site
Surface area
Climatological Database Choices
Default database (4-year record) for 102 U.S. cities
User supplied database
Synthetic weather generator for 139 U.S. cities
Soil and Dredged Material Properties
Default soil option (yes/no)
Manual soil option
Porosity
Field capacity
Wilting point
Initial water content
Saturated hydraulic conductivity
Other
Evaporative zone depth
Type of vegetative cover
Simulation period (1 to 20 years, depending on climatological database)
Type of output
Daily
Monthly averages
Annual totals

precipitation will pond on the dredged material surface. The "closed" option will allow runoff.

The user has the choice of using a default climatological database, user-supplied database, or a synthetic weather generator. The default climatological database is a 5-year record (1974 through 1978) for 104 U.S. cities. The

user can choose to input a climatological database consisting of daily temperature, precipitation, solar radiation, and other parameters. The HELP model synthetic weather generator is applicable to 139 U.S. cities. The default soil database in the HELP model is based on the USCS. The user specifies the type of soil according to one of 15 possible USCS classifications. There are also default soil data for two types of barrier soils that may be further specified as compacted or uncompacted. The user can also specify soil and dredged material properties for each layer as follows: wilting point, porosity, saturated hydraulic conductivity, initial water content, and field capacity.

Other model inputs include evaporative zone depth, leaf area index, simulation period, and type of output. The evaporative zone depth is the depth beginning at the soil cover (or dredged material) surface affected by evaporative drying. The leaf area index is zero for a primary settling facility since the dredged material will be removed for treatment before vegetation has a chance to establish. The maximum simulation period is 20 years and depends on the length of record in the climatological database. Longer periods can be simulated by restarting the HELP model using water budget information from the last output.

The types of output data provided by the HELP model when the user specifies daily output are listed below:

Julian date
Precipitation, inches
Runoff, inches
Evapotranspiration, inches
Head on barrier soil liners, inches
Percolation through barrier soil liners, inches
Lateral drainage from surface of any barrier soil, inches
Water content in evaporative zone, dimensionless

The following types of output are provided when the user specifies monthly totals:

Precipitation, inches
Runoff, inches
Evapotranspiration, inches
For each layer:
- Percolation, inches
- Lateral drainage, inches
- Monthly average daily head, inches
- Monthly standard deviation of daily heads, inches

Annual totals for the parameters listed below are presented for both daily and monthly output options in three types of units: inches, cubic feet, and percentage of annual precipitation:

Precipitation
Runoff
For each layer
    Percolation
    Lateral drainage
Soil water in storage at beginning of year
Soil water in storage at end of year
Snow water in storage at beginning of year
Snow water in storage at end of year
Annual change in total water storage

Leachate quality. Techniques for predicting leachate quality in primary settling facilities and CDFs are discussed in this section. Two types of predictive techniques for leachate quality are discussed. The first technique is an a priori technique, and the second technique involves laboratory leach tests. Both techniques are based on equilibrium partitioning theory. Application of this theory to dredged material leaching is described by Hill, Myers, and Brannon (1988); Myers, Brannon, and Price (1992), Brannon, Myers, and Tardy (1994).

Equilibrium partitioning as used in this report is a simple representation of a variety of contaminant interphase transfer processes. The complexity of the problem is shown in Figure 10. As shown in Figure 10, interphase contaminant transfer is a complicated interaction of many elementary processes and factors affecting these processes. A complete description of all these processes, their interactions, and factors affecting these processes is not presently possible. Instead, a lumped parameter, the equilibrium distribution coefficient, is used to describe the distribution of contaminant between aqueous and solid phases.

At equilibrium, the net transfer of contaminant across the solids-water interface is zero, and the mass of contaminant in each phase is constant, but not necessarily equal. Thus, only the relative distribution of contaminant between solid and aqueous phases is needed to predict leachate quality. This distribution of contaminant mass between solid and aqueous phases is represented by the equilibrium distribution coefficient defined as follows:

$$K_d = \frac{\frac{M_{cs}}{M_s}}{\frac{M_{cw}}{M_w}} \tag{23}$$

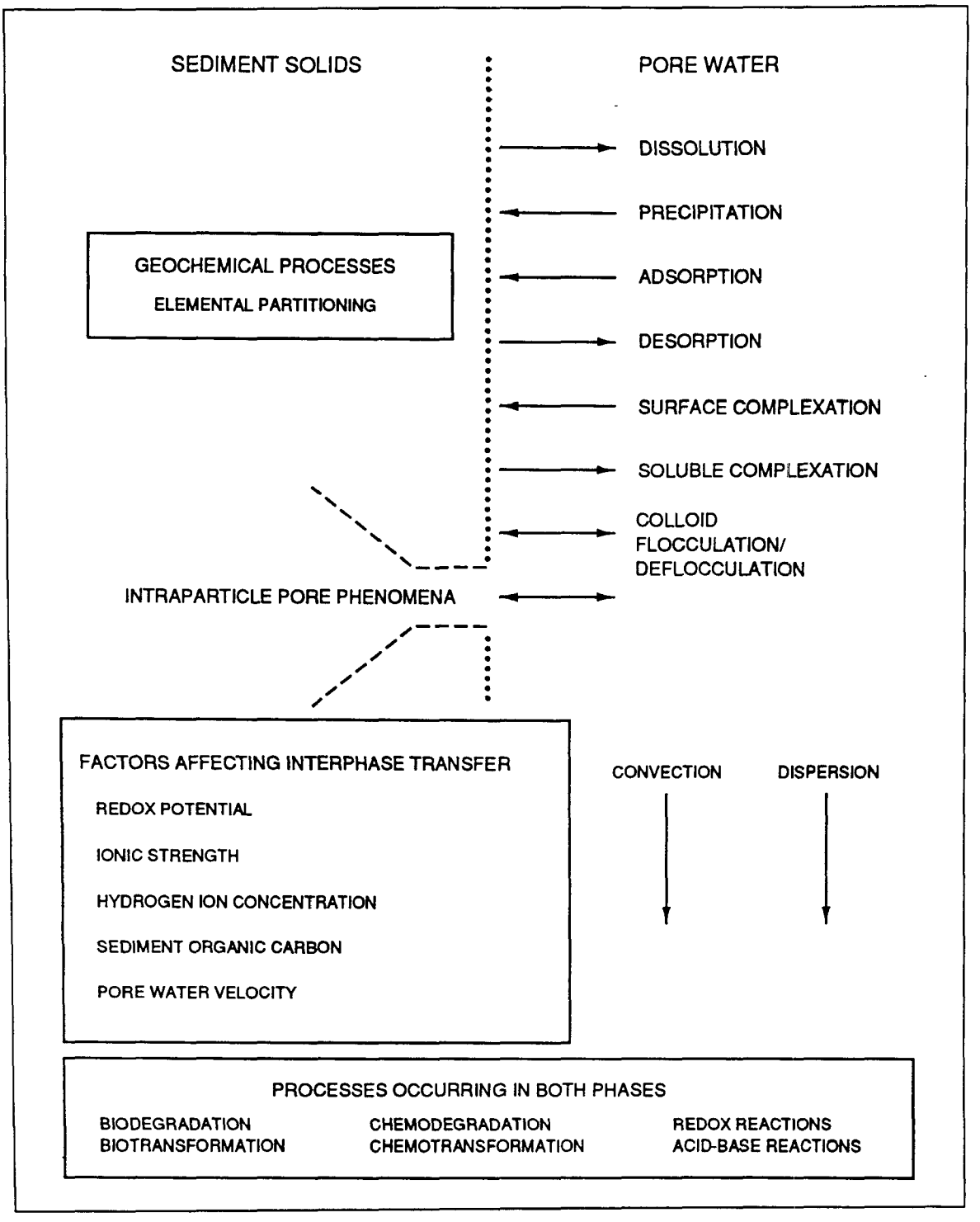


Figure 10. Interphase transfer processes and factors affecting interphase transfer processes (from Myers, Brannon, and Price 1992)

where

K_d = contaminant-specific equilibrium distribution coefficient, dimensionless

M_{cs} = mass of contaminant in solid phase, kg

M_s = mass of solids, kg

M_{cw} = mass of contaminant in aqueous phase, kg

M_w = mass of water, kg

The mass fractions in Equation 23 can be replaced with phase contaminant concentrations without any loss of generality so that Equation 23 becomes

$$K_d = \frac{C_s}{C_w} \tag{24}$$

where

K_d = contaminant-specific equilibrium distribution coefficient, ℓ/kg

C_s = contaminant concentration in sediment at equilibrium, mg/kg

C_w = aqueous phase concentration at equilibrium, mg/ℓ

Equations 23 and 24 describe the equilibrium distribution of a single contaminant in dredged material; that is, equilibrium distribution coefficients are contaminant and dredged material specific. In addition, the distribution of contaminant mass is affected by various factors, such as pH, ionic strength, redox potential, and sediment organic carbon. Varying these factors during leaching can shift the equilibrium position of the system and change K_d.

The local equilibrium concept is illustrated in Figure 11. When the rate at which water moves is slow relative to the rate at which equilibrium is approached, a local chemical equilibrium exists between the pore water and the sediment solids. Thus, the local equilibrium assumption implies that as a parcel of water passes a parcel of dredged material solids, the water and solids come to chemical equilibrium before the parcel of water moves to contact the next parcel of dredged material solids. Thus, leachate quality at the surface can differ from leachate quality at the bottom of a primary settling facility, while leachate in both locations will be in equilibrium with the dredged material solids.

Application of the equilibrium assumption to prediction of leachate quality in dredged material is based on two arguments: (a) the argument that the interphase transfer rates affecting leachate quality are fast relative to the

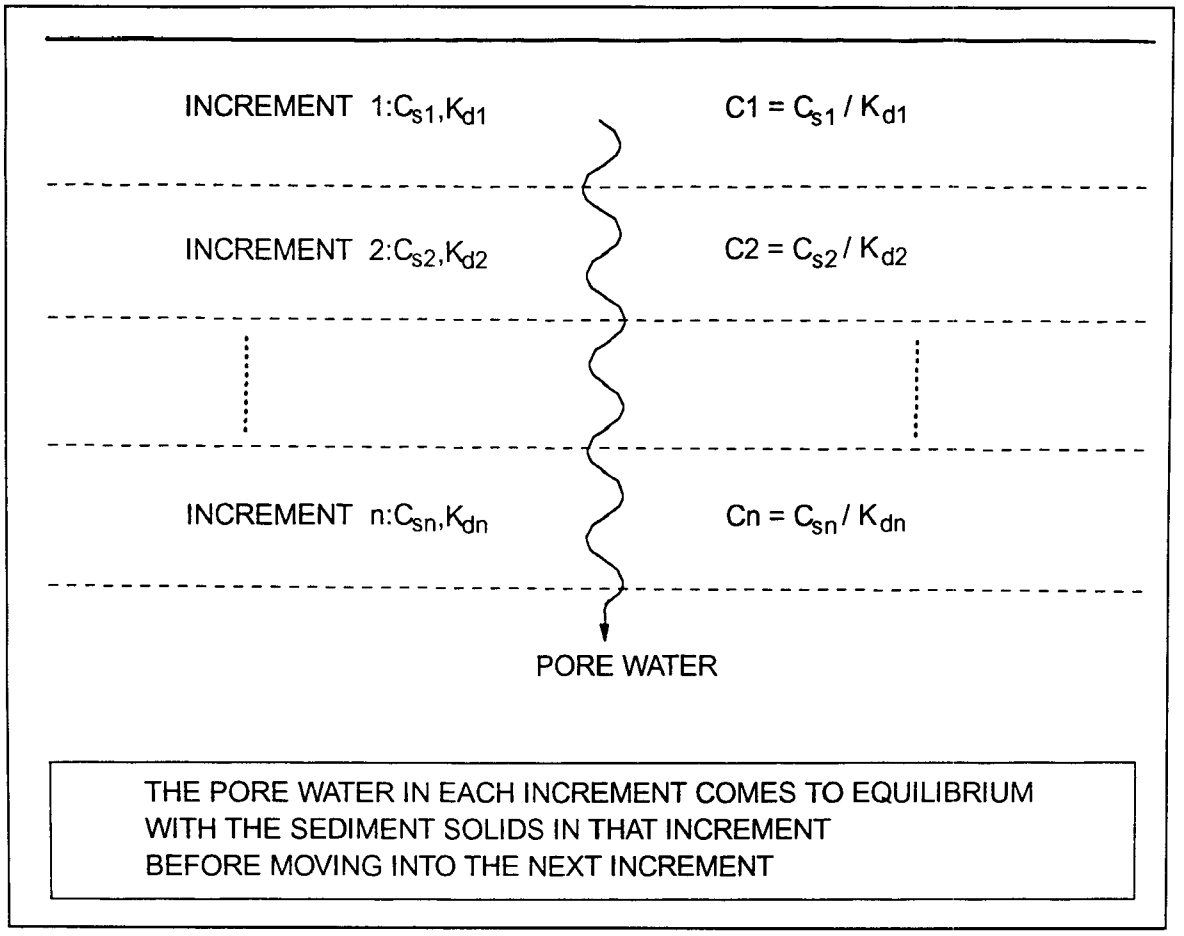


Figure 11. Illustration of local equilibrium assumption for leaching in a pretreatment facility

volumetric flux of water and (b) the argument that equilibrium-controlled desorption provides conservative predictions of leachate quality. These arguments are discussed below.

The equilibrium assumption is valid when the seepage velocity is slow relative to the rate at which contaminants desorb from dredged material solids. This is a realistic assumption for fine-grain dredged material because seepage velocities are usually very low due to the low hydraulic conductivity of fine-grain dredged material. The hydraulic conductivity of fine-grain dredged material is usually in the range of 10^{-8} to 10^{-5} cm/sec. In primary settling facilities and CDFs, the hydraulic gradient is usually about one, so that pore water velocities are usually in the range of 10^{-8} to 10^{-5} m/sec. Some soil column studies have indicated that the local equilibrium assumption is valid for pore water velocities as high as 10^{-5} cm/sec (Valocchi 1985). Theoretically, equilibrium-controlled desorption requires an infinitely fast desorption rate. However, if the critical interphase transfer rates are sufficiently fast, the equilibrium assumption can yield results indistinguishable from full kinetic modeling (Jennings and Kirkner 1984; Valocchi 1985; Bahr and Rubin 1987).

In addition to being a good approximation, the assumption of equilibrium-controlled desorption is conservative; that is, predictions based on the equilibrium assumption will overestimate leachate contaminant concentrations if pore water velocities are too high for local equilibria to become established. The equilibrium assumption is conservative because interphase transfer is from the dredged material solids to the pore water, and equilibrium means that all of the desorption that can occur has occurred. Thus, for clean water entering dredged material, pore water contaminant concentrations cannot be higher than the equilibrium value.

Rearrangement of Equation 24 yields

$$C_w = \frac{C_s}{K_d} \tag{25-a}$$

Equation 25-a uses the bulk sediment contaminant concentration, C_s, and a contaminant-specific distribution coefficient, K_d, to predict dissolved leachate contaminant concentration.

Organic contaminants sorb to the humic and fluvic acids that make up dissolved organic carbon. Since dissolved organic carbon is mobile, dissolved organic carbon enhances advective transport of contaminants. Equation 25-b includes a factor to account for facilitated transport by colloidal-bound contaminant.

$$C_{pw} = C_w (1 + K_c C_c) = \frac{C_s (1 + K_c C_c)}{K_d} \tag{25-b}$$

where

C_{pw} = pore water contaminant concentration, mg/ℓ

C_c = dissolved organic carbon, kg/ℓ

K_c = equilibrium distribution coefficient for partitioning of contaminant between dissolved organic carbon and water, ℓ/kg

Empirical equations that relate distribution coefficients to sediment organic carbon and octanol-water partitioning coefficients are available (Karickhoff, Brown, and Scott 1979; Means et al. 1980; Karickhoff 1981; Schwarzenbach and Westall 1981; Chiou, Porter, and Schedding 1983; Lyman, Reehl, and Rosenblatt 1990). These relationships were developed mainly through batch adsorption tests using soils, sediments, and aquifer materials. The generality of these relationships for desorption of contaminants from dredged material is uncertain, but the basic technique is widely accepted. A priori estimation of

distribution coefficients is described in Appendix B. Caution should be exercised when choosing and using K_d values. If K_d is estimated from empirical relationships based on sediment organic carbon content and octanol-water partitioning coefficients, Equation 25-b is recommended. Equation 25-a is valid, but facilitated transport will not be included. If K_d is determined using the sequential batch leach test discussed later, Equation 25-a should be used because the K_d obtained from this test included facilitated transport. Equation 25-b should not be used in this case.

An example a priori prediction of organic chemical concentrations in dredged material leachate is presented in Table 7. The estimates provided in Table 7 are sediment and contaminant specific. The predictions are sediment specific because the C_s values used in the predictions are for a sediment from Norfolk, VA, and the K_d values are based, in part, on the organic carbon content of that sediment. The predictions are contaminant specific because the octanol-water partitioning coefficients used to calculate K_d values are contaminant specific.

Table 7
A Priori Prediction of Selected Organic Chemical Concentrations in Dredged Material Leachate From Norfolk, VA

Organic Contaminant	C_s, mg/kg	C_w, µg/ℓ	K_d, ℓ/kg
p,p-DDD	0.0004	7.2 E-07	55,770
p,p-DDE	0.0022	6.9 E-06	3.2 E+05
p,p-DDT	0.0012	6.8 E-05	17,600
Heptachlor	0.0022	0.0025	869
Dieldrin	0.0007	0.0057	123
Endosulfan sulfate	0.0014	0.0571	24.5
Endrin	0.0003	0.0004	807
Endrin Aldehyde	0.0011	7.9 E-06	1.4 E+05
Heptachlor Epoxide	0.0007	0.0440	15.9
Methoxychlor	0.0017	0.0003	5,794

Note: From Palermo et al. (1993).

Equilibrium partitioning theory with some modification can also be used to develop a priori predictions of metal concentrations in dredged material leachate (Palermo et al. 1993). The theoretical and experimental basis for a priori estimation of metal pore water concentrations is not as well developed as that for organic contaminants. The basic approach for metals is the same as the approach for organic contaminants except that Equation 25-a as stated is not applicable to metals. Equation 25-a is not applicable because the total metal concentration in the dredged material solids is not leachable (Environmental Laboratory 1987).

A significant fraction of the total metal concentration in sediments is in geochemical phases that are not mobilized by aqueous extraction (Brannon et al. 1976; Steneker, Van Der Sloot, and Das 1988).

Modification of Equation 25-a for the leachable metal concentration provides a method for estimating pore water metal concentrations. Assuming a modification of the equilibrium approach discussed previously applies, metal pore water concentration is given by

$$C_w = \frac{C_{sL}}{K_d} \tag{26}$$

where C_{sL} is the leachable metal concentration in the dredged material solids (mg/kg).

Empirical relationships for estimating the water leachable concentration, C_{sL}, and the distribution coefficient, K_d, for metals are not available. These parameters are sediment specific, as well as metal specific. They are affected by a variety of factors including oxidation-reduction potential, pH, and organic carbon, sulfur, iron, and salt contents of the sediment. For these reasons, K_d and C_{sL} are difficult to estimate a priori.

Data from Brannon et al. (1976), Environmental Laboratory (1987), Brannon, Myers, and Price (1992) on leachable metal fractions in three freshwater sediments are presented in Table 8. As indicated in Table 8, between

Table 8
Percent Leachable Metal Concentrations in Selected Sediments

	Sediment		
Metal	1	2	3
Arsenic	0.34	6.5	1.37
Cadmium	<0.01	5.2	0.40
Chromium	*	*	0.17
Copper	<0.01	0.55	*
Nickel	<0.01	2.4	*
Lead	*	1.3	0.33
Zinc	0.87	3.0	0.27

[1] Ashtabula Harbor, Ohio; sum of interstitial and exchangeable phases, from Brannon et al. (1976).
[2] Hamlet City Lake, North Carolina; total extracted in anaerobic sequential batch leach test, from Brannon, Myers, and Price (1992).
[3] Indiana Harbor, Indiana; total extracted in anaerobic sequential batch leach test, from Environmental Laboratory (1987).
* No data.

about 0.3 and 7 percent of the total arsenic, about 0.01 and 6 percent of the total cadmium, and 0.2 and 3 percent of the total zinc in these freshwater sediments were leachable. These ranges in leachable metal fractions can be used to estimate ranges of C_{sL} for metals in freshwater sediments. The leachable concentration is given by multiplying the bulk sediment metal concentration by the percent leachable divided by 100.

Data on other metals is too limited to provide guidance on estimating leachable fractions in sediments. Mercury was investigated by Environmental Laboratory (1987) and Palermo et al. (1989), but detectable amounts did not leach in sequential batch leach tests. Other studies have also shown that very little of the mercury in sediments is mobile (Brannon, Plumb, and Smith 1980).

Distribution coefficients are also needed to estimate pore water metal concentrations. Anaerobic sequential batch leach data from Environmental Laboratory (1987), Palermo et al. (1989), and Myers and Brannon (1988) indicated K_d values for metals range from 2 to 90 ℓ/kg, depending on the metal and the sediment. Conservative estimates are obtained when high values of K_d are avoided, that is, the lower end of the range in expected K_d values is used. For conservative estimation of metal pore water concentrations, a range of K_d values between 3 and 10 ℓ/kg is recommended.

Since specific values for the variables C_{sL} and K_d are not known a priori, a range of metal pore water concentrations should be estimated. Figure 12 is an example of the type of concentration envelope that can be developed using a range of values for C_{sL} and K_d. In Figure 12, arsenic concentrations in leachate for various C_{sL} and K_d values are shown as a concentration envelope bounded by $3 \leq K_d \leq 10$ and $0.005 \leq$ leachable fraction ≤ 0.1, where the leachable fraction (C_{sL}/C_s) is the percent leachable divided by 100. This figure is not a generic figure since C_{sL} is required in order to calculate a leachate concentration. Figure 12 is specifically for $C_s = 4.2$ mg/kg.

Predictions with less uncertainty than the a priori predictions discussed above can be made if process descriptors, such as distribution coefficients, are determined experimentally. Currently, USACE has a research activity within the LEDO program at WES that is developing laboratory procedures for investigating interphase transfer processes, testing alternative formulations of interphase process mathematics, and quantification of interphase process descriptors. The basic approach of the LEDO leachate research is a semi-empirical approach that uses a theoretical framework based on mass transport theory (Figure 13) to guide experimental design and data interpretation (Hill, Myers, and Brannon 1988). The theoretical framework couples mass transport theory with both batch and column testing in an integrated approach (Figure 14) (Louisiana Water Resources Research Institute 1990). In the integrated approach, process descriptors from batch tests, such as distribution coefficients, are used to predict column elution curves. If predicted and observed elution histories agree, the conclusion may be reached that the processes governing transfer of contaminants from dredged material solids to

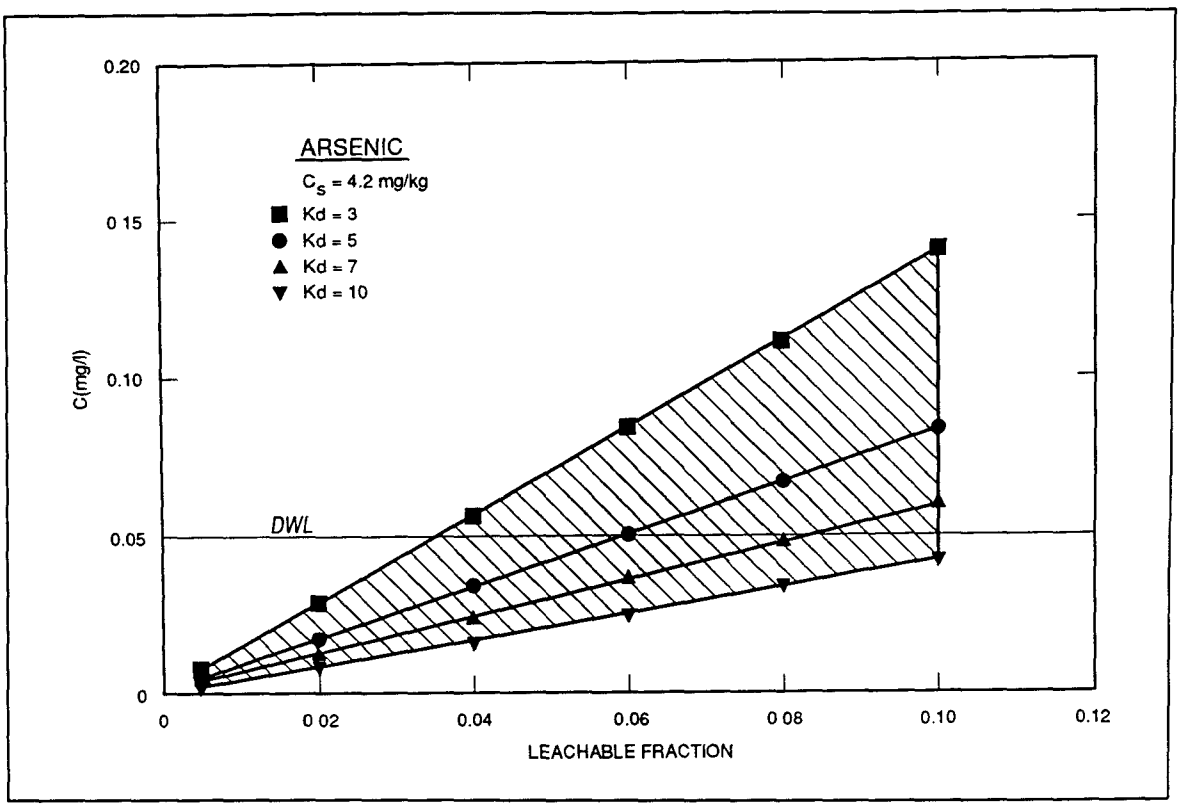


Figure 12. Predicted arsenic concentration in leachate for C_s = 4.2 mg/kg and 3ℓ/kg ≤ K_d ≤ 10ℓ/kg and 0.005 ≤ C_{sL}/C_s ≤ 0.1 (DWL = drinking water limit)

water have been adequately described. Once interphase transfer has been adequately described, contaminant migration by leaching can be evaluated for the flow conditions that apply in the field.

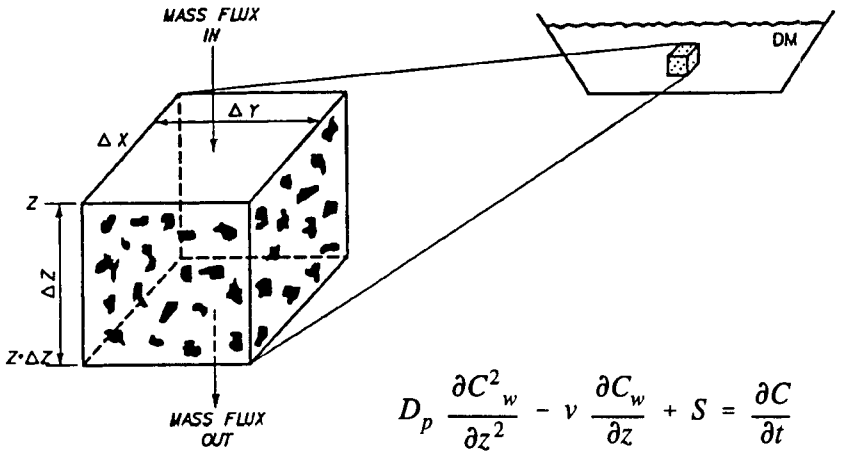
Laboratory procedures for both batch and column tests are under development (Myers, Brannon, and Price 1992). The batch test involves sequential leaching of sediment solids in a quick and relatively easy procedure that provides quantitative interphase transfer process descriptors.

The sequential batch leach procedure used to investigate sediments and dredged material (Myers and Brannon 1988) is presented below:

STEP 1: Load sediment into appropriate centrifuge tubes and add sufficient deoxygenated distilled-deionized water to each tube to bring final water-to-sediment ratio to 4:1 by weight (dry sediment solids). All operations should be conducted in a glove box under a nitrogen atmosphere.

STEP 2: Shake or tumble tubes for 24 hr.

STEP 3: Centrifuge for 30 min at 6500 × g for organics and 9000 × g for metals.



$$D_p \frac{\partial C^2{}_w}{\partial z^2} - v \frac{\partial C_w}{\partial z} + S = \frac{\partial C}{\partial t} \tag{27}$$

$$S = - \frac{\rho_b}{\epsilon} \frac{\partial C_s}{\partial t} \tag{28}$$

where

D_p = dispersion coefficient, cm^2/sec
z = distance along main axis of flow, cm
v = average pore water velocity, cm/sec
C_w = aqueous phase contaminant concentration, mg/ℓ
S = interphase contaminant transfer, mg/ℓ sec
t = time, sec
ρ_b = bulk density, kg/ℓ
ϵ = porosity, dimensionless
C_s = solid phase contaminant concentration, mg/kg

Figure 13. Mathematical model of dredged material leaching (from Hill, Myers, and Brannon 1988)

STEP 4: Filter leachate through 0.45-μm membrane filters for metals. Filter leachate through a Whatman GD/F glass-fiber prefilter followed by a Gelman AE glass-fiber filter of 1.0-μm nominal pore size for organics.

STEP 5: Preserve and store samples in the dark at 4 °C until analyzed.

STEP 6: Return to Step 2 after replacing leachate removed in Step 4 with fresh deoxygenated distilled-deionized water. Repeat the entire procedure desired number of complete cycles.

Research to date has included investigations of factors affecting leachate quality, such as liquid-solids ratio and the shake time required to réach

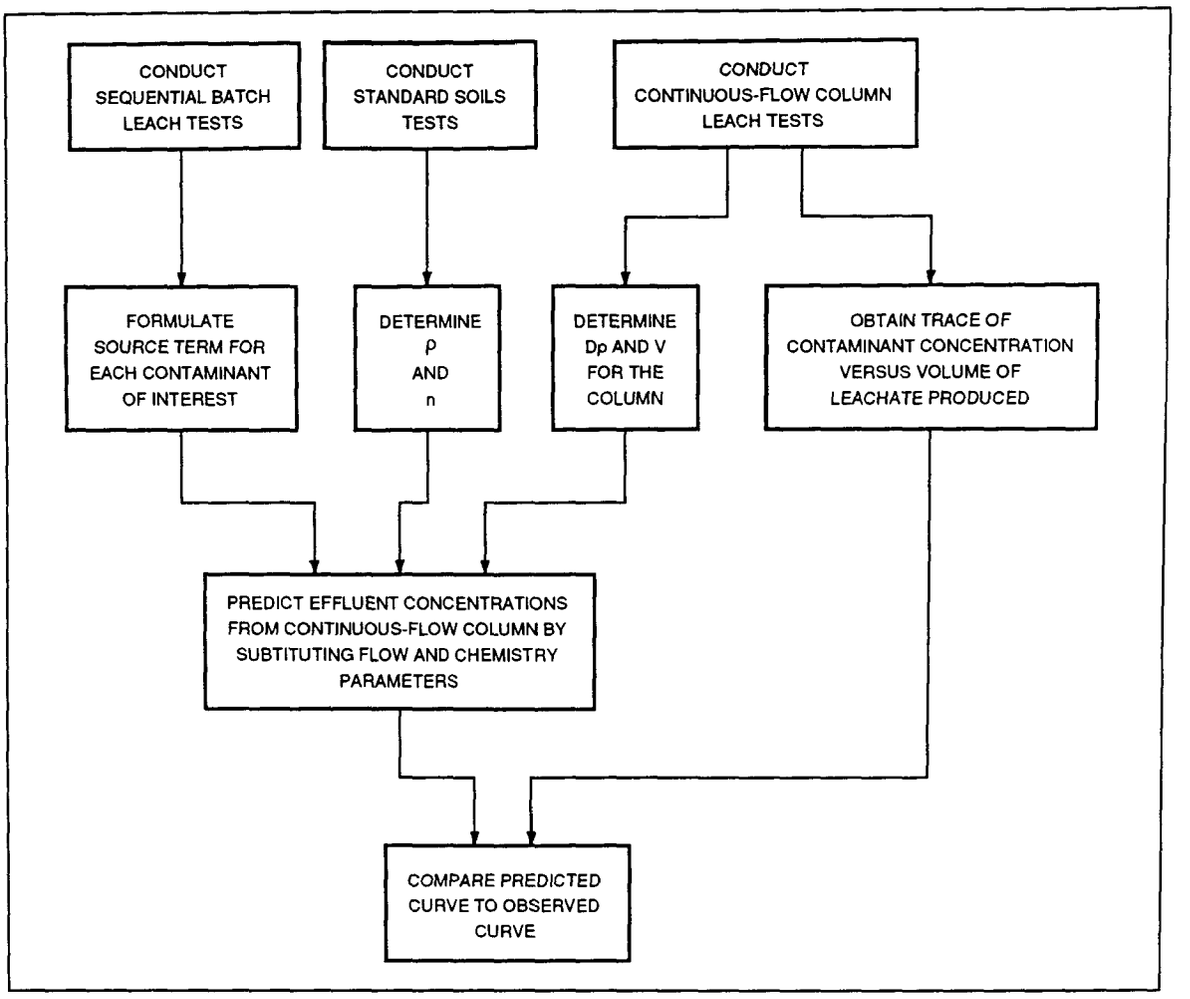


Figure 14. Integrated approach for examining interphase mass transfer (from Louisiana Water Resources Research Institute 1990)

steady-state leachate concentrations. Results indicate that a four-to-one ratio of water-to-solids by weight (dry sediment solids) is best, and 24 hr of shaking time is sufficient to achieve steady-state conditions during batch leaching of sediments (Brannon et al. 1989; Brannon, Myers, and Price 1990; Myers, Brannon, and Price 1992).

Sequential batch leach tests were used in three major dredged material disposal alternative evaluations (Environmental Laboratory 1987; Myers and Brannon 1988; Palermo et al. 1989) to determine how the equilibrium solid phase contaminant concentration (C_s) was related to the equilibrium aqueous phase contaminant concentration (C_w) during leaching. A relationship between C_s and C_w is needed in order to evaluate the source term S in Equation 27. The source term S is obtained from the chain rule as follows:

$$S = -\frac{\rho_b}{\epsilon}\frac{\partial C_s}{\partial t} = -\frac{\rho_b}{\epsilon}\frac{\partial C_s}{\partial C_w}\frac{\partial C_w}{\partial t} \tag{29}$$

Sequential batch leach tests provide the information needed to evaluate $\partial C_s/\partial C_w$.

By sequentially leaching an aliquot of sediment, a table of solid phase contaminant concentration (C_s) versus aqueous phase contaminant concentration can be developed and plotted (successive batches have differing C_s and C_w concentrations). A plot of C_s versus C_w yields a desorption isotherm, the shape of which indicates the type of desorption. Several types of desorption isotherms have been observed in sequential batch leaching of sediments (Environmental Laboratory 1987; Myers and Brannon 1988; Palermo et al. 1989; Myers, Brannon, and Price 1992).

The desorption isotherms shown in Figure 15 are typical for metals in freshwater sediments. A key feature of these desorption isotherms is the constant slope. The slope is the distribution coefficient, K_d, and it can be shown that $\partial C_s/\partial C_w = K_d$. As previously discussed, K_d's obtained from sequential batch leach tests do not need an adjustment to account for facilitated transport. In this case, the source term formulation developed using Equation 29 is relatively simple, and when Equation 27 is solved, predicted metal concentrations in the leachate decrease as the dredged material solids are leached by percolating rainwater. This monotonic decrease in aqueous phase contaminant concentration as the solid phase contaminant concentration decreases is a characteristic of classical desorption processes.

A commonly observed feature of desorption isotherms for metals in freshwater sediments is that the isotherm does not go through the origin. The intercept is the amount of metal in geochemical phases that is resistant to aqueous leaching. The difference between C_s and the intercept is equivalent to the C_{sL} discussed previously. Accurate measurement of C_{sL} is important because the initial metal pore water concentration needed to set the initial condition for Equation 27 is calculated using Equation 25(a or b) for organics and Equation 26 for metals.

Progress in developing a column leach test as a laboratory-scale physical model of contaminant leaching from dredged material has been slower than the development of sequential batch leach tests (Myers, Gambrell, and Tittlebaum 1991; Myers, Brannon, and Price, 1992). Problems with the time required to run column leach tests and the potential for sample deterioration during extended sample collection periods have been encountered. An improved column leaching apparatus has been designed (Figure 16) and is being used in current column leaching studies (Myers, Brannon, and Price 1992). The new column design increases the number of pore volumes that can be eluted in a given period of time, minimizes wall effects, and provides improvements in flow delivery and control.

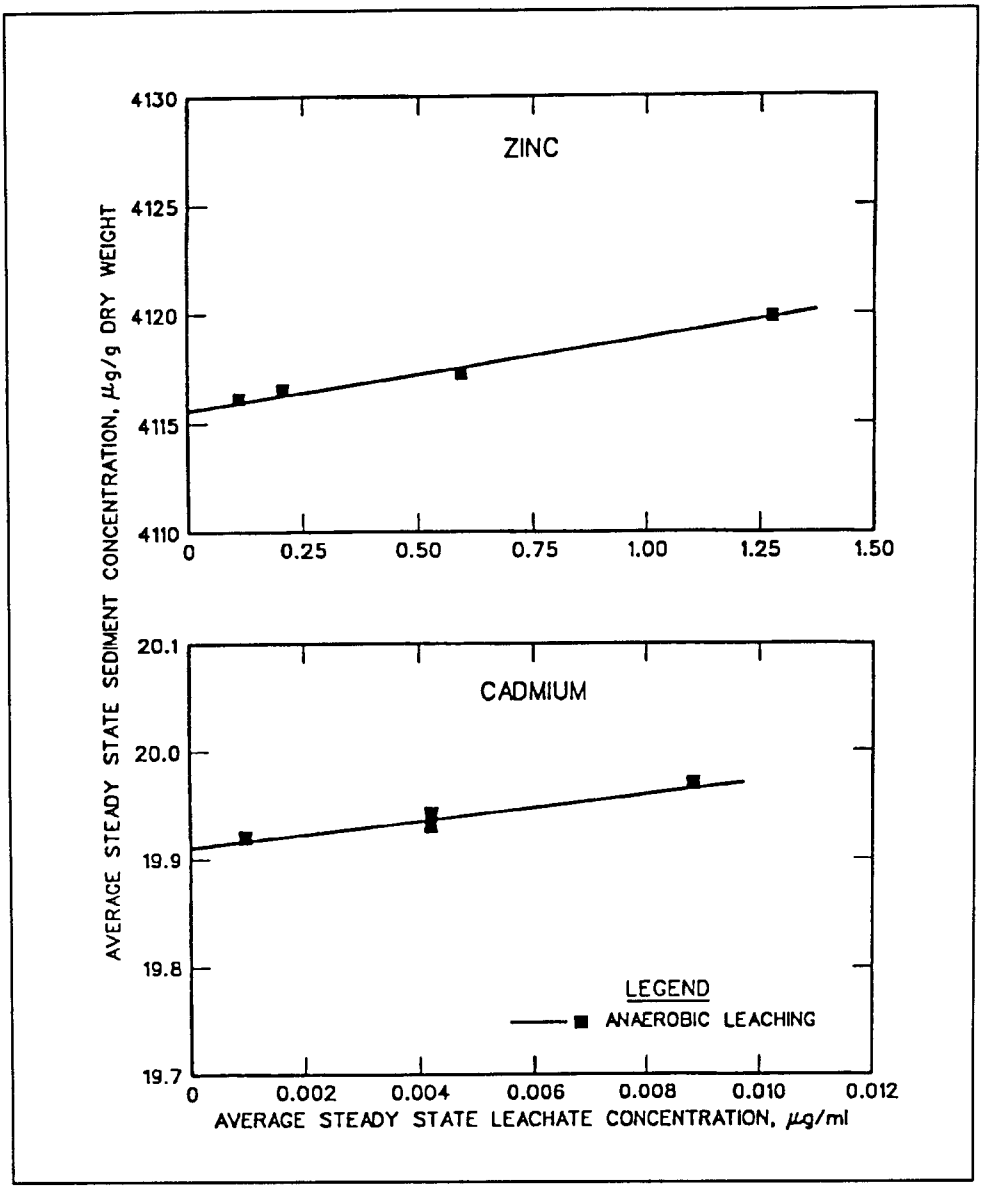


Figure 15. Desorption isotherms for zinc and cadmium in Indiana Harbor sediment (Environmental Laboratory 1987)

Elution curves obtained from column leach tests generally follow the trends indicated in sequential batch leach tests, although the sequential batch leach test usually overpredicts contaminant concentrations in column leachates (Environmental Laboratory 1987; Myers and Brannon 1988; Palermo et al. 1989). An example is shown in Figure 17. Several explanations for differences in predicted and observed contaminant concentrations in column leachate are possible, but no single explanation satisfactorily explains all the information available (Myers and Brannon 1988). Four explanations that have been considered are listed below:

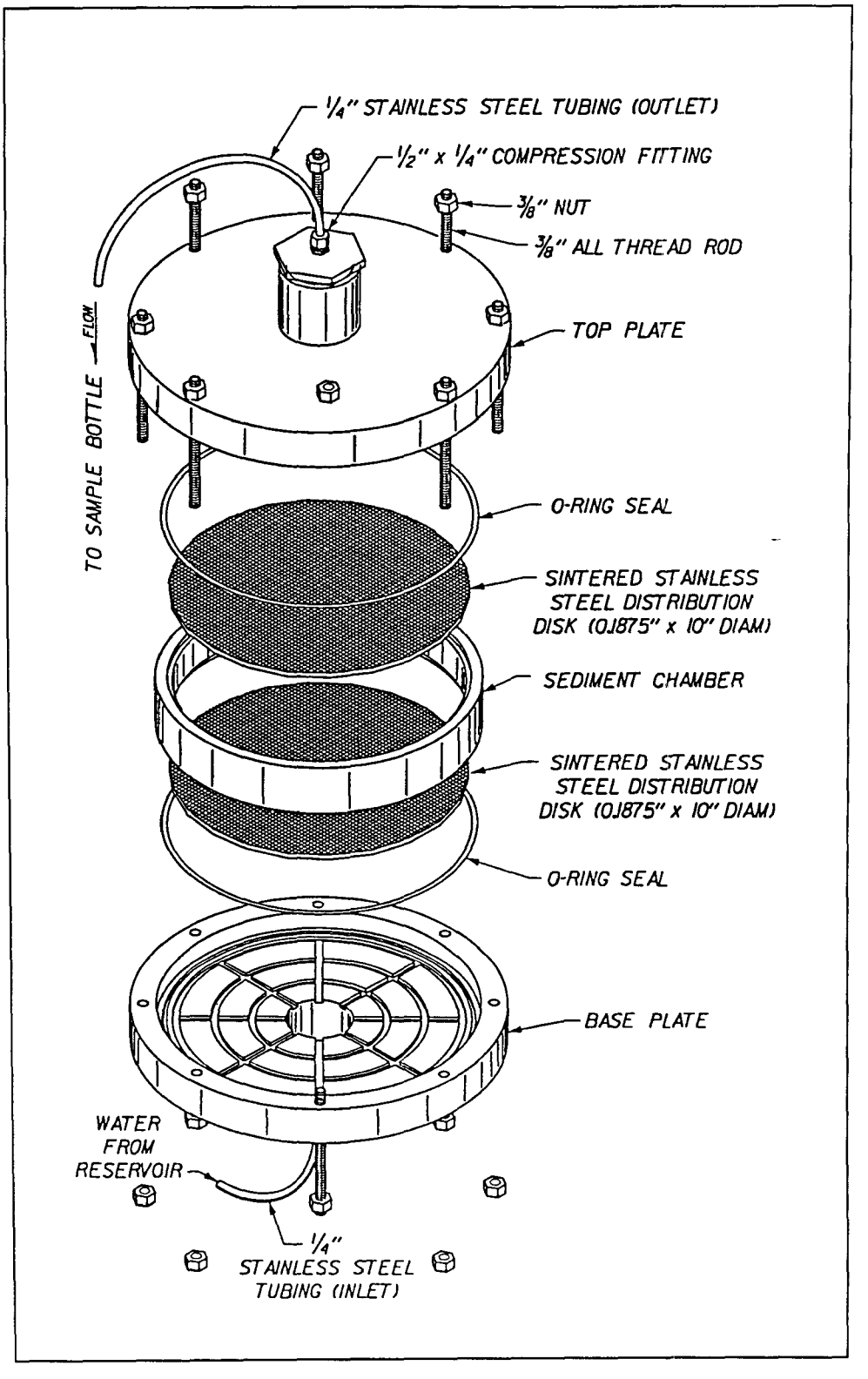


Figure 16. Schematic of improved column leaching apparatus for sediments and dredged material

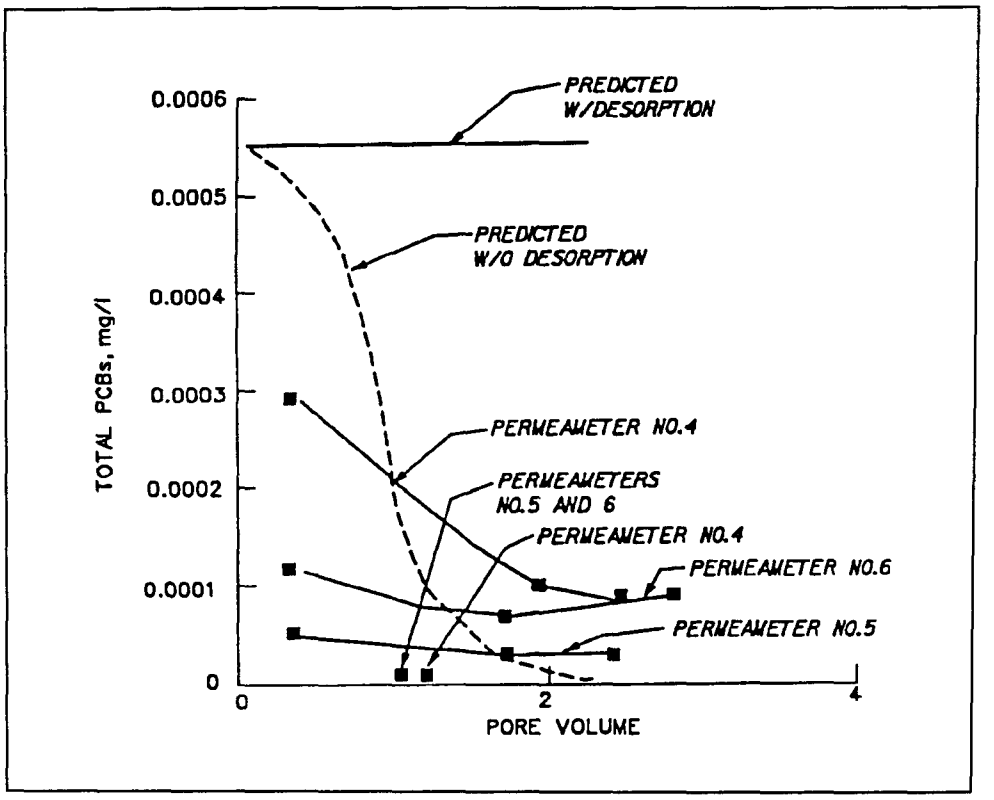


Figure 17. Total PCB concentrations in anaerobic column leachate for Indiana Harbor sediment (from Environmental Laboratory 1987)

a. Short-circuiting in the columns dilutes leachate with clean water.

b. Desorption in the columns is not equilibrium controlled.

c. Contaminant losses are not properly accounted for in collection vessels.

d. Particle disaggregation in batch tests leads to underestimation of distribution coefficients.

Research aimed at determining the cause or causes for the tendency of batch data to overpredict column data is continuing.

Because the equilibrium assumption used in designing the sequential batch leach test is a conservative assumption for contaminant desorption, and there are data from three studies indicating the sequential batch leach test to be a conservative predictor, the sequential batch leach procedure discussed is the recommended laboratory leach test for predicting dredged material leachate quality for freshwater sediments. Until the sequential batch leach test is fully developed and verified, column leaching and application of the integrated approach is also recommended. Additional discussion of dredged material leachate quality prediction including review of available field data is presented in Losses From Confined Disposal Facilities in the section on leachate.

Runoff

Runoff is not likely to be a significant contaminant loss pathway during pretreatment in primary settling facilities and flow equalization facilities that include engineering controls for runoff. During filling operations, water added by precipitation will become a minor component of the effluent flow. Contaminant losses associated with effluent have been previously discussed. After filling and while dredged material is being held for treatment or disposal, runoff will be a stochastic event that is low volume relative to effluent during hydraulic filling (a steady event). Runoff can be controlled by ponding water and allowing it to evaporate. It is, therefore, anticipated that engineering controls for containment of runoff will be implemented. If, however, pretreatment facilities are designed and operated such that runoff is not controlled, runoff will carry contaminants out of the facility. If necessary, the techniques discussed in Losses From Confined Disposal Facilities in the section on runoff can be applied to estimate contaminant losses in runoff during pretreatment.

Volatilization

Volatilization is the movement of a chemical into the air from a liquid surface. Volatilization from dredged material solids involves desorption through a water film covering the solids and then from the water to the air. Because chemicals must enter the water phase before they can volatilize from dredged material, the tendency of a chemical to volatilize from dredged material can be generally related to the Henry's constant. Henry's constant is the equilibrium distribution of a volatile chemical between air and water if true solutions exists in both phases (Thibodeaux 1979). There are various ways to express Henry's constant (Thibodeaux 1979). Two commonly used definitions that yield dimensionless Henry's constants are given below.

$$H = \frac{C_a}{C_w} \tag{30}$$

$$H = \frac{M_A \, p_A^*}{R \, T \, C} \tag{31}$$

where

C_a = dissolved concentration of chemical A in air, g/cm^3

C_w = dissolved concentration of chemical A in water, g/cm^3

H = Henry's constant, dimensionless

M_A = molecular weight of chemical A, g/mole

p^*_A = pure component vapor pressure of chemical A, atm

R = universal gas constant, 82.1 atm cm^3/mol K

T = temperature, K

C = solubility of chemical A in water, g/cm^3

Henry's constant and, therefore, volatilization tendency depend on aqueous solubility, vapor pressure, and molecular weight. Chemicals with high Henry's constant will tend to volatilize while chemicals with low Henry's constant will tend to dissolve in water. As indicated by Equation 30, Henry's constant is directly proportional to vapor pressure and inversely proportional to aqueous solubility. Chemicals with similar vapor pressures but different aqueous solubilities will have different volatilization tendencies. For example, the vapor pressures for lindane and Aroclor 1260 are 1.2×10^{-8} and 5.3×10^{-8} atm, respectively; but the Henry's constant for lindane is only 2.2×10^{-8}, while the Henry's constant for Aroclor 1260 is 0.3 (Thomas 1990a). Although the vapor pressures for both chemicals are very low, the Henry's constants differ by four orders of magnitude due to differences in aqueous solubility. The aqueous solubility of lindane and Aroclor 1260 are 7.3 and 2.7×10^{-3} g/cm^3, respectively (Thomas 1990a). This example shows that vapor pressure is not a good indicator of volatilization tendency from water. The actual direction of chemical movement across the air-water interface depends on chemical concentrations in aqueous and air phases and Henry's constant. The transfer rate (absorption for transfer to water and volatilization for transfer to air) depends on wind-induced turbulence at the air-water interface.

Theoretical chemodynamic models for volatile emission rates from dredged material were described by Thibodeaux (1989). Thibodeaux (1989) identified four emission locals, each with its own sources and external factors affecting emission rates. These four locales were as follows:

a. Dredged material transportation devices.

b. Ponded dredged material.

c. Exposed sediment.

d. Vegetation covered dredged material.

Locales b through d are shown in Figure 18. The first locale, volatile losses during transportation, was discussed previously. The last locale is not applicable to pretreatment facilities because it is anticipated that dredged material will be removed for treatment or disposal before vegetation can be established. This section, therefore, discusses volatile losses from two pretreatment

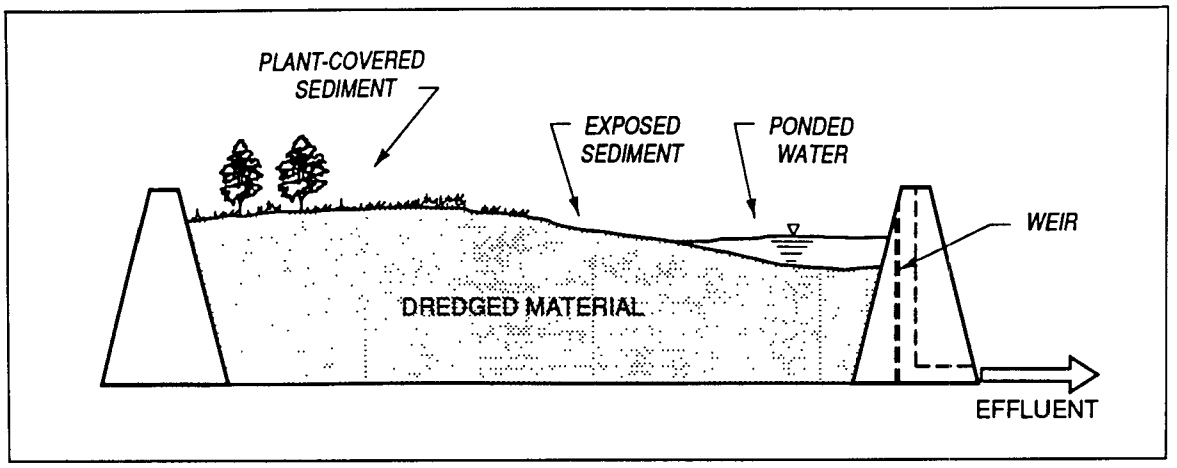


Figure 18. Volatilization locales for a CDF

volatilization locales, losses from ponded dredged material and exposed dredged material solids.

Locale b - ponded dredged material. Dredged material slurries pumped to primary settling facilities or CDFs undergo sedimentation, resulting in a thickened deposit of settled material overlain by clarified supernatant (Figure 4a). Thus, the ponded dredged material locale is characterized by water containing contaminated suspended solids and a thickened bottom deposit of dredged material. The volatilization pathway in this case involves desorption from the contaminated suspended solids followed by transport through the air-water interface.

The bottom deposit is not part of the pathway because suspended solids control dissolved contaminant concentrations, and it is dissolved chemicals that volatilize. While bottom deposits can contribute to dissolved contaminant concentrations, the contribution from bottom deposits is not important until the suspended solids concentration becomes negligible. In a primary settling facility, there is a continuous flux of suspended solids through the water column while dredged material is being pumped in. Diffusion from bottom deposits is, therefore, unimportant relative to desorption from suspended solids in controlling dissolved contaminant concentrations in primary settling facilities.

The model equation for volatilization from the ponded dredged material locale is given below (Thibodeaux 1989)

$$N_w = K_{OL}\left(C_w - C_w^*\right) \tag{32}$$

where

N_w = flux through air-water interface, g/cm^2 sec

K_{OL} = overall liquid phase mass transfer coefficient, cm/sec

C_w = dissolved contaminant concentration, g/cm^3

C^*_w = hypothetical dissolved chemical concentration in equilibrium with background air, g/cm^3

The dissolved contaminant concentration, C_w, can be estimated using Equation 25-a, or data on dissolved contaminant concentrations from the modified elutriate test can be used. The facilitated transport factor (Equation 25-b) should not be included because contaminants sorbed to colloidal organic matter must desorb before they can volatilize. For primary settling facilities, the ponded water area is known and the suspended solids can be predicted using the column settling tests previously discussed on losses for the effluent pathway. Equation 32 is applicable when the dissolved contaminant concentration is constant. Since volatilization continuously removes chemical mass from the dissolved phase, there is an implicit assumption for application of Equation 32 that either volatilization is so small that it does not affect dissolved chemical concentrations or there is a source(s) of chemical that replenishes the dissolved chemical mass as fast as it volatilizes. The effect that volatilization has on dissolved chemical concentrations depends on physical and chemical properties of the chemical of interest and site conditions. For these reasons, the relative significance of volatilization as a process affecting dissolved concentrations cannot be evaluated without applying a fate and transport model that simulates all the important processes. In primary settling facilities, however, there are two sources that can replenish chemical mass lost through volatilization. First, chemical is being continuously added in dissolved form by disposal operations. Second, there is a continuous solids flux through the water column that through partitioning processes tends to maintain constant dissolved chemical concentrations. For these reasons, the assumption of a constant dissolved chemical concentration is probably a good approximation of the field condition. It is also a conservative assumption since the gradient driving the volatilization process is not allowed to decrease.

Equation 32 has not been field verified for dredged material in pretreatment facilities or CDFs. The equation is, however, widely accepted and has been verified for volatile chemical emissions from various water bodies and waste impoundments (Liss and Slater 1974; Dilling 1977; Thibodeaux 1979; Thibodeaux, Parker, and Heck 1984). Probably the largest source of error in Equation 32 is estimation of the overall liquid phase mass transfer coefficient.

The overall liquid phase mass transfer coefficient depends on a variety of variable environmental and hydrodynamic factors that are difficult to quantify. Lunney, Springer, and Thibodeaux (1985) correlated overall liquid phase mass

transfer coefficients to wind speed and molecular diffusivity in water. Their correlation is presented below.

$$K_{OL} = 19.6\ V_x^{2.23}\ D_A^{.667} \tag{33}$$

where

K_{OL} = over-all liquid phase mass transfer coefficient, cm/hr

V_x = wind speed, mph

D_A = molecular diffusivity of chemical A in water, cm^2/sec

Other empirical equations are available for estimating K_{OL}, but the Lunney, Springer, and Thibodeaux (1985) equation is one of the most widely used equations. If the molecular diffusivity in water is not known, it can be estimated using Oldham's law as follows (Thibodeaux 1979):

$$\frac{D_A}{D_B} = \left[\frac{M_B}{M_A}\right]^{0.6} \tag{34}$$

where

A = chemical of unknown molecular diffusivity

B = model chemical of known molecular diffusivity

D_A = molecular diffusivity of chemical A in water, cm^2/sec

D_B = molecular diffusivity of chemical B in water, cm^2/sec

M_B = molecular weight of chemical B, g/mole

M_A = molecular weight of chemical A, g/mole

Equation 33 is an empirical model that lumps chemical property and environmental variables into just two parameters, wind speed and aqueous diffusivity. Since there are no field verification data for Equation 33 at dredged material pretreatment and disposal facilities, the range of error is not known. It is estimated that Equation 33 provides K_{OL} values within an order of magnitude. Part of the potential error is associated with selecting an average wind speed to represent a range of wind speeds over some period of time.

Thomas (1990a) describes some alternative techniques for estimating the overall liquid phase mass transfer coefficient that are based on two-resistance theory as follows (Liss and Slater 1974; Thibodeaux 1979):

$$\frac{1}{K_{OL}} = \frac{1}{K_L} + \left[\frac{1}{H\,K_G}\right] \tag{35}$$

where

K_L = liquid-side mass transfer coefficient, cm/sec

K_G = gas-side mass transfer coefficient, cm/sec

Although Equation 35 is a theoretical equation, estimation of K_G and K_L is highly empirical. Thomas (1990a) suggests using Southworth's correlations for volatilization of polynuclear aromatic hydrocarbons to estimate K_G and K_L as follows:

$$K_G = 0.32\,(V_x + V_{curr})\sqrt{\frac{18}{M_A}} \tag{36}$$

where

K_G = cm/sec

V_x = wind speed, m/sec

V_{curr} = water velocity, m/sec

For wind speeds less than 1.9 m/sec, K_L in cm/sec in given by

$$K_L = 0.0065\left[\frac{V_{curr}^{0.969}}{Z^{0.673}}\right]\sqrt{\frac{32}{M_A}} \tag{37}$$

where Z is water depth in meters, K_L in cm/sec. For wind speeds greater than 1.9 m/sec and less than 5 m/sec,

$$K_L = 0.0065 \left[\frac{V_{curr}^{0.969}}{Z^{0.673}} \right] \sqrt{\frac{32}{M_A}} \; e^{0.526\left(V_x^{-1.9}\right)} \tag{38}$$

When there exists no mean advective current in a CDF, wind-driven currents are of the order of 3 percent of wind speed, assuming continuity of shear stresses at the air-water interface. Thus, V_{curr} in Equations 36-38 can be replaced with 3 percent of the wind speed.

There are numerous empirical equations from stream reaeration studies that could also be adapted for estimating volatile emissions. Since the only consensus about these equations is that no one equation is superior for modeling reaeration, these equations are not discussed. It is recognized, however, that there are other estimation techniques available for mass transfer coefficients and that most of these techniques give approximately equivalent results.

Thomas (1990a) also discusses using rule-of-thumb values for K_G and K_L when making the type of a priori estimates discussed in this report. These rule-of-thumb values are presented in Table 9.

Table 9
Rule-of-Thumb Values for Liquid- and Gas-Side Mass Transfer Coefficients (cm/hr)

	K_L[1]	K_G[2]
V_x < 3 m/sec	3	--
3 m/sec < V_x < 10 m/sec	5-30	--
V_x > 10 m/sec	<70	--
Sea Surface Conditions		$K_G = 3{,}000\ (18/M_A)^{1/2}$

Note: V_x = Windspeed; M_A = Molecular weight of contaminant.
[1] From Cohen, Cocchio, and Mackay (1978) as cited by Thomas (1990a).
[2] Thomas (1990a).

The recommended estimation technique for K_{OL} is Equation 33 followed by a check against Equation 35 using values from Table 9 for K_G and K_L. If the value predicted by Equation 33 is substantially lower than the value predicted by Equation 35 using data from Table 9, an estimate should be made using Equations 35-38. If the value predicted by Equation 33 is within a factor of 3 of the value predicted by Equations 35-38, either value is appropriate. If the two predictions differ by more than a factor of 3, judgment has to be used. The alternatives are as follows: (a) select the one that seems most appropriate, (b) select the highest value (conservative approach), (c) use the value predicted by Equation 35 using data from Table 9, or (d) take the average of all the estimates.

In view of the lack of field data on volatilization from dredged material pretreatment and disposal facilities, it is not possible to determine which technique is the most accurate for estimating mass transfer coefficients. The correlations in Equations 36-38 were developed, however, for very similar situations of evaporation from surface impondments. For this reason, alternative predictive techniques including a rule-of-thumb approach were described above. The information from the literature suggests that the techniques discussed in this report should be accurate to within an order of magnitude (Thomas 1990a).

Locale C - exposed sediment. This volatilization locale is characterized by sediment that is exposed directly to air and void of vegetative or other cover. Exposed sediment is probably the most significant of the four volatilization locales as a source of volatile emissions (Thibodeaux 1989). Exposed sediment will be a source of volatile emissions during various stages of pretreatment and flow equalization as follows:

a. The delta formed during primary settling of dredged material slurries (Figure 4a).

b. The dredged material in filled primary settling facilities after ponded water is drawn off (Figure 4b).

c. The delta formed during mechanical placement of dredged material in in-water or nearshore flow equalization facilities.

d. The dredged material in upland flow equalization facilities for mechanically dredged material.

The rate at which chemicals volatilize from exposed sediment is affected by many factors. Geotechnical properties such as porosity and water content, chemical factors such as water and air diffusivities, and environmental factors such as wind speed and relative humidity all affect volatilization rates. In addition, processes such as air-water-solids chemical partitioning, diffusion of thermal energy, evaporation of water, and desiccation cracking of the sediment can have pronounced impacts on volatile emission rates for exposed sediment. Complete mathematical coupling of all these processes and the factors affecting these processes into a model equation(s) would lead to a very complex model requiring site-specific data that are usually unavailable. For this reason, the vignette models proposed by Thibodeaux (1989) are recommended for a priori prediction.

Dredged material begins evaporative drying and volatile chemical emission as soon as it is exposed to air. Initially, the chemical emission rate is affected by gas-side resistance. The top microlayer quickly becomes depleted of volatile chemicals (and water), so that, continuing losses of volatile chemicals come from the pore spaces within the dredged material. At this point, the emission process is transient and changes from being gas-side resistance

controlled to dredged material-side vapor diffusion controlled. The overall process is modeled by Equation 39 below (Thibodeaux 1989).

$$n_e = \frac{\left[\frac{C_s H}{1000 K_d} - C_{ai} \right]}{\sqrt{\frac{\pi t}{D_{a3} \left[\epsilon + \frac{K_d}{\rho_b} \right]}} + \frac{1}{K_G}} \tag{39}$$

where

n_e = instantaneous flux of chemical A through the dredged material-air interface at time t, mg/cm^2 sec

H = Henry's constant, dimensionless

K_d = contaminant specific equilibrium distribution coefficient, cm^3/g

C_{ai} = background concentration of chemical A in air at dredged material-air interface, mg/cm^3

π = 3.14159 ....

t = time since initial exposure, sec

D_{A3} = effective diffusivity of chemical A in the dredged material pores, cm^2/sec

ϵ_1 = air-filled porosity, dimensionless

ρ_b = bulk density, g/cm^3

K_G = gas side mass transfer coefficient, cm/sec

Equation 39 is an idealized diffusion model that describes chemical movement in the unsaturated zone near the air-dredged material interface. The emission pathways modeled include surface depletion, desorption from particle surfaces into a water film surrounding the particle surfaces (hence, the appearance of K_d), desorption from the water film into the pore gas (hence, the appearance of H), and vapor phase diffusion in the dredged material pore spaces (hence, the appearance of D_{A3}, ϵ_1, and ρ_b).

The instantaneous flux predicted by Equation 39 decreases with time as shown in Figure 19. Decreasing flux with time is a characteristic of

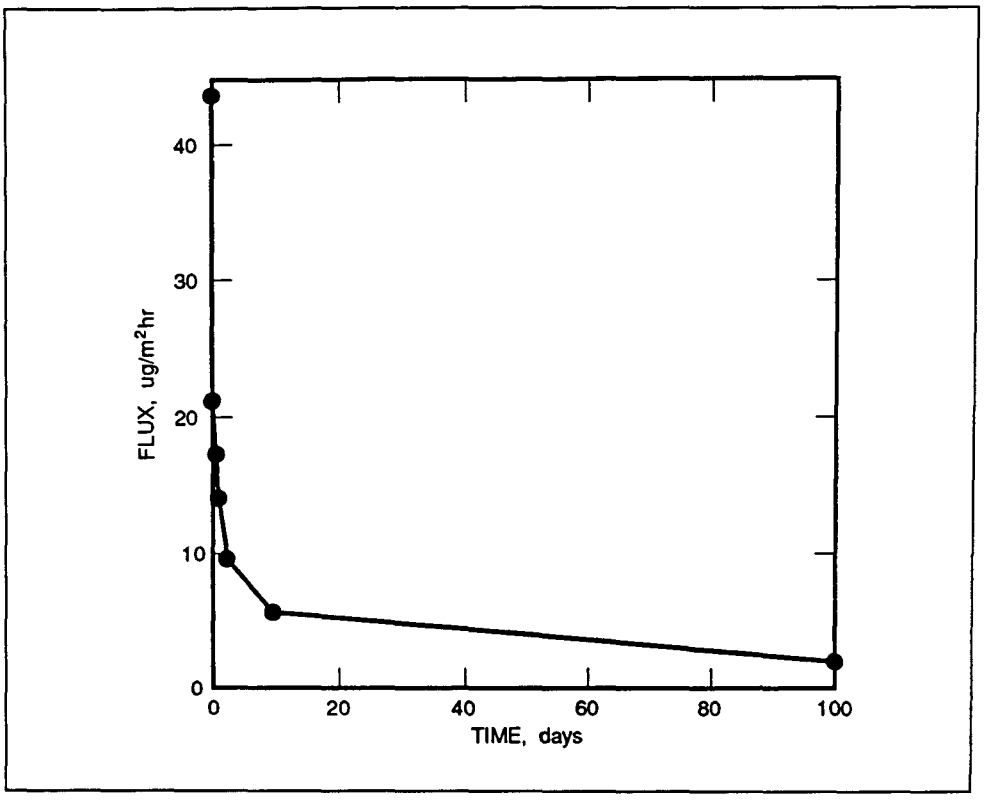


Figure 19. Predicted Aroclor 1242 flux from exposed New Bedford Harbor Superfund sediment

contaminant volatilization from soils that is often observed in controlled laboratory studies (Mayer, Letey, and Farmer 1974). The total mass loss is the area under the curve multiplied by the surface area of exposed sediment. The area under the curve is the integral of Equation 39 with respect to time. A number that is useful for estimating mass loss is the average flux over some time t' given by

$$\bar{n}_a = \frac{\int_0^{t'} n_a \, dt}{\int_0^{t'} dt} \tag{40}$$

Simple numerical techniques can be used to perform the integrations indicated in Equation 40. If the top microlayer depletion is neglected, the K_g term disappears from Equation 39. For this simplification, performing the indicated integrations yields the approximate solution

$$\bar{n}_a = 2\, n_a \tag{41}$$

Thus, the average volatile flux over some time t is just twice the instantaneous flux at time t. Average flux multiplied by the area of exposed sediment and the exposure time yields the total volatile loss.

The diffusion equation on which Equation 39 is based is well established for pesticide volatilization from soil surfaces (Hamaker 1972; Mayer, Letey, and Farmer 1974; Thomas 1990b) and has been successfully applied to modeling emissions from landfarming operations (Thibodeaux and Hwang 1982) and hazardous waste impoundments (Dupont 1986). Solutions to the diffusion equation involving different boundary conditions than those used in deriving Equation 39 are available (Carslaw and Jaeger 1959) and have been applied to modeling volatilization of pesticides from soil (Thomas 1990b).

Extrapolation of models for soils to dredged material has not, however, been verified, and there are aspects of the simple model previously discussed that need further development. For example, the effects of water content and water evaporation on volatilization rates are not included in Equation 39. The effective diffusion coefficient D_{A3} can be estimated by

$$D_{A3} = D_{A1} \frac{\epsilon_1^{10/3}}{\epsilon^2} \tag{42}$$

where

D_{A1} = air diffusivity of compound

ϵ_1 = air-filled porosity

ϵ = total dredged material porosity

This relationship shows that the effective diffusion coefficient is very sensitive to changes in the water content and porosity of the dredged material. Fully saturated dredged material exhibits a very low diffusion coefficient. The effects of desiccation and the subsequent reduction of porosity on volatile emissions from dredged material have not been systematically investigated. Since porosity is an important parameter, the assumption of constant porosity could lead to substantial errors in volatile emission estimated from exposed dredged material.

Thibodeaux (1989) and Taylor and Glotfelty (1988) discuss the importance of water content and evaporation of water as factors and processes affecting volatilization. Major differences in diurnal volatilization rates have been

observed that are related to water content. Volatilization rates decrease during the day as the soil surface dries and increase at night as soil moisture losses during the day are replaced by subsurface soil moisture. Volatilization rates have also been observed to increase significantly following rainfall. The effect is probably due to competitive adsorption between water molecules and contaminant molecules for sorption sites on soil particles.

Evaporation induces an upward movement of water that results in convective flow of the bulk pore gas. Thibodeaux (1989) presented an enhancement factor approach to account for evaporation that simplifies coupling convective movement of water and diffusive movement of volatile chemicals. Convective movement of water, however, distorts diffusive gradients, and evaporation is not a continuously steady process. Evaporation varies greatly under field condition and may cease at high relative humidity.

Thibodeaux (1989) also recognized desiccation cracking of the dredged material surface as a process likely to affect volatilization and suggested some approaches to developing volatile emission models that include the effects of desiccation cracking. Figure 20 shows the type of desiccation cracking that takes place in fine-grain dredged material. Such cracks can encompass up to 20 percent of the volume of the surface crust that develops by evaporative drying (Haliburton 1978).

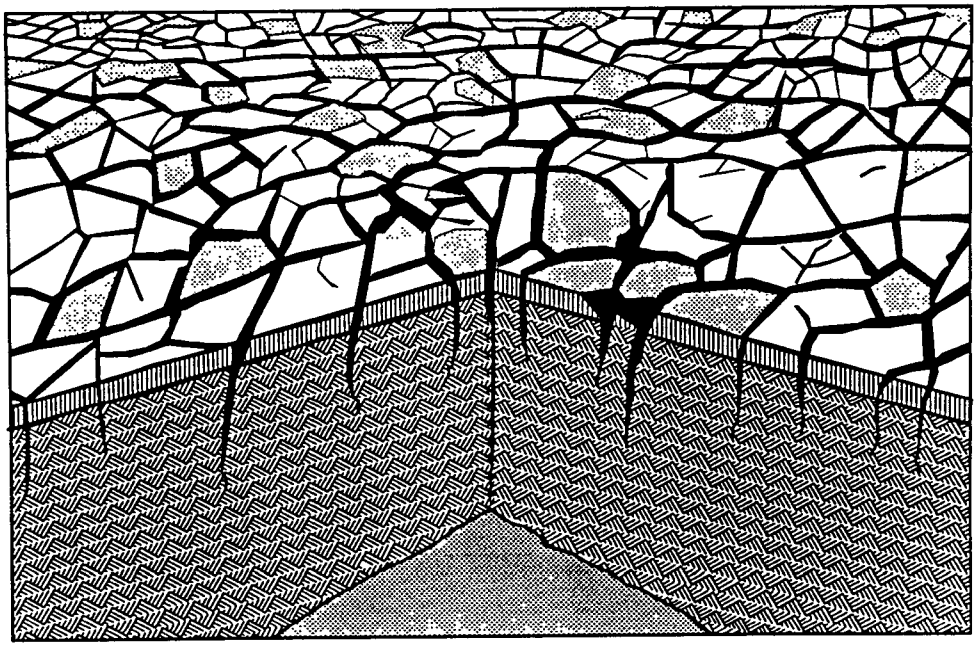


Figure 20. Desiccation cracking of exposed dredged material

Volatile emission summary. Predictive techniques for the ponded dredged material and the exposed sediment volatilization locales were described. The predictive techniques, however, are based on simple models that in some cases do not account for important factors and/or processes. Development of

predictive models that take into account water content, water evaporation, and desiccation cracking is a critical need for estimating volatilization losses from exposed dredged material. Laboratory and field testing is needed to build a higher degree of confidence in the predictive capability of the available volatilization models.

5 Losses From Confined Disposal Facilities

Background

Confined disposal facilities[1] are often used in the Great Lakes for disposal of dredged materials that are unsuitable for open-water disposal. When contaminated dredged material is placed in a CDF, contaminants may be mobilized and transported away from the CDF by a variety of physical, chemical, and biological processes. Release rates vary depending on the chemical and engineering properties of the dredged material, the method of dredging and dredged material placement, CDF location, stage of filling, and CDF design, operation, and management.

Pathways involving movement of large masses of water, such as CDF effluent, have the greatest potential for moving significant quantities of contaminants out of CDFs (Brannon et al. 1990). Other water-related migration pathways include ponded water seepage through permeable dikes, seepage of leachate through permeable dikes, seepage of leachate through foundation soils, and surface runoff. Pathways such as volatilization may also result in movement of substantial amounts of volatile organic chemicals at certain stages in the filling of a CDF. Internal contaminant cycling can also be important in the long-term mass balance for CDFs (Brannon et al. 1990).

This section begins with an overview of CDF disposal technology, followed by a review of the literature on contaminant losses from CDFs. Predictive techniques for effluent, leachate, and volatile losses, major contaminant loss pathways for pretreatment facilities and CDFs, were discussed in Contaminant Losses During Pretreatment. CDFs have additional contaminant loss pathways that must be considered—losses associated with runoff and dike seepage. Predictive techniques for runoff and dike seepage losses are discussed in this section.

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[1] The terms confined disposal facility, confined disposal area, confined disposal site, diked disposal area, containment area, and diked dredged material containment area are used interchangeably in the literature.

Overview of Confined Disposal Facility Technology

Contaminant releases from CDFs depend on a number of factors including CDF design, operation, and management, nature and level of contamination in the dredged material, and the physicochemical environment of the disposal site. Factors related to site location and CDF design, operation, and management are discussed in this section.

CDF siting locales

CDFs can be located in three disposal environments: upland, nearshore, and in-water (Figure 21). Upland CDFs may be formed by construction of earthen dikes or the use of existing pits or depressions. Nearshore and in-water CDFs may be constructed with soil, stone, or combination soil and stone-filled dikes. There are numerous modifications of these dike design themes such as back-filling with stone on either side of sheet piling, cellular sheet pile construction, placement of grout-filled fabric mattresses on rock-filled dikes, use of geotextiles in soft foundation soils, and the use of sand blankets and/or clay cores in the dike design.

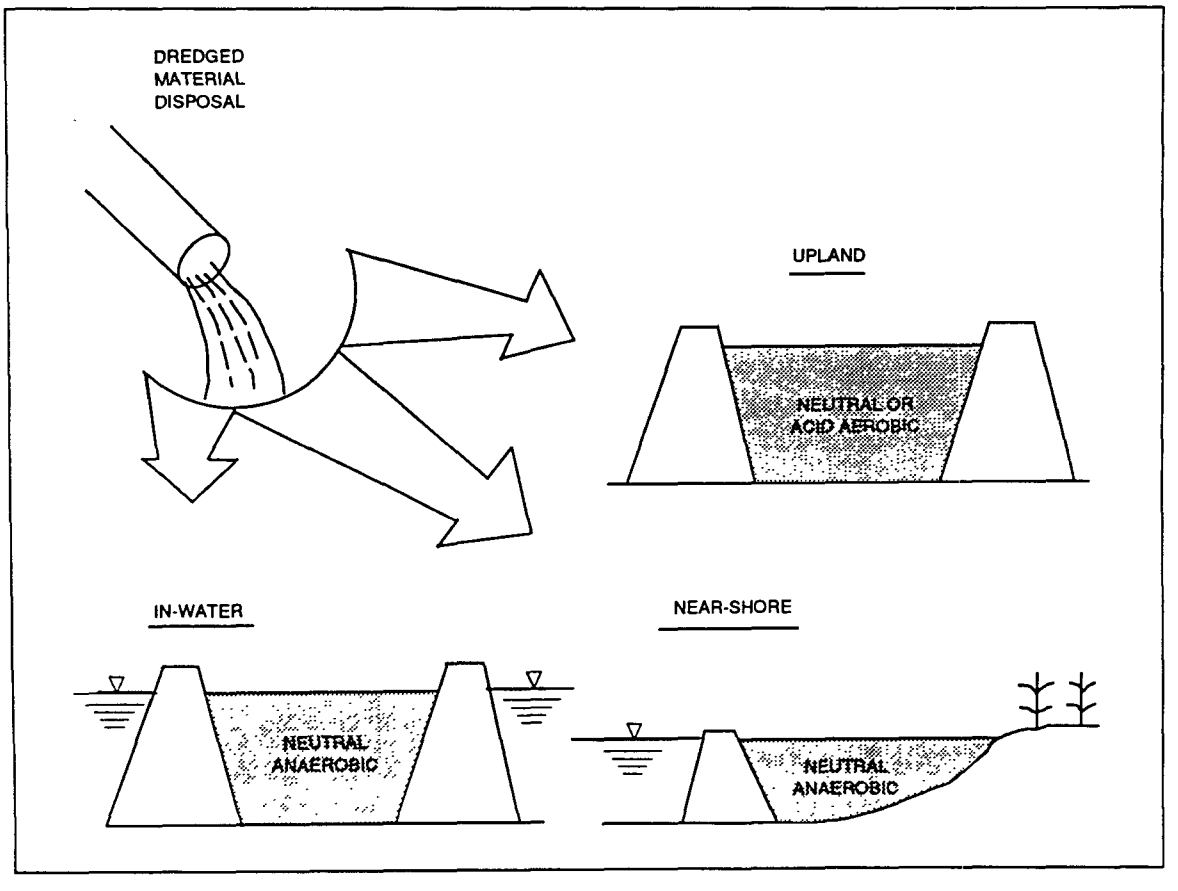


Figure 21. Three general locales for siting CDFs

CDFs are rarely cited far away from the waterway for which the CDF serves as a dredged material disposal site. Transportation costs for remote sites are frequently a prohibitive factor as is the difficulty of finding suitable remote sites. For these reasons, CDFs are usually located near or in the waterway. Upland CDFs are generally located adjacent to the waterway for which the CDF serves as a dredged material disposal site, with one side usually bordering the waterway. Nearshore CDFs are located along a shoreline with three sides bounded by water. In-water CDFs are surrounded on all sides by water.

Physicochemical conditions. Contaminant mobilization is regulated to a large extent by physicochemical conditions (oxidation-reduction potential, pH, and salinity) in the sediment or dredged material (Gambrell, Khalid, and Patrick 1978). In situ sediments normally encountered in highly industrialized ports are fine grained, anaerobic, and near neutral pH. A thin surface layer, usually 1 cm or less thick, may be oxidized. Beneath this surface layer, microbial activity results in a depletion of oxygen, nitrate, and oxidized forms of iron and manganese and accumulation of ammonia nitrogen and reduced forms of iron and manganese. When hydraulic dredging occurs, the sediment is vigorously mixed with overlying site water. The resulting influent to a CDF is a mixture of reduced sediment and oxic site water. Field studies indicate that influents have little or no dissolved oxygen (Hoeppel, Myers, and Engler 1978), probably because the high biochemical oxygen demand of dredged material rapidly depletes the dissolved oxygen in the site water entrained during hydraulic dredging.

Because of the oxygen demand imposed by microbial metabolism, the settled solids in a CDF quickly revert to the anaerobic, near neutral pH conditions previously existing in the in situ sediment and remain anaerobic and near neutrality as long as the dredged material is flooded or saturated. Contaminants in dredged material are generally less mobile under anoxic (flooded) conditions than under oxidized (dewatered) conditions (Peddicord 1988).

Since the physicochemical conditions in a CDF depend on site locale and management, there are some important differences in the long-term mobility of some chemicals in CDFs. The basic difference between physicochemical conditions in an upland CDF and those in nearshore and in-water CDFs is the extent of the penetration of oxic (dewatered) conditions. Disposal in an unlined upland CDF with permeable foundation soils results in dewatering and oxidation of the upper portion of the dredged material profile. Complete dewatering and oxidation is rarely achieved except with sandy sediments placed above the water table. Upland disposal of dredged material high in sulfur (e.g., pyrites) can result in mobilization of metals in the surface crust as the dredged material becomes oxic (dewatered) and the pH drops due to sulfur oxidation. These conditions are not common in CDFs containing freshwater dredged material. Since a major portion of the dredged material profile in most CDFs remains saturated (anoxic, neutral pH), metal mobilization is minimized and is less significant relative to the fully drained condition (Peddicord 1988).

Groundwater interactions. The three CDF siting locales differ significantly in their interaction with groundwater (Yu et al. 1978). Figure 22 is a generalized sketch of groundwater-CDF interactions for the three CDF siting locales shown in Figure 21. In the upland locale, the hydraulic gradient between inside and outside of the CDF tends to drain the CDF and create oxic conditions in a portion of the dredged material profile. The hydraulic gradient is much smaller in the nearshore and in-water locales, so that saturated conditions are more likely to persist in the dredged material profile. For upland and nearshore sites, groundwater impacts are possible depending on site conditions. For an in-water site, groundwater, except in unusual cases, is not significantly impacted.

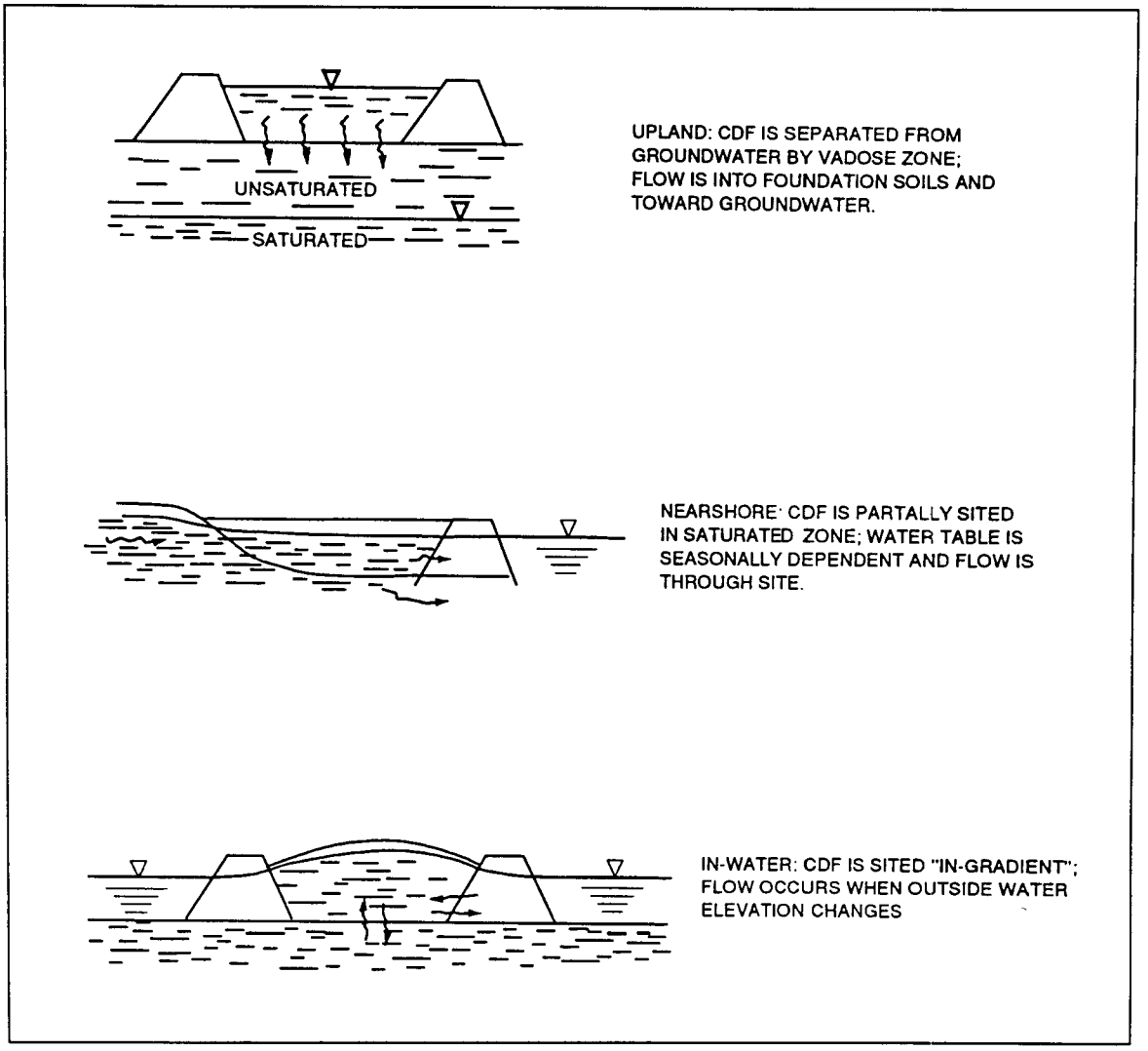


Figure 22. Groundwater-CDF interactions

CDFs are not usually located in groundwater recharge zones. This is because CDFs are cited along waterways, and most waterways receive some groundwater discharge. The impacts of groundwater discharge contaminant losses at CDFs have not been studied extensively because contaminant losses at CDFs are primarily governed by surface hydrology (rainfall, etc.), dredged material properties, and CDF design.

Placement methods

Dredged material is placed in CDFs hydraulically by pipeline dredge, hopper dredge, or scow pumpout and mechanically by bucket dredges. Hydraulic disposal operations involve pumping dredged material into the CDF as a slurry that is 10- to 20-percent solids by weight. Solids settle (Figure 4) and consolidate, and water is discharged through an outlet structure or permeable dikes or both to make room for additional dredged material. Mechanical dredging usually involves dredging and transfer of material to a scow using a bucket. Dredged material may then be transferred from the scow to the CDF by hydraulic or mechanical methods. Because mechanical disposal does not use water for conveyance, the volume of water introduced into a CDF that must later be discharged is significantly reduced when mechanical dredging and disposal methods are used compared with hydraulic dredging and disposal.

Design and operation

CDFs are built by raising dikes around a prescribed area and are designed to retain dredged material solids while allowing the carrier water and/or water initially present in the CDF to be released as the CDF fills with solids. The primary design objectives are as follows: (a) provide adequate storage capacity to meet dredging requirements, and (b) attain the highest possible efficiency in retaining solids during filling operations (Palermo, Montgomery, and Poindexter 1978; USACE 1987).

Solids retention. Solids retention is important because the major fraction of the contaminants in dredged material is bound to sediment solids (Burks and Engler 1978). During hydraulic disposal, water and solids separate in the CDF by gravity sedimentation, and the clarified water is the effluent that potentially impacts surface water quality. The design fundamentals for solids retention during hydraulic filling of CDFs were developed by Montgomery (1978) and refined by Shields et al. (1987). Verification studies of CDF design procedures for solids retention were conducted by Averett, Palermo, and Wade (1988). The settling characteristics of dredged material depend on many variables and must be determined experimentally in laboratory settling tests for each dredging project (Montgomery 1978; Palermo, Montgomery, and Poindexter 1978; Palermo 1986; USACE 1987). Based on the settling characteristics determined in laboratory tests, the residence time required for

clarification to a target effluent suspended solids concentration can be determined. This information is used to size CDFs.

Water release. Release of the water that must be discharged during filling operations is accomplished in three basic ways. Effluent may be released through an outlet structure(s), pervious dikes, or both. There are many ways that these basic methods for water release are implemented, some simple and some complicated. Outlet structures include simple overflow weirs, sand-filled weirs, and multimedia filter cells. Pervious dikes are rock-filled structures that can be built with sand blankets, sheet pile crowns, and other modifications designed to control flow and/or quality of the water released. Outlet structures and pervious dikes are not mutually exclusive, that is, a CDF can be designed to release water through pervious dikes for a period of time, typically until the dikes clog with dredged material solids. After that, water is released through an outlet structure.

Literature on Effluent Losses During Hydraulic Disposal

As previously discussed, influent and effluent flows are approximately equal during hydraulic disposal in most CDFs. During active disposal operations at upland, nearshore, and in-water CDFs, effluent is probably the most significant pathway through which contaminant losses occur. Assuming inflow equals outflow and losses associated with pathways other than effluent are negligible, the containment efficiency equation is

$$CEF_{EFF} = \frac{C_{INF,TOT} - C_{EFF,TOT}}{C_{INF,TOT}} \tag{43}$$

where

CEF_{EFF} = containment efficiency based on effluent pathway only

$C_{INF,TOT}$ = total concentration of contaminants in influent, mg/ℓ

$C_{EFF,TOT}$ = total concentration of contaminants in effluent, mg/ℓ

Equation 43 has been applied in several field studies to individual contaminants. The data are reported as contaminant-specific removal efficiencies in percent. This literature is reviewed below.

Hoeppel, Myers, and Engler (1978)

Influent and effluent samples from nine confined disposal sites collected during hydraulic disposal were studied by Hoeppel, Myers, and Engler

(1978). The nine sites investigated included four on the Atlantic coast, two on the Gulf coast, one on the Pacific coast, one in the Great Lakes, and one inland site. Field measurements included salinity, conductivity, dissolved oxygen, and pH. Laboratory measurements included particle size, solids, alkalinity, combined nitrogen (organic nitrogen, ammonia nitrogen, nitrate nitrogen, and nitrite nitrogen), total and ortho-phosphate phosphorous, total and inorganic carbon, selected pesticides (DDT, DDE, DDD, dieldrin, aldrin, lindane, heptachlor, heptachlor epoxide, and chlordane), PCBs, oil and grease, sulfides, major ions (calcium, magnesium, potassium, sodium, chloride, and sulfate), and trace metals (iron, manganese, zinc, copper, cadmium, lead, nickel, chromium, mercury, arsenic, vanadium, selenium, and titanium). This study showed that most chemical constituents in dredged material were associated with the solids fraction, and the efficiency of contaminant containment during filling operations was directly related to the efficiency of solids retention.

Application of Equation 43 to influent and effluent data for eight of the nine sites is summarized in Figure 23. Reduction in total metal concentrations for iron, zinc, cadmium, copper, nickel, arsenic, vanadium, and lead closely followed total solids removal (96 percent). The metals that showed average retention efficiencies of less than 90 percent included titanium (89 percent), manganese (88 percent), potassium (78 percent), and mercury (46 percent).

Most total nutrient concentrations (total organic carbon, organic nitrogen, and total phosphorus) showed retention efficiencies approximating total solids removal (96 percent). Total ammonia-nitrogen removal was only 57 percent.

Oil and grease, most pesticides, and PCBs showed very efficient removal when adequate solids retention was maintained. Almost all of the oil and grease, pesticide, and PCB was associated with solids in both the influent and effluent samples. Although oil and grease were efficiently removed during dredged material containment, sediments with high contents of petroleum residues seemed to settle more slowly, often forming an oil-water-sediment layer near the bottom of ponded areas in the CDF.

Lu et al. (1978)

Lu et al. (1978) carried out studies similar to those conducted by Hoeppel, Myers, and Engler (1978) at two sites, one in Mobile, AL (Pinto Island), and one in Detroit, MI (Grassy Island). This study placed major emphasis on size fractionation of influent and effluent suspended particulate matter. The results showed that most trace metals, oil and grease, chlorinated pesticides, and PCBs were almost totally associated with settleable solids (>8 μm) in influent and effluent samples. A significant fraction of total calcium, magnesium, sodium, potassium, ammonia-nitrogen, total carbon, and organic carbon was associated with the dissolved phase (<0.05 μm). Containment efficiencies for these parameters were low relative to the solids retention efficiency.

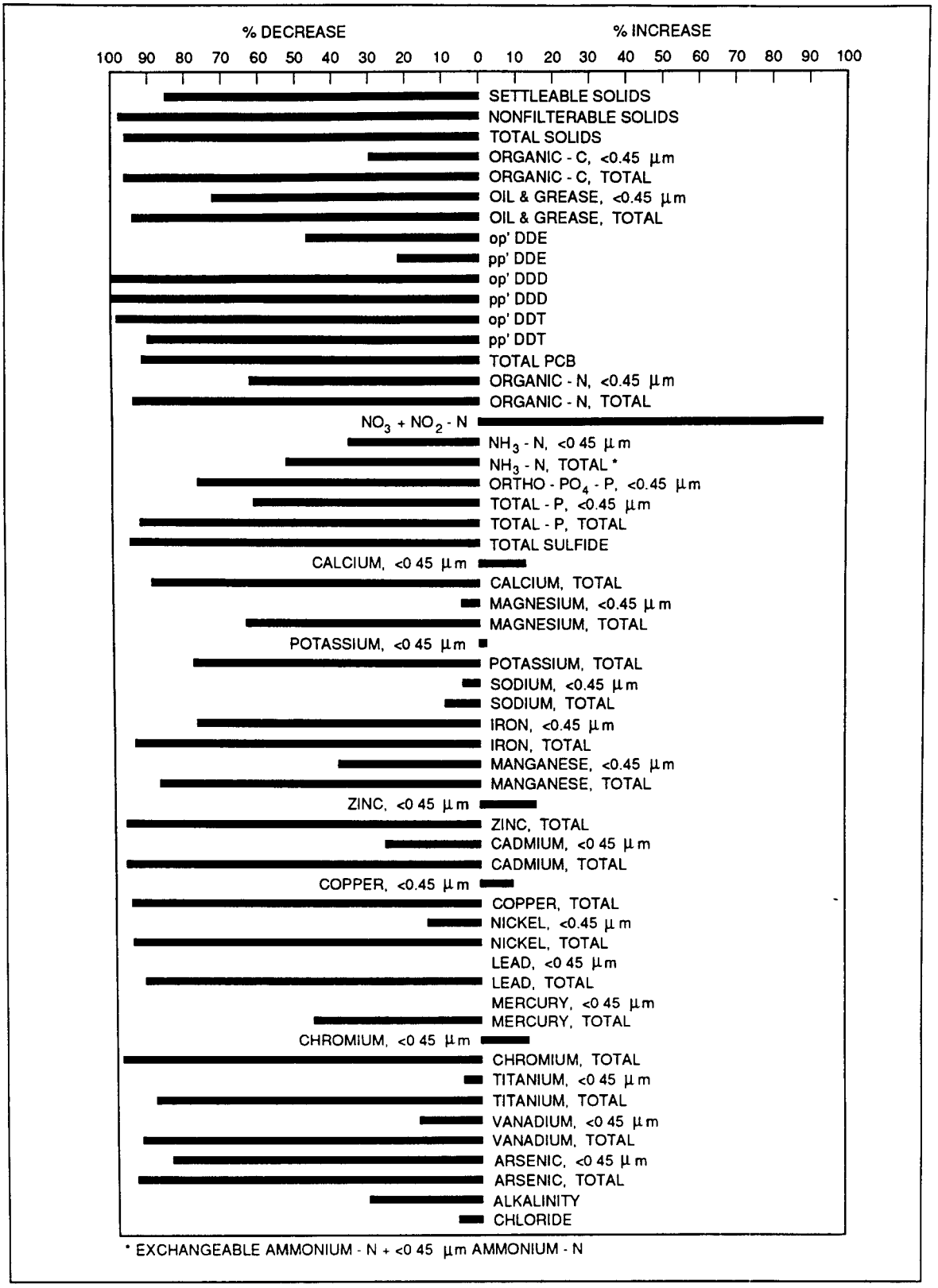


Figure 23. Contaminant containment efficiencies for eight CDFs (Hoeppel, Myers, and Engler 1978 as cited by Palermo 1988)

The Grassy Island CDF is located in the Detroit River and discharges to the Detroit River. Sampling was conducted during hopper dredging and disposal of material from the Rouge River in Detroit, MI. Retention efficiencies for most trace metals, oil and grease, chlorinated pesticides, and PCBs were very close to the total solids retention (99.7 percent) at the Grassy Island CDF. Parameters with retention efficiencies less than 90 percent included ammonia nitrogen (83 percent), total organic carbon (62 percent), potassium (61 percent), total carbon (55 percent), calcium (44 percent), and magnesium (10 percent).

The Pinto Island CDF is located in Mobile Bay, Alabama. Sampling was conducted during hydraulic dredging and direct pipeline disposal. The retention efficiencies at the Pinto Island CDF were generally lower than those at the Grassy Island CDF for trace metals (cadmium, 18 percent; copper, 52 percent; mercury, 35 percent; nickel, 67 percent; lead, 35 percent; selenium, 39 percent; and zinc, 35 percent). Retention efficiencies for organics were much better than for metals at Pinto Island. PCB retention efficiencies for Aroclors 1242, 1254, and 1260 were 96, 97, and 99 percent, respectively.

Palermo (1988)

Palermo (1988) evaluated the predictive capability of the modified elutriate and companion settling tests for effluent quality during hydraulic filling of CDFs. Field data from five sites, four on the Atlantic coast and one on the Gulf coast, were compared with predictions made on the basis of laboratory data. Average containment efficiencies (for all sites) for most contaminants were very close to the total solids retention (99.91 percent). The average containment efficiency for metals for the five sites was 98.56. Results for nutrients were generally similar to those for metals at most sites. PCBs were measured at only one site, and the containment efficiency for PCBs at this site was 99 percent.

For all five sites, the laboratory tests adequately predicted the dissolved concentration of contaminants and the contaminant fractions of the total suspended solids in the effluent. The predictions were within a factor of 1.5 of the field data for a total of 64 of the 84 parameters measured. The modified elutriate test was also a generally conservative predictor, that is, predictions of effluent contaminant concentrations were generally higher than the measured field results.

Palermo (1988) obtained detailed statistical data on the predictive capability of the modified elutriate and companion settling tests for sites studied. Results for both the laboratory predictions and the field data are shown in Figures 24 and 25. In most cases, the mean of the modified elutriate was within the standard deviation for the field data. These data provide the scientific basis for recommending the modified elutriate test and companion settling tests as the predictive techniques for estimating contaminant losses associated

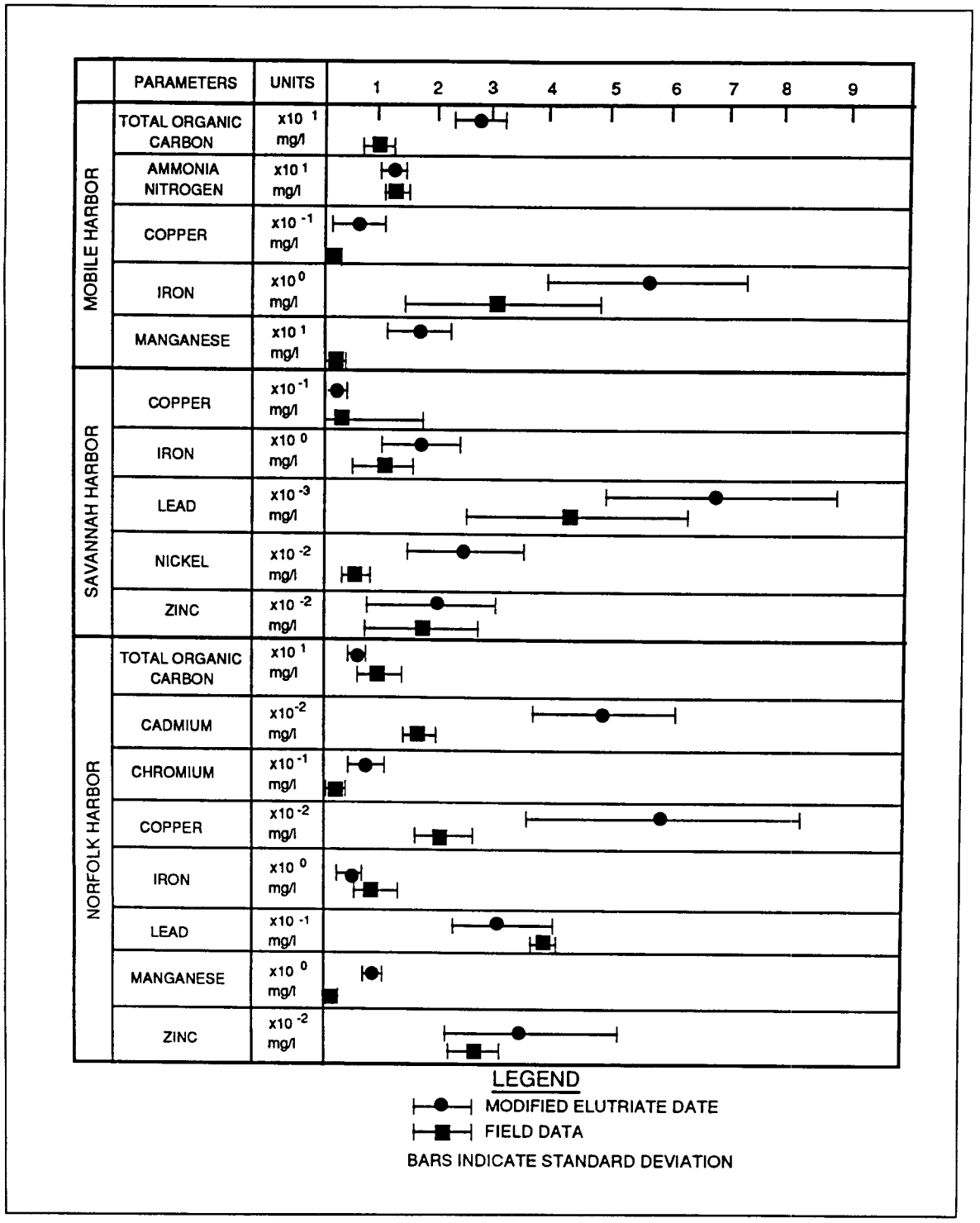


Figure 24. Means and standard deviations of predicted and observed effluent quality at Mobile Harbor, Savannah Harbor, and Norfolk Harbor CDFs

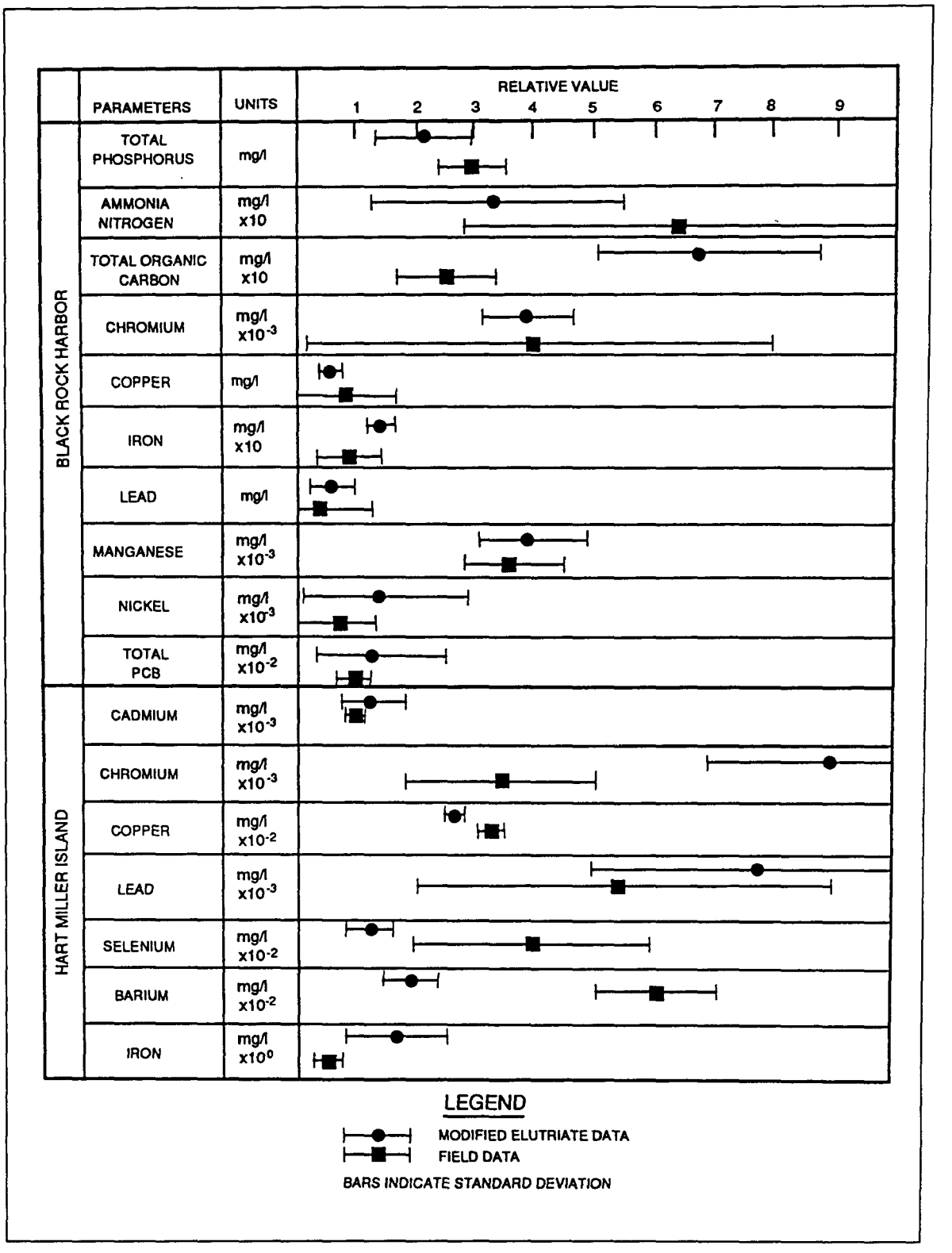


Figure 25. Means and standard deviations of predicted and observed effluent quality at Black Rock Harbor and Hart Miller Island CDFs

with effluent. The data, however, are primarily nutrients, dissolved oxygen, pH, organic carbon, and metals concentrations in effluent. The dissolved and total organic carbon estimates provided by the laboratory tests were in good agreement with the field data. The modified elutriate and companion settling tests should, therefore, be a good predictor of dissolved and total organic chemical concentrations in effluent. Sediment from one site, Black Rock Harbor, Connecticut, contained high enough concentrations of PCBs (14.3 mg/kg total PCB) for PCBs to be found in the effluent during disposal operations. The mean total PCB concentration in effluent from the Black Rock Harbor CDF was 0.0099 mg/ℓ, versus a predicted value of 0.013 mg/ℓ.

Thackston and Palermo (1990)

Thackston and Palermo (1990) applied the modified elutriate and companion settling tests to prediction of effluent quality from a CDF for the Houston Ship Channel, Texas, during hydraulic filling. This study was designed to fill data gaps on freshwater sediments and organic contaminants. Additional information on effluent quality during disposal of a freshwater sediment was obtained, but the organic chemical contamination of the sediment was too low to obtain information on the predictive capability of the modified elutriate and companion settling test for organic contaminants in CDF effluent. In this study, the mean ratios of predicted to observed effluent nutrient and metals concentrations was near 1.0, and the range in predicted total to observed total effluent contaminant concentrations was 0.2 to 2.6. Total ammonia-nitrogen concentration was underpredicted (ratio of predicted to observed = 0.2), and total chromium was overpredicted (ratio of predicted to observed = 2.6). On balance, the data set obtained again showed that the modified elutriate and companion settling tests comprise a useful and reasonably accurate predictive technique.

Thackston and Palermo (1992)

Additional verification work on PCBs was conducted by Thackston and Palermo (1992) at the New Bedford Harbor Superfund Demonstration CDF, New Bedford, MA. The PCB concentrations in sediments from the site ranged from a few milligrams per kilogram to over a gram per kilogram (Averett 1988). A demonstration-scale CDF for hydraulic dredging and disposal of 1,680 m^3 of contaminated sediment was constructed as part of a pilot study of dredging and disposal alternatives. The total PCB concentration in the composite sample used for modified elutriate testing was 2.2 g/kg. The predicted value for dissolved PCB (0.0075 mg/ℓ) was very close to the observed mean value for dissolved PCB in the CDF effluent (0.0045 mg/ℓ).

Myers (1991)

Myers (1991) measured PCB congener concentrations in influent and pond water in the Saginaw CDF, Saginaw, MI. Sampling was conducted during hopper dredging and disposal of material from the Saginaw River near Saginaw, MI. The perimeter dike at the Saginaw CDF was designed to be permeable. Effluent monitoring was not practical because the discharge through permeable dikes is diffuse and is quickly diluted to background concentrations. Based on PCB congener concentrations in pond water collected on the inside face of the perimeter dike, the containment efficiency of the Saginaw CDF for PCBs was 99.82 percent. This estimate neglects filtration and sorption in the dike and assumes that the dike is transparent to both dissolved and particulate PCB.

Myers (1991) also compared PCB congener concentrations in the modified elutriate test with observed pond water PCB congener concentrations during disposal operations. The results of this study were consistent with the verification studies of Palermo (1988) and Thackston and Palermo (1992), which involved sediments with higher contamination levels and used total PCB as the model parameter. Of the 60 PCB congeners analyzed, 16 were detected in the unfiltered modified elutriates, compared with 13 detected in unfiltered pond water samples. The predicted total concentrations for 4,4'-dichlorobiphenyl and 2,2',5,5'-tetrachlorobiphenyl, the two most abundant PCB congeners in the dredged material influent, were 0.02 and 0.07 $\mu g/\ell$, respectively, compared with observed concentrations in the CDF pond water of about 0.05 and 0.003 $\mu g/\ell$ for 4,4'-dichlorobiphenyl and 2,2',5,5'-tetrachlorobiphenyl, respectively. Dissolved PCB congener concentrations were generally below or just above the chemical analytical detection limit (0.01 $\mu g/\ell$) in both the modified elutriate test and CDF pond water.

Krizek, Gallagher, and Karadi (1976)

Krizek, Gallagher, and Karadi (1976) studied influent and effluent samples collected during hydraulic filling of the Penn 7 CDF in Toledo, OH. The experimental design was similar to that used by Hoeppel, Myers, and Engler (1978) and Lu et al. (1978) in that numerous influent and effluent samples were collected. Samples were analyzed for metals, nutrients, chemical oxygen demand (COD), and biochemical oxygen demand (BOD).

Containment efficiencies for most parameters were very close to the total solids retention (99.7 percent). Average retention efficiencies were as follows: iron (99.8 percent), COD (99.1 percent), potassium (98.8 percent), total phosphate (98.7), BOD (98.4 percent), calcium (97.5 percent), manganese (96.7 percent), zinc (95.9 percent), sodium (87.5 percent), cadmium (63.5 percent), copper (45.0 percent), and lead (45.0 percent). Effluent nitrate-nitrogen showed a 10-fold increase over influent nitrate-nitrogen. The authors attributed this increase to nitrification of nitrogenous compounds in the CDF.

MacKnight and MacLellan (1984)

MacKnight and MacLellan (1984) described disposal of PCB-contaminated sediment at Petit-de-Grat, Nova Scotia, Canada. Sediment containing 2- to 25-mg/kg PCB was hydraulically dredged and disposed in a CDF. A cationic polymer was used to improve solids removal in the CDF. Samples collected and analyzed for suspended solids and PCBs showed that the effluent met effluent water quality guidelines of less than 300-mg/ℓ suspended solids and less than 0.05-μg/ℓ PCBs. The authors concluded that hydraulic disposal in a CDF is an economically and environmentally acceptable method of disposal.

Khan and Grossi (1984)

Khan and Grossi (1984) presented results from a single round of effluent quality tests conducted during hydraulic disposal of contaminated sediment from Hamilton Harbor, Ontario, Canada. During this disposal operation, dredged material was pumped into a primary sedimentation cell. The supernatant from the primary cell traveled through three more cells before discharge to Hamilton Harbor. The results showed solids retention of 98.5 percent and effluent water quality comparable with ambient water conditions outside the CDF.

Effluent Losses During Mechanical Disposal

Predictive techniques for effluent quality during mechanical disposal of dredged material are currently unavailable. Mechanical placement of dredged material in a CDF differs from hydraulic placement, not only in the way placement is accomplished, but also in the way dredged material behaves once it is in the CDF. Mechanically dredged and disposed sediments have a markedly different character from hydraulically dredged sediment due to the fact that they have not been slurried with water as part of the dredging process. Since placement is at a much higher solids concentration, there is less effluent. In many instances, fine-grain mechanically dredged sediments have a paste-like cohesive character. In the mechanical placement process, dredged material primarily stays where it is initially placed, and only a very small proportion of the solids are actually released to water that may have been in the CDF prior to filling operations. It is therefore inappropriate to use a test like the modified-elutriate test, which involves slurrying sediment and water to estimate contaminant release during mechanical disposal.

Dredged material mechanically placed in upland CDFs should generate little to no effluent for discharge. Mechanical placement of dredged material in nearshore and in-water CDFs will displace the water initially present as filling proceeds. Because mechanical disposal rates are much lower than hydraulic disposal rates and most of the material stays where it is initially placed, only weak currents from placement point to discharge point are

generated. The advective velocity imparted by mechanical disposal operations is essentially negligible when the discharge point is a long distance from the placement point.

There are three primary mechanisms by which pollutants in mechanically placed dredged material are released to ponded water in nearshore and in-water CDFs. These are diffusion, release of pore water by consolidation, and resuspension of fine solids. Probably the most important process is wind-induced resuspension. Wind-induced currents resuspend sediment solids and disperse contaminants released by diffusion and pore water released by consolidation. Without wind-induced currents, migration of contaminant to an effluent discharge point is extremely slow.

Jones and Lee (1978) proposed development of a plop test for estimating contaminant release during mechanical disposal of dredged material. A plop test has never been developed, and the amount of testing conducted by Jones and Lee (1978) was limited. Some Corps of Engineers Districts have estimated effluent quality during mechanical disposal in an in-water CDF as dilution of pore water by ponded water in the CDF. This method is maybe better than no method at all; but since resuspension is not accounted for, this method underestimates pollutant releases.

Basically, there are two approaches to developing a predictive test, and the approach taken significantly affects test design and the basis for extrapolating laboratory data to the field. One approach is strictly empirical. It uses statistical analysis to establish correlation between laboratory and field data. The other approach is deterministic. In the deterministic approach, a mathematical model is derived from the physical-chemical laws that govern important processes. The mathematical model will require some parameter estimation and is therefore not purely deterministic. Most predictive techniques embody a combination of approaches with one being the primary basis for experimental design.

In the case of an empirical approach, a laboratory test should simulate the placement process, release of pollutants, and transport to the discharge point. In general, a laboratory test can never fully simulate all the minutia of field phenomena. With sufficient laboratory and field data, however, correlation functions can be developed that provide a basis for prediction. The cost of obtaining enough reliable data is a disadvantage of the empirical approach. Another disadvantage is that unless the laboratory test simulates important processes, the correlation functions may be too statistically weak to be of practical value.

The deterministic approach involves describing the pollutant release-transport-discharge process mathematically, assigning coefficients or variables to each part of the overall process, and estimating the magnitude of each coefficient or variable. The entire process may never be experimentally simulated as a whole, but, instead, each step is simulated or analyzed separately; the steps are then combined logically and/or mathematically. In order to

successfully implement this type of approach, the overall process must be understood well enough so that it can be broken down into a small number of steps that can be isolated and measured in a laboratory test. The feasibility of this type of approach is enhanced if the overall process depends primarily on one or two mechanisms such that other mechanisms are unimportant or they have a range of variability so low that they can be assumed to be constant.

Development of the procedures for predicting effluent quality during hydraulic disposal previously discussed is an example of a successful combination of empirical and deterministic approaches. Several factors contributed to the successful development of these procedures. First, hydraulic dredging tends to homogenize variations in sediment chemical and physical properties so that the use of average values is consistent with the physics of the process. In addition, many of the variables affecting contaminant release during hydraulic filling are understood because of considerable experience with hydraulic dredging. Flocculation and sedimentation have been studied for many years so that there was a large knowledge base from which to initiate test development. Further, the time scale of the flocculation-sedimentation process is large relative to the time required for many individual chemical or physical reactions so that minor errors in variable estimation are not critical.

The above discussion describes technical aspects of developing a predictive technique for effluent quality during mechanical disposal. The problem is not sufficiently understood to determine which of the two approaches discussed should be recommended. Since mechanical dredging and disposal is an alternative that is sometimes selected for contaminated sediments, development of a predictive technique for effluent quality during mechanical disposal in nearshore and in-water CDFs is needed to fully evaluate this alternative.

Seepage Through Permeable Dikes: Nearshore and In-Water CDFs

Pond water seepage through dikes

Some nearshore and in-water CDFs use permeable dikes to release the carrier water used in hydraulic dredging. Figure 26 is a typical cross section of the perimeter dike at the Saginaw CDF, Saginaw, MI. Dredged material solids clog permeable dikes as CDFs fill so that an outlet structure(s) is usually necessary for release of carrier water in the latter stages of filling.

During disposal operations, the flow through the dike is the volumetric influent flow if the influent flow is continuous. In between disposal operations and when influent flow is not continuous, there is a potential for lake water to move through the dike into the CDF and then back out again as lakeside water levels fluctuate. The direction of flow depends on water elevations inside and outside the CDF. Flow through the dike can be estimated using Dupuit's equation, Equation 44.

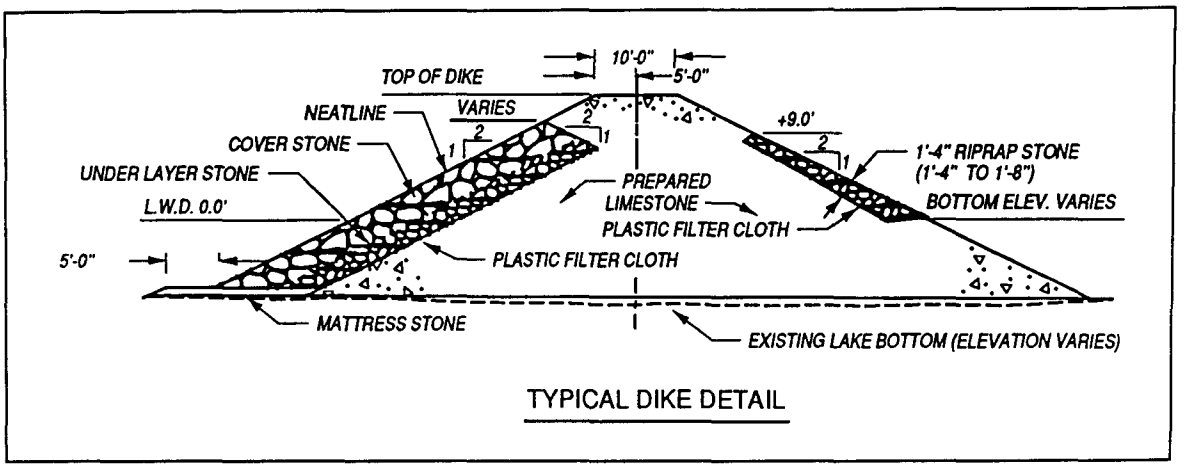


Figure 26. Cross section of perimeter dike at Saginaw CDF

$$q = \frac{K\,(h_1^2 - h_2^2)}{2\,L} \tag{44}$$

where

q = discharge per unit length of dike, m^2/sec

K = hydraulic conductivity of dike, m/sec

h_1 = pond water elevation above base of dike, m

h_2 = outside water elevation above base of dike, m

L = horizontal distance separating surface of pond and surface of outside water body, m

A definition sketch for Dupuit's equation is given in Figure 27. The assumptions on which Equation 44 is based are discussed by Harr (1962).

To use Dupuit's equation, water level fluctuations outside the CDF are needed. These data are not easily obtained for preproject analysis of contaminant losses. There may be several ways of dealing with this problem. Two are briefly mentioned as follows: use historical data or develop a synthetic water level generator based on historical data. In either case, the time scale for the lakeside water elevations must properly represent the dispersion effect that changing water elevations in the lake have on contaminant movement from the pond water through the dike. There are, however, no data on contaminant movement through permeable dikes due to fluctuating lake levels that can be used as guidance. Engineering judgment in the selection and use of water level data is therefore required.

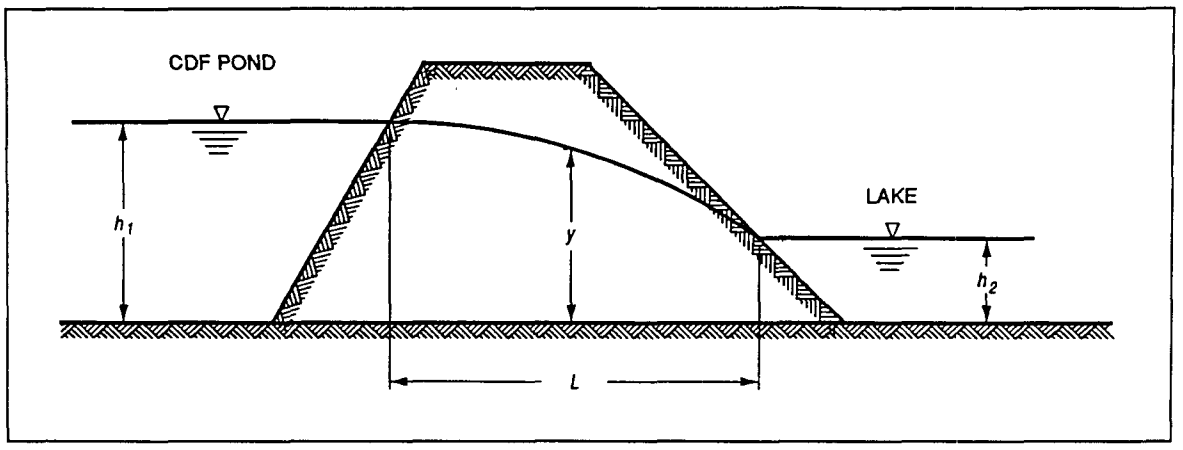


Figure 27. Definition sketch for application of Dupuit's equation

To estimate the mass of contaminant released, predicted flows must be coupled with estimates of pond water contaminant concentrations. If pond water contaminant concentrations are assumed to be constant and equal to the concentrations predicted by the modified elutriate test, then the mass of contaminant released is total flow out of the CDF times the contaminant concentrations predicted by the modified elutriate test. This type of estimate is probably a crude overestimate of contaminant losses. Simple techniques for predicting the time dependency of pond water contaminant concentrations are not available.

Prediction of contaminant losses due to changing lake levels can also be developed by modeling the dispersive effect of water moving in and out of the CDF as a diffusion process. This approach is well established in estuary modeling where the overall flow is out to sea but there is substantial mixing by tidal effects. Martin, Ambrose, and McCutcheon (1988) incorporated algorithms into the Water Analysis Simulation Program, Version 4 (WASP4) that parameterized the dispersive effects of changing water level elevations in a dispersion coefficient. This model does not require time-dependent lake elevations as input and can simulate some of the processes affecting contaminant concentrations in pond water. There are, however, no data on the dispersion effects of fluctuating water levels in permeable dike CDFs on which to base estimation of dispersion coefficients, nor are data available on processes affecting contaminant concentrations in pond water. Application of WASP4 and similar models, therefore, requires engineering judgment in the selection of dispersion and other transport process coefficients.

Leachate seepage through dikes

As previously mentioned, some nearshore and in-water CDFs use permeable dikes to release the conveyance water used in hydraulic dredging. Once the CDF is filled above the high-water datum, exchange of water between the CDF and the outside water body is restricted by the dredged material that fills

the voids in the inside face of permeable dikes. The HELP model previously discussed provides an estimate of the total seepage likely to occur but does not indicate the fraction that seeps through dikes. HELP model leachate flow predictions have been interpreted to represent the total leachate released through all boundaries of the CDF without implying that leachate only flows vertically (Averett et al. 1988). However, when flow is two- or three-dimensional, caution must be exercised when using a one-dimensional tool such as the HELP model to estimate flow. If a barrier soil with a hydraulic conductivity lower than that of the dredged material is constructed, leachate flow into the foundation soils can be reduced. However, flow through the dikes may be increased, depending on the hydraulic conductivity of the dikes. The HELP model is an appropriate tool for predicting total leachate flow and evaluating the effectiveness of a barrier soil to reduce flow into the foundation soils, but it is not designed to provide information on potential flow through the dikes.

Unconfined-saturated flow groundwater models are available that could be used to model dike seepage. Such models require substantial site-specific data on local hydrogeology. Although not described in this report, two- and possibly three-dimensional models may be needed to fully describe dike seepage at CDFs. Simplified models could also be developed for comparison with HELP model estimates. An example of the type of simple seepage models that could be developed is described below.

At some point in time, the amount of water entering the dredged material as percolation and the amount leaving as leachate flow will tend to balance so that a quasi-equilibrium exists. When a quasi-equilibrium exists, flow averaged over an extended time scale is steady and, under certain conditions, is parallel to the bottom of the CDF. Definition sketches for horizontal-steady flow in upland and in-water CDFs are given in Figure 28. For homogeneous, isotropic, circular CDFs, flow is radially symmetric. Radially symmetric, steady flow in homogeneous and isotropic media is given by the following equation (Glover 1974; McWhorter and Sunada 1977):

$$Q = \pi K \frac{[H_1^2 - H_2^2]}{\ln\left[\frac{R_2}{R_1}\right]} \tag{45}$$

where

π = 3.1459...

H_1 = head at crown of water table mound (Figure 27), cm

H_2 = head outside CDF (Figure 28), cm

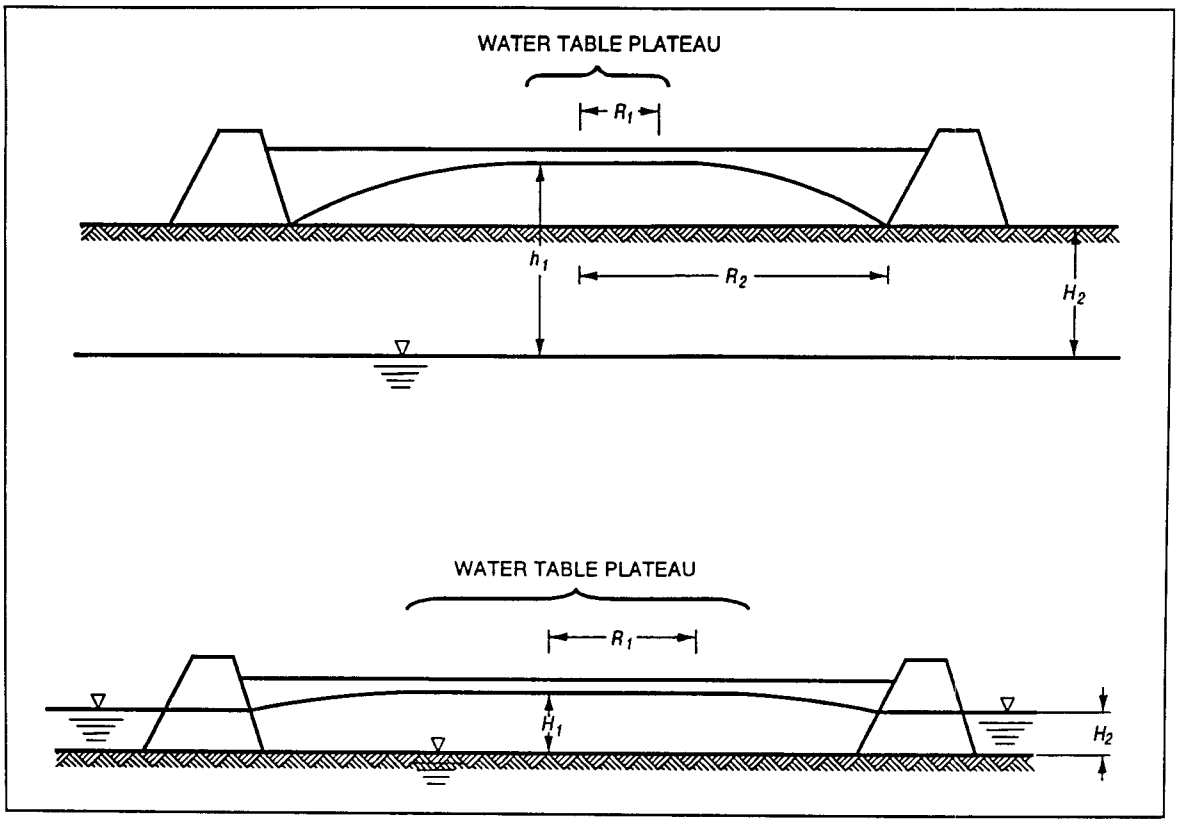


Figure 28. Definition sketches for horizontal-steady flow in CDFs

R_2 = distance from center of CDF to dike, cm

R_1 = distance from center of CDF to edge of water table crown, cm

Application of Equation 44 involves the following assumptions:

a. Isotropic and homogeneous medium.

b. Piezometric plateau in center of CDF.

c. Time invariant piezometric surface.

d. Time invariant dredged material hydraulic properties.

e. Radial symmetry with the center of symmetry coincident with the center of the CDF.

f. Dikes with infinite permeability relative to the permeability of the dredged material.

g. Dupuit-Forchheimer assumptions:

(1) Equipotential lines perpendicular to the bottom of the CDF.

(2) Hydraulic gradient is equal to slope of the free piezometric surface and invariant with depth.

These conditions are not always met, but when they are, flow is horizontal and modeled by Equation 45.

It is anticipated that leachate flow estimates provided by Equation 45 will be substantially less than leachate flow estimates provided by the HELP model. The differences are primarily due to differences in hydraulic gradients. Horizontal hydraulic gradients as indicated in Figure 28 are roughly a factor of 100 times lower than the vertical hydraulic gradients in the HELP model. Because site conditions that provide horizontal-steady flow minimize hydraulic gradients, estimates provided by Equation 45 are probably lower bounds on leachate flow.

The analysis of horizontal-steady flow discussed above assumes an equilibrium between surface recharge and seepage. Such conditions are rarely established, as there are seasonal as well as daily fluctuations in the piezometric surface in a CDF. Equation 45 is, therefore, limited to estimation of annual average flow in relatively old CDFs for which there is relevant site-specific information.

The flow domain and boundary conditions at many CDFs are such that leachate flow is not primarily vertical or steady-horizontal. Two- and three-dimensional flow in the subsurface environment has been considered in detail by many researchers (Harr 1962; McWhorter and Sunada 1977; Freeze and Cherry 1979; Bear and Verruijt 1987; Strack 1989; National Research Council 1990). The literature contains many different two- and three-dimensional numerical models of subsurface flow that could be used to analyze more complicated seepage conditions in CDFs.

The main difficulty with applying these models to CDFs is that local climatology and surface hydrology are not explicitly modeled. Infiltration is usually treated as an external input requirement without accounting for the stochastic character of rainfall events and resulting infiltration. As previously discussed, local climatology and surface hydrology are important because the water budget in a CDF is surface driven. Percolation to the saturated zone and the depth of the saturated zone depend on infiltration, which depends on the amounts of rainfall, runoff, and antecedent soil water. Since infiltration is the long-term source of water for leachate generation, climatologic and surface hydrologic modeling such as provided by the HELP model is a necessary component if the analysis of leachate flow is to be complete.

A careful scientific investigation calls for the complete use of the most up-to-date theoretical formulation and modeling tools. Preproject estimation of contaminant losses for planning level assessments sometimes may indicate the need for careful investigation of losses along some pathways. Losses

through permeable dikes is a contaminant loss pathway where the simple equations previously discussed are likely to be inadequate. As an alternative, the two-dimensional, finite element model, SEEPU, is available (Kuppusamy 1991) for estimating flow through dikes. This model has preprocessors and postprocessors to facilitate data input and output and runs on MS-DOS based personal computers.

Complex models are generally expected to have a greater predictive capability than simple models and increase the range of situations that can be described. Complex models require proper input information, as obtained from detailed field measurement. These measurements are usually quite extensive especially if a three-dimensional model is used.

In addition, the many models available differ from one another as a result of different objectives of the modeling effort. For this reason, a model should not be applied unless the objectives, model structure, type of output, and model precision are commensurate with the information needs and site conditions for a particular problem. For these reasons, development of predictive techniques for complicated flow problems in CDFs is not a search for one correct and completely general set of equations.

Estimation of contaminant losses associated with leachate seepage through dikes will involve coupling flow with leachate quality. Techniques for predicting leachate quality were discussed in Contaminant Losses During Pretreatment. These techniques are applicable to seepage from the anaerobic zone, that is, the saturated zone. Techniques are also available for predicting leachate quality from unsaturated, oxic dredged material crusts that develop in CDFs during evaporative drying (Environmental Laboratory 1987; Myers and Brannon 1988). These techniques can provide the leachate quality information needed to estimate losses during rainfall events that produce horizontal, saturated flow in the surface crust. These events must be short term in order for the aerobic leachate quality estimates to be applicable. The procedures for estimating leachate quality from aerobic dredged material are not discussed in this report because techniques for predicting the companion flow needed for contaminant loss estimation are not available.

Contaminant attenuation in permeable dikes

A parcel of water moving through a permeable dike takes a tortuous path before finally exiting the dike. The contaminants in such a hypothetical parcel of water are not likely to be conservatively transported. There are at least four processes that can attenuate transport of contaminants through dikes. These are filtration, adsorption, bioabsorption, and biodegradation. Filtration of solids is generally recognized as the primary removal process in permeable dikes. If dikes did not remove solids, permeable dike CDFs could not be filled. Adsorption can remove dissolved contaminants left after solids removal, but permeable dikes (sand and stone) have low sorption properties. Adsorption is probably insignificant in permeable dikes.

Bioabsorption and biodegradation are potentially significant removal processes that have not been investigated in permeable dikes. Ponded water in CDFs contains bacteria, protozoa, and other microscopic organisms that are also probably present in the dikes. Because filling operations are often intermittent, there is a potential for development of biofilms on dike materials. Biofilms in dikes potentially bioabsorb (remove) and degrade (treat) dissolved chemicals in pass-through water. Bioadsorption and biodegradation in permeable dikes have not been investigated. Consequently, removal and treatment of pollutants by biofilms in dikes have been generally ignored. Models, such as WASP4, that already have algorithms accounting for these processes need field data on process descriptors to improve their application to CDFs.

Literature on Leachate Losses From CDFs

There have been relatively few studies of the impacts of dredged material disposal in a CDF on groundwater and underlying soils. Some field and laboratory work was accomplished under the DMRP, but this research was limited in the number of sites investigated, duration of study, and number of chemical parameters studied. Recently, research toward development of predictive techniques for leachate quality in CDFs has been initiated under the LEDO program. This work, which involves both theoretical and laboratory studies, is still developmental. Some limited field data on leachate generation in a CDF have been reported by the U.S. Army Engineer District, Buffalo. The available information is reviewed below with emphasis on information for the Great Lakes.

Field studies

Yu et al. (1978). Yu et al. (1978) conducted field investigations of leachate impacts at four sites as follows: Sayerville, NJ; Houston, TX; Mobile, AL; and Grand Haven, MI. At each of the four sites, dredged material and soil samples were obtained from locations that would indicate lateral and vertical migration of contaminants. Groundwater samples were obtained from within the sites and directly below the sites, as well as from upgradient and downgradient locations. Groundwater samples were collected four times in 9 months; soil and dredged material samples were collected during the first sampling visit. Groundwater samples were filtered (0.45 μm) prior to analysis.

The general findings of Yu et al. (1978) indicated that leachate quality is a function of the physical and chemical nature of the dredged material, site-specific hydrogeological patterns, and environmental conditions of the area surrounding the site (e.g., physical and chemical nature of the adjacent soils). The study showed degradation of groundwater quality due to dredged material disposal in CDFs. Significant increases in chloride, potassium, sodium, calcium, magnesium, total organic carbon (TOC), alkalinity, iron, and manganese concentrations were measured in some downgradient groundwaters. Iron

and manganese appeared to be produced by localized environmental conditions, and their mobility was not considered directly related to dredged material disposal activities. Concentrations of chlorinated hydrocarbons, cadmium, copper, mercury, lead, zinc, phosphate, and nickel in CDF leachate were generally very low and did not appear to pose groundwater quality problems. Heavy metals were mostly in the parts-per-billion or subparts-per-billion range. No soluble chlorinated hydrocarbons were observed in groundwater.

Analysis of onsite dredged material and offsite soils failed to show systematic changes in chemical constituents. For most parameters, both increases and decreases in values occurred in different locations as well as at different depths. Total chlorinated hydrocarbons were higher in the dredged material than in offsite samples. The upper soil samples generally contained higher concentrations of chlorinated hydrocarbons than the samples obtained a few feet below. There was no evidence of chlorinated hydrocarbon migration from CDFs.

The Grand Haven CDF studied by Yu et al. (1978) is the same CDF studied by Hoeppel, Myers, and Engler (1978). This CDF is located on the bank of the Grand River, Michigan. Prior to filling, the site was used for disposal of construction debris. Onsite and offsite borings indicated that the foundation consists of fine to coarse sands contiguous to a depth of 6.1 m where a dense clay stratum (tens of meters thick) is encountered. Groundwater levels measured on four different dates at nine locations in and around the CDF indicated a gentle groundwater gradient through the site and toward the Grand River. Figure 29 shows groundwater contours and directions of flow for a typical survey. As shown in Figure 29, groundwater flows through the CDF from east to southwest.

There was no evidence of chloride, sodium, calcium, phosphate, magnesium, iron, manganese, mercury, lead, or zinc leaching from the Grand Haven CDF. Alkalinity was higher in the dredged material leachate than in downgradient samples. Comparison of samples collected beneath the site with upgradient samples showed that the average values were in decreasing order as follows: undersite, downgradient, and upgradient. This concentration gradient indicates an alkalinity plume beneath the CDF that is diluted as it moved downgradient. TOC was highly correlated with alkalinity in this study. At the Grand Haven site, TOC showed a concentration gradient similar to that for alkalinity, indicating leaching of TOC along with alkalinity from the CDF.

Downgradient cadmium concentrations were higher than in the dredged material leachate. The difference, 0.0006 mg/ℓ, was statistically significant, but such a small difference is probably not environmentally significant. The mean upgradient and downgradient cadmium concentrations were the same (0.0014 mg/ℓ), indicating no impact by the CDF. Copper was higher in the dredged material leachate (0.019 mg/ℓ) than in the downgradient samples (0.010 mg/ℓ). Upgradient copper concentrations were similar in copper

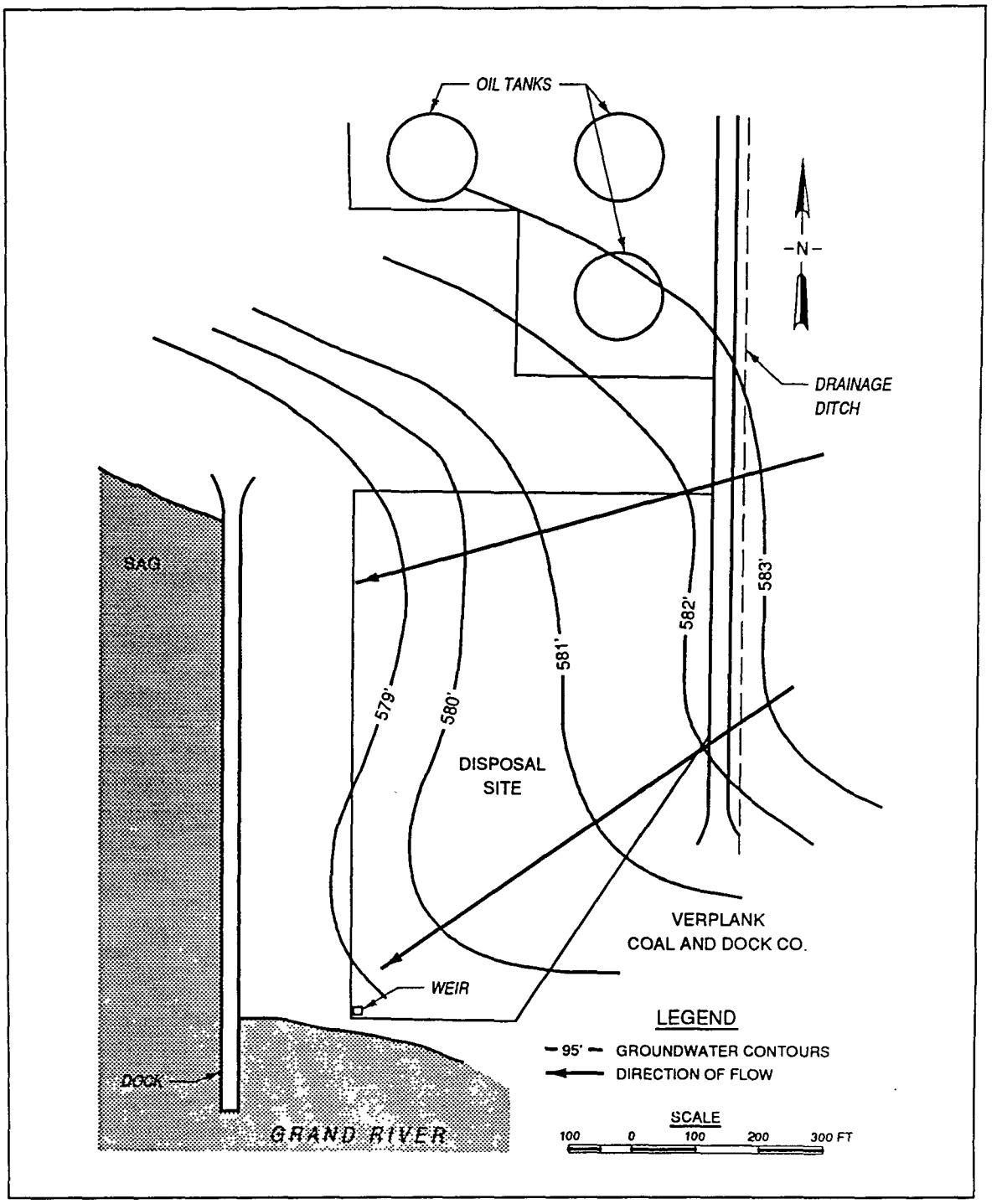


Figure 29. Water level contours at Grand Haven CDF (from Yu et al. 1978)

concentration to samples collected beneath the site, suggesting that copper was not leaching from the CDF.

A concentration gradient for nickel from the dredged material leachate to downgradient monitoring wells was found, indicating a potential for migration. Average nickel concentrations were higher in the dredged material leachate (0.127 mg/ℓ) than in the groundwater beneath the site (0.065 mg/ℓ) and the downgradient groundwater (0.027 mg/ℓ). However, because the average nickel concentrations in the upgradient wells (0.170 mg/ℓ) were higher than in the CDF leachate, the CDF may not be the primary source for nickel beneath the site and downgradient.

Leonard (1988). Leonard (1988) reported significant heavy metal and organic contamination in pore water in dredged material in the Times Beach CDF, Buffalo, NY. The Times Beach CDF is located on Lake Erie and was used for confined disposal of contaminated dredged material from the Buffalo River, the Buffalo Harbor, and the Black Rock Channel from 1972 to 1976. The site is underlain by fine sands, glacial till, and limestone. Upgradient monitoring wells showed evidence of arsenic, cadmium, and lead contamination. Groundwater beneath the site showed little evidence of contamination.

Krizek, Gallagher, and Karadi (1976). The field investigation conducted by Krizek, Gallagher, and Karadi (1976), previously discussed in the section on effluent losses, included some limited groundwater sampling within the vicinity of the Penn 7 CDF in Toledo, OH. The quality of the groundwater was found to be slightly worse than either the river water or the CDF effluent. Seepage from the CDF into the underlying soil was thought to be small due to the low permeability of the dredged material and the upper strata of the foundation soils.

Laboratory studies

Mang et al. (1978). Mang et al. (1978) investigated the generation of leachate from 16 large plexiglass lysimeters under various environmental conditions. The study used dredged material from five different locations and two native soils from California. Various leaching solutions were used including distilled water (rainwater leach), distilled water acidified to pH 4.5 with sulfur dioxide (acid rainfall leach), hard water buffered with bicarbonate (alkaline groundwater leach), and leachate obtained from a solid waste landfill. Parameters analyzed included major cations and anions, trace metals, PCBs, chlorinated pesticides, nutrients, and gross physicochemical parameters (Eh, pH, alkalinity, and conductivity).

The results showed that no single mechanism governs contaminant leaching from dredged material. During leaching some parameters increased (Eh, pH, TOC, alkalinity, and manganese), some remained relatively constant (phosphorus and magnesium), some decreased (organic and ammonia nitrogen, copper, calcium, sodium, and potassium), some parameters were highly

variable (cadmium and zinc), and some were consistently below detection limits (PCBs and chlorinated pesticides). This work showed that alkalinity, iron, manganese, zinc, and lead posed the greatest potential for dredged material disposal in a CDF to adversely impact groundwaters.

Soils placed beneath the dredged material tended to regulate pH, TOC, and alkalinity and serve as a source for iron, manganese, calcium, potassium, nitrate-nitrogen, and total Kjeldhal nitrogen. Adsorption onto soil solids seemed to be an important mechanism for attenuation of ammonia nitrogen, cadmium, copper, mercury, and lead.

Environmental Laboratory (1987). In this comprehensive study of dredging and disposal alternatives for PCB-contaminated sediment in Indiana Harbor, Indiana, batch and column leaching studies were conducted. The results showed that the metal and organic contaminants in Indiana Harbor sediment were tightly bound to the sediment solids. Less than 1 percent of the bulk metal concentrations were leachable in sequential batch leach tests. The overall batch equilibrium distribution coefficients for PCBs was very high, 256,000 ℓ/kg, indicating a low potential for leaching. Integration of batch and column test data using a mass transport equation showed that contaminant interphase transfer could be modeled using classical partitioning theory. Total PCB concentrations in leachate from a CDF containing Indiana Harbor dredged material were predicted to not exceed 0.0005 mg/ℓ. Metals were predicted to be near detection limits in leachate from CDFs filled with Indiana Harbor dredged material. The results also showed significant mobilization of metals in sediment that had been treated to simulate physicochemical conditions in the oxic crust that develops during evaporative drying.

Myers and Brannon (1988). Myers and Brannon (1988) conducted batch and column leach tests on New Bedford Harbor Superfund Site sediment. Desorption of PCBs and metals did not follow classical partitioning theory. Anaerobic desorption isotherms showed nonconstant partitioning for PCBs and metal during sequential leaching. Nonconstant partitioning in this sediment was due to salinity dependent release of sediment organic carbon (Brannon et al. 1991). Observed and predicted column elution curves qualitatively agreed, but quantitative agreement was not good. Predictions based on batch tests generally overpredicted observed column leachate contaminant concentrations. Salinity-dependent nonconstant partitioning is not expected to occur in the freshwater sediments and dredged materials in the Great Lakes.

Palermo et al. (1989). Palermo et al. (1989) conducted batch and column leach tests on sediment from Everett, WA. The contaminant levels in this sediment were low relative to those in Indiana Harbor and New Bedford Harbor sediments. Many contaminants leached in amounts below or near the chemical analytical detection limits. Results for contaminants that leached in amounts that could be reliably quantified were similar to those from New Bedford Harbor sediment. Salinity-dependent nonconstant partitioning was again observed.

Literature on Volatile Losses From CDFs

There are very few field data on volatile emissions from CDFs in the literature. Semmler (1990) did a desktop evaluation of the relative significance of PCB volatile losses from an upland and an in-water CDF filled with dredged material from Indiana Harbor, Indiana. This analysis indicated that volatile PCB losses from an upland CDF were approximately four times the volatile PCB losses from an in-water CDF. This analysis also indicated that volatile PCB losses from both disposal locales were three orders of magnitude higher than the PCB losses associated with leaching and four orders of magnitude higher than PCB losses associated with dike seepage. Semmler (1993) conducted field studies at a CDF in which PCB concentrations in sediment, water, and air compartments were monitored. The field results showed that the volatile pathway accounted for the majority of PCB mass loss from May to October. The studies of Semmler (1990) and Semmler (1993) serve notice that the volatile emission migration pathway could be of major significance for PCBs and other hydrophobic organic chemicals in CDFs.

EBASCO Services Incorporated (1990) conducted an ambient air monitoring program for the New Bedford Harbor Superfund pilot CDF. This study showed some of the pitfalls of attempting to measure emission rates by ambient air monitoring. PCB concentrations in ambient air around the site before, during, and after dredging and disposal activities were indistinguishable. These data should not be construed to imply that PCBs were not released to the air during dredging and disposal. Changing meteorological conditions, specifically wind velocities, generate turbulence that transports chemicals in all directions on a local scale. The result is a large and confusing data set when surface samplers are placed around a site with a large emission surface area and significant potential for high background levels. The upgradient and downgradient concepts applicable to groundwater and surface water monitoring are difficult to apply to air monitoring on a local scale.

Literature and Predictive Techniques for Runoff Losses

As previously discussed, when dredged material is placed in CDFs, physicochemical changes associated with evaporative drying affect contaminant mobility, including surface runoff quality. This section discusses techniques for predicting runoff quality from dredged material. Surface runoff flow predictions from CDFs can be obtained using the HELP model previously discussed.

Newly dredged sediment is generally anaerobic with near neutral pH and has high water content. During the wet, anaerobic stage, the transport of contaminants in surface runoff is mainly through the transport of suspended solids. As the material dries and oxidizes, the pH can decrease to sometimes

as low as 4 when high concentrations of sulfides are present. During the wet, anaerobic stage, metals tend to be bound as low solubility metal sulfides. As the dredged material oxidizes, some of these metals may increase in solubility and be released during storm events.

WES Rainfall Simulator-Lysimeter System

The WES Rainfall Simulator-Lysimeter System (RSLS) combines a rainfall simulator with a lysimeter bed containing dredged material (Figure 30). With the WES RSLS, runoff samples can be collected for analysis during simulation of selected storm events. By allowing the material placed in the lysimeters to age, changes in runoff quality as dredged material dries can be determined.

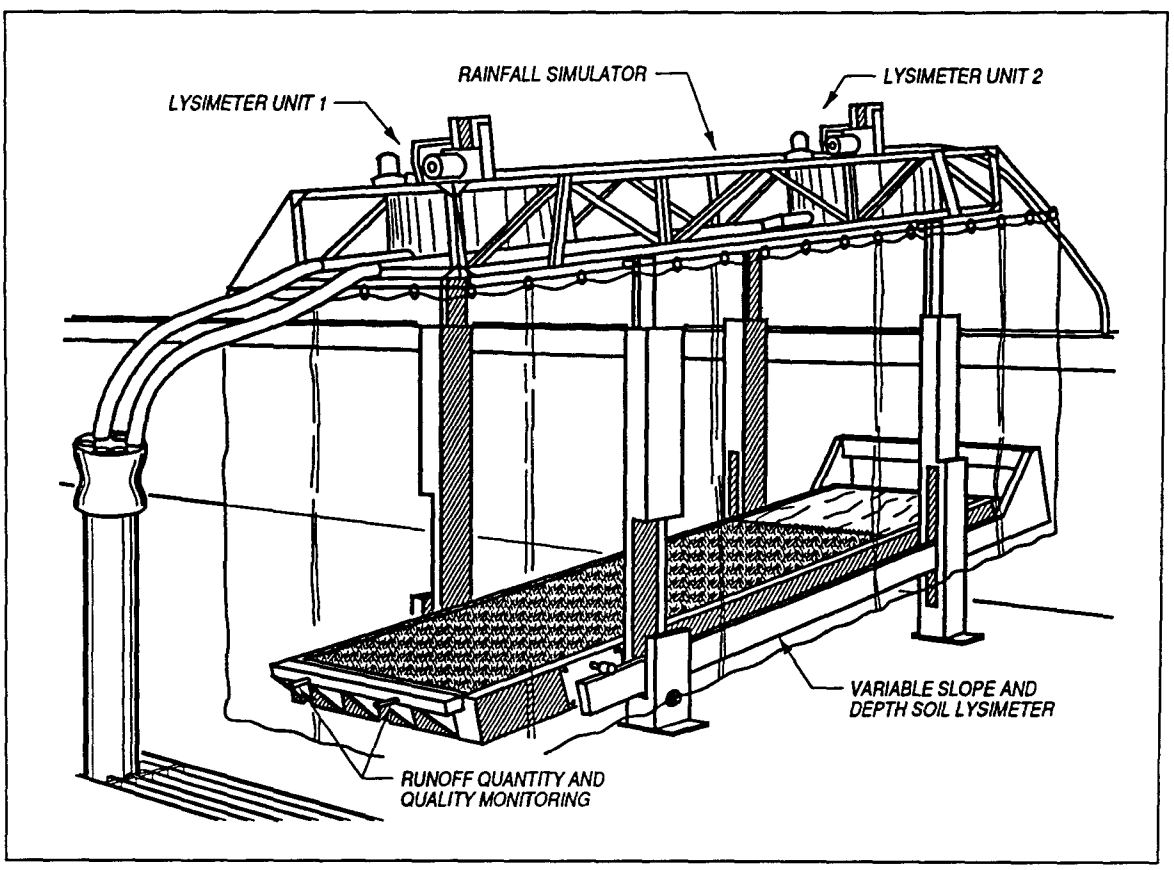


Figure 30. Schematic of WES Rainfall Simulator-Lysimeter System (from Skogerboe et al. 1987)

The rainfall simulator is a modified version of a rotating disk rainfall simulator originally developed at the University of Arizona (Morin, Goldberg, and Seginer 1967). Until the rotating disk-type simulator was developed, rainfall simulators were unable to simulate the kinetic energy of natural rainfall (Morin, Cluff, and Powers 1970). The rainfall simulator used in the WES RSLS is equipped with several important design modifications, including a

programmable slit disk opening that can instantly change rainfall intensity (Westerdahl and Skogerboe 1982). The WES rainfall simulator has been tested and calibrated to optimize drop-size distribution, terminal drop velocity, and rainfall intensity distribution (Skogerboe et al. 1987).

The lysimeters used in the WES RSLS are constructed of aluminum with surface dimensions of 4.6 by 1.2 m. Depth is adjustable in 15-cm increments to 1.2 m, and slope can be varied from 0 to 20 percent.

Runoff quality studies using WES Rainfall Simulator-Lysimeter System

Verification studies. A series of field verification tests were conducted by Peters, Lee, and Bates (1981) and Lee and Skogerboe (1984) that showed that the WES RSLS could accurately simulate surface runoff from natural storm events under a variety of conditions. The effect of plant biomass on runoff suspended solids concentrations was a major focus of these studies.

Skogerboe et al. (1987). Skogerboe et al. (1987) evaluated surface runoff water quality impacts from an upland dredged material disposal site at Black Rock Harbor, Bridgeport, CT, using the WES RSLS. This work was conducted as part of the U.S. Army Corps of Engineers/U.S. Environmental Protection Agency Interagency Field Verification of Testing and Predictive Methodologies for Dredged Material Disposal Alternative Program (Field Verification Program (FVP)). Sediment was collected from Black Rock Harbor and tested at WES to predict surface runoff water quality. Similar material was also dredged from Black Rock Harbor and disposed in an upland disposal site. Laboratory and field results showed significant increases in the mobilities of cadmium, copper, nickel, zinc, and manganese as the dredged material aged. Statistical analysis of observed and predicted runoff quality showed no significant differences. Results of this study, therefore, demonstrated that the WES RSLS can simulate the physicochemical changes and resulting changes in runoff quality that take place when contaminated dredged material is placed in upland environments.

Environmental Laboratory (1987). In the comprehensive study of dredging and disposal alternatives for PCB-contaminated sediment in Indiana Harbor, Indiana (Environmental Laboratory 1987), the WES RSLS was used to evaluate potential runoff water quality impacts. The results showed that during the early, wet, anaerobic stages, contaminants were primarily bound to the suspended solids in runoff. Filtered concentrations during this period were low compared with unfiltered concentrations, but were still of concern when compared with the USEPA Maximum Criteria for the Protection of Aquatic Life. As the sediment dried, the suspended solids concentrations decreased, thereby decreasing the unfiltered contaminant concentrations.

After the sediment dried and aged for 6 months, water quality constituents in runoff changed. Organic contaminants were no longer a concern because most of these compounds had been lost by volatilization and/or

biodegradation. No PCBs were detected in runoff from dry, oxidized sediment. Heavy metals concentrations also decreased; however, many became more soluble. Filtered concentrations of cadmium, copper, nickel, zinc, manganese, and lead were not significantly different from unfiltered concentrations, indicating that these metals were primarily present in soluble form. Filtered concentrations of cadmium, copper, zinc, and lead were greater than or equal to the USEPA Maximum Criteria for the Protection of Aquatic Life.

Palermo et al. (1989). In the evaluation of dredged material disposal alternatives at Everett, WA (Palermo et al. 1989), the WES RSLS was used to evaluate potential runoff water quality impacts. The results showed that during the early, wet, anaerobic stages, contaminants were primarily bound to the suspended solids in runoff. All filtered metal concentrations were significantly less than the USEPA Maximum Criteria for the Protection of Aquatic Life and were not considered a problem as long as the dredged material remained wet and anaerobic. Organic contaminant concentrations were also low, especially in filtered samples. PCBs were below the detection limits in both unfiltered and filtered samples.

After 6 months of drying and aging, the sediment did not form the hard crust with large cracks typical of many sediments. The material remained light and fluffy and was highly susceptible to erosion with suspended solids in runoff averaging 1,000 mg/ℓ. The sediment pH also remained high. Heavy metal concentrations in filtered samples were significantly lower than concentrations in unfiltered samples, indicating that the major fraction was in particulate form. However, filtered concentrations of some metals were high. Filtered concentrations of cadmium were significantly greater than the USEPA Maximum Criteria for the Protection of Aquatic Life, and filtered concentrations of copper and zinc were not significantly different from the criteria. Unfiltered and filtered concentrations of polynuclear aromatic hydrocarbons (PAHs) were very low, and PCBs were below the detection limit.

In addition to providing information on runoff quality, Palermo et al. (1989) made estimates of yearly mass release for an upland CDF. These predictions were calculated using the Universal Soil Loss Equation (Wischmeier, Johnson, and Cross 1971). Annual losses for cadmium, copper, zinc, and lead were estimated to be 6.2, 2.4, 115, and 0.7 kg/ha, respectively. The estimates involve using a soil erodibility factor obtained from the RSLS tests and a site-specific rainfall erodibility factor in the Universal Soil Loss Equation.

Skogerboe, Price, and Brandon (1988). Skogerboe, Price, and Brandon (1988) conducted surface runoff tests on New Bedford Harbor Superfund Site sediment with PCB concentrations of 100 mg/kg or less using the WES RSLS. Results of the surface runoff tests conducted immediately after placement of sediment in the lysimeter showed that contaminants were primarily in the particulate phase. Suspended solids concentrations were high (>7,000 mg/ℓ), resulting in high unfiltered concentrations of contaminants. Copper was the only contaminant exceeding the U.S. Environmental Protection Agency Acute

Water Quality Criteria for the Protection of Marine Aquatic Life in filtered samples. Filtered PCB concentrations were statistically less than the criteria.

After 6 months of drying and aging, a hard crust formed that reduced the erosiveness of the sediment. Results of surface runoff tests conducted 6 months after drying and aging showed that filtered cadmium, copper, and zinc concentrations were not significantly different from unfiltered concentrations, indicating that these metals were primarily present in soluble forms. Filtered copper and zinc were statistically greater than or equal to the U.S. Environmental Protection Agency Acute Water Quality Criteria for the Protection of Marine Aquatic Life. Both unfiltered and filtered PCB concentrations decreased in surface runoff after drying and aging.

Simplified laboratory tests

The WES RSLS described previously requires substantial quantities of sediment for testing and to properly simulate the physicochemical changes that are associated with drying and oxidation, 6 months to complete a test. The Indiana Harbor studies (Environmental Laboratory 1987) included investigation of laboratory batch extractions for predicting runoff quality from wet, anaerobic dredged material and dry, oxidized dredged material. The tests for wet, anaerobic dredged material involved serial dilution of suspended solids. The tests for dry, oxidized dredged material included various short-term drying and chemical extraction procedures. The results for predicting wet, anaerobic dredged material runoff quality by solids dilution and predicting dry, oxidized dredged material runoff quality by peroxide oxidation were promising. Additional testing and verification on a number of different sediments were recommended.

6 Contaminant Losses for In Situ Capping and Capped Disposal

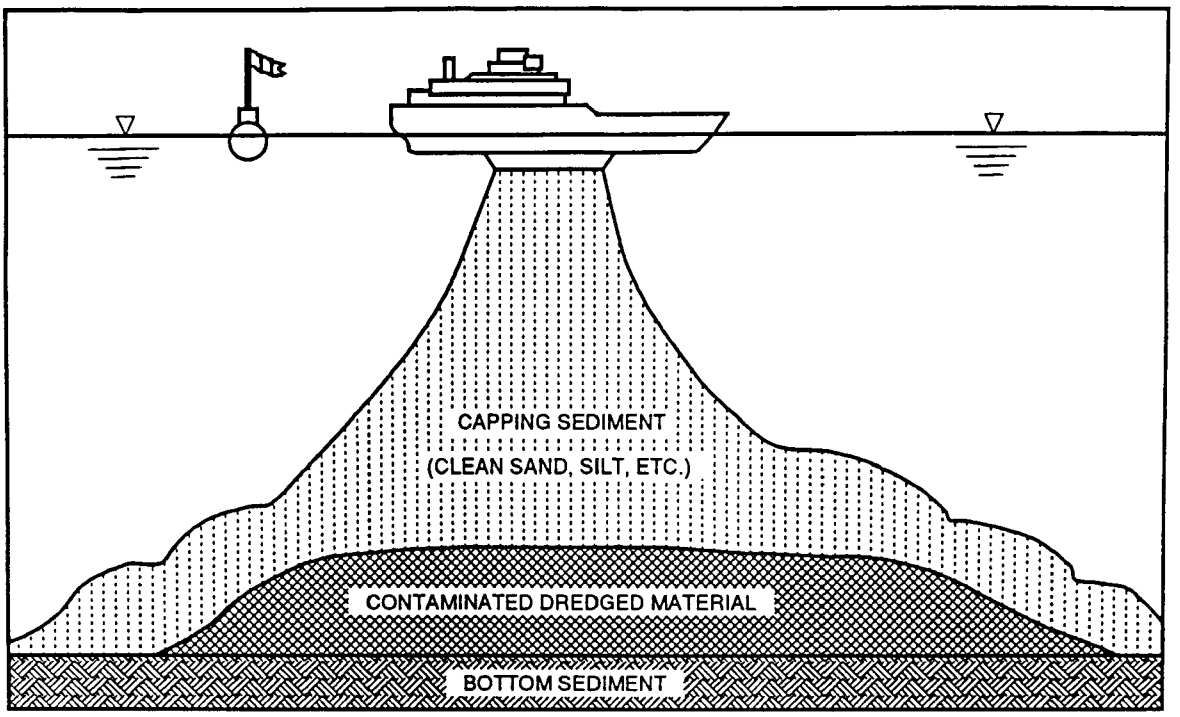
Background

General

In situ capping (ISC) is the placement of a covering or cap of clean material over an existing deposit of contaminated sediment. Capping is also a disposal alternative that can be considered when contaminated sediments are removed as a cleanup measure. For the case of removal, capping is the controlled accurate placement of contaminated material at an open-water disposal site, followed by a covering or cap of clean material. For purposes of this report, the term "contaminated" refers to material that is unacceptable for unrestricted open-water disposal and the term "clean" refers to material that is acceptable for such open-water disposal. Level bottom capping (LBC) is the placement of a contaminated material on the bottom in a mounded configuration and the subsequent covering of the mound with clean sediment. Contained aquatic disposal (CAD) is similar to LBC but with the additional provision of some form of lateral confinement (e.g., placement in bottom depressions or behind subaqueous berms) to minimize spread of the materials on the bottom.

Capping is considered an appropriate contaminant control measure for benthic effects in the Corps dredging regulations (33 CFR 335-338) and supporting technical guidelines (Francingues et al. 1985). An illustration of ISC, LBC, and CAD is shown in Figure 31.

Capping, a technology for isolating contaminated material, was developed as a control measure for contaminant effects on benthic organisms. The clean material in a cap isolates benthic organisms that recolonize a site from the contaminants in the material beneath the cap. The release of contaminants into the water column is not generally viewed as a significant problem for dredged material from most navigation projects. However, when capping is



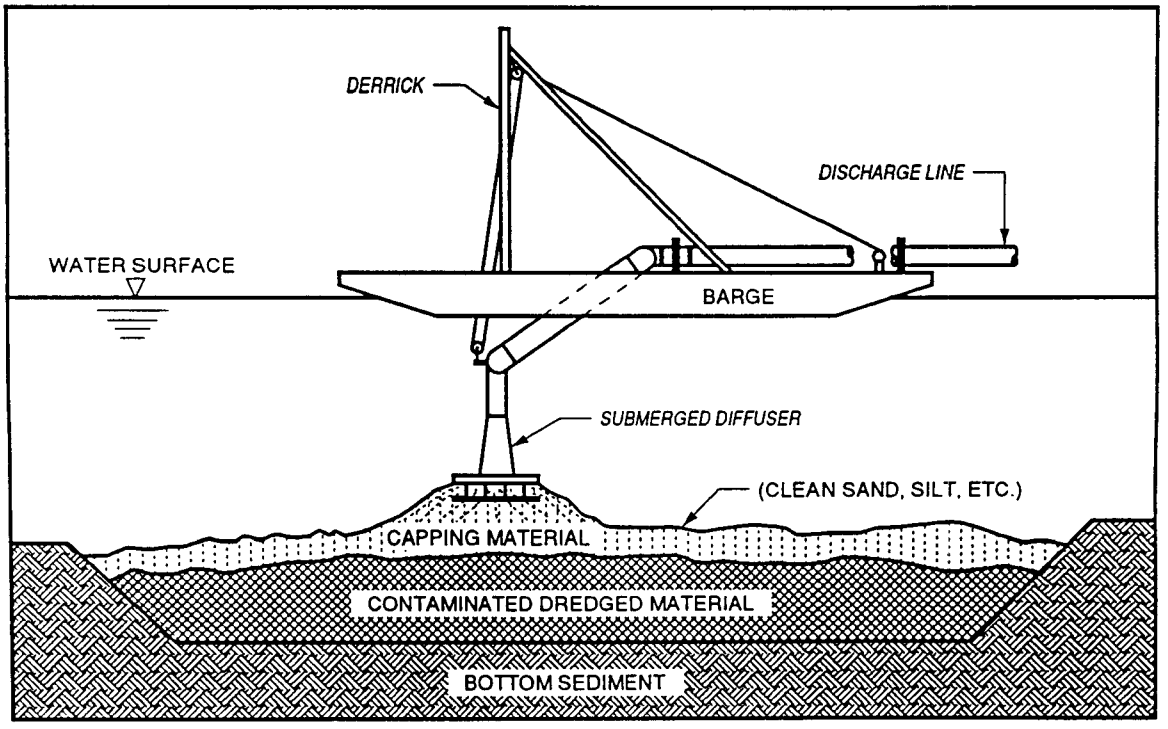


Figure 31. Capping alternatives

considered as an alternative for sediment remediation, contaminant release to the water column must be considered.

Design requirements for capping

Capping should not be viewed merely as a form of restricted open-water disposal. A capping operation is an engineered project with carefully considered design, construction, and monitoring. The basic criterion for a successful capping operation is simply that the cap thickness required to isolate the contaminated material from the environment be successfully placed and maintained.

Guidelines are available on planning and design concepts (Truitt 1987a, b), design requirements (Palermo 1991a), site selection considerations (Palermo 1991b), equipment and placement techniques (Palermo 1991c),and monitoring (Palermo, Fredette, and Randall 1992) for capping projects. These guidance documents were developed primarily for capping projects associated with navigation dredging; however, they are also applicable to capping associated with sediment remediation to include ISC, LBC, and CAD projects. A capping guidance document is being prepared specifically for in situ subaqueous capping of contaminated sediments that should be consulted when it becomes available (Palermo et al., in preparation).

Influence of Capping Materials, Site, and Operations

The nature of the material to be capped, the nature of the capping site, and the dredging and placement equipment and techniques used will have direct influence on the potential contaminant releases associated with capping. These essential components of the design must be examined as a whole with compatibility in mind.

A major consideration in compatibility is an acceptable match of equipment and placement techniques for contaminated and capping material. For example, if the contaminated material were mechanically dredged and released from barges, the capping material could be similarly placed or could be placed hydraulically. However, if the contaminated material were hydraulically placed, then only hydraulic placement of the capping material may be appropriate due to the potentially low shear strength of the hydraulically placed material.

Compatible scheduling of the contaminated material placement and capping operation is essential. The exposure of the contaminated material to the environment and need to allow consolidation of the contaminated material to occur prior to cap placement must be balanced in scheduling both placement operations. Availability of equipment and funding and the possibility of equipment

breakdowns or other delays should be considered in determining if the capping schedule is compatible with the contaminated material placement schedule.

Mechanisms for Contaminant Loss During Capping

For capping projects, the mass release is that total contaminant mass that is not initially capped or that does not remain isolated by the cap. This definition implies that both short-term losses during contaminated and capping material placement and long-term losses following completion of the construction of the cap must be considered.

Mechanisms for contaminant loss associated with capping therefore include the following:

a. Water column during placement of contaminated material.

b. Resuspension during placement of cap.

c. Pore water expulsion during cap consolidation.

d. Long-term diffusion and advection.

e. Long-term bioturbation.

f. Long-term erosion.

For LBC and CAD, contaminated material is dredged, transported, and placed at a capping site; therefore, losses for these components must be considered. It is anticipated that the majority of capping projects for sediment remediation will be in situ. For ISC, there is no dredging or placement of contaminated material and, therefore, no contaminant loss associated with contaminated material placement. Resuspension of the contaminated material and associated loss and long-term losses associated with diffusion, advection, bioturbation, and erosion processes must be considered for ISC, LBC, and CAD alternatives.

Water Column Contaminant Loss During Placement

Mass release of contaminants

Prediction of water column losses in terms of mass release for capping during placement of the contaminated material for LBC and CAD alternatives can be made using similar approaches as normally used for prediction of water column releases for open-water disposal operations (USEPA/USACE 1992). The approach taken is to determine contaminant concentrations associated with both dissolved and suspended particulate phases by standard elutriate testing.

Modeling of the fluid and suspended solids plumes is then used to predict the losses.

Standard elutriate testing

The prediction of dissolved and particle-associated releases of contaminants relies on the standard elutriate test. This test was developed in the early 1970s as a regulatory tool, and its utility and accuracy have been extensively field verified (Burks and Engler 1978; Brannon 1978; Jones and Lee 1978). In normal practice, the test is used as a predictor of dissolved contaminant releases resulting from open-water discharge of dredged material for purposes of comparison with applicable water quality criteria or standards, and to develop an appropriate medium for conducting water column bioassays (USEPA/USACE 1992). If total concentrations of contaminants are measured in the test, the results can be used in conjunction with modeling to calculate mass release of contaminants associated with the suspended solids (Palermo et al. 1989).

The standard elutriate test consists of the following steps as illustrated in Figure 32:

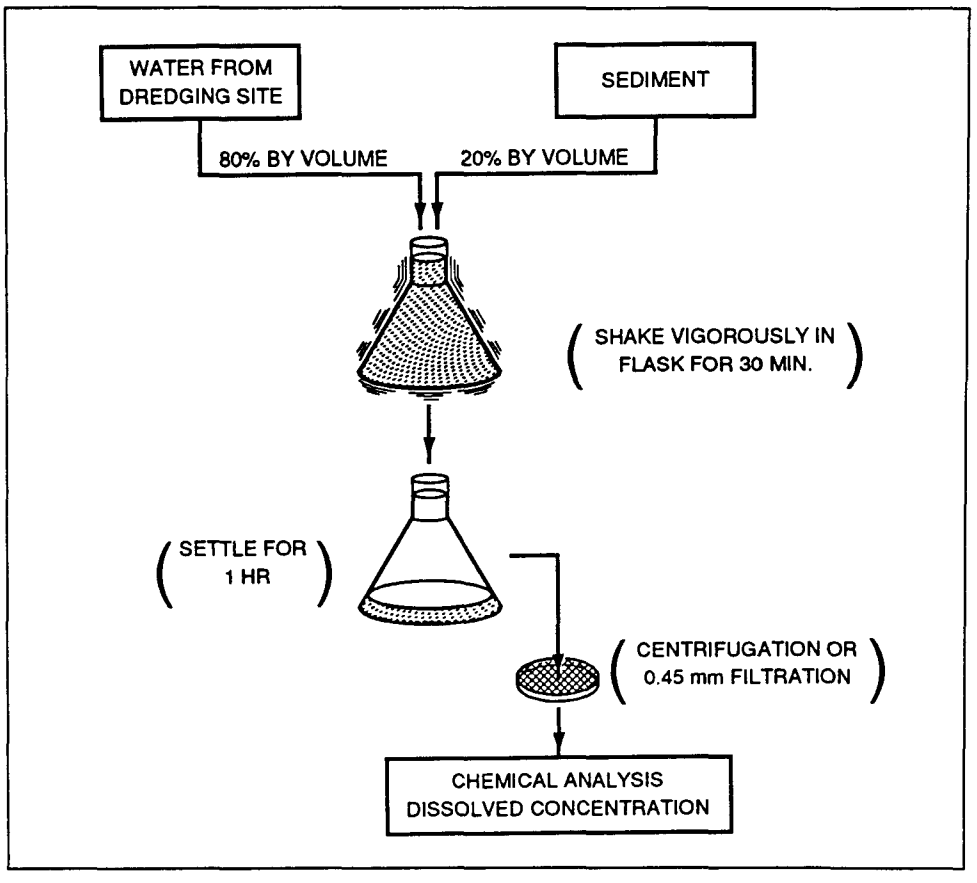


Figure 32. Standard elutriate test procedure

a. Mix dredging site sediment and water to a sediment-to-water ratio of 1:4 on a volume basis at room temperature.

b. Stir the mixture vigorously for 30 min with a magnetic stirrer. At 10-min intervals the mixture is also stirred manually to ensure complete mixing.

c. Allow the mixture to settle 1 hr.

d. Siphon off the supernatant and centrifuge or filter (0.45 μm) to remove particulates prior to chemical analysis for dissolved contaminant concentrations.

e. If particle-associated concentrations are desired, split the supernatant immediately after siphoning into subsamples for dissolved and total concentrations of contaminants and concentration of total suspended solids.

The dissolved concentrations from the test are the predicted dissolved concentrations in the discharge. The contaminant concentrations associated with suspended solids is the difference between total contaminant concentrations in whole water samples and dissolved contaminant concentration in the filtered water samples (Equation 46 below).

$$C_{ps} = \frac{C_{total} - C_w}{C_p} \tag{46}$$

where

C_{ps} = suspended solids contaminant concentration, mg/kg

C_{total} = whole water contaminant concentration, mg/ℓ

C_w = dissolved contaminant concentration, mg/ℓ

C_p = suspended solids concentration of elutriate sample, kg/ℓ

It should be noted that C_w and C_s in the above equation are not necessarily equilibrium concentrations. They could be equilibrium concentrations, but equilibrium is not a necessary condition in the standard elutriate test.

Open-water disposal modeling

Computer models are available for predicting water column dispersion and mixing (USEPA/USACE 1992 and Johnson 1990). The models also predict the amount of material that would be lost to the water column during placement. The use and limitations of the models along with theoretical discussions

are presented in detail in Johnson (1990). If barge release or hopper dredge release is used, these models also indicate the initial spread of a single barge load. This information is needed for evaluating mounding characteristics for the material volume to be placed.

The models are available as a part of the Automated Dredging and Disposal Alternatives Management System (ADDAMS) (Schroeder and Palermo 1990) and can be run on a microcomputer. ADDAMS is an interactive computer-based design and analysis system for dredged material management. The general goal of the ADDAMS is to provide state-of-the-art computer-based tools that increase the accuracy, reliability, and cost-effectiveness of dredged material management activities in a timely manner.

Model descriptions. The models account for the physical processes determining the short-term fate of dredged material disposed at open-water sites. The models provide estimates of water column concentrations of suspended sediment and contaminant and the initial deposition of material on the bottom.

Two of the models were developed by Brandsma and Divoky (1976) under the Corps Dredged Materials Research Program to handle both instantaneous dumps and continuous discharges. A third model that utilized features of the two earlier models was constructed later to handle a semicontinuous disposal operation from a hopper dredge. These models are known as DIFID (Disposal From an Instantaneous Dump), DIFCD (Disposal From a Continuous Discharge), and DIFHD (Disposal From a Hopper Dredge). Collectively, the models are known within ADDAMS as the Open-Water Disposal (DUMP) Models.

For evaluation of initial mixing for ocean disposal, the models need only be run for the contaminant requiring the greatest dilution to meet the respective water quality criteria. A data analysis routine is contained in the models for calculating the required dilutions and determining which contaminant(s) should be modeled.

In all three models, the behavior of the material is assumed to be separated into three phases: convective descent, during which the dump cloud or discharge jet falls under the influence of gravity and the initial momentum of the discharge; dynamic collapse, occurring when the descending cloud or jet either impacts the bottom or arrives at a level of neutral buoyancy where descent is retarded and horizontal spreading dominates; and passive transport-dispersion, commencing when the material transport and spreading are determined more by ambient currents and turbulence than by the dynamics of the disposal operation.

These models simulate movement of disposed material as it falls through the water column, spreads over the bottom, and finally is transported and diffused as suspended sediment by ambient currents. DIFID is designed to simulate the movement of material from an instantaneous dump that falls as a hemispherical cloud. Thus, the total time required for the material to leave

the disposal vessel should not be greater than the time required for the material to reach the bottom. DIFCD is designed to compute the movement of material disposed in a continuous fashion at a constant discharge rate. Thus, it can be applied to pipeline disposal operations in which the discharge jet is below the water surface or discharge of material from a single bin of a hopper dredge. If the initial direction of disposal is vertical, either the disposal source must be moving or the ambient current must be strong enough to result in a bending of the jet before the bottom is encountered. DIFHD has been constructed to simulate the fate of material disposed from stationary hopper dredges. Here, the normal mode of disposal is to open first one pair of doors, then another, until the complete dump is made, which normally takes on the order of a few minutes to complete. DIFHD should not be applied to disposal operations that differ significantly from the stationary hopper dredge operations described above.

DIFID, DIFCD, and DIFHD model disposed dredged material as a dense liquid. This model assumption will be satisfied if the material is composed of primarily fine-grained solids. Thus, the models should not be applied to the disposal of sandy material. A major limitation of these models is the basic assumption that once solid particles are deposited on the bottom, they remain there. Therefore, the models should only be applied over time frames in which erosion of the newly deposited material is insignificant.

The passive transport and diffusion phase in all three models is handled by allowing material settling from the descent and collapse phases to be stored in small Gaussian clouds. These clouds are then diffused and transported at the end of each time step. Computations on the long-term grid are only made at those times when output is desired.

Model input. Input data for the models are grouped into the following general areas: (a) description of the disposal operation, (b) description of the disposal site, (c) description of the dredged material, (d) model coefficients, and (e) controls for input, execution, and output.

Ambient conditions include current velocity, density stratification, and water depths over a computational grid. The dredged material is assumed to consist of a number of solid fractions, a fluid component, and a conservative contaminant. Each solid fraction must have a volume concentration, a specific gravity, a settling velocity, a void ratio for bottom deposition, and information on whether or not the fraction is cohesive. For initial mixing calculations, information on initial concentration, background concentration, and water quality criteria for the constituent to be modeled must be specified. The description of the disposal operations for the DIFID model includes position of the disposal barge on the grid, the barge velocity, and draft, and volume of dredged material to be dumped. Similar descriptions for hopper dredge and pipeline operations are required for the DIFCD and DIFHD models. Coefficients are required for the models to accurately specify entrainment, settling, drag, dissipation, apparent mass, and density gradient differences. These coefficients have default values that should be used unless other site-specific

information is available. Appendix C - Table C1 lists the necessary input parameters with their corresponding units. More detailed descriptions and guidance for selection of values for many of the parameters is provided directly on-line in the system.

Model output. The output starts by echoing the input data and then optionally presenting the time history of the descent and collapse phases. In descent history for the DIFID model, the location of the cloud centroid, the velocity of the cloud centroid, the radius of the hemispherical cloud, the density difference between the cloud and the ambient water, the conservative constituent concentration, and the total volume and concentration of each solid fraction are provided as functions of time since release of the material. Likewise, the location of the leading edge of the momentum jet, the center-line velocity of the jet, the radius of the jet, the density difference between material in the jet and the ambient water, the contaminant concentration, and the flux and concentration of each solid fraction are provided as functions of time at the end of the jet convection phase in DIFCD and DIFHD.

At the conclusion of the collapse phase in DIFID and DIFHD, time-dependent information concerning the size of the collapsing cloud, its density, and its centroid location and velocity as well as contaminant and solids concentrations can be requested. Similar information is provided by DIFCD at the conclusion of the jet collapse phase. These models perform the numerical integrations of the governing conservation equations in the descent and collapse phases with a minimum of user input. Various control parameters that give the user insight into the behavior of these computations are printed before the output discussed above is provided.

At various times, as requested through input data, output concerning suspended sediment concentrations and solids deposited on the bottom can be obtained from the transport-diffusion computations. With Gaussian cloud transport-diffusion, only concentrations at the water depths requested are provided at each grid point. The volume of each sediment fraction that has been deposited in each grid cell is also provided. At the conclusion of the simulation, the thickness of the deposited material is given.

For evaluations of initial mixing for ocean disposal, results for water column concentrations can be computed in terms of milligrams per liter of dissolved constituent or in percent of initial dredged material suspended phase concentration. The maximum concentration within the grid and the maximum concentration at or outside the boundary of the disposal site are tabulated for specified time intervals.

Calculation of mass release

Estimation of both concentrations and volumes are required to compute mass release. Estimation of concentrations using standard elutriate results as described above is fairly straightforward. However, the estimation of fluid

and solids fractions released based on the model results requires a definition of what is considered a release.

The mass release of dissolved contaminants can be determined from the dissolved contaminant concentrations as defined by the standard elutriate test and the total volume of water entrained during dredging and released during the discharge. A conservative approach is to assume that the total volume will be released (Palermo et al. 1989). The volume of the fluid fraction is dependent on the in situ density of the sediment dredged and the volume of water entrained during dredging. This is a conservative approach, especially for mechanically dredged material discharged from barges, because a large portion of the fluid fraction will descend to the bottom as interstitial water with the solids and will be capped.

The mass release associated with the particle fraction is more difficult to calculate. Several factors must be considered and several approaches can be taken. The model results include an estimation of the total fraction of material remaining in suspension as a function of space and time. The "footprint" of the deposit of contaminated material can also be determined from both model results for a single discharge and the anticipated evolution of the mound size for the total volume of material to be placed, including the capping material.

One approach is to assume that all material remaining in suspension after a given time period is released. The appropriate time period used can be determined by the frequency of discharges from the barge or hopper, current conditions, and the disposal site size and anticipated size of the overall capped mound or deposit, considering the total volumes placed. Time periods on the order of 30 min have been used for such estimates (Palermo et al. 1989). Another approach is to examine the total volume of solids deposited within the anticipated footprint of the deposit to be capped and assume that all solids not settling within that footprint will be released. In either case, the results of the model should be carefully considered in making the estimates. Past field data have indicated that only a small fraction (a few percent) of the total mass of material will not quickly settle to the bottom and therefore could not be initially capped (Truitt 1986).

Based on the above considerations, the following steps should be followed in calculating the mass release during placement of contaminated material:

a. From standard elutriate test, determine dissolved and particle-associated concentrations for the open-water discharge.

b. Determine the volume of the water fraction of the discharge based on predredging sediment water content and anticipated water entrainment during dredging.

c. Calculate the total mass release of the dissolved fraction as the product of the dissolved concentration and the volume of water released (for

pipeline discharges, the mass release is the product of the concentration, flow rate, and time duration of the discharge).

d. Determine the total mass of suspended solids considered a release based on model results.

e. Calculate the mass of contaminants associated with the suspended solids as the product of the particle-associated concentration and the mass of solids released.

f. Calculate the total mass release as the sum of the dissolved and particle-associated releases.

Water column control measures

If the total mass release to the water column during placement is unacceptable, control measures could be considered to reduce the potential for water column effects or other dredging equipment and placement techniques, or use of another capping site could be considered. Control measures could include use of a submerged discharge point, submerged diffuser, tremie pipe, hopper dredge pumpdown, or similar equipment (Truitt 1987b).

Resuspension During Cap Placement

Resuspension of contaminated material already on the bottom by impact of discharges of capping material is a potential contaminant release mechanism for ISC, LBC, and CAD alternatives. However, the design of caps (Palermo 1991a) normally requires an excess thickness of capping material to account for inaccuracies in the placement process. The placement technique for the cap must be carefully chosen to minimize displacement and mixing of the contaminated and capping material. In general, the choice of capping materials and placement techniques is intended to result in a cap with an initial density less than or equal to the deposit of contaminated material.

Resuspension of contaminated material during cap placement will be located near the bottom and highly localized. Resuspended material should settle back to the bottom almost immediately. The overall size of the deposits laid down during capping and the gradual manner in which capping material is placed tend to result in capping of material displaced in the early stages of the capping operation. However, loss of contaminated material during cap placement has not been extensively monitored, and there are no techniques available for preproject estimation of potential resuspension.

Losses During Consolidation

Contaminant losses during consolidation after cap placement may be important especially for BLC and CAD. Pore water expressed through the cap will result in the release of contaminants to the overlying water unless the cap has sufficient sorption capacity to retain the contaminants. The release of contaminants via the expression of pore water through consolidation can be modeled as a short-term advective process using the methods of the next section. For organic contaminants, retention during consolidation is more likely if the cap material contains significant organic matter.

Long-Term Contaminant Release Through Cap

Determine required cap thickness and exposure time

The cap must be designed to chemically and biologically isolate the contaminated material from the aquatic environment. Determination of the minimum required cap thickness is dependent on the physical and chemical properties of the contaminated and capping sediments, the potential for bioturbation of the cap by aquatic organisms, and the potential for consolidation and erosion of the cap material. Laboratory tests have been developed to determine the thickness of a capping sediment required to chemically isolate contaminated sediment from the overlying water column (Sturgis and Gunnison 1988). These tests can also be performed in the presence of bioturbating organisms (Brannon et al. 1985). An evaluation of the potential for colonization of the capped site by bioturbating organisms and the behavior of those organisms with respect to intensity and depth of burrowing must be made. The minimum required cap thickness is considered the thickness required for chemical isolation plus that thickness of bioturbation associated with organisms likely to colonize the site in significant numbers.

The integrity of the cap from the standpoint of physical changes in cap thickness and long-term migration of contaminants through the cap should also be considered. The potential for a physical reduction in cap thickness due to the effects of consolidation and erosion can be evaluated once the overall size and configuration of the capped mound is determined. The design cap thickness can then be adjusted such that the minimum required cap thickness is maintained.

Most of the consolidation of the contaminated material will occur within a few weeks of placement. Cap placement could be delayed an appropriate time period to allow the majority of consolidation to occur. Such a delay also holds advantage from the standpoint of resistance of the contaminated deposit to displacement during cap placement. However, a delay exposes the contaminated material to the environment. An appropriate delay between contaminated material placement and capping must balance environmental exposure

with the engineering requirements of stability and the scheduling constraints of the dredging required for capping.

There is potential for long-term migration of contaminants through the cap due to consolidation of the contaminated material and diffusion and advection. The techniques for evaluation of consolidation (Poindexter-Rollings 1990) can be used to estimate the cap thickness potentially affected by the movement of contaminated pore water. Theoretical models for evaluation of long-term cap releases is discussed in the following section.

Models for long-term capping releases

The goal of capping is containment for a sufficiently long period of time that natural degradation processes have the opportunity to render the contaminant harmless or to reduce the contaminant flux to levels that are protective of ecological and human health. Due to the uncertainty associated with the rate and existence of natural degradation processes, this discussion will assume no irreversible fate processes and focus on the estimation of the undergraded contaminant losses through a cap.

Potential long-term contaminant loss mechanisms for capped sediment are essentially identical to the original uncapped sediments. The pore water transport processes of diffusion and advection, perhaps enhanced by the presence of colloidal particles in the pore water, are present. Particulates that remain suspended can also enhance the transport of contaminants, but a cap should act as an effective filter or scavenger of noncolloidal particulates. In addition, and especially important for strongly sorbed contaminants, particle movement processes such as erosion and deposition as well as bioturbation occur.

In the capped system, the bioturbating organisms at the original sediment-water interface are buried, but recolonization of the upper cap layer occurs. Over much of the capped depth, pore water processes such as molecular diffusion and advection dominate transport processes. Erosion of the cap can eliminate resistance to mass transfer provided by the cap by allowing deeper penetration of the bioturbation layer. In the long-term models discussed in this section, the cap is assumed stable or replaced as necessary to maintain sufficient depth to avoid bioturbation of the original sediments. The effects of slow depositional and erosional processes on contaminant transport through caps are considered, but the effects of storm events on cap stability are not included in the models discussed in this section. The long-term stability of a cap can be assessed via the methods presented by Dortch et al. (1990) and Maynord (1993).

Despite the similarity of transport processes in the capped and uncapped sediment, the cap serves to reduce the net contaminant transport over the uncapped situation as a result of the following:

a. Destruction of bioturbating organisms at original sediment-water interface.

b. Increase of diffusion path length or advective path length before contaminants are transported to the water column.

c. Elimination of erosion at original sediment-water interface, at least until erosion of cap.

d. Introduction of thermodynamic limitations due to elimination of particle transport processes in the contaminated zone.

e. Retardation of pore water processes through the cap due to the presence of unfilled sorption sites.

In the following sections, processes affecting long-term cap effectiveness will be discussed, and a quantitative analysis of these processes will be presented.

Molecular diffusion. Molecular diffusion is the process of random molecular motion leading ultimately to equalization of chemical potentials everywhere within the system. In free water, the diffusive flux is written as proportional to the concentration gradient in the water

$$N_A = - D_{A2} \frac{\partial C_w}{\partial z} \tag{47}$$

where

N_A = flux of contaminant A in free water, g/m^2 sec

D_{A2} = diffusivity of A in water, m^2/sec

C_w = dissolved concentration of A, g/m^3

z = distance through water, m

The diffusion coefficient is of the order of 10^{-5} cm^2/sec (10^{-9} m^2/sec) in water. The minus sign is needed in Equation 47 because contaminants diffuse from regions of higher concentrations to regions of lower concentrations by random molecular motion. The random motion of molecules that leads to diffusion generally occurs at significant rates only within the pore spaces of the sediment or the overlying cap. Therefore, diffusivity must be corrected for the available pore space in the media (ϵ = porosity) and the fact that the diffusion paths are not straight (τ = tortuosity = actual path length/straight-line path link). In a saturated, unconsolidated granular sediment, the tortuosity is approximately $\epsilon^{-1/3}$ (Millington and Quirk 1961) suggesting

$$N_A = - D_{A3} \frac{\partial C_w}{\partial z} = - D_{A2} \epsilon^{4/3} \frac{\partial C_w}{\partial z} \tag{48}$$

where

N_A = flux of contaminant A into cap, g/m^2 sec

D_{A3} = effective diffusivity of A in sediment, m^2/sec

C_w = water concentration of A, g/m^3

z = distance into sediment or cap, m

D_{A2} = diffusivity of A in water, m^2/sec

ϵ = sediment porosity, m^3 voids/m^3 total volume

Flux is positive when the movement is toward positive z, that is, into the cap or sediment.

Advection. Advection is a process associated with the bulk movement of the pore water in response to pressure or head gradients in the sediment. Advective processes should be especially important near the banks of rivers, shores of lakes, and in estuarine systems subject to significant tidal variations. In many regions, there is insufficient information on the permeability and hydraulic gradient to adequately assess the advective contaminant transport. If such information is available, however, the advective flux is written as follows:

$$N_A = U C_w \tag{49}$$

where

N_A = flux of contaminant A, g m^{-2} sec^{-1}

U = Darcy water velocity, m/sec

C_w = dissolved concentration of A, g/m^3

The Darcy velocity used to define the advective flux is averaged explicitly over the entire cross-sectional area of the medium and implicitly over some volume. This averaging fails to identify the variations in velocity that occur both within a pore and between adjacent pores in the medium. The variation in velocities on the microscale results in additional mixing of the contaminant above what would result from molecular diffusion alone. By analogy with molecular mixing, microscale dispersive mixing is parameterized as follows:

$$N_A = - E_{A3} \frac{\partial C_w}{\partial z} \tag{50}$$

where

N_A = flux of contaminant A, g/ m^{-2} sec

E_{A3} = effective dispersion coefficient in medium, m^2/sec

C_w = dissolved concentration of A, g/m^3

z = distance through water, m

Although the effective dispersion coefficient can be estimated from medium properties, better estimates are obtained from laboratory contaminant transport or tracer experiments that simulate field conditions.

The dispersion coefficient is often taken as approximately proportional to the Darcy velocity. The constant of proportionality, the dispersivity, is related to the characteristic size of microscale heterogeneities. For a homogeneous, granular medium, the dispersion coefficient is expected to be approximately half of the particle diameter, that is

$$E_{a3} = \tau U = \frac{d_p}{2} U \tag{51}$$

where

E_{a3} = dispersion coefficient in medium, m^2/sec

τ = dispersivity, m

U = Darcy velocity, m/sec

d_p = particle diameter, m

Very low advective velocities can control contaminant transport when compared with diffusion. The importance of advection relative to diffusion can be quantified by the Peclet number (Pe), which is defined

$$Pe = U L / D_{A3} \tag{52}$$

where

U = advective velocity

L = transport length scale

D = effective diffusion coefficient

Since capping relies on reducing (by design) contaminant transport to diffusion, evaluation of the Peclet number is very important. For example, the effective diffusion coefficient in the cap is typically of order 10 cm^2/year. For a chemical isolation layer of only 10 cm, advection at only 1 cm/year is approximately equal in importance to diffusion for transport. Due to the potential importance of advective processes, the prevailing groundwater velocities must be ascertained before confidence can be placed in the ability of a cap to contain contaminants.

Facilitated transport. Advection, dispersion, and diffusion are pore water processes that may be enhanced by the presence of colloidal particles in the pore water. Colloidal organic matter in the pore water may be especially important. Due to natural degradation processes, there typically exists colloidal organic carbon, for example, large molecular weight humic and fulvic acids, at concentrations of the order 10 to 100 mg/ℓ. Hydrophobic organic contaminants can effectively sorb to this dissolved organic carbon in the same manner that they sorb to organic carbon on the sediment surface. Since the dissolved organic carbon (DOC) is mobile, however, the presence of colloidal organic matter essentially increases the capacity of pore water to carry contaminants. The DOC moves at the velocity of the pore water and with a diffusivity of the same order of magnitude as the free water diffusivity of the contaminant.

If the partition coefficient between pure water and the colloidal species is K_c, then the advective and diffusive flux for a contaminant can be written

$$N_A = U\left(1 + K_c C_c\right) C_w - D_{A2}\epsilon^{4/3}\left(1 + D' K_c C_c\right)\frac{\partial C_w}{\partial z} \tag{53}$$

where

N_A = flux of contaminant A is direction of bulk flow, g/m^{-2} sec

U = Darcy velocity, m/sec

K_c = colloid - water partition coefficient of A, m^3/g

C_c = colloid concentration in water, g/m^3

C_w = dissolved concentration of A, g/m^3

D_{A2} = diffusivity of A in water, m^2/sec

D' = ratio of colloidal species diffusivity to D_{A2}, m^2/sec

ϵ = sediment porosity, m^3 voids/m^3 total volume

z = distance into sediment, m

For hydrophobic organic species, K_c should be of the same order as the partition coefficient between water and sediment organic carbon, K_{oc}. In addition, C_c should be approximately defined by the DOC for hydrophobic organic contaminants if the particulate organic carbon is effectively scavenged by the sediment. Finally, the diffusivity of the colloidal species in water is likely to be approximately the same as the diffusivity of the contaminant species, that is, D' is approximately equal to one since almost all organic species exhibit a water diffusivity of the order of 10^{-9} m^2/sec. With these assumptions, Equation 53 can be written as follows:

$$N_A = U\left(1 + K_{oc}C_{doc}\right) C_w - D_{A2}\epsilon^{4/3}\left(1 + K_c C_C\right) \frac{\partial C_{w2}}{\partial z} \tag{54}$$

where

N_A = flux of contaminant A is direction of bulk flow, g/m^2 sec

U = Darcy velocity, m/sec

K_c = colloid - water partition coefficient of A, m^3/g

C_w = dissolved concentration of A, g/m^3

D_{A2} = diffusivity of A in water, m^2/sec

ϵ = sediment porosity, m^3 voids/m^3 total volume

C_C = colloidal species concentration, g/m^3

z = distance into sediment, m

Equation 54 is based on equilibrium partitioning concepts and is, therefore, primarily applicable to organic contaminants. Guidance on applying a modification of equilibrium partitioning to metals is available in Chapter 4 in the section on a priori prediction. However, there is no guidance available for colloidal species that might sorb metallic or elemental species.

Slow deposition and erosion. Deposition and erosion processes move contaminants by exposing contaminated pore water and by movement of

contaminants sorbed to the depositing or eroding particles. For a particle deposition velocity U_d, the flux of contaminants by this process can be written

$$N_A = U_d C_s = U_d \left(\epsilon + \rho_b K_d + \epsilon C_c K_c\right) C_w \tag{55}$$

where

N_A = flux of contaminant A in free water, g/m^2 sec

U_d = net deposition velocity, m/sec

C_s = local sorbed concentration of A, typically concentration of A in cap material at cap-water interface, g/m^3

ϵ = local porosity, m^3 voids/m^3 total volume

ρ_b = local bulk density, g/m^3

K_d = solids-water partition coefficient, m^3/g

K_c = colloidal-water partition coefficient, m^3/g

C_c = colloidal species concentration, g/m^3

C_w = local dissolved water concentration of A, g/m^3

The first term in parenthesis in Equation 55 is that portion of the flux associated with the pore water movement. The third term in parenthesis represents that portion of the flux associated with the colloidal motion. The second term in parenthesis represents the movement of contaminants sorbed to the depositing or eroding particles.

Bioturbation. Bioturbation is an effective means of moving dissolved and sorbed contaminants near the sediment-water interface. Bioturbation is probably the most significant mechanism for chemical transport from noneroding bottom sediments. For lack of a better estimation method, bioturbation fluxes are often modeled as an effective diffusion process. For example, it has been estimated that bioturbation has resulted in an effective particle diffusion coefficient of about 10 cm^2/year in New Bedford Harbor (Thibodeaux 1990). This is approximately a factor of 10 smaller than the estimated molecular diffusivity. Since bioturbation is a particle movement process, however, the ratio of bioturbation to molecular diffusion is the order of $D_b K_d / D_{A2}$; for a contaminant with a sediment-water partition coefficient of the order of 10^4 ℓ/kg (for example, PCBs in Indiana Harbor sediment (Environmental Laboratory 1987)), bioturbation in this case is approximately 10^3 times more rapid than molecular diffusion. The bioturbation flux, assuming that it can be represented by a diffusion model, can be written as follows:

$$N_A = D_b \frac{\partial C_s}{\partial z} = - D_b \left(\epsilon + \rho_b K_d + \epsilon C_c K_c\right) \frac{\partial C_w}{\partial z} \tag{56}$$

where

N_A = flux of contaminant A out of sediment, g/m^2 sec

D_b = effective bioturbation diffusion coefficient, m^2/sec

C_s = sorbed concentration of A, g/m^3

z = distance into sediment, m

ϵ = sediment porosity, m^3 voids/m^3 total volume

ρ_b = bulk density, g/m^3

K_d = sediment-water partition coefficient, m^3/g

K_c = colloidal-water partition coefficient,m^3/g

C_c = colloid concentration in water, g/m^3

C_w = dissolved concentration of A, g/m^3

Elimination of the organisms at the original sediment-water interface is a very effective means of reducing the migration of contaminants from the sediment into the overlying water as well as an effective means of isolating the contaminants from bottom-dwelling organisms. Recolonization of the new sediment-water interface, however, reduces the effective cap thickness. Bioturbating species are limited to the upper sediment, and many species are limited to aerated sediments in the upper few centimeters. Some species, however, burrow deeply into the sediment, and the occurrence of these organisms may require a deeper cap or elimination of the capping alternative in particular areas. Assessment of this problem requires a survey of the type and density of the organisms in a particular contaminated sediment area prior to remediation planning.

Combined process model. The combination of all of the processes discussed above into a dynamic mass balance on the capped sediment allows estimation of the contaminant flux through the cap. The transient accumulation of the contaminant includes accumulation in the pore water, on the colloidal fraction in the pore water, and in the sorption sites in the cap. If it is assumed as before that sediment-water partitioning is reversible, instantaneous and linear, the conservation equation for contaminant transport in the cap can be written as follows:

$$\epsilon R_f \frac{\partial C_w}{\partial t} + \left[U_d \epsilon R_f + U(1 + C_c K_c)\right] \frac{\partial C_w}{\partial z} = \left[D_b \epsilon R_f + D_{A2} \epsilon^{4/3} \left(1 + D' C_c K_c\right) + E_{A3}\right] \frac{\partial^2 C_w}{\partial z^2} \tag{57}$$

where R_f is a retardation factor defined by

$$R_f = 1 + \frac{\rho_b K_d}{\epsilon} + C_c K_c \tag{58}$$

and

ϵ = sediment porosity, m^3 voids/m^3 total volume

C_w = dissolved concentration of A, g/m^3

t = time, sec

U_d = net deposition velocity, m/sec

U = Darcy velocity, m/sec

K_c = colloid-water partition coefficient, m^3/g

C_c = colloid concentration in water, g/m^3

z = distance into sediment, m

D_b = effective biotubation diffusion coefficient, m^2/sec

D_{A2} = diffusivity of A in water, m^2/g

D' = ratio of colloidal species diffusivity to D_{A2}, m^2/sec

ρ_b = bulk density, g/m^3

K_d = sediment-water partition coefficient, m^3/g

Dividing Equation 57 by R_f gives

$$\frac{\partial C_w}{\partial t} + \left[U_d + U\frac{(1 + C_c K_c)}{R_f} \right] \frac{\partial C_w}{\partial z} = \left[D_b + D_{A2}\, \epsilon^{1/3} \frac{(1 + D' C_c K_c)}{R_f} + \frac{E_{A3}}{R_f} \right] \frac{\partial^2 C_w}{\partial z^2} \tag{59}$$

where U is the interstitial velocity, or U/ϵ, and all other terms are as defined for Equation 57. The significance of the retardation factor, R_f, in Equation 59 is evident from its appearance in the denominator of several terms in the equation. As indicated by Equation 58, R_f is always greater than or equal to one. Thus, retardation reduces the significance of the terms that R_f appears in, and thereby retards the effective velocity or diffusion of a strongly sorbing contaminant. The effective velocity is the bracketed term on the left-hand side of the equation while the effective diffusion coefficient is the bracketed term on the right-hand side of the equation. Equation 59 assumes that the partition coefficients and colloid concentration are not spacially dependent. Solutions of Equation 59 can be used to define concentration gradients in caps or, through the previously defined flux equations, determine contaminant fluxes at any time out of capped material.

Use of Equation 59 requires determination of the indicated parameters and an appropriate means of using these parameters to define concentration or fluxes as a function of time or position. Porosity and bulk density are sediment or field parameters that are often measured or are available. Molecular diffusivity is a chemical-specific property that is tabulated or for which estimation methods are available (Lyman, Rheel, and Rosenblatt 1990; Reid, Prausnitz, and Sherwood 1977). Net deposition velocities and effective bioturbation diffusivities are site specific and difficult to measure since field data are often limited to a small number of samples over short time periods. The time evolution of vertical contaminant concentration profiles in sediments is needed before accurate estimates of bioturbation diffusion coefficients can be made. Generally, groundwater gradients and hydraulic conductivities in the vicinity of a stream or lake are not known with sufficient resolution to accurately predict groundwater flow velocities directly. In most large lake systems, however, significant convective velocities are likely to be confined to the nearshore environment. Finally, chemical partitioning data are chemical and sediment specific, and accurate determination of these terms require laboratory tests such as batch or continuous leaching tests as discussed in Contaminant Losses During Pretreatment. In the absence of specific laboratory characterization of the contaminant partitioning, estimation techniques can be employed for hydrophobic organic chemicals as discussed in Appendix B.

As previously indicated, in the absence of direct measurements, C_c and K_c are approximated by the dissolved organic carbon concentration and K_{oc}, respectively, for hydrophobic organic chemicals. A priori estimation of K_{oc} is discussed in Appendix B. Dissolved organic carbon concentration is difficult

to estimate without data from laboratory leach tests. D', the ratio of the colloidal diffusivity to the effective contaminant diffusivity in the medium, can be estimated from information on the size of the colloidal matter or assumed to be approximately equal to 1. Thus, all of the parameters in Equation 59 can be estimated from sediment or cap chemical-physical properties determined in laboratory testing or from field data.

Use of the parameters as defined by either field, laboratory, or predictive estimation techniques to the estimation of concentrations or fluxes with or without a cap requires numerical or analytical solution of Equation 59. Analytical solutions will be preferred here recognizing that simple physical systems amenable to analytical solution are as sophisticated as can normally be justified by the precision of the input parameters. Consistent with this goal, analytical solutions will be described for the following:

a. Advective transport through a cap.

b. Steady-state diffusive flux through a capping layer.

c. Diffusive flux through capping layer at any time.

d. Time to diffusive breakthrough.

e. Time to diffusive steady-state flux.

In each case, a cap is assumed to be placed on a contaminated sediment as shown in Figure 33. The result of the capping process is a layer of thickness L of initially clean capping material that isolates the contaminants from the bottom dwelling organisms and slows their release back into the water column. The sediments will be assumed to be sufficiently contaminated that the contaminant concentrations in the material below the original sediment-water interface remains essentially constant. This assumption provides an upper bound to the actual contaminant release rate. The total depth, L, of cap is assumed to be composed of two layers, L_{cap}, a layer in which advection or molecular diffusion dominate, and L_{Bio}, a layer in which bioturbation is the dominant transport process. As will be indicated later, the rate of contaminant transport in the bioturbated layer is likely to be much greater than that through the remainder of the cap. Therefore, the effective thickness of the cap is essentially equal to the total cap thickness minus the bioturbation layer.

Significant advection is an indication that capping may not be an appropriate containment mechanism. For compounds that can be sorbed by the capping layer, a cap will provide containment for long periods of time, even in the presence of advection. If advection is the dominant transport process, the contaminant migrates through the cap at a rate given by U/R_f. A breakthrough time, or the time until contaminants are observed in the water above the cap, can thus be defined as

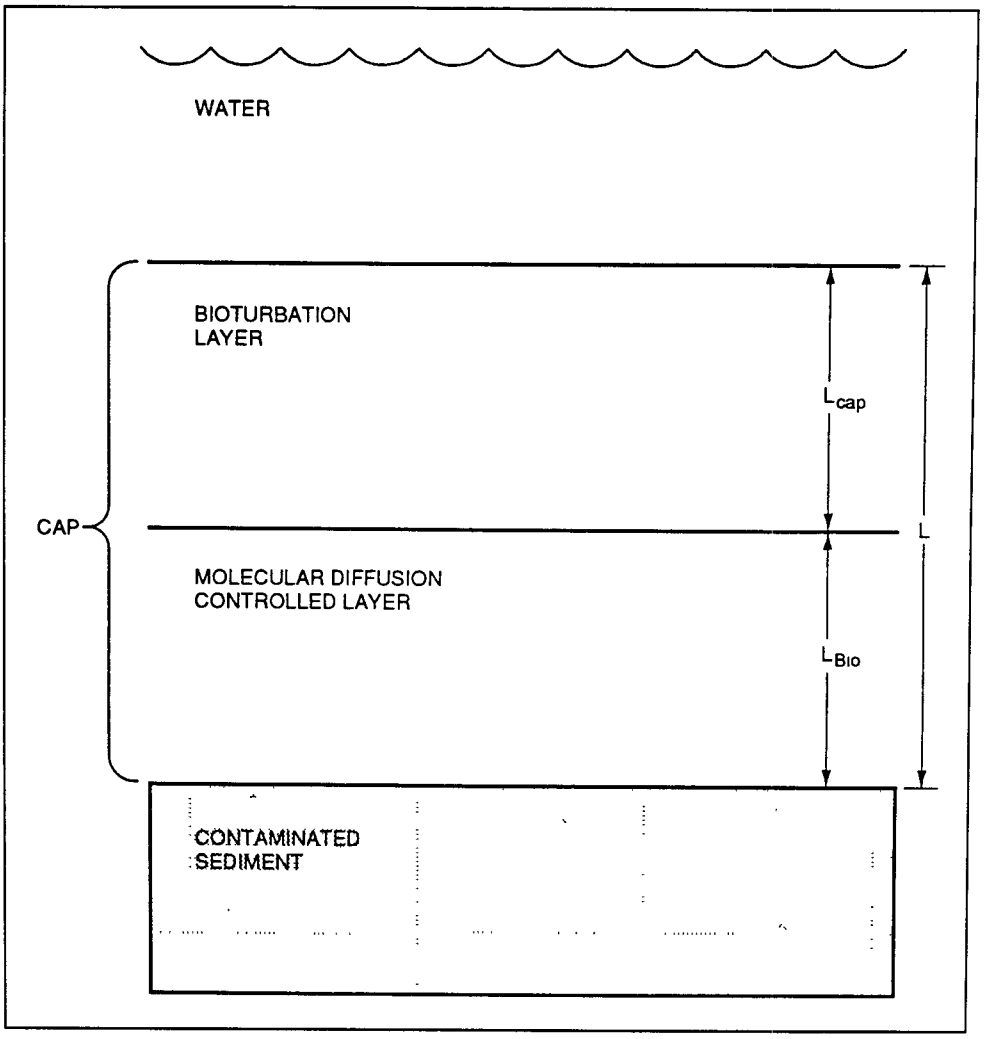


Figure 33. Definition sketch for in situ capping losses

$$\tau_{b,adv} = \frac{L_{cap} R_f}{U} \tag{60}$$

For highly sorbing compounds such as PCBs or PAHs, advective transport through the cap is still orders of magnitude smaller than the groundwater flow velocities as long as the cap retains some sorption capability. A sand or gravel cap, however, will be relatively permeable and will exhibit little or no sorption, resulting in rapid breakthrough if advective transport should occur. Caps composed of fine-grained material containing organic carbon will be both more sorptive and less permeable. In addition, the extra resistance to flow posed by the presence of the capping layer is likely to divert groundwater flows to regions other than the capped sediment. Finally, permeability control can always be achieved in particular situations by placement of a low permeability layer such as a bentonite-impregnated fiber mat that will reduce the expected advective flows to very low levels. Thus, it is expected that the sites most suitable for capping will have adequately low groundwater

velocities or can be modified to reduce groundwater flows. In these cases, molecular diffusion is expected to be the primary transport process in the cap, and the subsequent discussion of contaminant losses will focus on that process.

In the molecular diffusion layer, the effective cap height is L_{eap} and not L. Within this layer of the cap, that is below the bioturbation layer, bioturbation is negligible. In the region L_{eap}, therefore, Equation 58 becomes

$$\frac{\partial C_w}{\partial t} = \left[D_{A2}\, \epsilon^{1/3} \frac{\left(1 + D' C_c K_c\right)}{R_f} \right] \frac{\partial^2 C_w}{\partial z^2} \tag{61}$$

The bracketed term on the right-hand side of Equation 61 is the effective diffusion coefficient in the cap. This term accounts for facilitated transport and sorption. As indicated by Equation 61, the contaminant transport rate through the cap is reduced if sorption occurs in the cap, that is, $R_f > 1$. For hydrophobic organic contaminants, this suggests that a high organic content cap should be chosen.

In the region L_{Bio}, bioturbation is expected to be a much more rapid transport process for sorbing contaminants than molecular diffusion so that molecular diffusion can be neglected, and Equation 59 becomes

$$\frac{\partial C_w}{\partial t} = \left[D_b\, \epsilon^{1/3}\, R_f \right] \frac{\partial C_w^2}{\partial z^2} \tag{62}$$

Solutions of Equations 61 and 62 can be used to describe concentrations and fluxes of contaminants from the cap. Crank (1975) and Carslaw and Jaeger (1959) present solutions to equations of the form of Equations 61 and 62 under a wide variety of boundary and initial conditions. In the sections that follow, selected solutions will be presented that describe contaminant flux through a cap initially clean of contaminants overlying a contaminated sediment layer of essentially constant concentration.

The maximum release rate will occur after contaminants have penetrated through the entire cap. Since the amount of contaminant in the original sediment is assumed constant, steady-state solutions to Equations 61 and 62 exist that represent this upper bound flux. Steady-state forms of Equation 61 (molecular diffusion layer) are given by

$$0 = \left[D_{A2}\, \epsilon^{1/3} \left(1 + D' C_c K_c\right) \right] \frac{d^2 C_w}{dz^2} \tag{63-a}$$

and for $D' = 1$,

$$0 = D_{A2}\ \epsilon^{1/3}\ \frac{d^2C_{pw}}{dz^2} \tag{63-b}$$

where

D_{A2} = diffusivity of A in water, m^2/sec

ϵ = porosity, m^3 voids/m^3 total

D' = ratio of colloidal species diffusivity to D_{A2}, m^2/sec

C_c = colloid concentration in water, g/m^3

K_c = colloid-water partition coefficient, m^3/g

C_w = water concentration of A, g/m^3

z = distance up into cap, m

C_{pw} = pore water concentration of A, including colloidal bound, g/m^3

Under steady conditions, all sorption sites in the cap are filled and no transient accumulation occurs. As a result, the retardation factor, which represents this transient accumulation, does not appear in Equations 63-a and 63-b.

Steady-state forms of Equation 62 (bioturbation layer) are given by

$$0 = \left(D_b\ \epsilon\ R_f\right) \frac{d^2C_w}{dz^2} \tag{64-a}$$

and

$$0 = \left[\frac{D_b\ \epsilon\ R_f}{(1 + K_cC_c)}\right] \frac{d^2C_{pw}}{dz^2} \tag{64-b}$$

From Equations 63-b and 64-b, steady-state flux through the cap is given by:

$$N_{ss} = K_{ov}\left(C^o_{pw} - C^*\right) \tag{65}$$

where

$$K_{ov} = \left[\frac{L_{cap}}{D_{A2}\,\epsilon^{1/3}} + \frac{L_{Bio}(1 + K_c\,C_c)}{\epsilon\,R_f\,D_b} + \frac{1}{K_b} \right]^{-1} \tag{66}$$

and

$$K_b = 0.036 \left[\frac{D_{A2}}{\sqrt{A}} \right] \left[\frac{v\sqrt{A}}{\nu} \right]^{0.8} S_c^{1/3} \tag{67}$$

and

N_{ss} = steady-state flux, g/m^2•s

K_{ov} = overall mass transfer coefficient, m/year

C_{pw}^{o} = pore water concentration in original sediment, g/m^3

C^* = background water concentration above cap, g/m^3

D_{A2} = diffusivity of A in water, m^2/sec

ϵ = porosity, m^3 voids/m^3 total

K_c = colloid-water partition coefficient, m^3/g

C_c = colloid concentration in water, g/m^3

K_b = benthic mass transfer coefficient, cm/year

A = surface area of cap, yd^2

ν = kinematic viscosity of water, cm^2/sec

v = current speed above cap, m/sec

S_c = Schmidt number, dimensionless = ν/D_{A2}

h_d = effective depth of cap, diffusive layer depth, m

h_b = depth of bioturbation layer, m

Equation 66 can be simplified by defining a coefficient R such that

$$R = \frac{\epsilon\,R_f}{1 + K_c C_c} \tag{68}$$

so that Equation 66 becomes

$$K_{ov} = \left[\frac{L_{cap}}{D_{A2}\,\epsilon^{1/3}} + \frac{L_{Bio}}{R\,D_b} + \frac{1}{K_b} \right]^{-1} \tag{69}$$

Techniques for predicting the pore water contaminant concentration, C_w, below the original sediment-water interface were previously discussed in the section on leachate quality. If a low solubility chemical is present as a pure phase in the original uncapped sediment, C_w is limited by that solubility. As indicated previously, one advantage of the cap is that direct exposure of chemicals in a pure phase is eliminated, and the pore water processes that control are thermodynamically limited in their capacity for contaminant transport.

Equation 65 is written with pore water concentration (dissolved plus colloidally bound) as the input variable. The dissolved plus colloidally bound concentration is operationally defined as the dissolved fraction in that it is the concentration that is measured after filtering the water. Thus the normally available dissolved concentration contains both dissolved and colloidally bound contaminant, and Equation 65 is the appropriate equation to use. In addition, partition coefficients between sediment and water usually are measured by employing the operational definition of dissolved. That is, the water concentration predicted by such a partition coefficient would be total pore water concentration or the sum of the truly dissolved and the colloidal contaminant, and again Equation 65 would be the appropriate equation to use with that concentration.

An equivalent equation could be written with truly dissolved concentration as the input variable and modified definition of the overall mass transfer coefficient to include facilitated transport. If only the truly dissolved concentration is used, that is, if empirical relationships from the literature are used to estimate distribution coefficients, the pore water concentration is given by

$$C_{pw} = \frac{C_s}{K_d}\,(1 + K_c C_c) \tag{70}$$

and the retardation coefficient is as previously defined. As discussed in Appendix B, K_d is given by

$$K_d = f_{oc}\,K_{oc} \tag{71-a}$$

and

$$K_c \approx K_{oc} = \frac{K_d}{f_{oc}} \tag{71-b}$$

If $K_c = K_{oc}$ and C_c is approximated by DOC, then

$$C_{pw} = \frac{C_s}{K_d} \left[1 + \frac{K_d}{f_{oc}} C_{doc} \right] \tag{72}$$

If pore water concentrations are estimated from sequential batch leach tests as previously described, then there is no need to adjust for facilitated transport. Leachate concentrations provided by this test include colloidally bound contaminant. Distribution coefficients obtained from sequential batch leach tests also include the influence of colloids. Retardation factors obtained from sequential batch leach tests, therefore, should not be corrected to account for facilitated transport. In this case, Equations 63-b or 64-a should be used, and the retardation factor in these equations becomes

$$R_f = 1 + \frac{\rho_b K_d}{\epsilon} \tag{73}$$

The steady-state flux given by Equation 65 is an upper bound to the actual release rate. If significant sorption occurs in the cap, the time required to reach steady state can be very long. Solution of the transient flux equation, Equation 61, in the molecular diffusion layer of the cap suggests that the ratio of the release rate from the top of the cap at any time to the steady-state rate is given by (Thoma et al. 1993)

$$\frac{R_A(t)}{R_A(t \to \infty)} = 1 + 2 \sum_{n=1}^{\infty} (-1)^n \exp \left[- \frac{D_{eff}\, n^2 \pi^2 t}{h_d^2} \right] \tag{74}$$

where

$R_A(t)$ = release rate of contaminant, at time t, g/sec

$R_A(t \to \infty)$ = release rate of contaminant, at steady state, g/sec

D_{eff} = effective diffusivity, bracketed term Equation 59, m^2/sec

h_d = effective depth of cap diffusive layer, m

From this solution, the time required to achieve a breakthrough flux that is 0.05 percent of the steady-state flux is given by

$$\tau_b = 0.54 \frac{h_d^2}{D_{eff}^2} \tag{75}$$

and the time required to achieve 95 percent of the steady-state diffusive flux through the cap without bioturbation is given by

$$\tau_{ss} = 3.69 \frac{h_d^2}{D_{eff}\, \pi^2} \tag{76}$$

where τ_{ss} is the time required for the instantaneous flux to approach the maximum value, that is, the steady-state flux given by Equation 74. Since the effective diffusivity for a sorbing compound may be of the order of 10^{-9} cm^2/sec, this suggests that it could take thousands of years to achieve the steady-state release rate defined by Equation 65. It should be realized that even the steady-state release rate is still orders of magnitude lower than the release rate from the uncapped contaminated sediment.

The model equations presented have received experimental validation in small laboratory test cells in which the release rate of trichlorophenol was monitored (Wang et al. 1991; Thoma et al. 1993). Field demonstrations of capping have been conducted, and preliminary evaluations of capping effectiveness have been published (O'Connor and O'Connor 1983; Brannon et al. 1985; Truitt 1986b; Brannon et al. 1986). The information presented in these evaluations is insufficient to determine the field validity of the in situ model equations, primarily due to the long time required for measurable contaminant migration. In addition, the model equations discussed for in situ capping provide estimates of minimum losses because they do not account for losses during placement and cap consolidation and erosion.

Long-term capping model summary. The general theoretical framework for modeling long-term capping effectiveness was presented. The general model includes the following transport processes: molecular diffusion, advection, dispersion associated with advection, low-order deposition/erosion (excludes storm events), bioturbation, and sorption by capping material. Simple model equations that neglect deposition/erosion, bioturbation, advection, and dispersion were presented. These model equations indicate that hydrophobic organic chemicals in sediments can be isolated from the overlying water column as long as the cap is stable, cap thickness is sufficient to eliminate bioturbation, and advective transport is less than diffusive transport.

7 Contaminant Losses During Effluent and Leachate Treatment

Background

After contaminated sediment has been removed by dredging, effluent and leachate discharges may be generated during pretreatment, treatment, and disposal operations. Effluent is generated during pretreatment by dewatering processes, during hydraulic disposal in CDFs, during mechanical placement in nearshore and in-water CDFs, and as a process waste stream during dredged material treatment. Leachate is generated at pretreatment and CDFs as a result of consolidation of dredged materials and infiltration and percolation of rainfall.

Both effluent and leachate may be collected for treatment and/or disposal, or may be allowed to dissipate to the surrounding soil and waters. This chapter addresses contaminant losses associated with various treatment alternatives for effluent and leachate and will not address potential losses associated with release of untreated effluent and leachate. Untreated effluent and leachate losses can be estimated using the predictive techniques discussed in Chapter 4.

Leachate and effluent from a single source will contain essentially the same contaminants, with the primary differences being the respective volumes generated, concentrations of contaminants, oxidation-reduction potential, and pH. Assuming the effects of the variable loading conditions can be effectively managed, process efficiency data are needed in order to estimate contaminant losses for treatment processes applied to effluent and leachate. Process efficiency is a function of initial contaminant concentrations, waste stream characteristics, process design, and unit operation and maintenance. At best, ranges in process efficiency can be estimated a priori. Bench- and pilot-scale testing is required to determine treatment effectiveness for specific processes and waste streams. In most cases, complete destruction of contaminants is not feasible, and some contaminant loss will occur in process waste streams.

The contaminants identified at the areas of concern under the ARCS program include PCBs, heavy metals, and PAHs. Other organic priority pollutants have been identified, but are generally present at concentrations of less than 1 mg/kg. Removal of suspended solids, organic contaminants, nutrients, ammonia, oxygen-demanding materials, oil and grease, and heavy metals can also be of concern for dredged material leachate and effluent. Three treatment technology types may be needed as follows: organic chemical removal or treatment, suspended solids removal, and heavy metals removal.

Contaminant Loss Estimation

Estimation of contaminant losses during effluent and leachate treatment is based on a materials balance of the process treatment train. A process flowchart should identify waste streams through which contaminants can escape treatment or control. An example is shown in Figure 34. Process flowcharts can be developed from site-specific bench- or pilot-scale treatability studies or from treatability studies conducted on similar wastewaters. Sediment sampling and appropriate laboratory tests as described in Chapter 4 are necessary to determine effluent and leachate characteristics and contaminant concentrations. Information on effluent and leachate characteristics, anticipated effluent and leachate flows, and treatment process efficiencies, is needed before treatment process trains and flowcharts can be fully developed.

Aqueous treatability data are available for many potentially applicable treatment technologies that can be used for a priori estimation of contaminant losses. These data, while suitable for planning level assessments, treatment process screening, and contaminant loss estimation, are not always suitable for site-specific design calculations. For this reason, bench- and/or pilot-scale treatability studies are usually needed to fully evaluate candidate treatment technologies. Treatability studies should be conducted such that the information needed to estimate contaminant losses is obtained in addition to the information needed for full-scale design.

Sources of information on treatment efficiency include Cullinane et al. (1986), Berger (1987), Corbitt (1989), and Averett et al. (1990). Emerging treatment technologies can be found in the USEPA site technology profiles (USEPA 1993b). In addition, computerized databases are available from the USEPA Risk Reduction Engineering Laboratory (RREL) (USEPA 1992), the Vender Information System for Innovative Treatment Technologies (VISITT) (USEPA 1993c), and SEDiment Treatment Technologies Database (SEDTEC) (Wastewater Technology Centre 1993). The RREL database contains 1,166 chemical compounds and over 9,200 sets of treatability data. It is available in diskette form for MS DOS personal computers and is menu driven and easy to use. Table 10 illustrates some of the information available on treatment processes available in the RREL database on aqueous waste steams. The data listed in Table 10 represent composite results for a variety of wastes

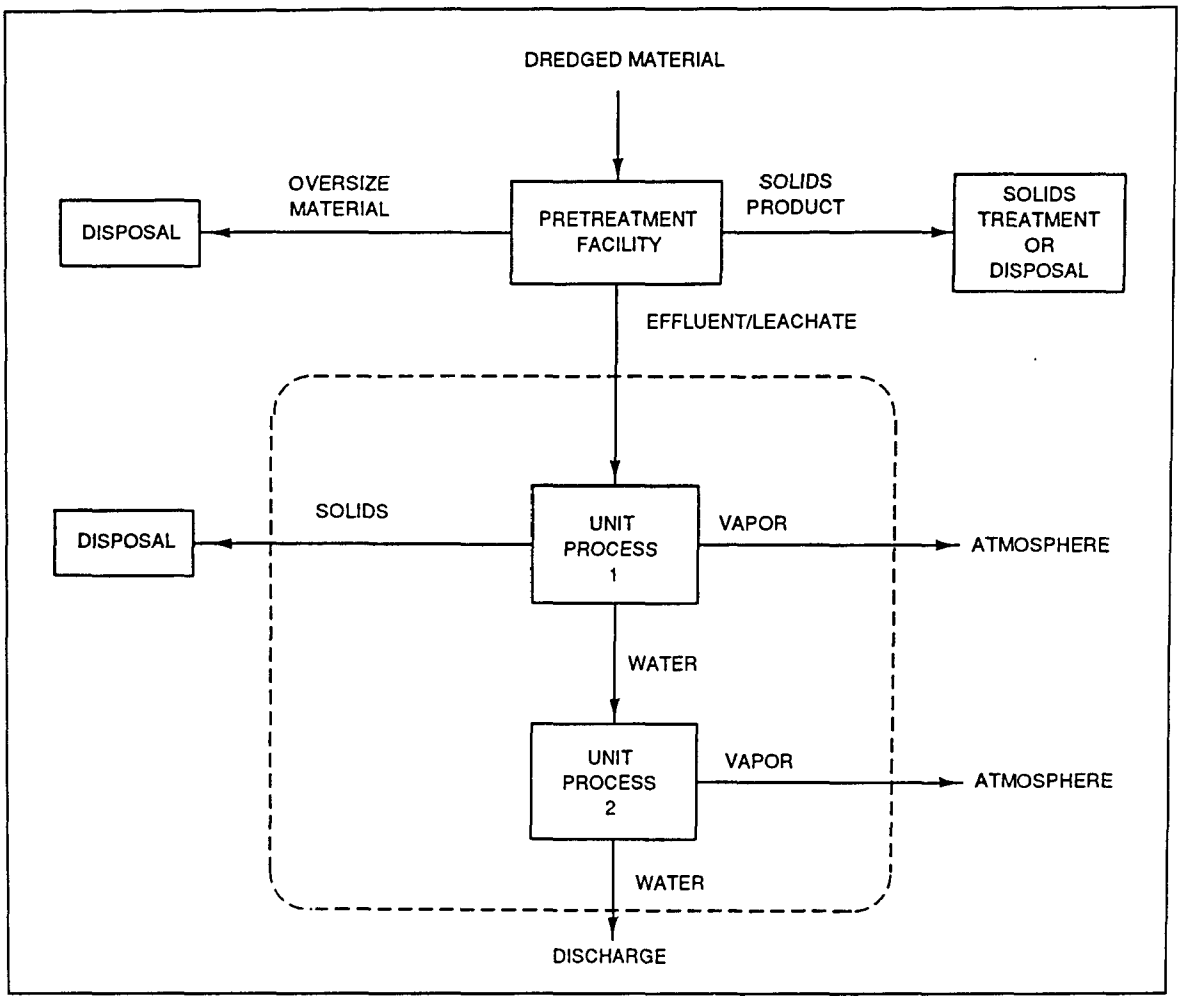


Figure 34. Example effluent/leachate treatment process flowchart

(domestic wastewater, industrial wastewater, synthetic wastewater, etc.), scales of treatment (bench, pilot, and full), and contaminant concentrations (low, medium, and high). Designers and planners should consult the RREL database or other sources for more detailed information on specific treatment performance. In the remainder of this chapter, selected treatment technologies identified by Averett et al. (1990) are briefly examined for process basics and information on treatment efficiencies. These technologies are listed in Table 11. The list of treatment technologies in Table 11 is not exhaustive, and designers of effluent and leachate treatment systems could consider other treatment technologies.

Organic Treatment Technologies

Carbon adsorption

Process description. Carbon adsorption is an effective treatment process for soluble organic compounds, and its use typically follows biological

Table 10
Selected Removal Efficiencies for Aqueous Waste Streams (From RREL Treatability Database (USEPA 1992))

Chemical	Treatment Process	Percent Removed
Arsenic	CHPT	30 - 90+
	FIL	17
	CAC	34 - 92
Cadmium	CHPT	38 - 99+
	FIL	25 - 49
	CAC	0 - 80
Copper	CHPT	27 - 99
	FIL	0 - 75
	CAC	19 - 95
Chromium	CHPT	0 - 99+
	FIL	47 - 90
	CAC	19 - 95
Lead	CHPT	0 - 99+
	FIL	21 - 66
	CAC	39 - 99
Aroclor 1254	API	18
Aroclor 1260	SED	52
Acenaphtnene	CHOX(CL)	48
	CHOX(OZ)	91
	PACT	90
Benzo(ghi)perylene	CHOX(CL)	73
Fluoranthene	CHOX(CL)	8 - 44
	CHOX(OZ)	99+
	PACT	77

Note: Composite information not intended for design calculations.
CHOX(CL): chemical oxidation using chlorine.
CHOX(OZ): chemical oxidation using ozone.
CHPT: chemical precipitation.
FIL: filtration.
CAC: chemically assisted clarification.
API: American Petroleum Institute oil/water separator.
SED: sedimentation.
PACT: powered activated carbon.

Table 11
Process Options for Effluent/Leachate Component Technology Types (From Averett et al. 1990)

Metal Removal	Organic Removal	Suspended-Solids Removal
Flocculation/coagulation	Carbon adsorption	Chemical clarification
Ion exchange	Oil separation	Granular media filtration
Permeable treatment Bed/dikes	Floating skimmers Gravity separation	Membrane microfiltration Constructed Wetlands
Precipitation	Coalescing plate separator	
Coagulation flocculation	Chemical oxidation of organics	
Constructed Wetlands	Ozonation	
	Resin Adsorption	
	Ultraviolet (UV) hydrogen peroxide	
	UV/ozonation	
	Constructed Wetlands	

treatment or granular media filtration. Oil and grease concentrations greater than 10 mg/ℓ in the influent necessitate pretreatment in order to protect the hydraulic and adsorptive capacity of the carbon. Air stripping may be utilized for this, but adds significantly to the overall cost of the treatment process. Oil skimmers and coalescing plate skimmers, discussed later, can be used to remove oil and grease, but may not be effective in reducing oil and grease concentrations to 10 mg/ℓ without the attendant use of de-emulsifying processes. Another alternative is to sacrifice the top layers of the carbon bed. Wastewaters with insoluble oil and grease concentrations as high as 50 mg/ℓ have been successfully treated in this manner. Suspended solids concentrations also influence process efficiency and column life. For fluids with a viscosity near that of water, downflow columns are suitable for influents containing suspended solids at concentrations of up to 65 to 70 mg/ℓ. Upflow columns can handle more viscous fluids, but require suspended solids concentrations less than 50 mg/ℓ (Cullinane et al. 1986). High concentrations of calcium carbonate or calcium sulfate will coat granular activated carbon which cannot then be regenerated. This can be dealt with by pH adjustment or by the addition of a scale inhibitor (Berger 1987).

Carbon adsorption processes will reduce BOD, COD, and TOC in addition to specific organic compounds. In general, carbon adsorption is not as effective for polar organic molecules as it is for nonpolar organic molecules. Nonpolar organics are hydrophobic and as a result have high adsorption potential and low solubilities. Low solubility humic and fulvic acids sorb readily and may exhaust the carbon. Carbon adsorption is reportedly very effective in the removal of PCBs, with tests resulting in levels less than 1 μg/ℓ (Carpenter 1986). Shuckrow, Pajak, and Osheka (1981) report percent reductions of

92.5 to 99.9+ percent reduction for PCBs. For multicontaminant systems, competitive adsorption can reduce the removal rates of some compounds by 50 to 60 percent (Shuckrow, Pajak, and Osheka 1981).

Process waste streams. Process waste streams from carbon adsorption units vary according to unit design. The waste streams common to all carbon adsorption units are spent carbon and process effluent. Other potential waste streams are offgases and backwash waters. Since contaminants are removed by sorption to the carbon, spent carbon is the primary waste stream. Regeneration of spent carbon is usually accomplished thermally and may involve a gas phase contaminant release. Losses associated with the process effluent should by design be acceptable, that is, the treatment unit should meet given performance standards. Performance standards can be met by additional treatment if necessary.

Oil separation

Process description. Oily compounds foul the surfaces of exchangers, sorbents, and filters diminishing process effectiveness and shortening the useful life of the equipment. Oil and grease must be removed prior to ion exchange, carbon adsorption, and filtration. Oil separation can be achieved with continuous or batch processes. Continuous processes such as floating skimmers and coalescing plate separators rely on gravity separation and require very low flow rates to be effective (Corbitt 1989; Averett et al. 1990).

The effectiveness of oil separation methods varies with the nature of the oil in solution, flow rate, temperature, and pH. Gravity separation can potentially be very effective in oil removal if a process train is developed that is appropriate for the characteristics of the fluid. Where the free oil concentration exceeds 1,000 mg/ℓ, a separator must precede coalescing units in order to prevent fouling with excess oil. Oil skimmers can potentially remove 99 percent of free oil at the water surface, provided oil loading rates do not exceed the capacity of the skimmer. Process efficiency will ultimately be determined by the distribution of soluble and emulsified oil and the effectiveness of flocculants and de-emulsifiers.

Process waste streams. Process waste streams for oil and grease removal technologies include removed oil and grease, process effluent, and gasses and vapors. The oil and grease that is removed may contain significant amounts of contaminants such as PCBs. For this reason, the oil and grease stream is usually subjected to further treatment, such as incineration.

Oxidation

Process description. Chemical oxidation is based on the reaction of chemical oxidants with wastewater constituents to transform and degrade contaminants. Oxidants include chlorine, ozone (discussed separately below),

permanganate, peroxide, fluorine, and hypochlorite. Chemical oxidation can be used for treatment of dilute influents containing oxidizable organics. It is not suitable for complex waste streams, due to the nonselectivity of many oxidants. Highly concentrated waste streams require large inputs of oxidizing agents for this reason. Chemical oxidation is also not suitable for highly halogenated organics. Its use has been reported for aldehyde, mercaptans, phenols, benzidine, unsaturated acids, cyanide, certain pesticides, and as a pretreatment to biological treatment for refractory compounds. It has limited application for slurries, tars, and sludges (Kiang and Metry 1982). Incomplete oxidation can occur, with the potential for the formation of toxic intermediate oxidation products.

Process waste streams. Process streams from chemical oxidation units are limited to the process effluent and, in some cases, vapors. Losses associated with the process effluent should by design be acceptable.

Ozonation

Process description. Ozonation is an oxidation process applicable to aqueous streams containing less than 1 percent oxidizable compounds. Many organic compounds and a few inorganic compounds are amenable to treatment with ozone. Ozonation is especially useful for those compounds that are resistant to biological treatment. Ozone is nonselective, oxidizing natural organics as well as contaminants (Averett et al. 1990). Ozonation is not suitable with sludges and solids. As with other types of chemical oxidation, toxic end products sometimes result. Ozone is an aggressive oxidant, acutely toxic and corrosive, requiring special handling, equipment, and safety measures. An incidental benefit to ozonation is the increase of dissolved oxygen.

Process waste streams. Ozone reactors are usually sealed reactors with only the inlet, outlet, and ozone piping present. As such, the only process waste streams for ozonation units is the process effluent. Losses associated with the process effluent should by design be acceptable, that is, the treatment unit should meet given performance standards. Additional treatment is usually not necessary.

UV/hydrogen peroxide and UV/ozone

Process description. Hydrogen peroxide and ozone in combination with ultraviolet (UV) light are effective in oxidizing a wide variety of chemicals. Process efficiency varies with the target chemical(s) and general quality of the water to be treated. Process efficiency is poorest with wastewaters that are highly colored or turbid.

Process waste streams. UV/hydrogen peroxide and UV/ozone oxidation units are usually sealed reactors with only the inlet, outlet, and oxidant addition piping present. As such, the only process waste streams from

UV/hydrogen peroxide and UV/ozone oxidation units is the process effluent. Losses associated with the process effluent should by design be acceptable. Additional treatment is usually not necessary.

Resin adsorption

Resin adsorption is applicable for the removal of color due to organic material and to high levels of dissolved organics (Cullinane et al. 1986). The mechanism of removal is primarily sorption, and organics are inhibitory to the function of ion exchange resins targeting other contaminants such as metals.

Performance data for resin adsorption are limited, and highly variable. Published efficiencies for dilute solutions containing PCB congeners range from approximately 20- to 100-percent removal. Shuckrow, Pajak, and Osheka (1981) reported 99-percent removal of PCBs at 100 $\mu g/\ell$ by Amberlite XAD-2. Other sources indicated similar efficiencies for this resin with PAHs. Data for other resins and solution concentrations were not readily available. Because of performance variability between resins and under different operating conditions, treatability studies are the most reliable method of determining potential efficiency for a particular waste stream.

Process waste streams. Process waste streams from resin adsorption units are similar to those from carbon adsorption units. Major waste streams are spent resin and process effluent. Other potential waste streams that are design and operation dependent are offgases and backwash waters. Since contaminants are removed by sorption to the resin, spent resin is the primary waste stream.

Constructed wetlands

Process description. Constructed pollution abatement wetlands can be designed to retain and degrade many pollutants, including toxic organic chemicals. Natural wetlands also potentially retain and degrade pollutants; but in the context of remediation, discharge of effluent or leachate to a natural wetland is not anticipated. Constructed pollution abatement wetlands have been primarily used in tertiary treatment of municipal wastewaters and for pH adjustment of acid mine drainage (Hammer 1989). The mechanisms of organic contaminant removal include adsorption, biodegradation, accumulation by microbes, and, to a lesser degree, plant uptake.

Process waste streams. Constructed wetlands are open systems with many contaminant migration pathways. They are also extremely complicated systems with many internal mechanisms for contaminant retention and degradation. Loss pathways include volatile emissions, leachate seepage, biotranslocation, and discharged waters. There are virtually no a priori and no laboratory-scale procedures for estimating contaminant losses from constructed wetlands. Mesocosm studies (pilot-scale wetlands) can be

conducted to obtain treatment process data needed for design (Rogers and Dunn 1992; Doyle, Myers, and Adrian 1993) and to estimate losses. Limited information on key wetland features, such as vegetative cover, vegetation type, area flooded, hydraulic retention time, etc., with organic chemical treatment process efficiency is available (Reed 1990; Phillips et al. 1993). Little information is available on the removal of PCBs, PAHs, and similar chemicals in constructed wetlands. A database on wastewater treatment using constructed wetlands (North American Wetlands for Water Quality Treatment Database) is available from USEPA (USEPA 1994c). The database includes 178 sites and 203 separate systems. Most of the treatment information in the database is limited to BOD and nutrients.

Suspended Solids Removal Technologies

Chemical clarification

Process description. Chemical clarification is utilized to enhance gravity separation of suspended solids by the addition of chemicals that cause aggregation of particles in solution. Organic polyelectrolytes are of primary interest as the flocculent for use under the ARCS program. Synthetic flocculants, while more expensive than natural inorganic compounds, require smaller doses to achieve the same treatment level.

Schroeder (1983) conducted studies to verify earlier results obtained in the use of polyelectrolytes and to develop guidelines in the design and operation of chemical clarification facilities for dredged material slurries and supernatant. As a result of these studies, all inorganic flocculants and all nonionic and anionic flocculants were eliminated in preliminary bench-scale tests, leaving 14 polymers that were tested on 0.84-, 1.26-, 1.69-, and 2.11-g/ℓ suspensions (suspended-solids concentrations representative of selected CDF effluents). The more highly cationic and higher molecular weight polymers were most effective in bench-scale tests.

Design of a system to achieve these treatment levels will be highly site and sediment specific. Schroeder (1983) developed laboratory testing procedures to facilitate determination of appropriate mixing intensity and duration, settling time and volume requirements, and polymer dosages. In general, polymer dosages are directly proportional to the turbidity to be treated, and inversely proportional to the amount of mixing. A properly designed and operated system can achieve average effluent suspended-solids concentrations on the order of 50 mg/ℓ under continuous operation. Results may be somewhat variable due to the dynamic nature of the system.

Process waste streams. Several waste streams are possible with chemical clarification systems depending on design. These waste streams include the process effluent, leachate, volatile losses, and solids removed from the secondary settling basin. If the secondary settling basin is designed for storage of solids as well as clarification, then there will be no contaminant losses

associated with removal and treatment/disposal of settled solids. Volatile and leachate losses can be estimated using the techniques described in Chapter 4. Leachate losses can be controlled by lining the settling basins. The process effluent will likely be the major pathway for contaminant loss with most designs, even those that do not include a liner. This loss can be reliably estimated using data from test procedures and design calculations described by Schroeder (1983).

Granular media filtration

Process description. Granular media filtration is a polishing step for water that has been pretreated by settling or chemical clarification. The water may be passed through permeable filter dikes or weirs, filter cells, or package filters. Filter cells and sand-filled weirs are vertical flow filters that can be replaced or regenerated when exhausted. Permeable dikes provide horizontal flow filtration and are nonrenewable once clogged. Package filters typically contain disposable cartridges that can be replaced when the solids loading capacity has been reached.

Granular media for suspended solids removal include fine gravel, sand, anthracite, and coal. Sand-filled weirs can remove 60 to 98 percent of suspended solids, reducing the concentration to 5 to 10 mg/ℓ for initial concentrations up to 1 g/ℓ. Efficiencies up to 90 percent have been achieved for concrete filter cells with sand and carbon filter media (Averett et al. 1990).

Process waste streams. Waste streams from granular media filters include the process effluent, backwash water, spent media, and volatile emissions. Volatile emissions can be estimated using the techniques described in Chapter 4. Design equations developed by Krizek, FitzPatrick, and Atmatzids (1976) can be used for a priori estimation of treatment efficiency for suspended solids and particulate-bound contaminants. For low-maintenance designs not requiring backwashing or media replacement, process water and volatile losses are the two loss pathways of concern. Systems in which the media is not contained in a chamber or vessel, such as porous dikes, may also have a leachate pathway. Systems that require periodic removal of spent media will have losses associated with the ultimate disposition of the spent media.

Membrane microfiltration

Process description. Membrane microfiltration can be effective for suspended solids concentrations of 10 to 5,000 mg/ℓ, with the incidental benefit of particle-associated contaminant removal (Averett 1990).

Process waste streams. Membrane microfiltration units produce two process waste streams, process effluent and the filter cake. The filter cake will probably contain most of the contaminant mass introduced into the unit.

Ultimate disposition of this material (landfilling or further treatment), therefore, is key to evaluating contaminant losses associated with membrane microfiltration. Spent membranes may also have to be considered. The process effluent may contain dissolved chemicals that can be removed by further treatment if necessary.

Constructed wetlands

Process description. Constructed pollution abatement wetlands can be very effective in removing sediment particles. Sediment removal effectiveness depends on sediment load and constructed wetland design and operation. The keys to effective removal are providing hydraulic retention times and quiescent conditions sufficient for settling. Establishment of emergent vegetation also plays an important role. Although the study of sedimentation of wetlands has been somewhat limited, sedimentation has been extensively studied in river, reservoir, and wastewater engineering. The design equations used for detention basins provide a suitable basis for estimating solids losses from wetlands constructed to treat effluent and leachate resulting from dredged material treatment.

Process waste streams. Suspended solids releases through water control structures is the primary mechanism for solids losses in constructed wetlands. Constructed wetlands properly designed to remove suspended solids routinely remove up to 90 percent of the total input (Reed 1990).

Metals Removal Technologies

Precipitation

Process description. Heavy metals can be precipitated from water as sulfides or hydroxides with the addition of lime or sodium sulfide. Flocculants can also be used to enhance agglomeration of precipitate particles and resulting sedimentation. Chemical precipitation is most effective following sedimentation and prior to filtration. Sulfides tend to be less soluble and more stable over a broad pH range than hydroxides. Theoretically, metals can be removed to their minimum solubility concentrations by adjusting the pH according to the behavior of a specific metal ion. Where more than one metal is present, more than one adjustment may have to be made, and a composite pH at which all or several of the metals present approach their minimum solubility is commonly used. Adequate process control can be difficult to achieve in precipitation units if influent flows and concentrations vary widely.

Process waste streams. Process waste streams from precipitation systems are similar to those from flocculation/coagulation systems. These waste streams include process effluent, volatile emissions, and solids removed from clarifiers. If the clarifier is designed for storage of solids as well as clarification, then there will be no contaminant losses associated with removal and

treatment/disposal of settled solids. For systems that involve solids removal from clarifiers, there may be contaminant losses associated with ultimate disposition of precipitated solids. Since precipitation systems are usually fabricated with steel or concrete, leachate is not a contaminant loss pathway for these systems. Volatile losses can be estimated using the techniques described in Chapter 4. Process effluent losses are best estimated from laboratory or pilot treatability studies.

Flocculation/coagulation

Process description. Of the two basic flocculants used to treat dredged material effluent, inorganic compounds and cationic polyelectrolytes are the most promising for freshwater slurries. Cationic, anionic, and nonionic polyelectrolytes are all potentially effective for use with saltwater slurries (Averett et al. 1990). As discussed previously in this chapter, suspended solids removals of 84 to 95 percent were achieved in field trials using polyelectrolyte flocculants. Given that heavy metals tend to associate with fine particles, metals-removal efficiencies are likely to be similar to suspended solids removal efficiencies. Flocculation added following precipitation treatment would remove precipitates formed from the soluble metals fractions as well.

Process waste streams. Several waste streams are possible with flocculation/coagulation systems, depending on design. These waste streams include process effluent, leachate, and volatile losses. In addition, there may be contaminant losses associated with ultimate disposition of solids removed from clarifiers. If the clarifier is designed for storage of solids as well as clarification, then there will be no contaminant losses associated with removal and treatment/disposal of settled solids. Leachate losses will be negligible from fabricated systems using steel or concrete. Earthen basins as clarifiers will have a leachate pathway that can be minimized or eliminated using a liner. Volatile and leachate losses can be estimated using the techniques described in Chapter 4. Process effluent losses are best estimated from laboratory or pilot treatability studies.

Ion exchange

Process description. Of the three major operating modes (fixed-bed concurrent, fixed-bed countercurrent, and continuous countercurrent), the fixed bed countercurrent system is most common (Cullinane et al. 1986). Use of a hydrogen exchange resin facilitates removal of anions, and the hydroxide form facilitates removal of cations. For a mixed waste, resins in series targeting first the organics (polar and nonpolar resins) and then the ionic species (cationic and anionic resins) are effective (Cullinane et al. 1986). Ion exchange is valuable because of the selectivity exhibited by exchange resins (Corbitt 1989). This selectivity varies with ionic strength, the relative concentrations of ions in solution, and to a lesser extent temperature and other factors. Natural ion exchange mediums include clay, zeolites, sulfonated coal, and

peat. Synthetic resins have the advantage of controllable properties and high capacity.

Process waste streams. Process waste streams from ion exchange resins are spent resin and process effluent. Other potential waste streams that are design and operation dependent are offgases and backwash waters. Since contaminants are removed by ion exchange with a resin, spent resin is the primary waste stream. Depending on the ultimate disposition of spent resin, there may be losses associated with disposal of this material. Losses associated with the process effluent should by design be acceptable. These losses can be controlled by additional treatment if necessary. Data from bench- or pilot-scale treatability studies are needed for design and estimation of contaminant losses.

Permeable treatment beds/dikes

Process description. Permeable treatment beds and dikes were previously discussed under suspended solids treatment technologies. Under optimum conditions, filtration through these structures will remove 60 to 98 percent of the suspended solids and sediment-bound contaminants (Cullinane 1986). They may be constructed using limestone, crushed shell, activated carbon, glauconitic green sands (zeolites), or synthetic ion-exchange resins at the core to effect ion exchange or precipitation reactions in addition to simple filtration. Permeable treatment beds and dikes are capable of handling suspended solids concentrations up to 1 g/ℓ (Averett et al. 1990).

Process waste streams. Process waste streams for permeable treatment beds and dikes are the same as previously discussed under suspended solids treatment technologies.

Constructed wetlands

Process description. As previously discussed, constructed pollution abatement wetlands are capable of removing a wide spectrum of waterborne pollutants, including metals. Metals can be immobilized in constructed wetland soils and sediments by biologically mediated reduction-oxidation (redox) and pH reactions. Microbes in constructed wetlands soils and sediments utilize available electron acceptors (oxygen, nitrate, ferric iron, sulfate, manganic manganese, and carbon dioxide) to accomplish electron transfer reactions required for obtaining energy from substrates (Turner and Patrick 1968). In this process, pH is raised or lowered depending on starting conditions to near neutral. Coupling of oxidation-reduction reactions with pH is a chemical thermodynamic requirement for these reactions (Ponnamperuma 1972). Many metals are relatively insoluble at near neutral pH and low redox potential. Aerobic (high redox), acidic wastewaters introduced as subsurface flow to constructed wetlands is neutralized with concomitant reduction in dissolved metals. This basic principle has been effectively used to treat acid mine

drainage at numerous sites (Hammer 1989). To date, however, sufficient data are not available for development of design equations or contaminant loss estimation algorithms.

Process waste streams. Loss pathways include leachate seepage, biotranslocation, and discharged waters. Discharged waters probably represent the major loss pathway for metals. Wetlands constructed to process wastewaters from mining activities vary widely in their metals removal efficiencies (Phillips et al. 1993). Wetlands can be very effective in removing metals (removal efficiencies greater than 90 percent) or can be completely ineffective. Mesocosm studies (pilot-scale wetlands) can be conducted to obtain treatment process data needed for design (Rogers and Dunn 1992; Doyle, Myers, and Adrian 1993) and to estimate losses.

Summary

Treatability data needed for screening candidate treatment processes are in some cases difficult to locate depending on the contaminants and treatment processes of interest. Sources of information include Cullinane et al. (1986), Averett et al. (1990), Corbitt (1989), Berger (1987), USEPA (1993b), and Wastewater Technology Centre (1993). In addition, treatment technology databases are available that provide information on treatment process performance (USEPA 1992; USEPA 1993c; USEPA 1994c). Process treatment efficiencies are usually given in terms of percent of contaminant removed. In some cases, this is a function of initial concentrations of contaminants. From percent removal data, planning level assessments of contaminant losses during effluent and leachate treatment can be made.

Sediment sampling and appropriate bench-scale testing are necessary to determine effluent and leachate characteristics and concentrations of contaminants present. From this information and information on expected flow, candidate treatment processes for effluent and leachate can be evaluated in bench-scale treatability studies. Treatment efficiencies and contaminant concentrations in process streams can be calculated on a case-by-case basis once site-specific treatability data are available.

Treatability studies are considered to be a requisite part of any treatment design activity. The chemical and physical interactions of waste components and treatment processes require careful evaluation for effective implementation of any treatment program. Attention to design and scale-up principles including consideration of process control is a key element in achieving optimum removal efficiencies and minimum contaminant releases.

8 Contaminant Losses for the No-Action Alternative

Background

The no-action alternative, as used in this report, describes an assessment of current contaminant concentrations in sediments at a site and of potential danger that may occur in the future if no remedial action is taken. The assessment assumes that the natural events expected in a water body will be allowed to run their course with no changes made in the water body management plan (no changes in loads, dredging practices, etc.). The no-action alternative also may be referred to as a baseline exposure assessment because it serves as a basis from which to compare all action alternatives. With this baseline, the relative benefits of remediation programs can be compared, and the time required for the system to cleanse itself can be estimated.

Procedures for developing a no-action alternative

The general steps in establishing a no-action alternative are described below. These steps are modified from guidance provided for conducting remedial investigations and feasibility studies under the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) (USEPA 1989).

Step 1. The first step in the assessment of the no-action alternative, or baseline exposure assessment, is identification of potential pathways by which contaminants may migrate from a source to a point of contact that is considered hazardous to humans or to terrestrial or aquatic life. This assessment includes identifying the mechanisms affecting the release of the chemical (e.g., from contaminated sediments) as well as the processes that may affect the environmental transport of the chemical (e.g., via sediment resuspension and food chain uptake). This identification step serves to focus the assessment on critical exposure pathways.

Step 2. Once the source(s) and release mechanisms have been identified for contaminants in surface waters, the no-action exposure assessment then

turns to an analysis of the environmental transport and transformation of the contaminants of concern. This fate analysis considers the potential environmental transport (e.g., surface water and groundwater); transformation (e.g., biodegradation, hydrolysis, and photolysis); and transfer mechanisms (e.g., sorption and volatilization) to provide information on the potential changes in the magnitude and extent of environmental contamination.

Step 3. Next, the actual and potential exposure points for receptors (e.g., humans and aquatic life) are identified. As part of this evaluation, a reasonable maximum exposure scenario should be developed that reflects the type(s) and extent of the exposure that could occur based on the likely or expected use of the site (or surrounding areas) in the future.

Step 4. Information developed in the first three steps then is integrated to produce quantitative and/or qualitative estimates of the expected exposure level(s) from the actual or potential release of contaminants from the site.

Step 5. A final step in the assessment is to establish the uncertainty associated with projections of contaminant fate. This uncertainty assessment may be both qualitative and quantitative.

Mathematical models are commonly used as the integrating tools to provide estimates of the expected exposure level(s) under future conditions. Configuring contaminant fate and transport models to provide predictions requires the projection of environmental and physical/chemical conditions into the future. Because of the persistence of some chemicals, these modeling projections may extend to 30 years or longer.

The uncertainty associated with future projections severely complicates the identification of the reasonable maximum exposure scenario and the use of contaminant transport and fate models in the evaluation of the no-action alternative. Application of mathematical models over the long time periods required for the assessment of persistent chemicals is particularly difficult. Because the response time of some chemicals is on the order of 20 to 100 years, the no-action alternative modeling scenario would have to be simulated for that period of time. This introduces a level of uncertainty on how projections are made. For example, an assessment of flows could be made using the period of record flows for the simulations. However, the historical flow pattern may not be a good estimator of flow conditions for the future. The system could experience a major flood, the equivalent of which was not represented in the period of record. Alternatively, a truly stochastic approach could be used based on the historical distribution of hydrology. However, a completely stochastic approach is usually not feasible unless relatively simple models of contaminant transport and fate are used, due to the computational burden imposed by complex models. Different approaches used in the modeling scenario to evaluate the no-action alternative may produce different estimates of the time-to-recovery or potential exposure levels in the future.

Because most persistent contaminants in aquatic systems are associated with sediments, they are moved or dispersed in association with major sediment resuspension events. Properly accounting for these events is often very important. For example, a 100-year flood event could be responsible for movement of 90 percent of the total contaminants, all in the course of a few days, with the contaminated sediments either being exposed or buried. Trying to account for the effects of large events is difficult. Seldom is there enough information to allow for a complete analysis of these infrequent conditions.

Not only must the flow and sediment transport be predicted for the future, but future land uses and exposure pathways must be considered during the evaluation of the no-action alternative. The determination and the ultimate acceptance of the no-action alternative would be based on the reduction of contaminants in the water body and the subsequent reduction in the associated risk of exposure. This no-action reduction can be used as a comparison of the effectiveness of some proposed remedial action plans, where the calculated risks can be compared with that of the baseline risk assessment.

Levels of study complexity and uncertainty

The level of effort required in the analysis of the environmental fate and transport of contaminants in the no-action assessment depends largely on the complexity of the site. The goal is to gather sufficient information to adequately and accurately characterize the potential exposure from the site, while at the same time conducting the study as efficiently as possible. Factors that may affect the level of effort required include (USEPA 1988): (a) the number, concentration, and types of chemicals present and the areal extent of the contamination, (b) the quantity and quality of available supporting data, (c) the number and complexity of the exposure pathways (including the complexity of release sources and transport media), and (d) the required precision of analyses, which in turn depends on site conditions.

Evaluation of the no-action alternative usually requires the use of hydrodynamic/sediment transport and contaminant transport and fate models. However, the level of complexity of the modeling study may vary for the reasons cited above. There are basically three levels in which a no-action alternative can be conducted:

a. *Screening Level*—A simplified modeling method or analytical equations can be used to give rough estimates of contaminant mobility and concentrations under a set of conditions. This level is useful in addressing broad management questions over long time periods.

b. *Descriptive Modeling*—A contaminant transport and fate model could be set up on the water body using flows derived from historical records and sediment transport derived from sedimentation records. This approach bypasses the use of the hydrodynamic and sediment transport model, but still provides insight in how the contaminant will react over

long periods of time under variable flow and sediment transport conditions.

c. *Fully Predictive Modeling*—A hydrodynamic and sediment transport model could be utilized in predicting the flows and sediment transport for the period of record. This information would then be linked with the fate and transport model.

Most large water bodies in the United States have U.S. Geological Survey gauging stations or National Oceanic and Atmospheric Administration water elevation gauges from which one can obtain measured continuous flow or water surface elevations for the period of record required for the modeling study. This type of information can be used in all three levels of modeling. These data can be analyzed to determine the variability of the water movement, and a probability distribution function (PDF) can be generated. PDFs are used to determine the probability of different flow regimes occurring that might cause major scour events. These major events and their associated probabilities can be incorporated into the no-action alternative modeling study.

Degradation processes are known, or can be estimated reasonably well, for most contaminants. Values for these processes would be entered into the contaminant transport and fate model and used throughout the simulation period. However, the site-specific parameters used to describe the degradation processes are usually determined using data available only over relatively short time periods in comparison to the time over which the no-action alternative will be evaluated.

The determination of the exposure pathway of concern will dictate the spatial and temporal resolution needed in the contaminant transport and fate model. If the only interest is in reduction of downstream loadings, large spatial compartments and temporal information may be adequate.

The processing and averaging of data may affect the conclusions that result from the evaluation of the no-action alternative. To illustrate, downstream contaminant concentrations using mean monthly flow data versus daily flow data are compared in Figure 35. As illustrated, the differences between the calculated downstream concentrations are minimal. However, if the exposure pathway is bioaccumulation of the contaminant in fish, the spatial and temporal prediction to model may be very important. If this information was taken a step further and used in a bioaccumulation study, the arbitrary selection of mean monthly flow data over daily flow data could lead to differences in the predicted contaminant concentration in the biota. Figure 36 shows the predicted contaminant concentrations in fish for a heavy organic-like PCB. Although there is a difference in the predicted fish concentration, the error is not large compared with the predicted exceedance of the U.S. Food and Drug Administration (FDA) action limit. But, in the case of a light organic illustrated by Figure 37, the selection of the flow criteria can have an impact on the remediation decisions for the water body. In this case, the error is significant compared with the predicted exceedance of the FDA action limit.

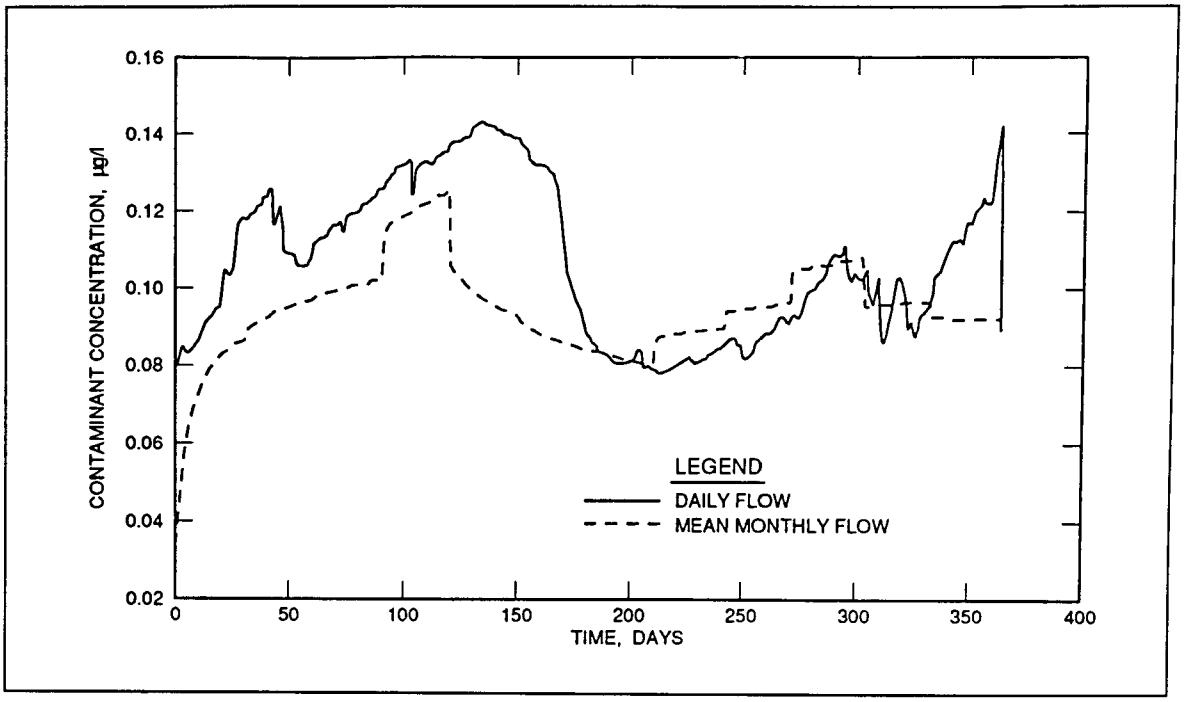


Figure 35. Daily versus mean monthly contaminant concentrations

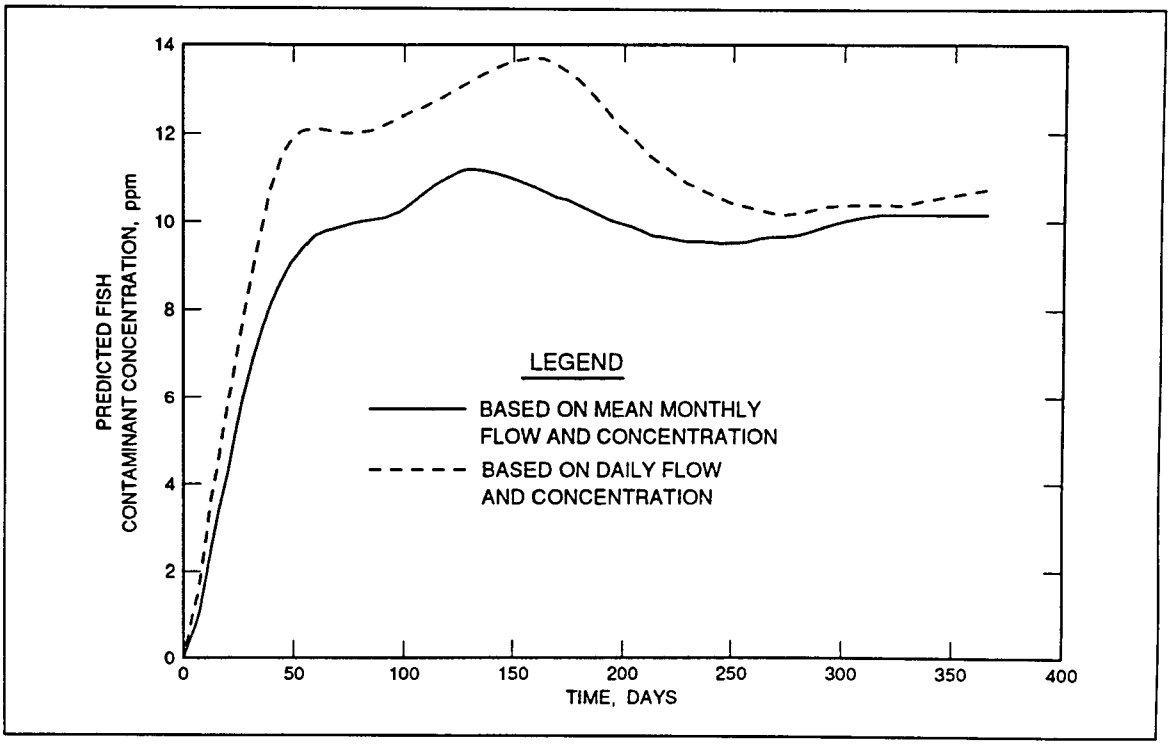


Figure 36. Heavy organic bioaccumulation: mean monthly versus daily flow and contaminant concentrations

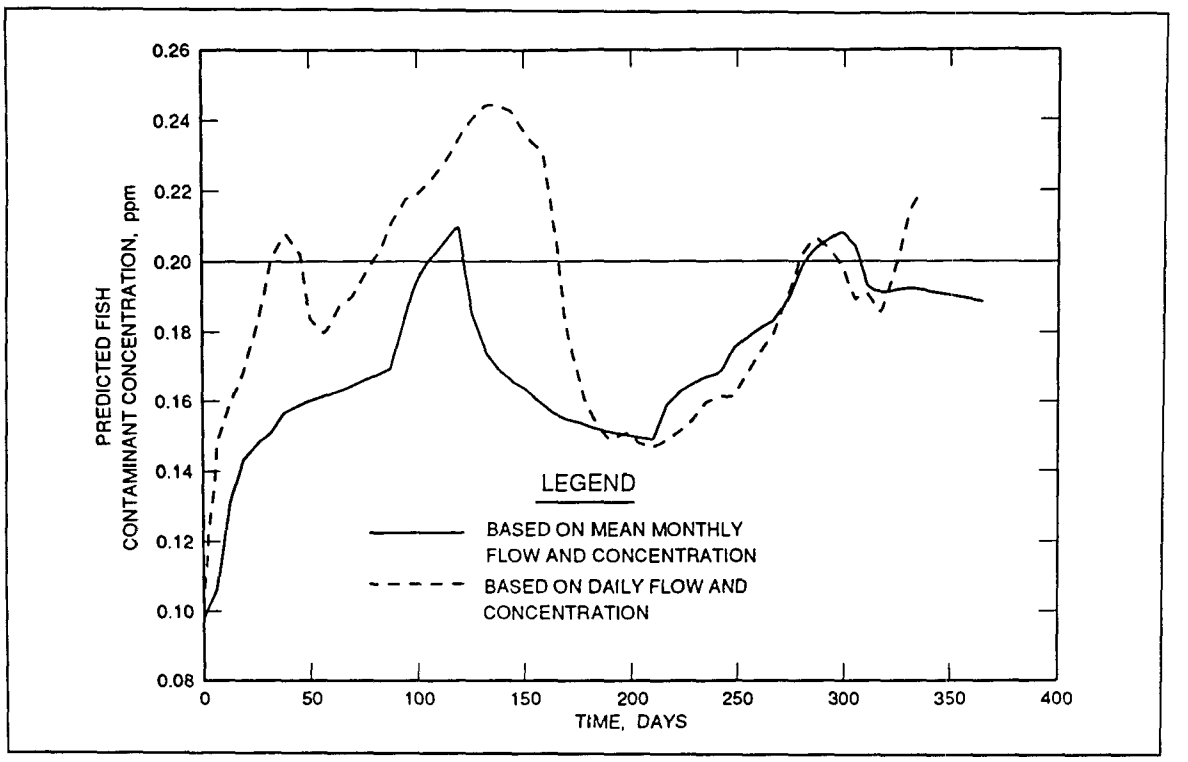


Figure 37. Light organic bioaccumulation: mean monthly versus daily flow and concentration

Modeling the No-Action Alternative

The application of mathematical models to conduct a no-action alternative is a multistep process. The steps for conducting a modeling study are illustrated in Figure 38. Water and sediment transport are first predicted so that this simulated information can be used by the contaminant transport and fate model. Next, the hydrodynamic and sediment transport predictions are used along with the estimates of contaminant loadings due to nonpoint/point source loadings to predict changes in chemical concentrations in water and sediments. This gives time-variable contaminant concentration profiles for sediments and water column that can be utilized by bioaccumulation/food chain models to predict contaminant body burden for fish.

Hydrodynamic models

To effectively predict the dissolved concentration of a contaminant, it is important to characterize the transport of water within the system. The variability and distribution of water column contaminant concentrations can often be largely explained by water transport alone. Water transport models are

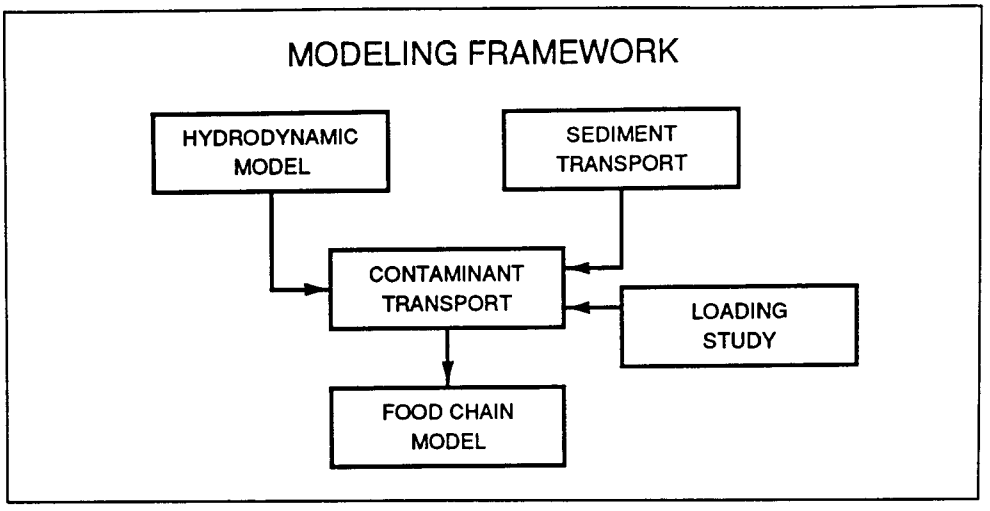


Figure 38. Steps in modeling no-action alternatives

based on a balance of water mass and, for hydrodynamic models, a balance of water momentum, which, like mass, is also a conservative property.

Characterization of water transport may be descriptive or predictive, depending on the modeling approach. In a descriptive approach, flow patterns are measured directly or inferred from measurements. The descriptive approach is often adequate where the system is very simple (hydraulically) or where only long-term, relatively crude estimates of water transport are required.

Hydrodynamic models are used to predict changes in volumes, depths, and velocities in response to changes in upstream flows, downstream flows, water surface elevations, or bottom morphometry. Hydrodynamic models can be used to predict flows for periods where direct measurements are not available. Hydrodynamic models also may be used to estimate changes in flows that may occur under future conditions, such as in evaluating the effects of changes in dredging patterns.

Sediment transport models

Adequately characterizing the movement of sediments is a critical step in the assessment of the no-action alternative. There are two primary goals of the sediment transport component: (a) to predict the movement of the sediments themselves in order to estimate changes that may occur in patterns of erosion, deposition, and transport, and (b) to estimate the transport of the particulate contaminant mass. Sediment transport models are based on a balance of sediment mass. As with water transport, sediment transport may be described or predicted in mass balance studies. The descriptive approach has proven useful in providing crude estimates of the effects of sediment transport on contaminant distributions. However, sediment transport is a very

dynamic process, and the assumption of steady-state solids behavior is a gross simplification.

In predictive sediment transport models, resuspension and transport are computed using the output of a hydrodynamic model and the characteristics of the sediments. The type of sediments of importance in contaminant studies are cohesive sediments (e.g., silts and clays) rather than noncohesive sediments (e.g., sands). The sediment transport model is used to predict changes in suspended solids concentrations, changes in sediment resuspension and deposition, and changes in the structure of the sediment bed. As with hydrodynamic models, sediment transport models can be used to interpolate among existing measurements or to estimate sediment transport for conditions for which data are not available. The majority of sediment transport occurs under extreme (rare) events, such as storms on lakes and large run-off events in rivers. Since data are often not available for these rare events, sediment transport models can be used to estimate transport under these conditions. For example, they may be used to estimate whether contaminated sediments may be buried or exposed under these conditions. This information can be used in the evaluation of remedial actions as well as the no-action alternative. Sediment transport models may also be used to evaluate the impact of removing or immobilizing sediments on subsequent erosion and deposition patterns. For example, if sediments are removed from a particular area, sediment transport models may be used to estimate how long it may take for the area to fill in as well as to predict changes that may occur in deposition and erosion areas. Table 12 suggests several hydrodynamic and sediment transport models that could be implemented in a no-action alternative modeling study. This table is restricted to models that are in the public domain. In addition to the models listed below, a wide variety of models are available in the private sector that may be suitable for use in the evaluation of the no-action alternative.

Table 12
Suggested Hydrodynamic and Sediment Transport Models

Hydrodynamic and Sediment Transport Models					
Name	**Source**	**Dimension**	**Sediment Transport**	**Cohesive Sediments**	**Linked w/WQ Models**
SED-3D	USEPA, Athens	3-D	Y	Y	N
SED-2D	USEPA, Athens	2-D	Y	Y	N
HEC-6	HEC, U.S. Army	1-D	Y	Y	N
RIVMOD	USEPA, Athens	1-D	Y	N	Y
DYNHYD	USEPA, Athens	1-D	N	N	Y
RMA	WES, U.S. Army	2-D	Y	Y	N
CE-QUAL-RIV1	WES, U.S. Army	1-D	N	N	N
DAM BREAK	US NWS	1-D	N	N	N

SED-3D.

General description: SED-3D is a circulation, sediment dispersion, resuspension, and deposition model for far-field transport in lakes, estuaries, coastal areas, and other water bodies. It employs approximate second-order closure scheme (Sheng and Eliason 1991).

Capabilities and strength: SED-3D can be used to simulate water flow and sediment transport in various water bodies under the forcing of winds, tides, freshwater inflows, and density gradients, as influenced by Coriolis acceleration, complex bathymetry, and shoreline geometry. The model can be run in a fully three-dimensional mode, a two-dimensional vertically integrated x-y mode, or a two-dimensional x-z mode. The model contains a free surface. A simplified second-order closure model of turbulent transport is used to compute the vertical eddy viscosity and diffusivity in the three-dimensional equations. The model contains six sediment transport processes—advection, turbulent diffusion, settling/flocculation, deposition, erosion, and bed evolution. It is a "process-based" model rather than a "conceptual," "descriptive," or "phenomenological" model.

Limitations: The model may need long computation times, which results in high computation costs. Detailed data are required for simulations and calibration. The model may have low efficiency when it is applied to a meandering river, as a rectangular domain is used in the model.

SED-2D.

General description: SED-2D is a finite element hydrodynamic, cohesive sediment transport model for vertically averaged estuaries, rivers, and other unstratified water bodies (Hayter 1987).

Capabilities and strengths: This model simulates two-dimensional surface water flow and cohesive sediment transport. The effects of bottom, internal, and surface shear stresses and the Coriolis force are represented in the equations of motion. The following sediment-related properties are calculated: sediment bed structure (bed density and shear strength profiles, bed thickness and elevation), net change in bed elevation over a given interval of time, net vertical mass flux of sediment over an interval of time, average amount of time sediment particles are in suspension, and the downward flux of sediment onto the bed. It can be efficiently applied to water bodies having complex geometries due to the employment of a finite element numerical scheme.

Limitations: SED-2D may not be suitable for long and continuous simulation application due to computation costs. This model has not been completely tested.

HEC-6.

General description: HEC-6 is designed to simulate one-dimensional, steady, gradually varied water and sediment flow problems.

Capabilities and strength: The model can predict long-term trends of scour and deposition in a stream channel. It can be used to predict reservoir sedimentation, degradation of channel bed downstream from a dam, and the influence of dredging activities. Local inflows and outflows of water and sediment from tributaries and/or diversions can be included. It can analyze channel contraction required to either maintain navigation depths or diminish dredging requirements. Its strength is its ability to simulate hydraulic sorting and bed armoring. This is done by sediment transport and scour/degradation computations performed by grain-size fraction.

Limitations: HEC-6 is unable to directly simulate meandering phenomenon, local scouring, bank erosion, and width adjustment. It is not suitable for rapidly changing flow conditions. Equilibrium sediment transport capacity is assumed. Density currents and bed forms are not accounted for.

RIVMOD.

General Description: RIVMOD is an unsteady, hydrodynamic and sediment transport riverine model that describes the longitudinal distributions of flow and sediment concentration in a one-dimensional water body through time (Hosseinipour and Martin 1991).

Capabilities and strength: The model allows prediction of gradually or rapidly varying flows through time and space. It includes time-varying lateral inflows. The sediment transport submodel predicts the transport of sediment through the channel network and the scour/deposition processes as well as bed level variations due to scour or deposition of materials. It can be applied to noncohesive sediments (sand) and/or cohesive (fine) materials.

Limitations: Flows are assumed to be advectively dominant, and the effect of eddy diffusivity is neglected. Water surface slope is assumed to be small. The model in its present form does not include armoring and channel stabilization. The cohesive sediment transport submodel does not account for suspended sediment deposition and resuspension.

DYNHYD.

General description: DYNHYD is a simple link-node hydrodynamic model that simulates variable tidal cycles, wind, and unsteady flows. It produces an output file that can be linked with the contaminant model WASP4 (described below) to supply the flows and volumes to the water quality model (Ambrose et al. 1987).

Capabilities and strength: When linked to WASP4, it simulates the movement and interaction of pollutants within the water. Driven by variable upstream flows and downstream heads, simulations typically proceed at 1- to 5-min intervals. The resulting unsteady hydrodynamics are averaged over large time intervals and stored for later use by the water quality program.

Limitations: No sediment transport simulations.

RMA(SED-2D).

General description: RMA (SED-2D) is developed for sediment problems in rivers, lakes, and estuaries.

Capabilities and strength: This is a two-dimensional, unsteady model. It can compute water surface elevations, current patterns, flow distributions around islands, flow at bridges having one or more relieving openings, flow in contracting and expanding reaches, flow into and off-channel storage for hydropower plants, flow at river junctions, and general flow patterns. The model can be used to compute sediment transport, deposition, and erosion in two-dimensional open channel flows. It is applicable to clay and/or sand bed sediments.

Limitations: Lengthy simulations are not feasible because of computation costs. It is not designed for nearfield problems where flow structure interactions are important. Variations in velocity or constituent concentration with depth are not predicted. Only a single grain-size sediment can be analyzed, and armoring is not addressed.

DAMBREAK.

General description: DAMBREAK is a dam-break flood forecasting model. The model consists of a breach component, which utilizes simple parameters to provide a temporal and geometrical description of the breach (Fred 1988).

Capabilities and strength: This model computes the reservoir outflow hydrograph resulting from the breach via a broad-crested weir flow approximation, which includes effects of submergence from downstream tailwater depths and corrections for approach velocities. Also, the effects of storage depletion and upstream inflows on the computed outflow hydrograph are accounted for through storage routing within the reservoir.

Limitations: No sediment transport simulations.

CE-QUAL-RIV1.

General description: CE-QUAL-RIV1 is a one-dimensional (longitudinal), water quality model for unsteady flows in rivers and streams. Output from the hydrodynamic part is used to drive the water quality model (Environmental Laboratory 1989).

Capabilities and strength: The model allows simulation of a branched river system with multiple hydraulic control structures, such as run-of-the-river dams, waterways, locks and dams, and regulation dams. The model was developed to simulate highly unsteady flows that can occur on regulated streams.

Limitations: No sediment transport simulations.

Contaminant transport models

Mass balance models for contaminants may be employed to estimate potential changes in contaminant concentrations for conditions prior to and after remediation. The mass balance models can be used to predict chemical concentrations in various media (water, sediments, and fish). These estimated concentrations can be used to calculate potential risks over time. The mass balance models vary in their complexity, from simple analytical calculations used to give rough screening level results to fully complex iterative models that can predict the transport and fate of chemicals throughout time.

In the application of the contaminant exposure models, the rate of change in mass (accumulation) is equated to the transport of a contaminant into, out of, and within the system (via water flows or sediment flows for those materials that sorb to sediments), the mass added to the system (via point and nonpoint loadings) minus the outputs and the quantities transformed and degraded within the system (via processes such as volatilization, biodegradation, and photodegradation). The output expected from the contaminant exposure model includes estimated contaminant concentrations in water and sediments (both particulate and dissolved forms) as well as estimates of mass fluxes due to inflows and loadings, outflows, degradation, and transformation processes. Depending on the level of the modeling effort, the transport (via water and sediments) may be described or predicted using hydrodynamic and sediment transport models, which are then coupled with the contaminant model. Table 13 suggests several contaminant transport and fate models that could be utilized in a no-action alternative modeling study.

WASP4.

General description: WASP is a generalized modeling framework for contaminant fate and transport in surface waters. Based on the flexible

Table 13
Suggested Fate and Transport Models

Fate and Transport Models for Organic Chemicals and Metals					
Name	**Source**	**Dimension**	**Solution Technique**	**Sediment Transport**	**Linked w/Hydro.**
WASP4	USEPA, Athens	1,2,3-D	Time Variable	Y	Y
EXAMS II	USEPA, Athens	1,2,3-D	Steady State	N	N
SMPTOX3	USEPA, Athens	1-D	Analytical	N	N

compartment modeling approach, WASP can be applied in one, two, or three dimensions. WASP4 predicts dissolved and sorbed chemical concentrations in the bed and overlying waters (Ambrose et al. 1987).

Capabilities and strength: This model is time variable and can simulate three chemicals and three sediment size fractions simultaneously. The model contains descriptive sediment resuspension/settling algorithms that allow for the modeling of sediment transport. The model provides linkages to hydrodynamic models that provide changing flows and volumes to WASP on a time-step-to-time-step fashion.

Limitations: The model does not have the kinetics for simulating metals and oily wastes, although metals can be simulated descriptively using empirical distribution coefficients.

EXAMS II.

General description: EXAMS is a generalized modeling framework based on the WASP4 transport system for contaminant fate and transport in surface waters. Based on the flexible compartment modeling approach, it can be applied in one, two, or three dimensions. EXAMS predicts dissolved and sorbed chemical concentrations in the bed and overlying waters (Burns and Cline 1985).

Capabilities and strength: This model can run in a steady state or a quasi-dynamic mode, three chemicals simultaneously. It is effective for doing rapid evaluations of contaminant fate and transport. The model executes in both an interactive and batch mode.

Limitations: The model is difficult to apply to a specific site. The model does not simulate solids settling and resuspension.

SMPTOX3.

General description: SMPTOX3 is a simplified analytical steady-state model that can calculate the distribution of contaminants in water and sediments.

Capabilities and strength: The model requires very few data to calculate the distribution of chemicals. The model uses travel times and calculates total chemical, sorbed chemical, and dissolved chemical. The model has interactive data entry and graphical simulation results. The model can be used for conducting screening-level calculations.

Limitations: The model is an analytical steady-state model with rudimentary sediment/benthos algorithms.

Food chain models

A food chain model is a mass balance model for contaminants where the rate of change in mass (accumulation) in each component of the food chain is equated to the transport of a contaminant into and out of that component (via ingestion, gill exchange, excretion, etc.) as well as internal changes that may occur due to growth (dilution). The food chain model enables one to assess the impact of remedial actions on contaminant concentrations within the food chain, given variations in concentrations derived from the contaminant exposure model. Outputs from a food chain model include time-varying estimates of contaminant concentrations in each component of the food chain (Suarez et al. 1986).

Summary

There are no fixed set of procedures for conducting No Action modeling exercises. The approach that is taken is site specific and requires various scenarios to be investigated and compared with the future exposure scenarios projected for the site.

9 Dredged Material Treatment

Dredged material that is contaminated to the extent that it requires decontamination or detoxification in order to meet environmental cleanup goals may be treated by one or more of a number of physical, chemical, or biological treatment options. Treatment technologies reduce contamination levels, contaminant mobility, or toxicity for the dredged material by one of four ways:

a. Destroying the contaminants or converting the contaminants to less toxic forms.

b. Separating or extracting the contaminants from the sediment solids.

c. Reducing the volume of contaminated material by separation of cleaner sediment particles from particles with greater affinity for the contaminants.

d. Physically and/or chemically stabilizing the contaminants in the dredged material so that the contaminants are fixed to the solids and are resistant to contaminant losses by leaching, erosion, volatilization, bioaccumulation, or other environmental pathways.

Destruction technologies include incineration, vitrification, chemical treatment, and biological treatment. Separation or extraction technologies include solvent extraction, soil washing, and thermal desorption. Particle separation technologies include hydrocyclones, classifiers, flotation, and screens. Stabilization or immobilization technologies include a variety of solidification techniques such as addition of lime and fly ash or addition of Portland cement to create a solid product without free water. A comprehensive discussion of process options for various treatment technologies is provided in Averett et al. (1990). Guidance on the selection and implementation of sediment treatment alternatives is available in the "ARCS Remediation Guidance Document" (USEPA 1994a).

Other components are always involved for remediation alternatives that involve treatment. Sediment is usually removed from the bottom of the waterway by dredging, transported to the disposal site, and conditioned for treatment and/or temporarily stored in a pretreatment facility prior to treatment.

Treatment processes generate solid and liquid residue, as well as air emissions. These streams may be subjected to further treatment or disposal. Estimation of contaminant losses from the steps leading up to treatment and from disposal have been discussed in previous chapters, and contaminant losses from liquid effluents were discussed previously. Most treatment processes include treatment of air emissions as an integral unit operation of the process with the treated gas stream being released to the atmosphere. For treatment processes, fractions of the contaminant in the feed to the process may end up in the following compartments:

a. Fraction destroyed or detoxified within the treatment process.

b. Fraction remaining in the treated dredged material.

c. Fraction released to the atmosphere.

d. Fraction associated with dilute liquid effluents.

e. Fraction associated with concentrated liquid effluents.

f. Fraction associated with solids enriched with contaminants due to separation of cleaner solids or adsorption media.

All treatment technologies do not generate all of the compartments listed above.

Contaminant Loss Pathways From Sediment Treatment Trains

Figure 39 illustrates the potential contaminant release points from sediment treatment processes. Not all treatment processes generate all of the air emissions and effluents shown in Figure 39. The components of the treatment train for a specific type of technology will dictate which pathways or compartments are important for that particular technology. The one component common to all treatment processes is the solids disposal block for the treated sediment and other solid residuals. These materials will generally be sent to a disposal site and are subject to the same pathways as disposal of untreated sediment. However, the contaminant levels in the treated sediment will be considerably reduced compared with untreated sediment, and the contaminant loads from treated sediment in a disposal site would be expected to show a corresponding reduction. Disposal pathway testing is recommended for the treated sediment to estimate the magnitude of these releases. Concentrated contaminant streams are usually transported to a hazardous waste treatment facility for destruction or disposal. These facilities are presumed to have best available treatment and state-of-the-art controls; therefore, contaminant losses from this phase of the treatment operation will be assumed to be minimal.

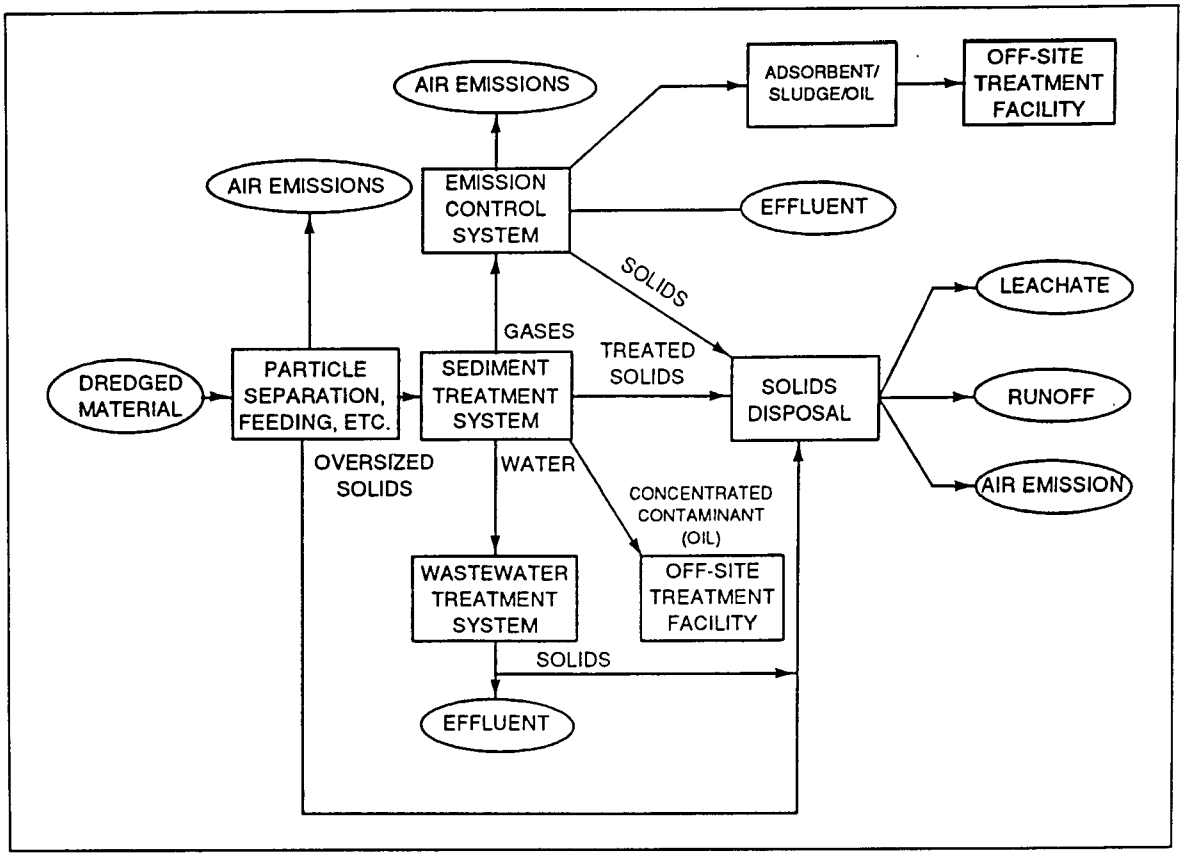


Figure 39. Contaminant losses from sediment treatment process trains

The applicability and importance of other emissions and effluents shown on Figure 39 for several technologies are discussed in the paragraphs that follow.

Thermal destruction

The most common type of thermal destruction technology is incineration, which has been demonstrated to be highly effective in destroying organic contaminants in soils and sediments. The process basically involves heating the sediment to temperatures ranging from 1200 to 2900 °F[1] in the presence of oxygen to burn or oxidize the organic compounds in the sediment. Most incinerators include a primary and a secondary combustion chamber. The primary chamber partially destroys the contaminants and volatilizes the remainder of the contaminants from the sediment, which are further oxidized in the secondary chamber. The sediment's residue after incineration is a dry ash or, for some innovative incineration processes, a dense slag or a glass-like product. The gases from the combustion chamber pass through an emission control system, which usually consists of a scrubber system, prior to release to the atmosphere from the stack. The stack emissions are the contaminant

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: C = (5/9) (F-32).

loss pathway of most concern for incineration systems. Environmental regulations may require destruction and removal efficiencies (DRE) of 99.9999 percent for incineration of organic contaminants. The DRE is calculated as the fraction of the contaminant mass fed to the incinerator that is released from the stack. It usually does not include the residual contaminant in the treated ash, nor does it include the wastewater and solids generated by the scrubber system. Residual organics in the ash and the scrubber releases are expected to be much less than 1 percent. However, volatile heavy metals, such as mercury and lead, may be volatilized in the combustion chambers and be released in the flue gas or concentrated in the scrubber wastewater. The wastewater and ash may receive further treatment to remove the contaminants or reduce contaminant mobility; whereas, the flue gas is released to the environment and constitutes a contaminant loss.

Thermal desorption

Thermal desorption physically separates volatile and semivolatile contaminants from sediment by heating the sediment to temperatures ranging from 200 to 1000 °F, usually in an inert atmosphere. Water, organic contaminants, and some volatile metals are evaporated from the sediment solids and are subsequently captured or destroyed by the emission control system. Condensation, scrubbing, adsorption, incineration, and particulate control processes are typical emission control system operations. Dust generated during the drying process is captured by cyclones or particulate filters. Potential contaminant loss streams include air from the emission control system stack, condensed oils and organic contaminants, condensed water or scrubber water, collected dust, the residue after treatment, and contaminated activated carbon or other adsorption media. Except for the stack gases, these streams will be subjected to further treatment or disposal practices. The oil will be sent to a hazardous waste treatment facility, the wastewater streams will be treated, probably onsite, and the particulates and residues will likely be placed in a disposal facility, either a landfill or a CDF. Since thermal desorption provides no treatment for heavy metals with the exception of mercury, metals in the solid residue will be a potential source for leachate contamination in the disposal site. Handling and transport of the dry, powdery residue will require control measures to minimize losses of contaminants as dust.

Biological treatment

Biological treatment processes use microbes to degrade or transform organic contaminants to less toxic or nontoxic compounds. Process options for bioremediation of sediments include bioslurry reactors, land-treatment systems, composting, and contained treatment facilities. Bioslurry systems produce a treated residue, air emissions, and wastewater. The other types of biotreatment systems generate a treated residue and may potentially generate air emissions and leachate. Wastewater, or leachate collection and treatment, and emission controls for bioslurry, contained land treatment, and composting

systems would likely be part of the treatment train. Emission estimates for land-treatment systems and for contained-treatment facilities without emission controls could be made using techniques described for CDFs.

Extraction processes

Extraction processes are nondestructive processes that generally separate contaminated sediment into solids, water, and an oily fraction containing the contaminants extracted from the sediment. A number of different solvents may be used for the extraction, including water with surfactants (soil washing), acetone, methanol, kerosene, triethylamine, and supercritical propane or carbon dioxide. Most of the solvents are recovered and recycled into the process. Potential contaminant losses for extraction processes are the wastewater separated by the processes, the contaminant-rich oil, and the solids residue. The wastewater would likely be treated onsite, and the oil phase would be sent to a hazardous waste treatment and disposal facility. Most of these processes can be closed to the atmosphere and do not have a positive gas release to the atmosphere. Extraction processes for organics usually do not affect heavy metals, and the metals remain with the residual solids. Removal of heavy metals may be accomplished by a separate extraction train using an acid or a chelating agent as the solvent. This train would require a concentration step for the heavy metals, which would have to be handled and disposed as a contaminated material.

Chemical processes

Chemical processes are destructive processes that use reagents, temperature, or pressure to drive a chemical reaction with the contaminants converting them to environmentally acceptable materials. A major class of chemical processes for sediment are the dechlorination processes for chlorinated hydrocarbons such as PCBs. These processes generally operate in a closed environment with a minimal release to the atmosphere. Wastewater will be generated by the process and will likely be treated onsite. The residual sediment will contain traces of the organic contaminants and most of the heavy metals originally present in the sediment. Other chemical processes that involve gas-phase reactions produce a stack emission that should be considered as a contaminant loss stream.

Immobilization processes

Immobilization processes alter a sediment's physical and/or chemical characteristics to reduce the potential for contaminants to be released from sediment when placed in a disposal site. Once placed in the disposal site, similar techniques as used for confined disposal may be used to estimate losses, particularly by leachates. Air emissions during the process of mixing sediment with reagents or binders are likely, particularly when the solidification

reaction generates heat, such as for Portland cement and pozzolan processes. Several years ago, solidification was reported to destroy PCBs, but later investigations proved that the PCBs were volatilized during processing rather than being changed by the process. One of the advantages of solidification processes is that they may virtually eliminate the effluent pathway and minimize the leachate pathway, since free water in the sediment is usually absorbed by the binder or becomes a part of a hydrated product.

Particle separation processes

Particle separation processes are usually considered as pretreatment processes rather than treatment processes, particularly where the objective is to remove oversized material from the sediments to avoid interference with subsequent processing steps. However, they also offer treatment advantages by separation of the clean fraction of a sediment from the more contaminated sediment fraction in order to reduce the volume of material requiring more costly treatment. Many commonly used options are available from the mining and materials processing industries. Likely choices for sediments are hydrocyclones, screens, classifiers, and froth flotation. Most of these operations process sediment as a slurry; therefore, a wastewater discharge or effluent will be produced by the process. With or without treatment, contaminant will be released with this effluent. Also, air emissions are possible due to the agitation created by most of these processes. If volatilization proves to be an important loss, the processing units may be housed and the emissions collected and treated. Furthermore, the "clean" fraction will not be contaminant-free and will represent a potential loss of organic and inorganic contaminants at the disposal site. The contaminant-rich fraction will be subjected to other treatment processes and the losses during these processes must be considered.

Techniques for Estimating Contaminant Losses During Treatment

The wide range of chemical and physical characteristics for contaminated sediment, the strong affinity of most contaminants for fine-grain sediment particles, and limited application of treatment technologies to contaminated sediment offer challenges to development of estimating or modeling techniques to estimate contaminant losses for various contaminant and treatment technology combinations. Basic mathematical models are likely available for simple process operations, such as extraction or thermal vaporization, applied to single contaminants in relatively pure systems. However, such models have not been validated for the sediment treatment technologies discussed here because of the limited database for evaluation of treatment technologies for contaminated sediment or soils and because of the wide range of sediment physical and chemical characteristics that impact treatment processes. Development of models for specific treatment technologies is beyond the scope of this study.

Standard engineering practice for evaluation of the effectiveness of treatment technologies for any type of contaminated media (solids, liquids, or gases) is to perform a treatability study for a sample that is representative of the contaminated material. In a management review of the Superfund Program, the U.S. Environmental Protection Agency (USEPA 1989) concluded "To evaluate the application of treatment technologies to particular sites, it is essential to conduct laboratory or pilot-scale tests on actual wastes from the site, including, if needed and feasible, tests of actual operating units prior to remedy selection." The performance data generated by treatability studies will usually provide the contaminant concentrations for the residual sediment following treatment. Contaminant concentrations and weights for side streams generated by a technology can also be determined from treatability studies, but the need for this information must be clearly identified as one of the objectives of the treatability study so that appropriate data will be collected. Treatability studies may be performed at bench-scale and/or pilot-scale level. Features of each of these treatability study types are discussed below.

Bench-scale treatability studies

Bench-scale studies simulate the basic operation of a treatment process, but are performed in a laboratory using a small volume (1 to 20 ℓ) of sediment. Individual operational parameters, such as chemical dosages, temperatures, or retention times, and variable waste characteristics can be evaluated for a number of different conditions. Bench-scale tests generally use laboratory glassware and carefully controlled conditions. The weights of solid or slurry and liquid streams can be accurately measured, which can be coupled with contaminant concentrations for each stream to provide a mass balance around the process for contaminants of concern. Side streams that include solid and liquid phases should be separated and each phase quantified to provide information needed to estimate the effectiveness of effluent treatment processes. One of the limitations of bench-scale testing is that the volumes of side streams generated may be too small for contaminant analysis at low concentrations. Gaseous emissions are more difficult to collect and measure, and air pollution control processes are more difficult to emulate in the laboratory in conjunction with the solids treatment processes. Other limitations of bench-scale studies include the volumes of the side streams produced may be insufficient to evaluate follow-on treatment technologies, and associated contaminant losses for the side streams, and contaminant losses for pretreatment and materials handling processes are difficult to evaluate.

Pilot-scale treatability studies

Pilot-scale treatability studies are performed using significantly larger volumes of sediment and using equipment that is similar to prototype processing equipment but reduced in scale. Pilot tests are of sufficient scale to minimize the physical and geometric effects of the test equipment on treatment performance and simulate effects such as mixing, wall effects, generation of

residues, heat transfer, or other factors in performance of the process. Information on performance, design, and cost are much improved over bench-scale tests. USEPA (1989) stated "Pilot-scale testing produces the most accurate data on residuals generation, cross-media impacts, and treatment train requirements." Contaminant controls and losses can be evaluated for the primary unit operation and for auxiliary unit operations used to control side streams produced by the process, including gas streams and materials handling operations. Pilot studies can be planned to provide a mass balance for contaminants of concern around the process train, thereby providing the information to predict contaminant losses. Pilot studies are much more expensive to perform, and are generally executed after selection of a technology for a particular site based on technology screening and bench-scale testing.

Important contaminant loss components for treatability testing

Table 14 summarizes the important components of the treatment technologies discussed previously that should be evaluated during treatability study testing in order to estimate contaminant losses. The ARCS program performed this type of testing for a number of process options, and the reader is referred to the reports for these tests for detailed information on the relative magnitudes of each of the components for each type of technology. As was stated earlier, these processes and treatability studies for these processes are strongly influenced by sediment chemical and physical characteristics. Generalization of the magnitude of these components into a table of guidance values can be misleading without complete information on how the treatability study was performed and complete laboratory data.

Table 14
Important Contaminant Loss Components for Treatment Technologies

Contaminant Loss Stream	Treatment Technology Type						
	Biological	Chemical	Extraction	Thermal Desorption	Thermal Destruction	Immobilization[1]	Particle Separation
Residual Solids	X	X	X	X	X	X	X
Wastewater	X	X	X	X			X
Oil/Organics			X	X			X
Leachate						X	
Stack gas				X	X		
Adsorption Media			X	X			
Scrubber water					X		
Particulates (Filter/ cyclone)				X	X		

[1] Immobilization is a special case for contaminant loss estimates in that its primary objective is to reduce leaching of contaminants from the sediment. Long-term contaminant losses must be estimated using leaching tests and contaminant transport modeling similar to that used for sediment placed in a CDF. Leaching could be important for residual solids for other processes as well.

10 Example Application to Contaminated Sediments in the Buffalo River

Introduction

The remedial project discussed in this section is provided only for discussion and illustration purposes—the loss calculations are "paper" exercises. No actual field implementation is endorsed nor has occurred as a consequence of this report.

This section describes example contaminant release calculations for a selected area of concern. The calculations illustrate the types of site-specific engineering assumptions that are required for implementation of the estimation techniques described in previous parts of this report. Example calculations are provided for losses from the following remediation components and remediation alternatives: sediment removal (dredging), in situ capping (non-removal remediation alternative), disposal without treatment in a CDF, and treatment by thermal desorption.

Depending on the remediation component or alternative, various types of results are obtained including concentrations, fluxes, and mass release rates. In each case, however, the results are reduced to one common denominator—contaminant mass loss per cubic meter of sediment remediated. Contaminant loss estimates were normalized with respect to the volume of sediment for remediation to facilitate comparison of losses among remediation components and alternatives. To put loss estimates on a common basis, judgment is needed about applicable time scales for analysis. Judgment about which contaminant loss mechanisms to include and a priori treatment process effectiveness also affects loss comparisons.

Most of the calculations were implemented on commercially available mathematical software (MATHCAD Version 4.0, Mathsoft, Inc., Cambridge, MA) that allows the user to present equations as if they were written on engineering paper. In one case, public domain software (the Hydrologic Evaluation of Landfill Performance computer model) was used to estimate leachate

seepage from an upland CDF. Preparation of this report did not involve computer model development, and no code was written to implement any of the estimation techniques. Readers are directed to the fact that a single computer code is not available for implementation of the various estimation techniques described in this report. Commercially available mathematical software is preferred by the authors over commercially available spreadsheet type software because of problems with confirming if user-developed spreadsheet algorithms are error free. With commercially available mathematical software, the user is freed from the tedious task of checking cell addresses, consistency of cell addressing, mysterious numbers in cells (unit conversion factors), and the sequential logic behind extensive calculation suites.

Site Description

The Buffalo River area of concern was selected for this effort. This area is shown in Figure 40. Two locations within the river were considered for demonstration of calculating the contaminant losses from the hypothetical implementation of remedial technologies, Dead Man's Creek and the Mobil Oil area. Contamination in the Dead Man's Creek area includes PAHs in the upper 50 to 100 cm of sediment. The volume of contaminated sediment at this site is approximately 10,000 yd^3. The Mobil Oil site includes about 40,000 yd^3 of contaminated sediment, again in the upper 50 to 100 cm.

The sediments in both areas are composed of silts and clays with some sands. Sediment samples from both sites also contain approximately 2 percent organic carbon, which can sorb PAHs. A summary of sediment properties at these sites is found in Figure 41.

Chemical analyses of the sediment from each site were used to identify target contaminants to be used in the analysis and their concentrations. Four PAHs (anthracene, benzoanthracene, benzopyrene, and phenanthrene) were selected as indicator contaminants. Figure 41 lists two levels of contamination, an average and a high level. Average concentrations were determined by simple averaging within the contaminated sediment regions. High concentration levels were determined by the average plus twice the standard deviation among the samples. The high concentration levels would represent the 95-percent confidence limit if the sample concentrations were distributed normally. The data, however, were not normally distributed with respect to concentration. Although the data were not normally distributed, the high level concentrations calculated were about the same as the highest observed concentrations. Because the concentrations at Dead Man's Creek were slightly higher than those at the Mobil Oil site and the sediments were otherwise similar, example contaminant loss release calculations focus on estimating contaminant losses from the Dead Man's Creek site. All other conditions being identical, the more concentrated sediment would be expected to result in higher contaminant loss rates.

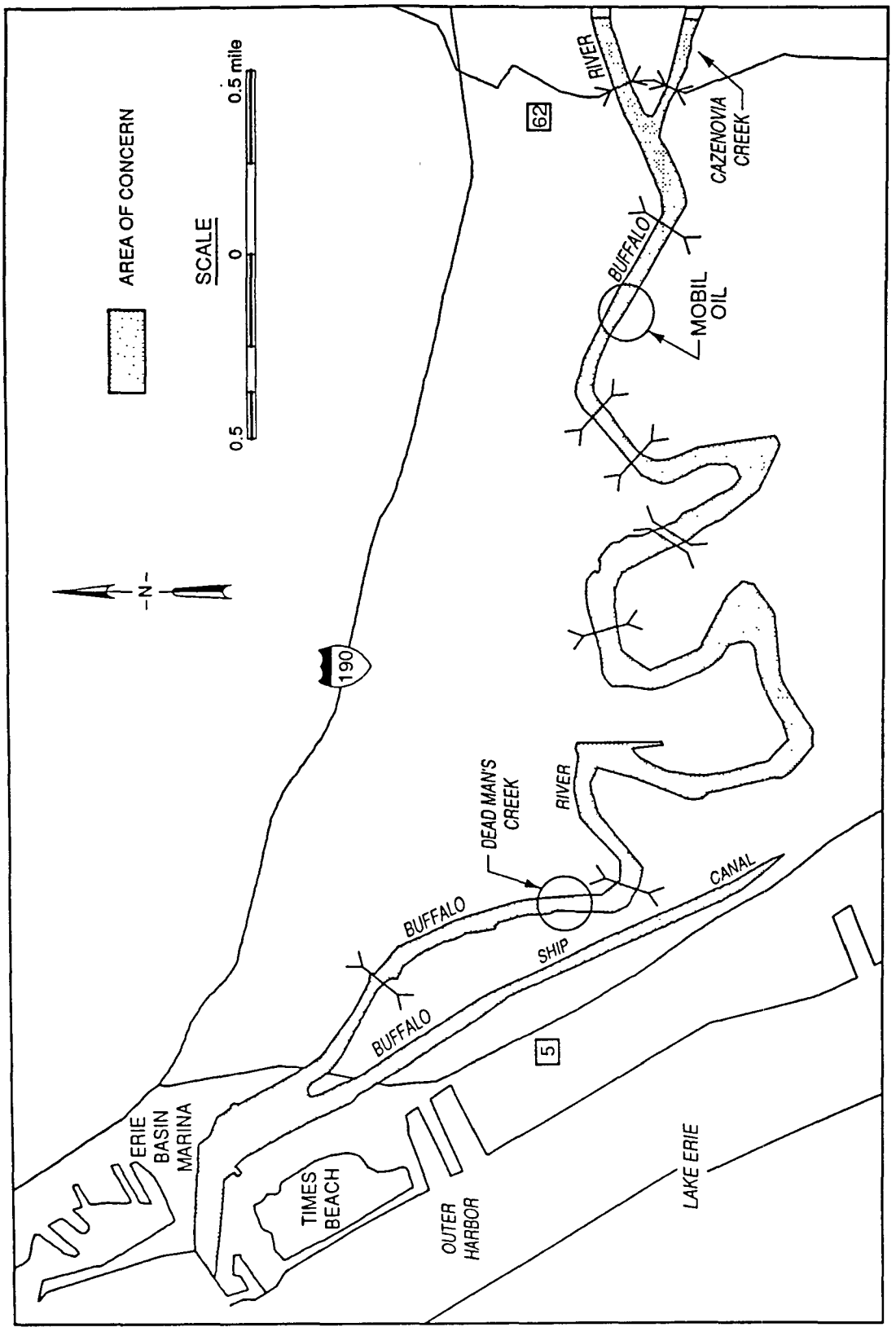


Figure 40. Buffalo River site map

SITE 1: DEAD MAN'S CREEK

Volume: 10,000 cu yd
Sediment: silty clay, 40% porosity, bulk density 1.5 g/cm^3, mean particle size 0.022 mm, 2% organic carbon

CONTAMINANT	LOW LEVEL (MEAN)	HIGH (MEAN+2σ)
Anthracene	860 μg/kg	2990 μg/kg
Benzoanthracene	1150 μg/kg	4450 μg/kg
Benzopyrene	770 μg/kg	2760 μg/kg
Phenanthrene	1780 μg/kg	5930 μg/kg

SITE 2: MOBIL OIL

Volume: 40,000 cu yd
Sediment: silty clay, 43% porosity, bulk density 1.4 g/cm^3, mean particle size 0.02 mm, 2.4% organic carbon

CONTAMINANT	LOW LEVEL (MEAN)	HIGH (MEAN + 2σ)
Anthracene	800 μg/kg	2200 μg/kg
Benzoanthracene	540 μg/kg	1800 μg/kg
Benzopyrene	340 μg/kg	780 μg/kg
Phenanthrene	1420 μg/kg	3790 μg/kg

Figure 41. Sediments and contaminants in Buffalo River AOC

In addition to the sediment and contaminant properties identified in Figure 41, river conditions influence contaminant losses during certain remedial activities such as capping in place. The median discharge in the Buffalo River has been estimated at 300 cfs (27.9 m^3/sec). A rating curve has been developed for various locations in the river. Dead Man's Creek is located about 2.8 km from the lake discharge of the river and the rating curves estimated at 2.4 km from the lake are[1]

$$Depth(m) \qquad h = 0.00258\ Q + 7.2 \tag{77}$$

$$Velocity(\frac{m}{sec}) \qquad v = \frac{1}{\left[0.4089 + \frac{1141}{Q}\right]} \tag{78}$$

where Q is flow (m^3/sec). These rating curves are valid for the region near Dead Man's Creek for river discharges up to about six times the median flow. The width of the river is about 82 m at this location. River conditions described by Equations 77 and 78 were used to estimate losses associated with in situ capping.

[1] Personal Communication, 1992, U.S. Army Engineer District, Buffalo, Buffalo, NY.

Mean and high-sediment contamination levels for Dead Man's Creek were selectively used in the contaminant-loss calculations that follow. Mean and high-sediment contamination levels were used to estimate losses during dredging to illustrate the range in loss estimates that can be obtained using real site characterization data as input. High sediment contamination levels were used to estimate losses for in situ capping. Using the high sediment contamination levels to estimate losses for in situ capping is a worst case scenario since surficial sediments are more recent and often times cleaner. Mean sediment contamination levels were used for estimating effluent and effluent treatment, leachate, and volatile losses from pretreatment and disposal facilities because dredging and dredged material placement/disposal tends to mix sediments. Mean sediment contamination levels were also used to estimate losses for thermal desorption processing of dredged material for the same reasons. Comparison of the alternatives was based on losses calculated using mean sediment contamination levels except for in situ capping.

Contaminant Losses During Dredging

Contaminant losses during dredging were estimated for both clamshell (mechanical) dredging and cutterhead (hydraulic) dredging of the Dead Man's Creek site.

Clamshell dredge

Calculations for contaminant losses during clamshell dredging are presented in Figures 42-45. Sediment parameters were either measured or estimated from the available data. A key parameter in the evaluation of contaminant losses during dredging is settling velocity, which was estimated from the mean grain size using Stoke's Law. This law is valid for dilute suspensions of uniform grain-size particles and a Reynold's number less than 1 (negligible inertial effects). Given the range of grain sizes in typical sediments, a measured settling velocity would be preferred.

A 10-yd^3 open clamshell bucket was assumed. A closed bucket could be used. Barnard (1978) estimated that closed buckets reduce turbidity by 30 to 70 percent compared with open buckets. Applicability of correction factors based on Barnard (1978) to the Collins (1989) equations for sediment resuspension, however, has not been demonstrated. Advancements in closed bucket technology that are currently available are not represented in either the Barnard (1978) or Collins (1989) data. In a remediation project, the most technologically advanced and cost-effective closed bucket would be preferred over a conventional open bucket. The techniques presented and illustrated in this report for conventional open buckets can be used to prepare loss estimates for comparison with vendor-supplied information on currently available closed-bucket technology.

Sediment parameters

Median grain diameter	$d = 0.022 \cdot mm$	
Particle density	$\rho_s = 2.65 \cdot \frac{gm}{cm^3}$	
Water density	$\rho_w = 1 \cdot \frac{gm}{cm^3}$	
Water viscosity	$\mu = 1.31 \cdot 10^{-2} \cdot poise$	
Settling velocity	$V_3 = d^2 \cdot g \cdot \frac{\rho_s - \rho_w}{18 \cdot \mu}$	$V_3 = 0.001 \cdot \frac{ft}{sec}$
Reynold's Number (N_{Re}<1, required)	$N_{Re} = \frac{V_3 \cdot d \cdot \rho_w}{\mu}$	$N_{Re} = 0.006$

Clamshell parameters

Bucket volume	$V_{cb} = 10 \cdot yd^3$	
Characteristic length of bucket	$L_{cb} = (2 \cdot V_{cb})^{\frac{1}{3}}$	$L_{cb} = 8.143 \cdot ft$
Clamshell cycle time	$\tau_{cb} = 120 \cdot sec$	
Dredged material production rate	$W = \frac{V_{cb}}{\tau_{cb}}$	$W = 300 \cdot \frac{yd^3}{hr}$
Minimum dredging time $V_{cont} \equiv 10000 \cdot yd^3$	$\frac{V_{cont}}{W} = 33.333 \cdot hr$	
Resuspended sediment conc. (near bucket) (Eq 10 of Text)	$C_p = 0.0023 \cdot 10^{-6} \cdot \rho_w \cdot \left(\frac{L_{cb}}{\tau_{cb} \cdot V_3}\right)^3$ $C_p = 555.526 \cdot \frac{gm}{m^3}$	

Figure 42. Clamshell dredge losses: sediment and dredge properties

Clamshell Operation

Fraction of cycle in various modes (not used in this analysis)

Falling = 40%

Out of water = 10%

Rising = 40%

Dredging depth $h_b = 20 \cdot ft$

Bohlen sweep area correction $\gamma = 4$

Calculation of sediment release

Particle resuspension rate (Eq 11 of Text) $R_p = \gamma \cdot L_{cb}^2 \cdot \frac{h_b}{\tau_{cb}} \cdot C_p$ $R_p = 695.43 \cdot \frac{gm}{sec}$

Normalized resuspension rate $\frac{R_p}{W} = 10.915 \cdot \frac{kg}{m^3}$

Nakai (1978) observed resuspension rate of 11.9 to 89 kg/m3 from bucket dredging

Figure 43. Clamshell dredge losses: resuspension calculations

4 contaminants	Avg Conc.	High Conc.	$i = 1..4$
1- Anthracene	$C_{s_1} = 860 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_1 = 2990 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
2- Benzoanthracene	$C_{s_2} = 1150 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_2 = 4450 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
3- Benzopyrene	$C_{s_3} = 770 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_3 = 2760 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
4- Phenanthrene	$C_{s_4} = 1780 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_4 = 5930 \cdot 10^{-6} \cdot \frac{gm}{kg}$	

Mean release rate $R_{M_i} = R_p \cdot C_{s_i}$ High release rate $R_{H_i} = R_p \cdot H_i$

Contaminant i	Mean Contaminant Release R_{M_i} $\left(\frac{gm}{hr}\right)$	High Contaminant Release R_{H_i} $\left(\frac{gm}{hr}\right)$	
1	2.15	7.49	<-- Anthracene
2	2.88	11.14	<-- Benzoanthracene
3	1.93	6.91	<-- Benzopyrene
4	4.46	14.85	<-- Phenanthrene

Figure 44. Clamshell dredge losses: contaminant release

Normalized contaminant loss in milligrams per cubic meter dredged.

$$N_{M_i} = \frac{R_p}{W} \cdot C_{s_i} \qquad N_{H_i} = \frac{R_p}{W} \cdot H_i$$

Normalized

Contaminant i	Mean Contaminant Loss N_{M_i} $\left(\frac{mg}{m^3}\right)$	High Contaminant Loss N_{H_i} $\left(\frac{mg}{m^3}\right)$	
1	9.4	32.6	<-- Anthracene
2	12.6	48.6	<-- Benzoanthracene
3	8.4	30.1	<-- Benzopyrene
4	19.4	64.7	<-- Phenanthrene

Figure 45. Clamshell dredge losses: normalized contaminant loss

The characteristic length of the bucket (8.14 ft)[1] was estimated by assuming that the bucket was triangular in shape. The cycle time from collection of sediment, raising the bucket, depositing the dredged material, and returning the bucket to the riverbed was assumed to be 120 sec. Resuspended sediment concentration near the bucket was estimated using Equation 10. Resuspended sediment concentration in the water immediately surrounding the clamshell bucket was estimated to be about 560 g/m^3 or 560 mg/ℓ. This concentration would fall off rapidly with distance from the clamshell due to dilution.

The depth of dredging was assumed to be 20 ft, river depth near Dead Man's Creek contaminated sediment area. The Bohlen sweep area correction factor (typically 2-3) was chosen to be 2. The particle resuspension rate was estimated (Equation 11) to be about 695 g/sec. Dividing the resuspension rate by the dredge production rate provides an estimate of 10.9 kg of resuspended sediment per cubic meter of sediment dredged. The resuspension estimate for

[1] A table of factors for converting non-SI units of measurement to SI units is presented on page xiii.

Dead Man's Creek is near the lower end of the range reported by Nakai (1978) for mechanical dredges.

The contaminant release rate (Figure 43) was estimated as the product of the sediment resuspension rate and contaminant concentrations in the sediment (Equation 12). Contaminant release rates were estimated for both the mean and high-level concentrations. The mean contaminant release rate ranged from 1.9 g/hr of benzopyrene to 4.6 mg/hr of phenanthrene. The high concentration release rate ranged from 6.9 g/hr of benzopyrene to 14.8 mg/hr for phenanthrene. Benzopyrene was the contaminant with the lowest concentration in Dead Man's Creek sediment, and phenanthrene was the contaminant with the highest concentration in Dead Man's Creek sediment.

Dredging losses normalized with respect to the volume of sediment dredged and were obtained as the product of normalized resuspension and sediment contaminant concentrations (Figure 44). Normalized losses for mean contamination levels were 8.4 mg/m^3 to 19.4 mg/m^3 for benzopyrene and phenanthrene, respectively, and for high contamination levels, normalized losses were 30 mg/m^3 to 65 mg/m^3 for benzopyrene and phenanthrene, respectively.

Cutterhead dredge

Calculations for contaminant losses during cutterhead dredging are presented in Figures 46-49. A cutterhead dredge with a cutterhead measuring 2.5 ft long and 3 ft high was selected. The intake suction velocity, cutterhead swing velocity, and cutterhead tangential velocity (rotational velocity) were selected to be 0.625, 1.25, and 5 ft/sec, respectively. The fractional depth of cut was selected to be 0.5. The production rate of the dredge of 371 yd^3/hr was estimated assuming that the suction velocity acted over the entire area of the cutterhead and that the dredged material was 25-percent dry solids. Operational parameters are listed in Figure 46.

Estimation of the sediment resuspension rate followed that outlined in Contaminant Losses During Dredging, specifically Equations 3 and 6. The various coefficients (α, β, F_D,F_F, etc.) were estimated and are shown in Figures 46 and 47. The estimated resuspended sediment concentration in the vicinity of the cutterhead (Figure 47) was about 8 g/m^3. This corresponds to a resuspension rate of about 18 g/sec, or, normalized with the estimated production rate, about 0.234 kg/m^3. This is again in the lower range of the resuspension rates from cutterhead dredges observed by Nakai (1978). Contaminant mass resuspension rates (Figure 48) were between 0.05 and 9.3 g/hr for mean sediment contaminant concentrations and between 0.18 and 99 g/hr for the high concentration sediment. Benzopyrene had the lowest release rate of the contaminants examined and phenanthrene had the highest. Normalized contaminant mass losses (Figure 49) were between 0.36 mg/m^3 and 0.83 mg/m^3 for mean sediment contamination and between 1.3 mg/m^3 and 2.8 mg/m^3 for high-sediment contamination.

Sediment parameters		
Median grain diameter	$d = 0.022 \cdot mm$	
Water density	$\rho_w = 1 \frac{gm}{cm^3}$	
Cutterhead parameters		
Length of cutterhead	$L_{ch} = 2.5 \cdot ft$	
Height of cutterhead	$H_{ch} = 3 \cdot ft$	
Cutterhead characteristic size	$D_{ch} = 2 \cdot \left(L_{ch} \cdot \frac{H_{ch}^2}{4} \right)^{\frac{1}{3}}$	$D_{ch} = 3.557 \cdot ft$
Cutterhead size factors	$\alpha = 1.75$	$\beta = 1.25$
Intake suction velocity	$V_i = 0.625 \cdot \frac{ft}{sec}$	
Ladder swing velocity	$V_s = 1.25 \cdot \frac{ft}{sec}$	
Blade velocity	$V_c = 5 \cdot \frac{ft}{sec}$	
Fractional depth of cut	$D_F = 0.5$	
Dredged material production rate (assuming 25% solids)	$W = V_s \cdot L_{ch} \cdot D_{ch} \cdot \frac{D_F}{4}$	$W = 185.255 \cdot \frac{yd^3}{hr}$
Minimum dredging time	$V_{cont} = 10000 \cdot yd^3$	$\frac{V_{cont}}{W} = 53.98 \cdot hr$

Figure 46. Cutterhead dredge losses: sediment and dredge parameters

Normalized mass loss estimates suggest that losses during cutterhead dredging are less than 3 percent of the losses during clamshell dredging. In general, contaminant release during cutterhead dredging is expected to be less than during clamshell dredging.

Calculation of sediment release

Empirical velocity significance factors: $a = 2.85$ $b = 1.02$

Burial coefficient (Eq 5 of Text)

$$F_D = 1 + \left[1.9 \cdot (D_F - 1)^2\right] + 0.41 \cdot (D_F - 1)^7$$

$$F_D = 1.472$$

Other factors coefficient (Eq 4 of Text)

$$F_F = \left[10^{\left[10^{\left[\left(\frac{D_{ch} \cdot 10^{-4}}{d \cdot 13.32}\right)^{7.04}\right] - 2.05}\right]}\right]$$

$$F_F = 0.09$$

Resuspended sediment conc. (Near cutterhead) (Eq 3 of Text)

$$C_p = \rho_w \cdot 10^{-6} \cdot F_F \cdot F_D \cdot \left(\frac{V_s}{V_i}\right)^a \cdot \left(\frac{V_c}{V_i}\right)^b$$

$$C_p = 7.926 \cdot \frac{gm}{m^3}$$

Resuspension rate

$$R_p = C_p \cdot V_c \cdot \alpha \cdot H_{ch} \cdot \beta \cdot L_{ch} \qquad R_p = 18.41 \cdot \frac{gm}{sec}$$

Normalized resuspension rate

$$\frac{R_p}{W} = 0.468 \cdot \frac{kg}{m^3}$$

Nakai (1978) observed sediment releases between 0.1 and 45.2 kg/m^3 for hydraulic cutterhead dredges

Figure 47. Cutterhead dredge losses: resuspension calculations

Contaminant Losses During In Situ Capping

An alternative to dredging and treatment or disposal of contaminated sediment is capping in place with a clean layer of sediment. In situ capping isolates contaminants from benthic organisms and the water column, significantly reducing ecological impacts and allowing time for natural processes to remediate contaminated sediment. In this example, times required for contaminants to break through the cap, times to steady-state flux through the cap, steady-state fluxes, and losses over the first 100 years normalized with respect to the volume of sediment capped were estimated. The breakthrough time is the time for the flux through the cap to reach 5 percent of the steady-state flux while the steady-state time was arbitrarily selected as the time required to reach 95 percent of the steady-state flux. The cap is assumed to be stable,

4 contaminants	Avg Conc.	High Conc.	$i = 1..4$
1- Anthracene	$C_{s_1} = 860 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_1 = 2990 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
2- Benzoanthracene	$C_{s_2} = 1150 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_2 = 4450 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
3- Benzopyrene	$C_{s_3} = 770\ 10^{-6} \cdot \frac{gm}{kg}$	$H_3 = 2760 \cdot 10^{-6} \cdot \frac{gm}{kg}$	
4- Phenanthrene	$C_{s_4} = 1780 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_4 = 5930 \cdot 10^{-6} \cdot \frac{gm}{kg}$	

Mean release rate $R_{M_i} = R_p \cdot C_{s_i}$ High release rate $R_{H_i} = R_p \cdot H_i$

Contaminant i	Mean Contaminant Release R_{M_i} $\left(\frac{gm}{hr}\right)$	High Contaminant Release R_{H_i} $\left(\frac{gm}{hr}\right)$	
1	0.057	0.198	<-- Anthracene
2	0.076	0.295	<-- Benzoanthracene
3	0.051	0.183	<-- Benzopyrene
4	0.118	0.393	<-- Phenanthrene

Figure 48. Cutterhead dredge losses: contaminant release

and contaminant transport through the cap is assumed to occur by diffusion, retarded by sorption in the capping layer, and facilitated by natural organic colloidal matter. Mass transfer processes driven by bioturbation were estimated to be sufficiently fast that the capped zone populated by benthic animals posed no effective mass transfer resistance. The benthic bioturbation mass transfer coefficient and overlying water conditions are listed in Figure 50.

The calculations focus on diffusion-controlled losses. After the loss calculations for a cap with diffusion-controlled mass transfer are presented, loss estimates are provided for advection-dominated mass transfer. The purpose of

Normalized contaminant loss in milligrams per cubic meter dredged.

$$N_{M_i} = \frac{R_p}{W} \cdot C_{s_i} \qquad N_{H_i} = \frac{R_p}{W} \cdot H_i$$

Contaminant i	Mean Contaminant Loss N_{M_i} $\left(\frac{mg}{m^3}\right)$	High Contaminant Loss N_{H_i} $\left(\frac{mg}{m^3}\right)$	
1	0.402	1.4	<-- Anthracene
2	0.538	2.08	<-- Benzoanthracene
3	0.36	1.29	<-- Benzopyrene
4	0.833	2.77	<-- Phenanthrene

Figure 49. Cutterhead dredge losses: normalized contaminant loss

the advection-dominated loss calculations is to compare diffusion-controlled and advection-dominated losses and show that if diffusion controls, capping can be a very effective remediation alternative.

Calculations were made for a cap with an effective depth of 50 cm. Effective depth is the actual depth of the cap minus the depth bioturbed by benthic organisms. Properties of the cap (Figure 51) were assumed identical to the properties of the underlying sediment (Dead Man's Creek). Although contaminant-specific diffusivities are available or can be estimated, chemical diffusivities in water do not vary widely and are all about 5×10^{-6} cm^2/sec. Diffusivities of the contaminants in water were, therefore, assumed to be 5×10^{-6} cm^2/sec. In the cap, this diffusivity is modified by porosity and tortuosity (Equation 48, Figure 51). Contaminant partitioning and reaction input parameters are listed in Figure 52. The calculations were arranged to include biodegradation by providing a characteristic reaction time, the compound half-life. For the calculation summary shown in Figure 53, the compound half-lives were assumed long enough (1 million to 100 million years) such that no significant reaction occurred over the time of the calculations. Although these half-lives may be too high to properly represent biodegradation, loss estimates based on these half-lives will be conservative, that is, losses to the overlying water column will be overestimated.

$\mu g \equiv 10^{-6} \cdot gm \quad \eta g \equiv 10^{9} \cdot gm$

Water column properties (Assuming 2000 cfs flow)

Current speed $\quad v = 0.002122 \cdot 2000^{0.8626} \cdot \frac{m}{sec} \quad v = 1.49 \cdot \frac{m}{sec}$

General contaminant/water properties

Diffusivity $\quad D_w = 5 \cdot 10^{-6} \cdot \frac{cm^2}{sec}$

Kinematic viscosity (Water) $\quad \nu = 910 \cdot 10^{-5} \cdot \frac{cm^2}{sec}$

Schmidt Number $\quad Sc = \frac{\nu}{D_w} \quad Sc = 1820$

Volume/Depth of contaminated sediment $\quad V_{cont} = 10000 \cdot yd^3 \quad d_{cont} = 0.5 \cdot yd$

Contaminated sediment area $\quad A = \frac{V_{cont}}{d_{cont}}$

Benthic bl m-t coefficient (Turbulent boundary layer) $\quad K_b = 0.036 \cdot \frac{D_w}{\sqrt{A}} \cdot \left(\frac{v \cdot \sqrt{A}}{\nu} \right)^{0.8} \cdot Sc^{\frac{1}{3}}$

$K_b = 2.81 \cdot \frac{cm}{hr}$

Figure 50. Contaminant losses for in situ capping: water/contaminant properties

Cap pore waters were assumed to contain natural organic colloidal material at a uniform concentration of 25 mg/ℓ (Figure 51). This colloidal material can sorb contaminants, effectively increasing their "solubility" in the pore water. This factor ($1 + K_{oc}C_{doc} = 1 + K_dC_{doc}/f_{oc}$) was incorporated into the estimation of the pore water concentration (Figure 54). For transient calculations, that is for the calculation of the breakthrough and steady-state times and the transient flux-steady-state flux quotient, effective diffusivity was retarded by sorption onto the immobile sediment phase. A retardation coefficient (R) was defined that represents the total concentration of contaminant in the system to the concentration in the water phase (Figure 53). This retardation

Sediment/cap properties

Bulk density	$\rho_b := 1.5 \cdot \frac{gm}{cm^3}$	
Porosity	$\varepsilon := 0.4$	
Dissolved organic carbon	$C_{doc} := 25 \cdot \frac{mg}{liter}$	
Bioturbed layer depth	$L_{bio} := 10 \cdot cm$	
Bioturbed layer diffusivity	$D_{bio} := 10 \cdot \frac{cm^2}{yr}$	
Cap thickness (Total thickness - bioturbed depth)	$L_{cap} := 50 \cdot cm$	
Cap fraction organic carbon	$f_{oc} := 0.02$	
Cap effective diffusion coefficient	$D_{eff} := D_w \cdot \varepsilon^{\frac{4}{3}}$	$D_{eff} = 1.47 \cdot 10^{-6} \cdot \frac{cm^2}{sec}$

Contamination Levels

4 contaminants	Avg Conc.	High Conc.
$i := 1..4$		
1- Anthracene	$C_{s_1} := 860 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_1 := 2990 \cdot 10^{-6} \cdot \frac{gm}{kg}$
2- Benzoanthracene	$C_{s_2} := 1150 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_2 := 4450 \cdot 10^{-6} \cdot \frac{gm}{kg}$
3- Benzopyrene	$C_{s_3} := 770 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_3 := 2760 \cdot 10^{-6} \cdot \frac{gm}{kg}$
4- Phenanthrene	$C_{s_4} := 1780 \cdot 10^{-6} \cdot \frac{gm}{kg}$	$H_4 := 5930 \cdot 10^{-6} \cdot \frac{gm}{kg}$

Figure 51. Contaminant losses for in situ capping: sediment/cap/contaminant properties

coefficient was adjusted by the factor $1 + K_d C_{doc}/f_{oc}$ to account for facilitated transport by colloidal organic material.

Dissolved contaminant concentrations, which define the concentration difference in the determination of the contaminant flux, was estimated by assuming the pore water was in equilibrium with the sediment. If the sediment is above the critical loading, the predicted dissolved concentration could exceed the solubility of the contaminant in water. The critical loading is the sediment concentration at which equilibrium concentrations calculated using

Contaminant Properties

4 contaminants	Solubility	Exchangeable	Half-life[1]	K_d in cap
1- Anthracene	$S_1 = 0.045 \cdot \frac{mg}{liter}$	$E_1 = 1.0$	$\tau_1 = 10^6 \cdot yr$	$K_{d_1} = 10^{4.27} \cdot \frac{liter}{kg} \cdot f_{oc}$
2- Benzoanthracene	$S_2 = 0.01 \cdot \frac{mg}{liter}$	$E_2 = 1.0$	$\tau_2 = 10^8 \cdot yr$	$K_{d_2} = 10^{6.14} \cdot \frac{liter}{kg} \cdot f_{oc}$
3- Benzopyrene	$S_3 = 0.004 \cdot \frac{mg}{liter}$	$E_3 = 1.0$	$\tau_3 = 10^8 \cdot yr$	$K_{d_3} = 10^{6.0} \cdot \frac{liter}{kg} \cdot f_{oc}$
4- Phenanthrene	$S_4 = 1.0 \cdot \frac{mg}{liter}$	$E_4 = 1.0$	$\tau_4 = 10^6 \cdot yr$	$K_{d_4} = 10^{3.72} \cdot \frac{liter}{kg} \cdot f_{oc}$

[1] Half-life of 10^6 years represents essentially no reaction

Figure 52. Contaminant losses for in situ capping: contaminant partitions and reaction

partitioning approach the solubility limit. The calculations were constructed to limit dissolved concentrations to solubility limits (Figure 54).

The time to breakthrough calculations (Figure 53) showed estimated breakthrough times between 165 years (phenanthrene) and 1,373 years (benzoanthracene). Steady-state would not be reached for 1,127 years in the case of phenanthrene and for 9,384 years for benzoanthracene.

Contaminant fluxes under steady-state conditions were estimated assuming that the concentration in the sediment remained constant, that is, loss due to reaction and diffusion over the time required to reach steady state were neglected. Predicted dissolved contaminant concentrations, pore water contaminant concentrations (including that sorbed to colloidal matter), and the resulting steady-state fluxes are presented in Figure 54. The effective mass transfer coefficient under steady-state conditions is essentially the effective diffusion coefficient in the cap divided by its effective depth. Bioturbation and benthic boundary mass transfer resistances can be neglected as indicated in Figure 54 if the overall cap design provides for bioturbation and benthic boundary layers above the chemical isolation layer. Steady-state flux for phenanthrene was the highest of the four PAHs at about 0.6 mg $m^{-2}year^{-1}$.

Retardation factor defined to account for colloidal transport

$R = (\text{epilson } R_f)/(1+K_{oc}C_{doc})$

where R_f is as defined in Eq 57 of Text

C_{doc} = dissolved organic carbon conceentration

K_{oc} = organic carbon partitioning coefficient

epilson is as defined in EQ 56 of Text

$$R_f = 1 + \rho_b \cdot K_d + \frac{K_d}{f_{oc}} \cdot C_{doc} \qquad \overrightarrow{R = \frac{\varepsilon \cdot R_f}{1 + \frac{K_d}{f_{oc}} \cdot C_{doc}}}$$

Breakthrough time (5% of steady flux)

$$\tau_b = 0.54 \cdot L_{cap}^2 \cdot \frac{R}{D_{eff}\pi^2}$$

Steady state time (95% of steady flux)

$$\tau_{ss} = 3.69 \cdot L_{cap}^2 \cdot \frac{R}{D_{eff}\pi^2}$$

Fraction of compound remaining after reaction at breakthrough

$$\overrightarrow{f_{rxn} = e^{-0.69 \cdot \frac{\tau_b}{\tau}}}$$

Results

Contaminant i	f_{rxn_i} (1= no reaction)	τ_{b_i} / yr	τ_{ss_i} / yr	
1	1	450	3073	< Anthracene
2	1	1373	9384	< Benzoanthracene
3	1	1359	9285	< Benzopyrene
4	1	165	1127	< Phenanthrene

Figure 53. Contaminant losses for in situ capping: calculation of tansient times

Contaminant loss estimates based on steady-state fluxes are unrealistic for time frames significantly less than the time required to reach steady state. The fraction of the steady-state flux occurring at times less than the steady-state time was used to estimate fluxes over a time period of 100 years. Equation 72 provides the transient flux-steady-state flux quotient as a function

Overall mass transfer coeff:

$$K_{ov} = \overrightarrow{\left(\frac{L_{cap}}{D_{eff}} + \frac{1}{R}\cdot\frac{L_{bio}}{D_{bio}} + \frac{1}{K_b}\right)^{-1}}$$

Note virtually all of the resistance to mass transfer is in the cap, e.g.

$$K_{ov_1} = 0.92\cdot\frac{cm}{yr} \qquad \left(\frac{L_{cap}}{D_{eff}}\right)^{-1} = 0.93\cdot\frac{cm}{yr}$$

Calculated contaminated zone dissolved pore-water concentration cannot exceed solubility.

$$C_{w_i} := \text{if}\left[\left(\overrightarrow{\left(\frac{\rho_b\cdot H}{R_f} > S\right)}\right)_i, S_i, \left(\overrightarrow{\frac{\rho_b\cdot H}{R_f}}\right)_i\right]$$

Pore water concentration includes colloidally-sorbed contaminant.

$$C_{pw} = \overrightarrow{\left[C_w\cdot\left(1 + \frac{K_d}{f_{oc}}\cdot C_{doc}\right)\right]}$$

Steady-state flux through cap

$$Flux_{ss} = \overrightarrow{\left(K_{ov}\cdot C_{pw}\right)}$$

Dissolved and pore water (dissolved plus colloidal) concentrations and steady-state fluxes.

Contaminant i	C_{w_i} $\left(\frac{\mu g}{liter}\right)$	$\left(C_{pw_i}\right)$ $\left(\frac{\mu g}{liter}\right)$	$Flux_{ss_i}$ $\left(\frac{mg}{m^2\cdot yr}\right)$	
1	8.01	11.74	0.108	< Anthracene
2	0.16	5.72	0.053	< Benzoanthracene
3	0.14	3.58	0.033	< Benzopyrene
4	56.09	63.45	0.581	< Phenanthrene

Figure 54. Contaminant losses for in situ capping: steady-state flux—high concentrations

of time. Figure 55 shows the calculation setup for the first 100 years following cap placement. Caution should be exercised when using Equation 72 as indicated in Figure 55. The infinite series in Equation 72 is unstable for times significantly less than the breakthrough time. This is indicated in two of the four graphs in Figure 55. Anthracene and phenanthrene curves behave as

Ratio of the flux at some time t to the steady-state flux is calculated using equation 72. Define the quotient given in Eq 72 of the Text as the Greek letter PHI, PHI = $R_A(t) / R_A(t\text{->infinity})$. Use 200 terms of the infinite series and set up calculations for first 100 years.

$A = 1\ yr$ $j = 1..100$ Note: i is the contaminant index and j is the year index.

$t_j = A \cdot j$ Examples of A*j and t_j $t_{20} = 20 \cdot yr$ $t_{50} = 50 \cdot yr$

$$\Phi_{i,j} = 1 + 2 \sum_{n=1}^{200} (-1)^n \exp\left[\left(\left(\frac{-D_{eff}}{R_i} \cdot n^2 \cdot \pi^2 \cdot \frac{t_j}{L_{cap}^2}\right)\right)\right]$$

Note: the series converges withas few as 50 terms for times approaching or beyond the breakthrough time. When times are significanly less than the breakthrough time, the series may oscillate around zero. $j = 3..100$

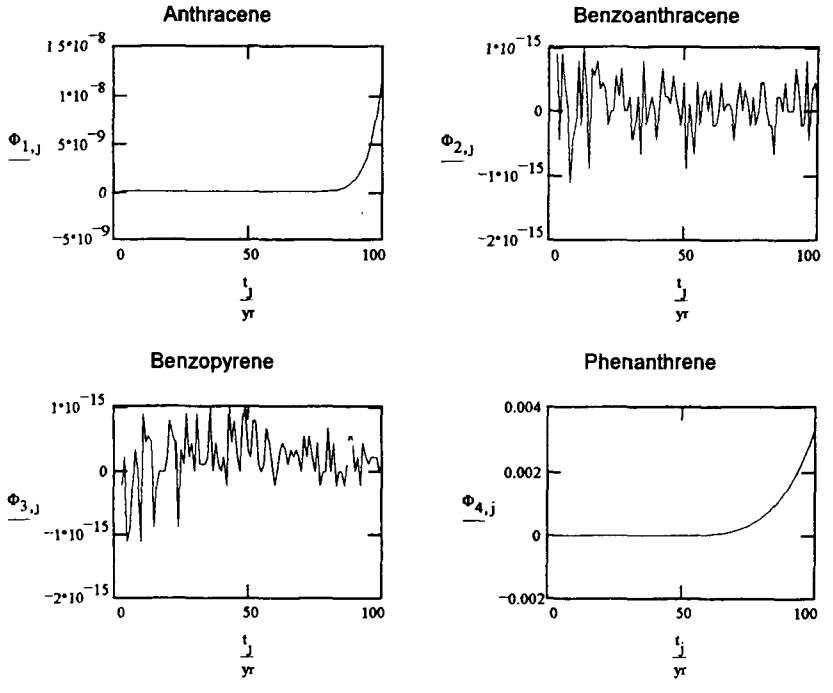


Figure 55. Contaminant losses for in situ capping: release rate ratios

expected because the period of analysis (100 years) is sufficiently close to the breakthrough times, 450 and 165 years, respectively. The curves for benzoanthracene and benzopyrene oscillate around zero because the breakthrough times are so large, 1,373 and 1,359 years, respectively. These results indicate that the transient flux over the first 100 years is approximately zero for benzoanthracene and benzopyrene.

Oscillations in the benzoanthracene and phenanthrene curves increase in magnitude as time zero is approached from the positive direction. For this reason, results for Years 1 and 2 are not plotted. Increasing the number of

terms in the series is marginally effective when using single precision arithmetic. Roundoff error begins to degrade the results as the number of terms in the series is increased. Actually, only 50 terms are needed to obtain convergence for phenanthrene. Since breakthrough time is directly proportional to the retardation factor, contaminants with low retardation factors may need only few terms for the series to converge. For contaminants with high retardation factors, the series is slow to converge.

Transient fluxes of anthracene and phenanthrene were integrated using the trapezoidal rule to obtain the total emission per square meter for the first 100 years (Figure 56). Transient fluxes for benzoanthracene and benzopyrene were not calculated because the transient flux was approximately zero for the first 100 years. The results in mass per area are shown in Figure 56. These results were then normalized with respect to the volume of contaminated sediment capped (Figure 57). Normalized mass losses for in situ capping were 1.3×10^{-8} and 0.05 mg/m^2 for anthracene and phenanthrene, respectively, and approximately zero for benzoanthracene and benzopyrene.

When an advective component is present, the above diffusional analysis of contaminant losses for in situ capping can be seriously misleading. As previously discussed in Chapter 6, the significance of advection relative to diffusion can be evaluated using the Peclet number (Equation 52). Figure 58 shows anthracene breakthrough curves for Peclet numbers of 1, 10, and 50. Cap thickness was used as the characteristic length.

Breakthrough curves were calculated using the Cleary and Adrian (1973) finite length model for advection/dispersion with linear equilibrium-controlled retardation. The same cap thickness (50 cm), same retardation coefficient for anthracene (156, Figure 53), and same effective diffusion coefficient (Figure 51) used in the diffusional analysis were used to prepare the breakthrough curves shown in Figure 58. The Peclet numbers represent three average pore water velocities as follows: Pe = 1 and U = 10^{-7} cm/sec, Pe = 10 and U = 10^{-6} cm/sec, and Pe = 50 and U = $5 \bullet 10^{-6}$ cm/sec. The instantaneous advective flux is the product of average pore water velocity and contaminant concentration at the cap-overlying water interface. Instantaneous fluxes at Year 100 are shown in Table 15.

The instantaneous advective fluxes for Peclet numbers 1 and 10 are lower and the instantaneous advective flux for Pe = 50 is larger than the steady-state diffusional flux for anthracene shown in Figure 54. However, as shown in Figure 53, the time to reach steady-state diffusional flux for anthracene is over 3,000 years. The times to breakthrough for advection, as defined by 5 percent of the steady-state advective flux, are also shown in Table 15. Note that the advective breakthrough occurs much more rapidly than for diffusion (Figure 58). In addition, the ultimate steady-state advective flux is U C_0, or identical to the advective flux without a cap. Thus, even a very small advective flux can completely alter the contaminant loss picture for in situ capping. In an advection-dominated system, the objective of capping is

The instantaneous flux at some time t is the product of the steady-state flux (Table 29) and the flux ratio at time t (Table 30).

Instantaneous flux of contaminant i at time t is given by the following, where j is the time index (1 to 100 years).

Anthracene at 100 yr

$$Flux_{1,j} = Flux_{ss_1} \cdot \Phi_{1,j} \qquad \text{Example:} \quad Flux_{1,100} = 1.32 \cdot 10^{-3} \cdot \frac{\eta g}{m^2 \cdot yr}$$

Phenathrene at 100yr

$$Flux_{4,j} = Flux_{ss_4} \cdot \Phi_{4,j} \qquad \text{Example:} \quad Flux_{4,100} = 1.92 \cdot \frac{\mu g}{m^2 \cdot yr}$$

Trapezoidal Rule:

$$j = 2..100$$

$$IFlux_i = Flux_{i,1} \cdot yr + \frac{1}{2} \cdot \sum_j \left(Flux_{i,j-1} + Flux_{i,j}\right) \cdot yr$$ <— IFlux is the integrated result.

Anthracene

$$IFlux_1 = 6.05 \cdot 10^{-9} \cdot \frac{mg}{m^2}$$

Benzoanthracene

$$IFlux_2 = 0 \cdot \frac{mg}{m^2}$$

Benzopyrene

$$IFlux_3 = 0 \cdot \frac{mg}{m^2}$$

Phenanthene

$$IFlux_4 = 2.16 \cdot 10^{-2} \cdot \frac{mg}{m^2}$$

Figure 56. Contaminant losses for in situ capping: flux integration over time

Time integrated results are multiplied by comtaminated area to obtain total mass loss for the time period of integration. To normalize total mass loss with respect to the volume, divide by the volume of contaminated sediment.

$$N_i = \frac{IFlux_i}{d_{cont}}$$ <-- Area / Volume = Depth

Contaminant i	Normalized 100 Year Mass Losses For Insitu Capping N_i $\left(\frac{mg}{m^3}\right)$	
1	$1.32 \cdot 10^{-8}$	<-- Anthracene
2	0	<-- Benzoanthracene
3	0	<-- Benzopyrene
4	$4.72 \cdot 10^{-2}$	<-- Phenanthrene

Figure 57. Contaminant losses for in situ capping: normalized mass losses

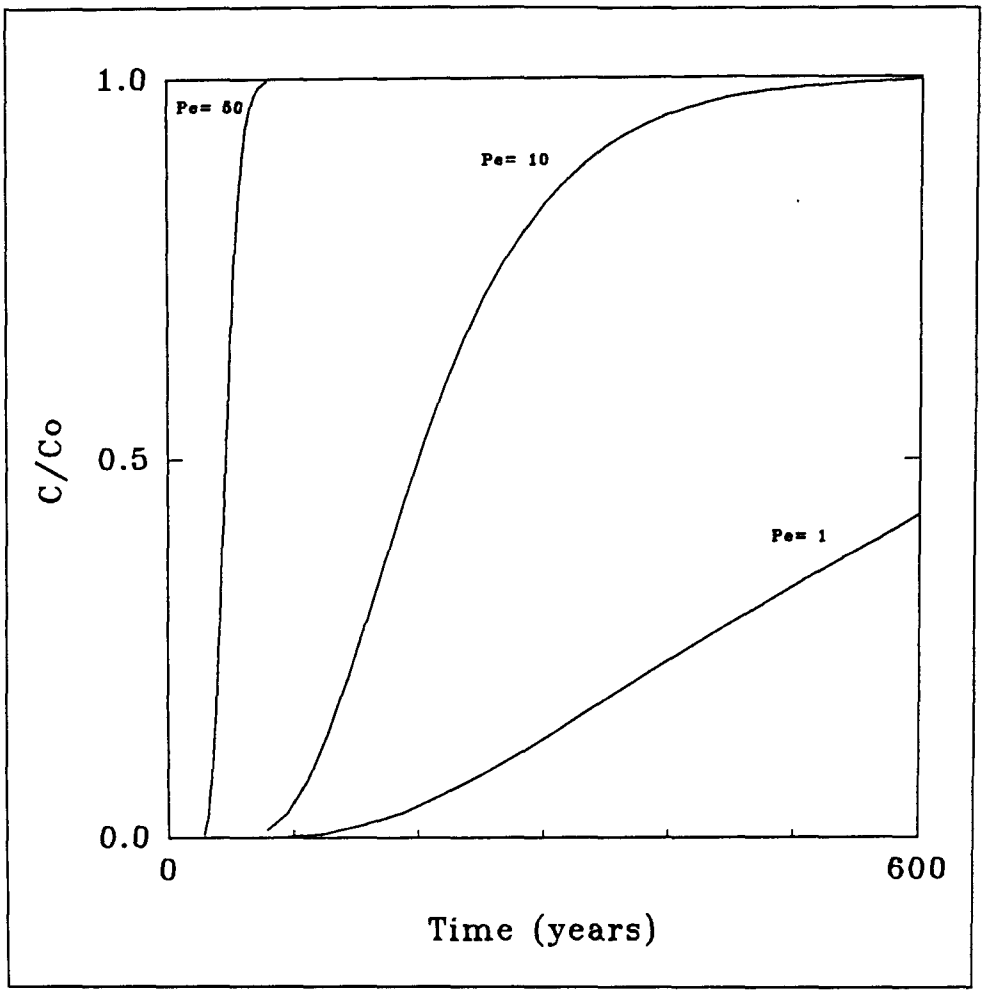


Figure 58. Anthracene breakthrough curves for a 50-cm cap, r = 156, and selected Peclet numbers

Table 15
Instantaneous Advective Anthracene Fluxes at Year 100 Through 50-cm Cap and Time Advective to Breakthrough (Based on 5 percent of steady-state flux)

Pe	U	Flux	τ_B
1	10^{-7}	0.002	≈235 years
10	10^{-6}	0.02	≈100 years
50	$5 \cdot 10^{-6}$	18	≈ 33 years

Note: Pe: Peclet number, dimensionless.
U: average pore water velocity, cm/sec.
Flux: $mg/m^2 \bullet year$.

containment until the contaminants are degraded or until a removal option can be implemented.

Losses for Pretreatment/Confined Disposal

Effluent

Mechanical dredging and placement of dredged material in pretreatment facilities for stockpiling and CDFs for disposal should result in minimal effluent losses since there is no conveyance water associated with mechanical dredging. Effluent losses for mechanical dredging and placement, therefore, are negligible. For mechanical dredging and hydraulic transfer to pretreatment or CDFs, the losses will be similar to those discussed below for hydraulic dredging and placement.

Effluent losses associated with hydraulic dredging and placement are best estimated from column settling and modified elutriate tests. These data can be applied to a specific facility design to predict losses or can be used in the design phase to design a facility for target effluent quality. Column settling and modified elutriate data are not available for materials from Dead Man's Creek. Therefore, the a priori technique for estimating effluent quality described in Contaminant Losses During Pretreatment was used to estimate effluent losses. The a priori techniques involves Equation 22 and CEFs from field studies to estimate effluent quality.

Palermo (1988) measured effluent quality and CEFs at five CDFs. The five-site average CEF for metals was 0.986 (98.6 percent). Organic contaminants were not investigated except for PCBs at one site. The one-site CEF for PCBs was 0.99 (99 percent). A CEF of 0.995 (99.5-percent containment) was used to estimate effluent losses. A CEF value higher than the previously measured CEFs is appropriate since the dredged material disposal operations for which CEF data are available were maintenance dredging projects, not remediation projects. It is assumed that remediation projects would put sufficient emphasis on facility design and operation that containment performance would be better than is typical for navigation maintenance projects.

Equation 22 in simple terms states that the fraction of contaminant mass placed in a pretreatment or disposal facility lost during hydraulic filling is 1 - CEF. Thus, for a CEF of 0.995, the mass fraction lost is 0.005. An estimate of mass loss was obtained by applying this factor to the sediment contaminant concentrations and bulk density for Dead Man's Creek. Normalized mass losses (product of contaminant concentration (mg/kg), bulk density (kg/m^3), and 0.005) are shown in Table 15. Sediment mean contaminant concentrations (Figure 41) were used for these estimates because effluent from hydraulic disposal operations tends to reflect the average dredged material contamination levels. Normalized contaminant mass losses for hydraulic

placement of dredged material from Dead Man's Creek ranged from 5.8 mg/m^3 for benzopyrene to 13 mg/m^3 for phenanthrene.

The field CEFs on which effluent a priori loss estimates are based were obtained using total (particulate plus dissolved) effluent contaminant concentrations. The effluent loss estimates in Figure 59, therefore, represent particulate and dissolved losses. Further, the a priori estimation technique for effluent losses does not account for contaminant chemical properties. A priori estimates are simply a faction of sediment contaminant concentrations, bulk density, and the applied CEF. In spite of these limitations, effluent a priori loss estimates are probably the most reliable a priori loss estimates that can be made at this time.

Mechanical Disposal

Contaminant	Normalized Mass Loss No Treatment	After Treatment
Anthracene	~ Zero	
Benzoanthracene	~ Zero	
Benzopyrene	~ Zero	
Phenanthrene	~ Zero	

Hydraulic Disposal

Contaminant	Normalized Mass Loss No Treatment	After Treatment*
Anthracene	6.4	1.5
Benzoanthracene	8.6	2.0
Benzopyrene	5.8	1.3
Phenanthrene	13	3.0

* Carbon adsorption, 77 percent treatment effectiveness (Table 10)

Figure 59. Effluent losses for placement of dredged material from Dead Man's Creek, Buffalo River

Effluent resulting from hydraulic placement of dredged material in pretreatment and disposal facilities can be treated to reduce effluent losses and associated water quality impacts. Effluent could be treated to reduce PAH losses. Normalized PAH losses after treatment by carbon adsorption are also shown in Figure 59. Normalized PAH losses after treatment were estimated by applying the 77-percent removal efficiency listed in Table 10 for fluoranthene by powdered activated carbon.

Leachate losses

Estimation of leachate losses requires estimation of leachate quality and flow and site-specific information on pretreatment or CDF design. For this example, upland pretreatment and CDFs were assumed.

Leachate flow was estimated using the HELP model in four simulations. These simulations were conducted to estimate leachate flow from mechanically placed dredged material in lined and unlined facilities and hydraulically placed dredged material in lined and unlined facilities. A simple liner consisting of a barrier soil (1-ft-thick) with a flexible membrane liner on top of the barrier soil was simulated. The synthetic weather generator in HELP was used to simulate climatological conditions for Buffalo, NY.

Spatial dimensions and dredged material properties for pretreatment and CDF simulations were identical. Time frames for simulation were different. The time for simulations of leachate flow from pretreatment facilities was 16 months, and the time for simulation of leachate flow from CDFs was 100 years.

Pretreatment and disposal facilities must be designed to handle dredge production and, in the case of pretreatment facilities, meet the requirements of the treatment process unit(s). For this example, a total processing time of 16 months for 10,000 yd^3 was assumed for the treatment process unit(s). Dredging could be scheduled in a variety of ways to satisfy this processing rate. Since only 10,000 yd^3 of material must be removed, the sediment could be removed in a single dredging project requiring 3 to 5 days. For both mechanical and hydraulic removal, it was assumed that all of the material would be dredged and placed at one time in either a pretreatment or a disposal facility.

Mechanical dredging and dredged material placement and hydraulic dredging and dredged material placement require different facility designs. Mechanical dredging and placement involves minimal increase in volume over the in situ volume of sediment. Hydraulic disposal, however, significantly increases the volume over the in situ volume. A rule of thumb is that four volumes of conveyance water becomes part of the dredged material for every volume of in situ sediment. Thus, facility dimensions are affected by the dredging method.

For mechanical disposal with negligible increase in dredged material volume over in situ sediment volume, a pretreatment or disposal facility must hold 10,000 yd^3 of material. Assuming an average depth of 6 ft, the facility surface area is 45,000 ft^2. The HELP model uses area to calculate total volume of seepage. General simulation parameters for facilities containing mechanical placed dredged material are listed in Table 16.

A two-layer simulation for mechanical placement in an unlined facility was conducted. The first or top layer is 6 ft of dredged material. The second or

Table 16
Design Parameters for Leachate Flow From Unlined and Lined Facilities Containing Mechanically Placed Dredged Material

Facility Design Parameters

- Layer one - 6 ft, dredged material, vertical percolation layer
- Layer two - Unlined Facility: 2 ft, foundation soil, vertical percolation layer.
  Lined Facility: 1 ft, constructed barrier soil with FML.
- Layer three - Lined Facility: 2 ft foundation soil, vertical percolation layer.

Soil and Dredged Material Properties

- Porosity
  Dredged Material = 0.40
  Foundation Soil = 0.50
  Constructed Barrier Soil = 0.4
- Field capacity
  Dredged Material = 0.32
  Foundation Soils = 0.30
  Barrier Soil = 0.32
- Initial water content
  Dredged Material = 0.40
  Foundation Soil = 0.35
  Barrier Soil = 0.35
- Saturated hydraulic conductivity
  Dredged Material = 1.0 E-06 cm/sec
  Foundation Soils = 1.0 E-04 cm/sec
  Barrier Soil = 1.0 E-07 cm/sec

Other

- Evaporative zone depth = 12 in.
- Type of vegetative cover - None
- No runoff, all water must percolate or evaporate.
- Area = 45,000 sq ft.

bottom layer is site foundation soil for which properties were assumed. When a specific site is under consideration, soil properties from the site should be used.

For hydraulic disposal with four volumes of water per volume of in situ sediment, a pretreatment or disposal facility accepting all the material at once must be able to store 50,000 yd^3. For a storage volume of 50,000 yd^3 and an assumed depth of 8 ft, the surface area is 168,750 ft^2. Hydraulically placed dredged material was assumed to rapidly consolidate to a porosity of 0.75. Further consolidation was not considered. Increasing the in situ sediment volume by the factor (0.75/0.4 = 1.875) and spreading this volume over 168,750 ft^2 yields an estimated dredged material depth of 3 ft. The HELP model simulations for hydraulic disposal were conducted as if the conveyance water used to place dredged material in the facility were all discharged as effluent, except for that retained in the dredged material. General simulation parameters for facilities containing hydraulically placed dredged material are listed in Table 17.

Table 17
Design Parameters for Leachate Flow From Unlined and Lined Facilities Containing Hydraulically Placed Dredged Material

Facility Design Parameters

- Layer one - 3 ft, dredged material, vertical percolation layer.
- Layer two - Unlined: 2 ft, foundation soil, vertical percolation layer.
  Lined: 1 ft, constructed barrier soil with FML.
- Layer three - Lined Only: 2 ft foundation soil, vertical percolation layer.

Soil and Dredged Material Properties

- Porosity
  Dredged Material = 0.75
  Foundation Soil = 0.50
  Constructed Barrier Soil = 0.4
- Field capacity
  Dredged Material = 0.32
  Foundation Soils = 0.30
  Barrier Soil = 0.32
- Initial water content
  Dredged Material = 0.75
  Foundation Soil = 0.35
  Barrier Soil = 0.35
- Saturated hydraulic conductivity
  Dredged Material = 1.0 E-06 cm/sec
  Foundation Soils = 1.0 E-04 cm/sec
  Barrier Soil = 1.0 E-07 cm/sec

Other

- Evaporative zone depth = 12 in.
- Type of vegetative cover - None
- No runoff, all water must percolate or evaporate.
- Area = 168,750 sq ft

Table 18 lists total percolation from facilities containing mechanically and hydraulically placed dredged material for 16-month and 100-year simulations. These leachate flow estimates are for percolation from the foundation soil layer. Although the percolation estimates were obtained using a vertical percolation simulation, leachate could move in all directions, including lateral movement through the confining dikes.

The pore water contaminant concentrations (Figure 60) were estimated using Equation 25-b. Equation 25-b includes the facilitated transport factor $(1 + K_{oc}C_{doc})$. Leachate contaminant concentrations were assumed to remain constant over time. For contaminants with high-distribution coefficients, such as PAHs, this is a good assumption. For contaminants with low-distribution coefficients, the assumption of constant-contaminant concentration overestimates contaminant losses.

Table 18
Totals for 16-Month and 100-Year HELP Model Vertical Percolation Simulations

Placement	Design	Total Percolation	
		16-Month (cu ft)	100-Year (Thousand cu ft)
Mechanical	Unlined	33,573 (0.31)	1,495 (2.2)
	Lined	42 (<0.1)	3.2 (<0.1)
Hydraulic	Unlined	75,250 (0.69)	5,555 (8.2)
	Lined	38 (<0.1)	5.6 (0.1)

Note: Numbers in parentheses are pore volumes of water displaced.

Figure 61 shows the sensitivity of contaminant concentration to the distribution coefficient. Dimensionless time (horizontal axis in Figure 61) is the number of pore volumes of water displaced. Large distribution coefficents (> 100 ℓ/kg) tend to keep contaminant concentrations low, but constant for long times. Small distribution coefficients (< 10 ℓ/kg) impose initially high contaminant concentrations that rapidly decline.

Normalized mass losses for the leachate pathway were obtained as the product of pore water contaminant concentration and total volume of leachate divided by the in situ volume of sediment requiring pretreatment or confined disposal. Normalized mass loss calculations for leachate are shown in Tables 18 and Figure 60. Mean sediment concentrations were used to estimate leachate losses because percolation tends to mix waters with varying contaminant concentrations. The LV matrices in Figure 62 are transpositions of the LV matrices (one column matrices) to one-row vectors, not the LV matrix raised to the T power. Elements in the N_m and N_n matrices (Figure 62) are normalized mass losses by leaching. The column headings for these matrices are the leachate volumes listed in Table 18, and the row designations are the four contaminants listed in Figure 41.

Volatile losses

Sediment and contaminant characteristics for volatile loss calculations are shown in Figure 63. Sediment characteristics include surface areas for hydraulic and mechanical filling and sediment organic carbon content (f_{oc}). Contaminant characteristics include mean sediment concentrations, distribution coefficients, molecular weights, molar volumes, solubilities in water, vapor pressures, dissolved water concentrations (estimated from sediment concentrations and distribution coefficients), Henry constants, diffusivities in water, overall liquid phase mass transfer coefficients (for an assumed wind speed of 15 mph), and gas-side mass transfer coefficients. Estimation of dissolved

$V := 10000 \cdot yd^3$ $\mu g \equiv 10^{-6} \cdot gm$ $f_{oc} := 0.02$ $C_{doc} := 25 \cdot \frac{mg}{liter}$

Mean Leachate Contaminant Concentrations (Dead Man's Creek)

$i := 1..4$

1-Anthracene	$C_{s_1} := 860 \cdot \frac{\mu g}{kg}$	$K_{oc_1} := 10^{4.27} \cdot \frac{liter}{kg}$
2-Benzoanthracene	$C_{s_2} := 1150 \cdot \frac{\mu g}{kg}$	$K_{oc_2} := 10^{6.14} \cdot \frac{liter}{kg}$
3-Benzopyrene	$C_{s_3} := 770 \cdot \frac{\mu g}{kg}$	$K_{oc_3} := 10^{6.0} \cdot \frac{liter}{kg}$
4-Phenanthrene	$C_{s_4} := 1780 \cdot \frac{\mu g}{kg}$	$K_{oc_4} := 10^{3.73} \cdot \frac{liter}{kg}$

$$C_{pw_i} := \frac{C_{s_i} \cdot \left(1 + K_{oc_i} \cdot C_{doc}\right)}{K_{oc_i} \cdot f_{oc}}$$

Leachate Volumes From HELP Model

Facility	Design	Time	Mechanical	Hydraulic
Pretreat	Unlined	16 mo	$LV_{m_1} := 33573 \cdot ft^3$	$LV_{h_1} := 75250 \cdot ft^3$
CDF	Unlined	100 yr	$LV_{m_2} := 1495000 \cdot ft^3$	$LV_{h_2} := 5555000 \cdot ft^3$
Pretreat	Lined	16 mo	$LV_{m_3} := 42 \cdot ft^3$	$LV_{h_3} := 38 \cdot ft^3$
CDF	Lined	100 yr	$LV_{m_4} := 3200 \cdot ft^3$	$LV_{h_4} := 5600 \cdot ft^3$

Figure 60. Contaminant losses by leaching: leachate concentrations and volumes

water concentrations did not include contaminant mass associated with colloidal organic matter because to volatilize from water, contaminants must be truly dissolved. Various constants, such as temperature, viscosity of water, atmospheric pressure, and molar volume of air, are also assigned values in Figure 63.

Calculations of volatile emission rates from ponded water are shown in Figure 64. The basic volatile flux equation for ponded water (Equation 32) was modified to an emission equation by multiplying flux by the ponded water

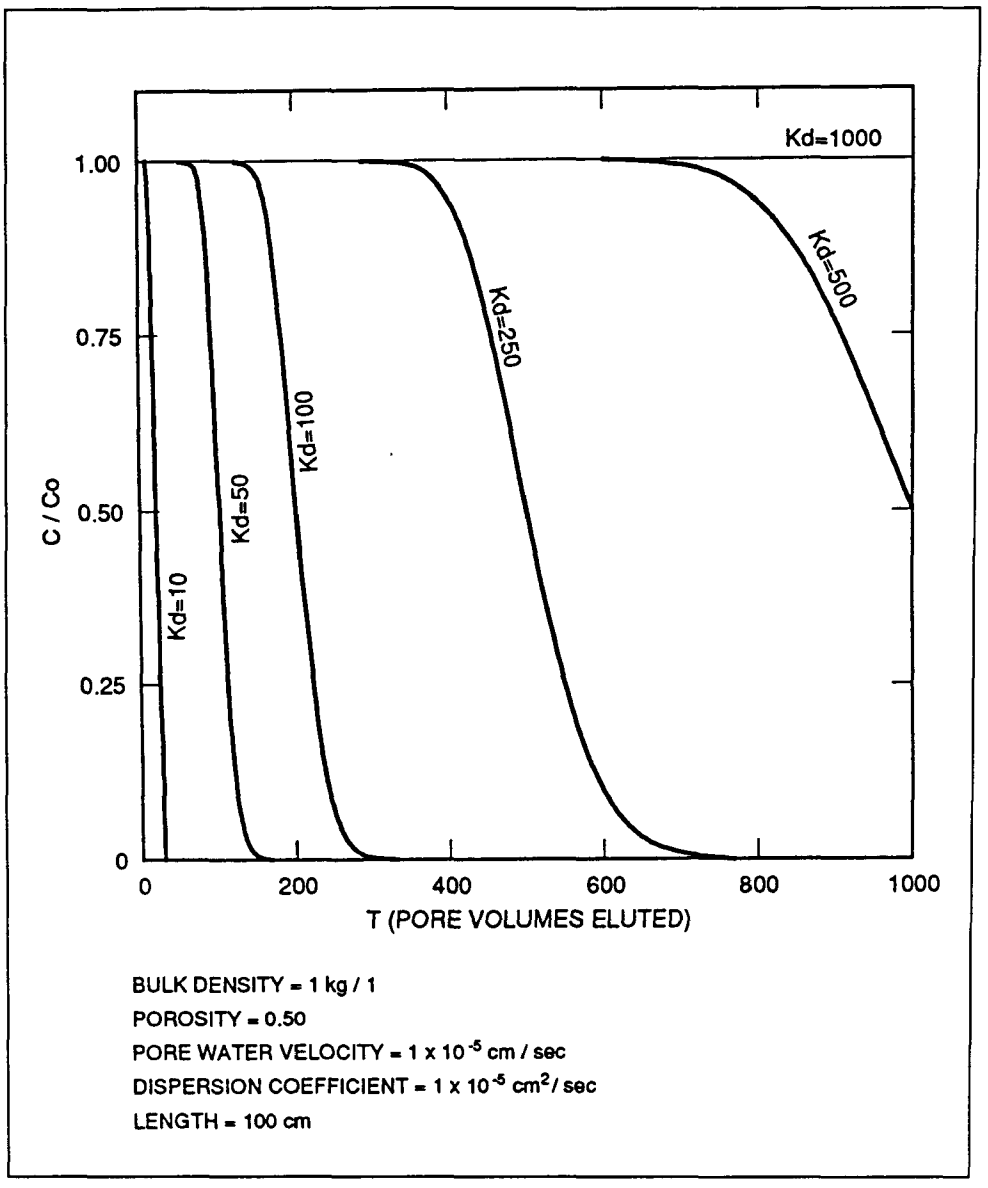


Figure 61. Fraction initial contaminant concentration remaining in leachate for various distribution coefficients

surface area for hydraulic filling. The surface area for hydraulic filling (A_2) as previously noted in the calculation of leachate losses is larger than the surface area for mechanical filling. In the example calculations shown in Figure 64, background air quality was assumed to be clean, that is, PAH concentrations in the background air were assumed to be negligible. Figure 64 also shows the calculation of normalized mass loss by volatile emission from ponded water for anthracene, benzoanthracene, benzopyrene, and phenanthrene. These calculations are applicable to pretreatment and disposal facilities because the ponded water holding time is about the same. Operation of each type of facility requires holding water long enough for adequate solids settling. For facilities of similar size, as assumed in these calculations,

$V = 10000 \cdot yd^3$ $\mu g \equiv 10^{-6} \cdot gm$ $f_{oc} = 0.02$ $C_{doc} := 25 \cdot \frac{mg}{liter}$

Mean Leachate Contaminant Concentrations (Dead Man's Creek)

$i = 1..4$

1-Anthracene	$C_{s_1} = 860 \cdot \frac{\mu g}{kg}$	$K_{oc_1} = 10^{4.27} \cdot \frac{liter}{kg}$
2-Benzoanthracene	$C_{s_2} = 1150 \cdot \frac{\mu g}{kg}$	$K_{oc_2} = 10^{6.14} \cdot \frac{liter}{kg}$
3-Benzopyrene	$C_{s_3} = 770 \cdot \frac{\mu g}{kg}$	$K_{oc_3} := 10^{6.0} \cdot \frac{liter}{kg}$
4-Phenanthrene	$C_{s_4} = 1780 \cdot \frac{\mu g}{kg}$	$K_{oc_4} = 10^{3.73} \cdot \frac{liter}{kg}$

$$C_{pw_i} = \frac{C_{s_i} \cdot \left(1 + K_{oc_i} \cdot C_{doc}\right)}{K_{oc_i} \cdot f_{oc}}$$

Leachate Volumes From HELP Model

Facility	Design	Time	Mechanical	Hydraulic
Pretreat	Unlined	16 mo	$LV_{m_1} = 33573 \cdot ft^3$	$LV_{h_1} = 75250 \cdot ft^3$
CDF	Unlined	100 yr	$LV_{m_2} = 1495000 \cdot ft^3$	$LV_{h_2} := 5555000 \cdot ft^3$
Pretreat	Lined	16 mo	$LV_{m_3} = 42 \cdot ft^3$	$LV_{h_3} := 38 \cdot ft^3$
CDF	Lined	100 yr	$LV_{m_4} := 3200 \cdot ft^3$	$LV_{h_4} := 5600 \cdot ft^3$

Figure 62. Contaminant losses by leaching: normalized mass losses

holding time requirements are similar. A 7-day holding time was used as previously discussed in the calculation of leachate losses. Normalized mass losses by volatilization from ponded water were highest for phenanthrene and lowest for benzopyrene. These estimates represent maximum potential losses

$V := 10000 \cdot yd^3$ $\mu g \equiv 10^{-6} \cdot gm$ $f_{oc} := 0.02$

$A_1 := 45000 \cdot ft^2$ <-- surface area of facility for mechanical filling

$A_2 := 168750 \cdot ft^2$ <-- Surface area of facility for hydraulic filling

$V_{is} := 1.139$ <-- viscosity of water (centipoise) $T := 288$ <-- temperature (K)

$M_a := 28.97$ <-- molecular wt of air $P := 1$ <-- pressure (atm)

$V_a := 20.1$ <-- molar volume of air (cc/mole)

Mean Sediment PAH Concentrations

4 PAHs	Avg Conc.	Distribution Coefficients	Mol Wt
1-Anthracene	$M_1 := 860 \cdot \frac{\mu g}{kg}$	$K_{d_1} := 10^{4.27} \cdot f_{oc} \cdot \frac{liter}{kg}$	$M_{b_1} := 178.24$
2-Benzoanthracene	$M_2 := 1150 \cdot \frac{\mu g}{kg}$	$K_{d_2} := 10^{6.14} \cdot f_{oc} \cdot \frac{liter}{kg}$	$M_{b_2} := 228.3$
3-Benzopyrene	$M_3 := 770 \cdot \frac{\mu g}{kg}$	$K_{d_3} := 10^{6.0} \cdot f_{oc} \cdot \frac{liter}{kg}$	$M_{b_3} := 352.3$
4-Phenanthrene	$M_4 := 1780 \cdot \frac{\mu g}{kg}$	$K_{d_4} := 10^{3.72} \cdot f_{oc} \cdot \frac{liter}{kg}$	$M_{b_4} := 178.24$

PAH Molar Volumes (Miller as cited by Mackay, Shiu, and Ma 1992) (cc/mole)

$V_{b_1} := 197$ $V_{b_2} := 248$ $V_{b_3} := 263$ $V_{b_4} := 199$

PAH Solubilities in Water (from Mackay, Shiu, and Ma 1992) (mg/L)

$S_1 := 0.075$ $S_2 := 0.014$ $S_3 := 0.004$ $S_4 := 1.2$

PAH Vapor Pressures (from Mackay, Shiu, and Ma 1992) (Pascals)

$a := .09357$ <-- factor for converting Pascals to mm Hg

$P_{a_1} := 0.00141 \cdot a$ $P_{a_2} := 4.1 \cdot 10^{-6} \cdot a$ $P_{a_3} := 7.32 \cdot 10^{-7} \cdot a$ $P_{a_4} := 0.0161 \cdot a$

Figure 63. Contaminant losses by volatilization: sediment and contaminant characteristics (Sheet 1 of 3)

Assume equilibrium for estimation of dissolved concentrations in ponded water.

$i = 1..4$

$$C_i = \frac{M_i}{K_{d_i}} \qquad C = \begin{bmatrix} 0.002 \\ 4.166 \cdot 10^{-5} \\ 3.85 \cdot 10^{-5} \\ 0.017 \end{bmatrix} \cdot \frac{mg}{liter}$$

Henry Constants (result is dimensionless)

$$H_i = 16.04 \cdot \frac{P_{a_i} \cdot M_{b_i}}{(T \cdot S_i)} \qquad H = \begin{bmatrix} 0.017 \\ 3.484 \cdot 10^{-4} \\ 3.36 \cdot 10^{-4} \\ 0.012 \end{bmatrix} \cdot 1$$

PAH Diffusivity in Water <-- result in cm^2/sec

$$D_{A2_i} = \frac{13.26 \cdot 10^{-5}}{V_{is}^{1.14} \cdot (V_{b_i})^{0.589}} \qquad D_{A2} = \begin{bmatrix} 5.089 \cdot 10^{-6} \\ 4.444 \cdot 10^{-6} \\ 4.293 \cdot 10^{-6} \\ 5.059 \cdot 10^{-6} \end{bmatrix}$$

Overall Liquid Side Mass Transfer Coefficient <-- results in cm/hr

$V_x = 15$ <-- assumed wind speed in mph

$$K_{OL_i} = 19.6 \cdot V_x^{2.23} \cdot (D_{A2_i})^{\frac{2}{3}} \cdot \frac{cm}{hr} \qquad K_{OL} = \begin{bmatrix} 2.432 \\ 2.222 \\ 2.172 \\ 2.423 \end{bmatrix} \cdot \frac{cm}{hr}$$

Figure 63. (Sheet 2 of 3)

Gas-Side Mass Transfer Coefficients

$$K_{g_i} = 3000 \cdot \sqrt{\frac{18}{M_{b_i}}} \cdot \frac{cm}{hr} \qquad K_g = \begin{bmatrix} 953.356 \\ 842.373 \\ 678.112 \\ 953.356 \end{bmatrix} \cdot \frac{cm}{hr}$$

PAH Diffusivities in air <— results in cm/sec

$$M_{r_i} = \frac{M_a + M_{b_i}}{M_a \cdot M_{b_i}}$$

$$D_{Al_i} = \frac{\sqrt{M_{r_i}} \cdot 10^{-3} \cdot T^{1.75}}{\left[V_a^{\frac{1}{3}} + \left(V_{b_i}\right)^{\frac{1}{3}} \right]^2 \cdot P} \cdot \frac{cm^2}{sec} \qquad D_{Al} = \begin{bmatrix} 0.055 \\ 0.049 \\ 0.047 \\ 0.055 \end{bmatrix} \cdot \frac{cm^2}{sec}$$

Figure 63. (Sheet 3 of 3)

because dissolved concentrations in ponded water were assumed to be equilibrium controlled and constant.

Calculations for volatile emissions from exposed dredged material solids for mechanical and hydraulic filling are shown in Figures 65 and 66, respectively. Calculations for mechanically filled and hydraulically filled facilities use the same basic equation (Equation 39). However, values for total porosity, air-filled porosity, and bulk density are different. The calculations involve piecewise integration of Equation 39 over time using the Romberg algorithm in MATHCAD. Piecewise integration was used to improve the accuracy of the results. The flux equation was integrated over 16 months and 100 years to simulate exposure times for pretreatment and disposal facilities,

Ponded Water Volatile Emission Rates - Hydraulic Filling

Ponded water volatile emission rates from mechanically placed dredged material are assumed to be negligible.

$$E_p := \overrightarrow{\left(K_{OL} \cdot C\right)} \cdot A_2 \qquad E_p = \begin{bmatrix} 2.114 \cdot 10^4 \\ 348.291 \\ 314.572 \\ 1.546 \cdot 10^5 \end{bmatrix} \cdot \frac{mg}{day}$$

Assume 4 day retention time for adequate solids settling plus 3 days for drawdown to yield total of 7 days emission time for ponded water.

Normalized mass losses by volatile emission from ponded water.

$$N_p = \frac{7 \cdot day \cdot E_p}{V}$$

<-- Applicable to both pretreatment and disposal facilities.

$$N_p = \begin{bmatrix} 19.351 \\ 0.319 \\ 0.288 \\ 141.544 \end{bmatrix} \cdot \frac{mg}{m^3}$$

<-- Anthracene
<-- Benzoanthracene
<-- Benzopyrene
<-- Phenanthrene

Figure 64. Volatile emission rates from ponded water—applicable to hydraulically filled pretreatment and disposal facilities

respectively. As previously discussed for leachate losses, the pretreatment facility will be needed for about 16 months, after which it will be closed. A disposal facility, however, is permanently maintained. A 100-year simulation time for a disposal facility was an arbitrary selection, influenced by uncertainty about applicability of the basic flux equation for long-term simulations.

For Mechanical Filling

$\varepsilon = 0.4$ <-- Total porosity

$\varepsilon_a = 0.1$ <-- Air filled porosity

$b = 1.5 \cdot \frac{gm}{cm^3}$ <-- Bulk density of Dredged Material

PAH Diffusivity in Soil Gas

$$D_{A3_i} = \frac{D_{A1_i} \cdot \varepsilon_a^{\frac{10}{3}}}{\varepsilon^2}$$

$$D_{A3} = \begin{bmatrix} 1.605 \cdot 10^{-4} \\ 1.422 \cdot 10^{-4} \\ 1.356 \cdot 10^{-4} \\ 1.598 \cdot 10^{-4} \end{bmatrix} \cdot \frac{cm^2}{sec}$$

Time-Integrated Flux From Mechanically Filled Dredged Material
MATHCAD's Romberg Integration, Tolerance set at 0.000001

TiFLUX = time integrated flux

Piecewise Integration Is Implemented Over Four Time Domains:

I: 0 - 1 day
II: 1 - 30 days
III: 30 days to 16 months
IV: 16 months to 100 years

$$TiFLUX_I_i = \int_{0 \cdot day}^{1 \cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$TiFLUX_I_1 = 0.123 \cdot \frac{mg}{m^2}$ <--Anthracene volatile flux from exposed sediment integrated over 0-1 day -- Mechanical Filling

Figure 65. Volatile emission from exposed dredged material—mechanical filling (Sheet 1 of 4)

$$TiFLUX_II_i = \int_{1 \cdot day}^{30 \cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$TiFLUX_II_1 = 0.179 \cdot \frac{mg}{m^2}$ <--Anthracene volatile flux from exposed sediment integrated over 1 to 30 days -- Mechanical Filling

$$TiFLUX_III_i = \int_{30 \cdot day}^{16 \cdot 30 \cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$TiFLUX_III_1 = 0.658 \cdot \frac{mg}{m^2}$ <--Anthracene volatile flux from exposed sediment integrated over 30 days to 16 months -- Mechanical filling

Figure 65. (Sheet 2 of 4)

$$TiFLUX_IV_i = \int_{16 \cdot 30 \cdot day}^{100 \cdot 365 \cdot day} \frac{\dfrac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\dfrac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \dfrac{1}{K_{g_i}}\right]} dt$$

$TiFLUX_IV_1 = 6.791 \cdot \frac{mg}{m^2}$ <--Anthracene volatile flux from exposed sediment integrated over 16 months to 100 years -- Mechanical filling

$$TiFLUX_16mo_i = TiFLUX_I_i + TiFLUX_II_i + TiFLUX_III_i$$

$$TiFLUX_100yr_i = TiFLUX_I_i + TiFLUX_II_i + TiFLUX_III_i + TiFLUX_IV_i$$

$TiFLUX_16mo_1 = 0.961 \cdot \frac{mg}{m^2}$ <-- Anthracene volatile flux from exposed sediment integrated over 16 months -- Mechanical Filling.

$TiFLUX_100yr_1 = 7.752 \cdot \frac{mg}{m^2}$ <-- Anthracene volatile flux from exposed sediment integrated over 100 years -- Mechanical Filling.

Figure 65. (Sheet 3 of 4)

Volatile Losses Normalized With Respect To the Insitu Volume of Sediment Dredged

Pretreatment Facility - Exposure Time = 16 months

$$NormVolLoss_Pt_{M_i} = \frac{TiFLUX_16mo_i \cdot A_1}{V}$$

$$NormVolLoss_Pt_M = \begin{bmatrix} 0.525 \\ 0.001 \\ 0.001 \\ 1.579 \end{bmatrix} \cdot \frac{mg}{m^3}$$

<--Anthracene
<-- Benzoanthracene
<-- Benzopyrene
<-- Phenanthrene

Disposal Facility - Exposure Time Infinite - Losses Estimated For 1st 100 Years

$$NormVolLoss_Dis_{M_i} = \frac{TiFLUX_100yr_i \cdot A_1}{V}$$

$$NormVolLoss_Dis_M = \begin{bmatrix} 4.239 \\ 0.012 \\ 0.009 \\ 11.897 \end{bmatrix} \cdot \frac{mg}{m^3}$$

<--Anthracene
<-- Benzoanthracene
<-- Benzopyrene
<-- Phenanthrene

Figure 65. (Sheet 4 of 4)

Volatile Fluxes From Exposed Dredged Material (mud)

Hydraulic Filling

$\varepsilon := 0.75$ <-- Total porosity

$\varepsilon_a := 0.2$ <-- Air filled porosity

$b := 0.86 \cdot \frac{gm}{cm^3}$ <-- Bulk density of Dredged Material

PAH Diffusivity in Soil Air

$$D_{A3_i} := \frac{D_{A1_i} \cdot \varepsilon_a^{\frac{10}{3}}}{\varepsilon^2} \qquad D_{A3} = \begin{bmatrix} 4.602 \cdot 10^{-4} \\ 4.076 \cdot 10^{-4} \\ 3.886 \cdot 10^{-4} \\ 4.581 \cdot 10^{-4} \end{bmatrix} \cdot \frac{cm^2}{sec}$$

Flux From Hydraulically Filled Dredged Material
MATHCAD's Romberg Integration, Tolerance set at 0.000001

$i := 1..4$

<u>Volatile Flux Integrated Over 0 - 1 day</u>

$$TiFLUX_I_i = \int_{0 \cdot day}^{1 \cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$$TiFLUX_I = \begin{bmatrix} 0.133 \\ 1.54 \cdot 10^{-4} \\ 1.132 \cdot 10^{-4} \\ 0.588 \end{bmatrix} \cdot \frac{mg}{m^2}$$

<-- Anthracene
<-- Benzoanthrancene
<-- Benzopyrene
<-- Phenanthrene

Figure 66. Volatile emissions from exposed dredged material—hydraulic filling (Sheet 1 of 4)

Volatile Flux Integrated Over 1 - 30 days

$$TiFLUX_II_i = \int_{1\,day}^{30\cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$$TiFLUX_II = \begin{bmatrix} 0.23 \\ 6.642 \cdot 10^{-4} \\ 4.918 \cdot 10^{-4} \\ 0.641 \end{bmatrix} \cdot \frac{mg}{m^2}$$

<-- Anthracene
<-- Benzoanthrancene
<-- Benzopyrene
<-- Phenanthrene

Volatile Flux Integrated Over 30 day - 16 mo

$$TiFLUX_III_i = \int_{30\cdot day}^{16\cdot 30\cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$$TiFLUX_III = \begin{bmatrix} 0.845 \\ 0.002 \\ 0.002 \\ 2.352 \end{bmatrix} \cdot \frac{mg}{m^2}$$

<-- Anthracene
<-- Benzoanthrancene
<-- Benzopyrene
<-- Phenanthrene

Figure 66. (Sheet 2 of 4)

Volatile Flux Integrated Over 16 mo - 100 yrs

$$TiFLUX_IV_i = \int_{30 \cdot 16 \cdot day}^{100\ 365 \cdot day} \frac{\frac{M_i \cdot H_i}{K_{d_i}}}{\left[\sqrt{\frac{\pi \cdot t}{D_{A3_i} \cdot \left(\varepsilon + K_{d_i} \cdot b\right)}} + \frac{1}{K_{g_i}}\right]} dt$$

$$TiFLUX_IV = \begin{bmatrix} 8.714 \\ 0.025 \\ 0.019 \\ 24.263 \end{bmatrix} \cdot \frac{mg}{m^2} \quad \begin{array}{l} \text{<-- Anthracene} \\ \text{<-- Benzoanthrancene} \\ \text{<-- Benzopyrene} \\ \text{<-- Phenanthrene} \end{array}$$

Summations of Piecewise Integrations for 16 Months and 100 Years

$$TiFLUX_16mo_i = TiFLUX_I_i + TiFLUX_II_i + TiFLUX_III_i$$

$$TiFLUX_100yr_i = TiFLUX_16mo_i + TiFLUX_IV_i$$

$$TiFLUX_16mo = \begin{bmatrix} 1.208 \\ 0.003 \\ 0.002 \\ 3.58 \end{bmatrix} \cdot \frac{mg}{m^2} \quad \begin{array}{l} \text{<-- Anthracene} \\ \text{<-- Benzoanthrancene} \\ \text{<-- Benzopyrene} \\ \text{<-- Phenanthrene} \end{array}$$

$$TiFLUX_100yr = \begin{bmatrix} 9.922 \\ 0.029 \\ 0.021 \\ 27.843 \end{bmatrix} \cdot \frac{mg}{m^2} \quad \begin{array}{l} \text{<-- Anthracene} \\ \text{<-- Benzoanthrancene} \\ \text{<-- Benzopyrene} \\ \text{<-- Phenanthrene} \end{array}$$

Figure 66. (Sheet 3 of 4)

Volatile Losses Normalized With Respect To the Insitu Volume Of Sediment Dredged.

Pretreatment Facility - Exposure Time = 16 months

$$NormVolLoss_Pt_{H_i} := \frac{TiFLUX_16mo_i \cdot A_2}{V}$$

$$NormVolLoss_Pt_H = \begin{bmatrix} 2.477 \\ 0.007 \\ 0.005 \\ 7.341 \end{bmatrix} \cdot \frac{mg}{m^3}$$

<--Anthracene
<-- Benzoanthracene
<-- Benzopyrene
<-- Phenanthrene

Disposal Facility - Exposure Time Infinite - Losses Estimated For 1st 100 Years

$$NormVolLoss_Dis_{H_i} := \frac{TiFLUX_100yr_i \cdot A_2}{V}$$

$$NormVolLoss_Dis_H = \begin{bmatrix} 20.346 \\ 0.059 \\ 0.044 \\ 57.092 \end{bmatrix} \cdot \frac{mg}{m^3}$$

<--Anthracene
<-- Benzoanthracene
<-- Benzopyrene
<-- Phenanthrene

Figure 66. (Sheet 4 of 4)

Figures 64 and 65 conclude with calculations of volatile losses normalized with respect to the in situ volume of sediment dredged. Comparison of normalized volatile losses in Figures 64 and 65 showed that losses were higher for hydraulically filled facilities than for mechanically filled facilities. This is due primarily to the larger surface area of hydraulically filled versus mechanically filled facilities. Losses for pretreatment facilities were substantially lower than losses for disposal facilities due to the lower exposure time for pretreatment facilities.

Contaminant Losses During Treatment By Thermal Desorption

Thermal desorption is one possible treatment option for removal of PAH compounds from Buffalo River sediment. An ARCS pilot study of thermal desorption treatment of Buffalo River sediment was performed in 1991, and a report has been prepared describing results of this study (USACE, Buffalo District 1993). In order to evaluate the effectiveness of thermal desorption and to collect design and operational data for future work, a monitoring program was implemented. The monitoring program included all streams entering and exiting the thermal desorption system. These data provide a basis for estimating mass of contaminant in each process stream, and, therefore, an estimate of contaminant losses.

A process flow diagram for the Buffalo River pilot thermal desorption (TD) unit is shown in Figure 67. Dredged material was screened prior to feeding the thermal desorption unit to remove oversize material. In this case, the oversize material consisted primarily of roots and debris. After screening, the sediment was stored in covered 208-ℓ, plastic-lined steel drums. Differences in contaminant concentrations before and after screening were not significant, suggesting that losses during screening were minimal. Major outputs from the thermal processor were the product solids, solids from a series of cyclones that removed particulates in the air stream, condensed liquids from the air stream, and a gas release from the stack. The system was designed to collect two separate liquid streams, one an oil residue small in volume and high in contaminant concentrations and the other a water stream high in volume and low in contaminant concentrations. During the Buffalo River Demonstration, these streams were difficult to separate and were similar in contaminant concentrations. A full-scale TD unit would require additional treatment of these liquid streams. Prior to release from the stack, the gas stream passed through an activated carbon bed. Spent carbon from a full-scale unit would require further treatment or disposal. The cyclone solids had PAH concentrations on the same order of magnitude as the dredged material, but the volume of cyclone solids collected was small relative to the volume of dredged material treated.

Nine separate runs were evaluated in the pilot demonstration. However, complete data sets are not available for every run. In particular, a limited

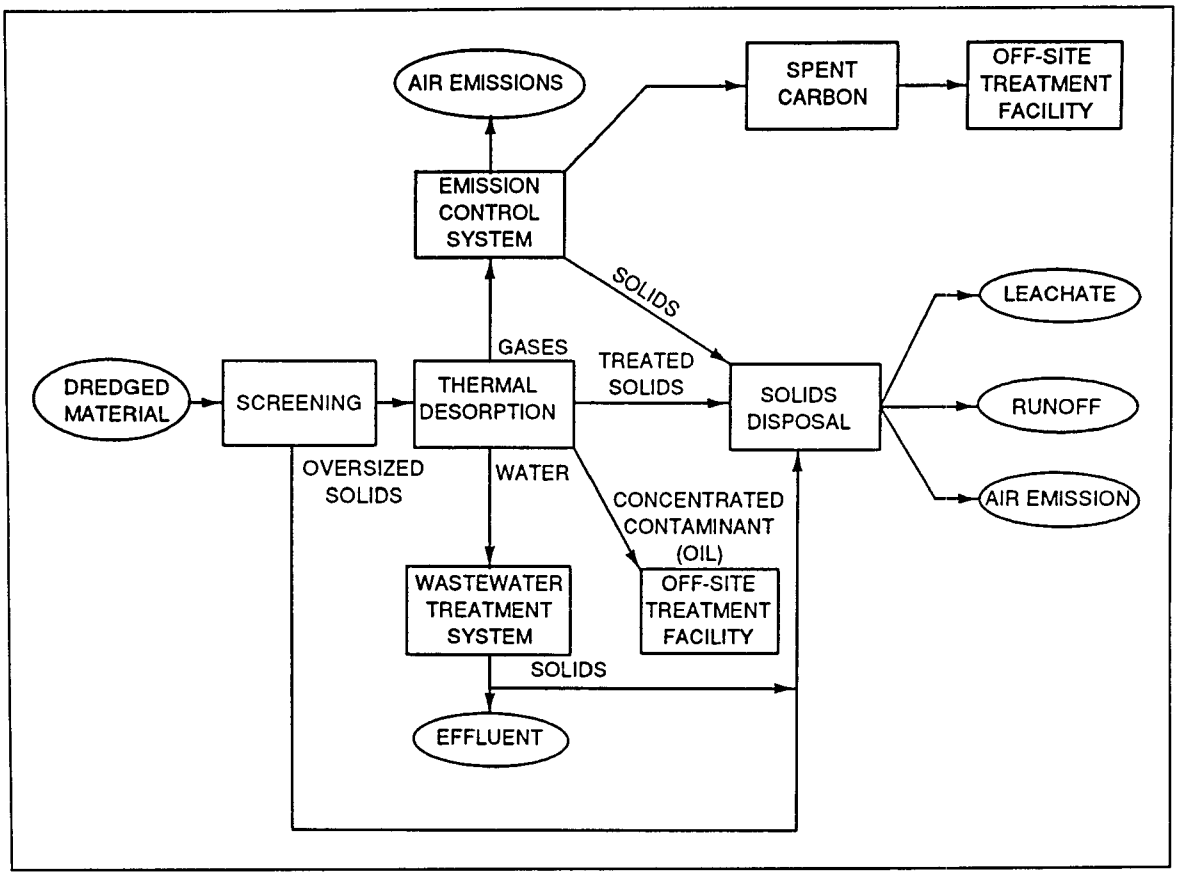


Figure 67. Process flow diagram for thermal desorption unit used in Buffalo River pilot demonstration

number of quality-ensured air analyses are available. A single run having a relatively complete data set was selected for estimating contaminant losses. This run was labeled as A2 and was conducted on 23 October 1991. Operating data for this run included a retention time of 60 min in the thermal processor and a soil exit temperature of 480 °F. Mass balance data are presented in Table 19. The mass of solids fed to the processor and exit streams were converted to pounds per hour dry solids since most analyses were reported on a dry weight basis. Table 20 provides the contaminant concentrations for each stream. The concentration and mass flow rate were multiplied to yield a contaminant mass emission per hour. Finally, Table 20 normalizes the mass of contaminant in each stream to the contaminant mass in the feed. This makes it convenient to extrapolate the results to a site-specific feed with different contaminant concentrations as shown in Table 21.

Table 21 also shows normalized contaminant concentrations in feed and process streams. The normalized contaminant concentrations in Table 21 represent contaminant mass in each stream per cubic meter of in situ sediment to be remediated. Most of the process streams could receive further treatment or could be placed in a secure facility with negligible contaminant losses. The results in Table 21 show that the cyclone catch represents the largest fraction

Table 19
Buffalo River Thermal Desorption Pilot Study, Mass Balance Data

Stream	Mass Rate Total lb/hr	Fraction Dry Solids	Mass Rate Dry solids lb/hr
Feed	502	0.545	273.59
Treated solids	238	0.998	237.52
Cyclone catch	25	0.862	21.55
Condensate	222	0.0095	2.11
Stack gas	NA	NA	

Table 20
Analysis of Buffalo River Thermal Desorption Pilot Study Data

Stream	Anthracene	Benzo(a)pyrene	Benzo(a)-anthracene	Phenanthrene
Concentration, ng/g (dry weight basis)				
Feed	133	545	542	670
Treated solids	5	10	5	37
Cyclone catch	101	260	228	618
Condensate	22.3	21.1	13.8	111
Stack gas	NA	NA	NA	NA
Contaminant Mass Flux, mg/hr				
Feed	16.5447	67.7958	67.4226	83.3453
Treated solids	0.5403	1.0805	0.5403	3.9979
Cyclone catch	0.9629	2.4788	2.1738	5.8920
Condensate	2.2476	2.1266	1.3909	11.1875
Stack gas	0.047	0.020	0.012	0.49
Fraction of Contaminant in Stream Compared with Mass in Feed				
Feed	1.0	1.0	1.0	1.0
Treated solids	0.0327	0.0159	0.0080	0.0480
Cyclone catch	0.0582	0.0366	0.0322	0.0707
Condensate	0.1359	0.0314	0.0206	0.1342
Stack gas	0.0028	0.0003	0.0002	0.0059
Estimated carbon load	0.7704	0.9158	0.9390	0.7412

Table 21
Extrapolation of Pilot Study Data to Contaminant Loss Example Problem

Fraction of Contaminant in Stream Compared With Mass in Feed				
Feed	1.0	1.0	1.0	1.0
Treated solids	0.0327	0.0159	0.0080	0.0480
Cyclone catch	0.0582	0.0366	0.0322	0.0707
Condensate	0.1359	0.0314	0.0206	0.1342
Stack gas	0.0028	0.0003	0.0002	0.0059
Estimated carbon load	0.7704	0.9158	0.9390	0.7412
Contaminant Concentrations in Example Sediment From Dead Man's Creek				
Contaminant concentration in feed, ng/g	Anthracene	Benzoanthracene	Benzopyrene	Phenanthrene
	860	1,150	770	1,780
Normalized Mass Concentration				
Contaminant concentration in feed, mg/m^3	1,290	1,725	1,155	2,670
Treated solids, mg/m^3	42.2	27.4	9.2	128.2
Cyclone catch, mg/m^3	75.1	63.1	37.2	188.8
Condensate, mg/m^3	175.3	54.2	23.8	358.3
Stack gas, mg/m^3	3.6	0.5	0.2	15.8
Estimated carbon load, mg/m^3	993.8	1,579.8	1,084.5	1,979

of contaminant in the residues requiring further treatment or disposal. The one process stream that would be difficult to further control is the stack gas (air emissions after carbon adsorption, Figure 67). The normalized concentrations in this process stream, therefore, are normalized contaminant losses for thermal desorption treatment, assuming other process residues receive further treatment or disposal without contaminant loss.

Comparison of Contaminant Losses

Overall

Figure 68 shows normalized PAH mass losses for five remedial alternatives. These alternatives are listed in Table 22. Alternative I involves mechanical dredging and mechanical disposal in an upland CDF. Controls for

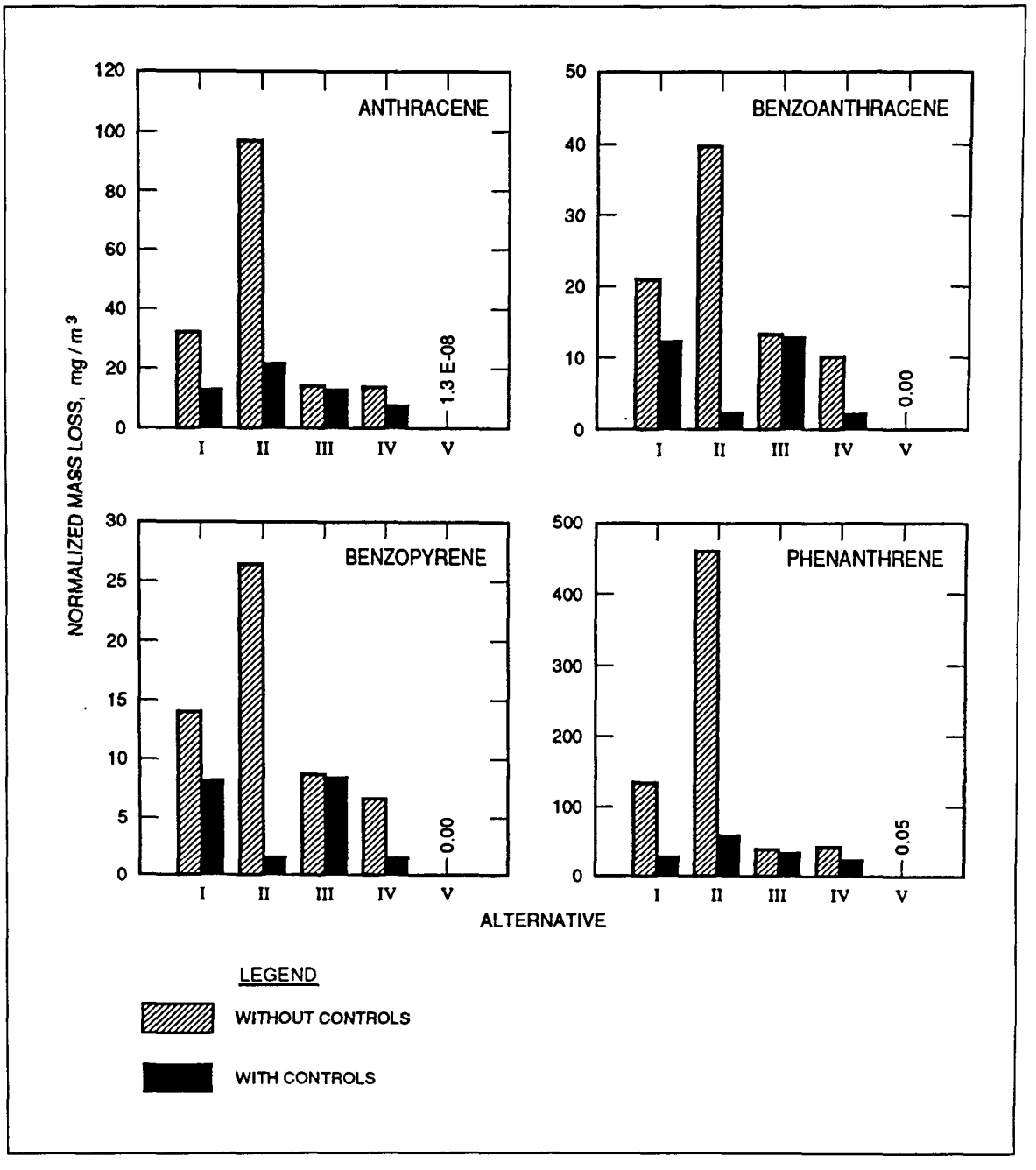


Figure 68. Normalized PAH mass losses

Table 22
Alternatives Considered for Remediation of Dead Man's Creek

Alternative	Description	Controls
I	Mechanical dredging and mechanical disposal in an upland CDF	Liner
II	Hydraulic dredging and disposal in an upland CDF	Effluent treatment by carbon adsorption and liner
III	Mechanical dredging and mechanical placement in a pretreatment facility (equalization) and thermal desorption processing of dredged material solids	Liner for pretreatment facility
IV	Hydraulic dredging and placement in a pretreatment facility (equalization and dewatering) and thermal desorption processing of dredged material solids	Carbon adsorption treatment of pretreatment effluent and liner for pretreatment facility
V	In situ capping	Assumes cap stability and isolation from bioturbation

Alternative I are limited to lining the CDF to minimize leachate losses. Effluent controls are not needed for Alternative I since dredging and disposal are mechanical. Alternative II involves hydraulic dredging and disposal in an upland CDF. Controls for Alternative II include effluent treatment by carbon adsorption and lining the CDF. Alternatives III and IV involve mechanical and hydraulic dredging, respectively, and include pretreatment (equalization and dewatering) and thermal desorption processing of sediment solids. Controls for Alternative III are limited to lining the pretreatment facility to minimize leachate losses. Effluent controls are not needed since dredging and placement are mechanical. Controls for Alternative IV include treatment of effluent from the pretreatment facility by carbon adsorption and lining the pretreatment facility to minimize leachate losses. Alternative V is in situ capping and does not involve dredging.

As indicated in Figure 68, contaminant loss calculations showed that in situ capping is superior to all other alternatives in terms of minimizing PAH releases over a 100 year period. Diffusion-controlled PAH release associated with in situ capping is estimated to be 1,000 to greater than 100,000 times less than the next best alternative. Alternatives are ranked in order of decreasing contaminant loss for each PAH in Table 23. Alternative IV with controls is second best in minimizing losses for all four PAHs. Alternative II without controls releases more PAH than any of the other alternatives, and Alternative I without controls was next worst. Between the second best and second worst alternatives, the relative order of the rankings vary with PAH. In general, the rankings for anthracene and phenanthrene are similar, and the rankings for benzoanthracene and benzopyrene are similar. Differences between rankings for the anthracene-phenanthrene pair and the

Table 23
Alternative Ranking by PAH

PAH	Ranking		Normalized Mass Loss, mg/m^3
Anthracene	V		1.32E-08
	IV	with controls	7.98
	III	with controls	13.51
	I	with controls	13.67
	IV	without controls	13.80
	III	without controls	13.93
	II	with controls	22.27
	I	without controls	32.33
	II	without controls	96.70
Benzoanthracene	V		0
	II	with controls	2.62
	IV	with controls	3.03
	IV	without controls	10.04
	I	with controls	12.63
	III	with controls	13.10
	III	without controls	13.28
	I	without controls	20.80
	II	without controls	39.59
Benzopyrene	V		0
	II	with controls	1.72
	IV	with controls	1.87
	IV	without controls	6.64
	I	with controls	8..42
	III	with controls	8.61
	III	without controls	8.73
	I	without controls	13.96
	II	without controls	26.80
Phenanthrene	V		0.05
	IV	with controls	26.98
	I	with controls	31.52
	III	with controls	36.78
	III	without controls	39.12
	IV	without controls	42.21
	II	with controls	61.3
	I	without controls	135.3
	II	without controls	457.9

benzoanthracene-benzopyrene pair are related to differences in chemical properties, as discussed below.

Alternative I

Figure 69 shows that most of the anthracene and phenanthrene losses for Alternative I without controls were through the leachate pathway. Dredging losses were second in relative significance for anthracene and phenanthrene, and volatile losses were third in relative significance for these two chemicals. Most of the benzoanthracene and benzopyrene losses were associated with dredging, and leachate losses made up the rest of the losses for these two

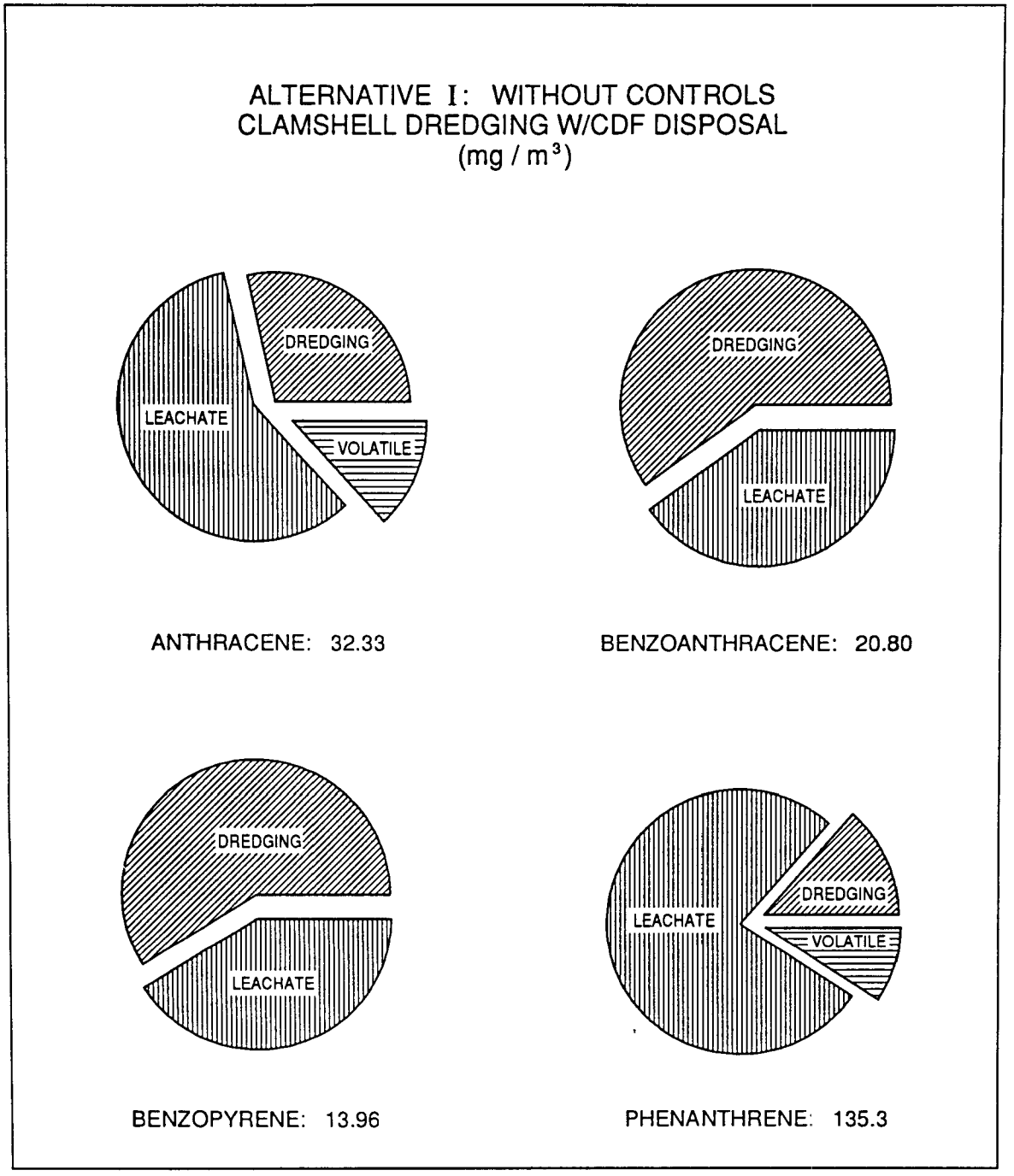


Figure 69. Alternative I without controls

chemicals. Volatilization was insignificant for benzoanthracene and benzopyrene.

Figure 70 shows losses for Alternative I with leachate controls (a lined CDF). Dredging losses dominate losses for all four PAHs, especially benzoanthracene and benzopyrene. Lining a CDF, of course, does not increase dredging losses. Because leaching losses have been significantly reduced,

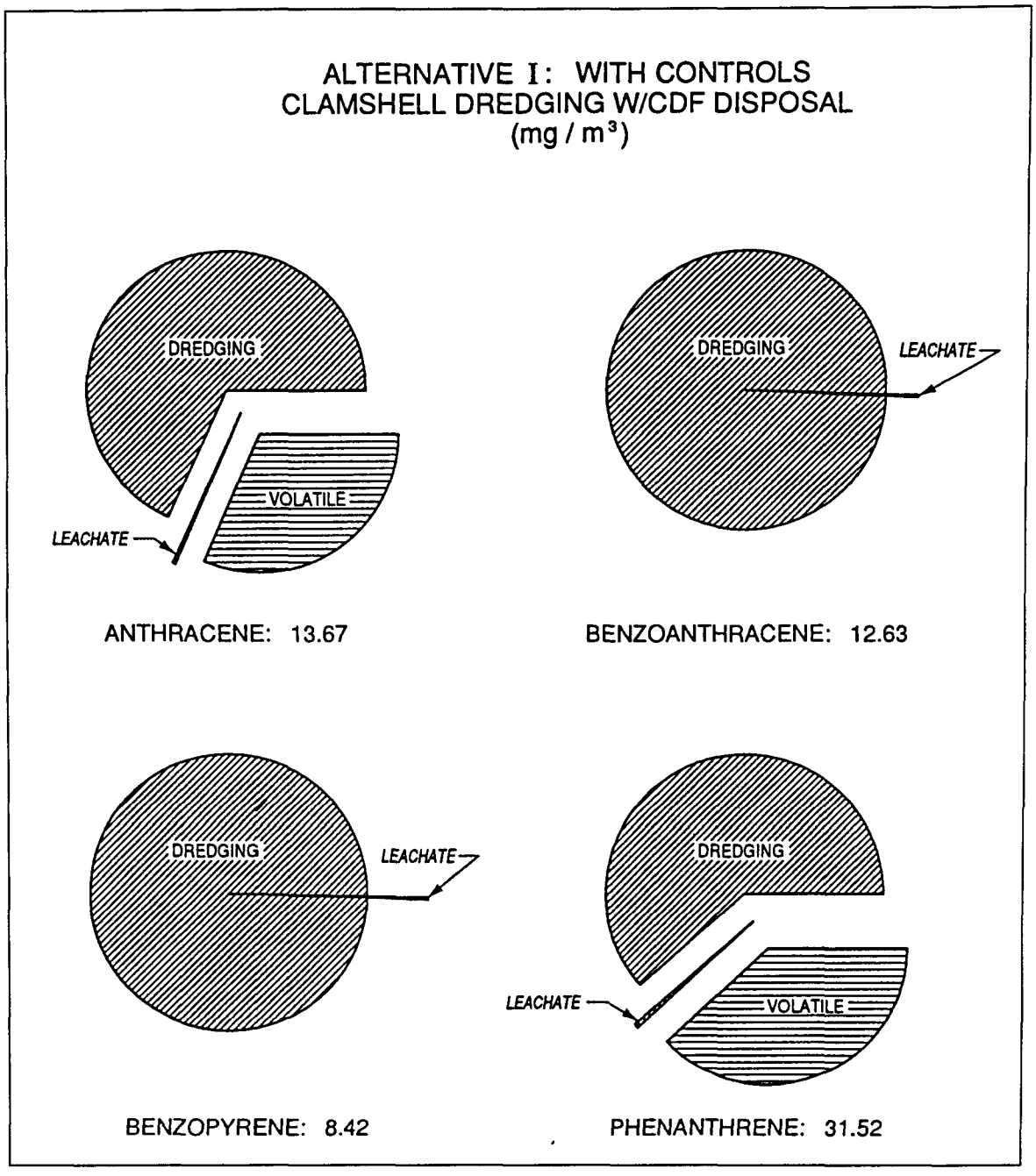


Figure 70. Alternative I with controls

dredging losses represent a proportionally larger share of the total loss estimate. Volatile losses are second in relative significance for anthracene and phenanthrene, and with leachate controls in effect for Alternative I, leachate losses of these two chemicals were of minor significance. Volatile losses were negligible for benzoanthracene and benzopyrene, and leachate losses were of extremely minor significance for these two chemicals.

PAH losses for Alternative I were reduced by 39 (benzoanthracene) to 77 (phenanthrene) percent by lining the CDF. Controls were less effective for benzoanthracene and benzopyrene than for anthracene and phenanthrene. The differences in control effectiveness is due to differences in the significance of dredging losses. For benzoanthracene and benzopyrene, dredging losses comprised a greater share of the total losses in the without-controls alternative than dredging losses for anthracene and phenanthrene. Therefore, implementing leachate controls (no impact on dredging losses) has less effect on benzoanthracene and benzopyrene losses.

The differences in primary loss pathways for different PAHs under Alternative I are related to differences in chemical properties. Anthracene and phenanthrene are more mobile than benzoanthracene and benzopyrene. Solubilities are higher (Figure 52), Henry constants are higher (Figure 63), and distribution coefficients (Figure 52) are lower for anthracene and phenanthrene than for benzoanthracene and benzopyrene. Thus, anthracene and phenanthrene were lost through pathways involving large masses of water (e.g., leachate) and volatilization. Benzoanthracene and benzopyrene were lost through pathways involving large masses of solids (dredging). Although the solubilities of anthracene, benzoanthracene, benzopyrene, and phenathracene are not high relative to many other chemicals, and distribution coefficients for these chemicals are not low relative to many other chemicals, leachate losses of these chemicals were significant for the unlined CDF option. Volatile losses were significant for anthracene and phenanthrene and insignificant for benzoanthracene and benzopyrene.

Alternative II

Figure 71 shows that most of the PAH losses for Alternative II without controls were through the leachate pathway. Volatile losses were second in relative significance for anthracene and phenanthrene, and effluent losses were third in relative significance for these two chemicals. For benzoanthracene and benzopyrene, effluent losses were second in relative significance, and dredging losses were third in relative significance for these two chemicals. Volatilization was insignificant for benzoanthracene and benzopyrene.

Figure 72 shows losses for Alternative II with controls (effluent treatment and a lined CDF). Most of the anthracene and phenanthrene were lost through volatilization. Effluent losses were second in relative significance for anthracene and phenanthrene, dredging losses were third in relative significance, and leachate losses were fourth in relative significance for these two chemicals with effluent and leachate controls. Effluent losses dominate losses for benzoanthracene and benzopyrene. Dredging losses were second in relative significance for benzoanthracene and benzopyrene, volatile losses were third in relative significance, and leachate losses were fourth in relative significance for these two chemicals with effluent and leachate controls.

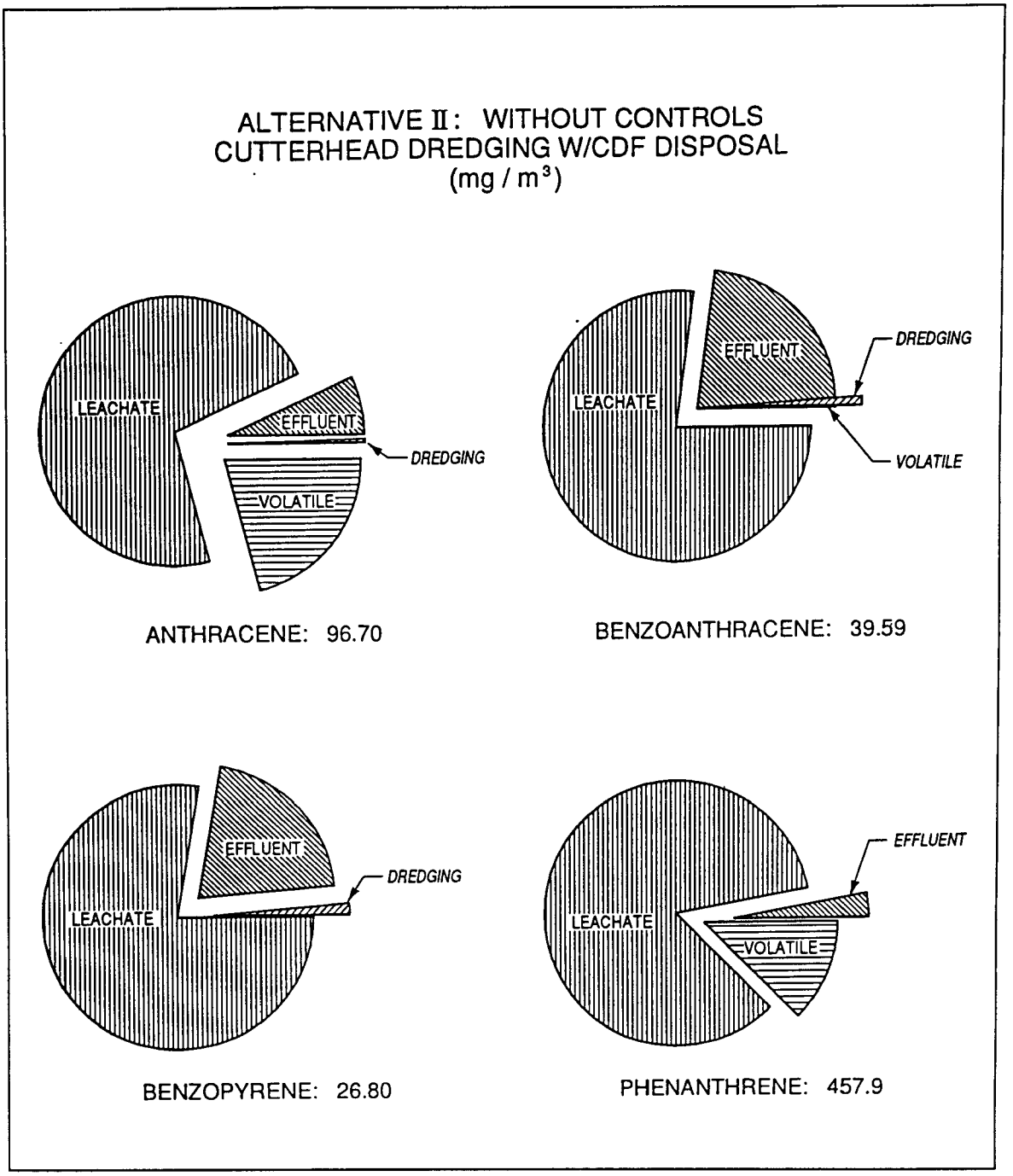


Figure 71. Alternative II without controls

PAH losses for Alternative II were reduced by 77 (anthracene) to 94 (benzopyrene) percent by implementing controls (Table 22). Losses were primarily reduced by restricting leachate flow. Controls were more effective for benzoanthracene and benzopyrene than for anthracene and phenanthrene. The differences in control effectiveness was due to differences in the relative significance of leachate losses. Benzoanthracene and benzopyrene leachate losses comprised a greater share of the total losses in the without controls-alternative

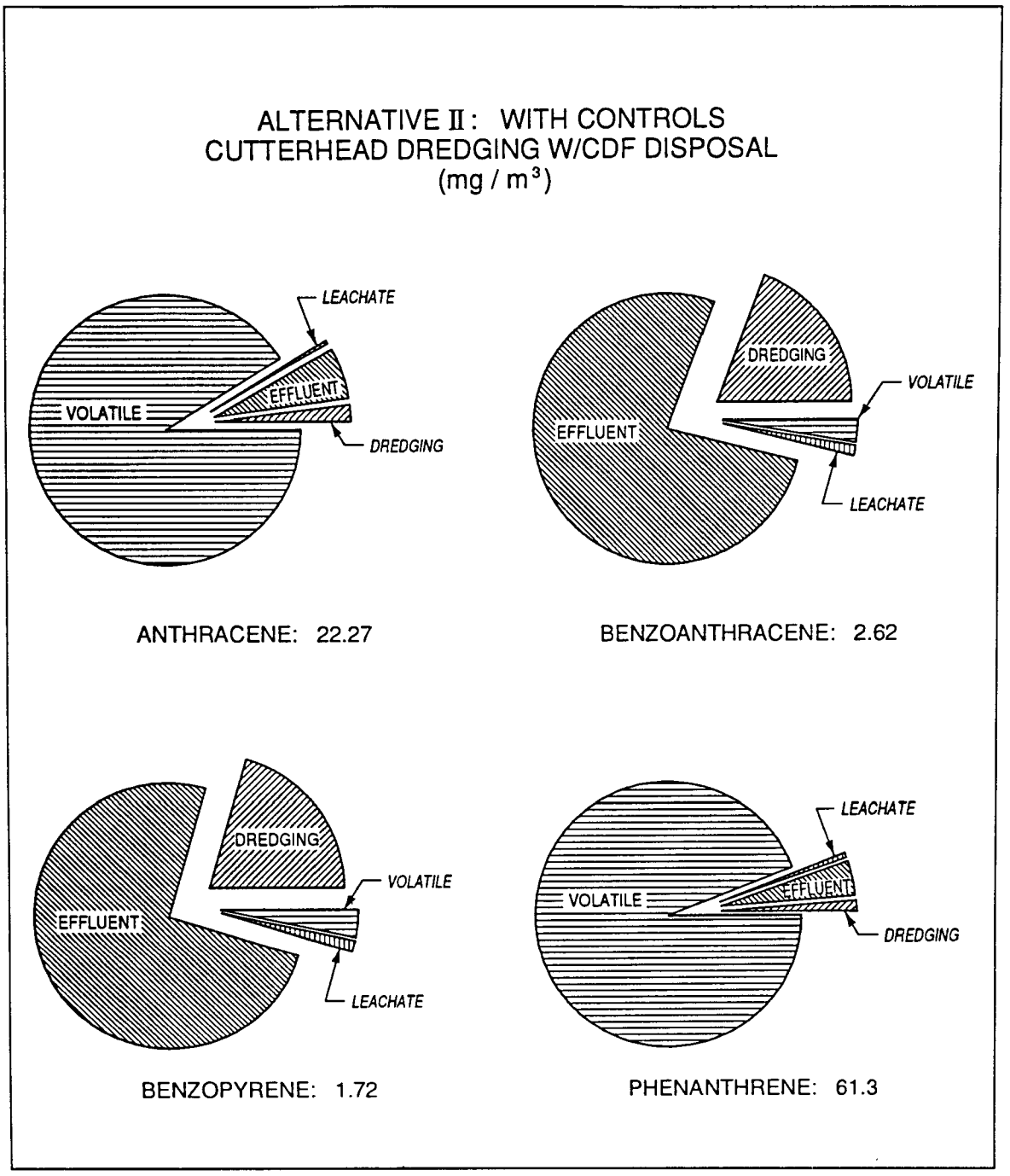


Figure 72. Alternative II with controls

than leachate losses for benzoanthracene and benzopyrene. Thus, leachate controls had more impact on those PAHs whose losses in the uncontrolled alternative were primarily leachate losses.

The relative significance of loss pathways for Alternative II with controls varied. For anthracene and phenanthrene, volatile and effluent pathways were

the primary loss pathways and volatile losses dominated. For benzoanthracene and benzopyrene, effluent and dredging pathways were the major pathways and effluent losses dominated. The differences in the significance of effluent and dredging losses is related to differences in the significance of the volatilization pathway. For Alternative II with controls, volatilization was the dominant loss pathway for anthracene and phenanthrene, but relatively insignificant for benzoanthracene and benzopyrene. The Henry constants are higher for anthracene and phenanthrene than for benzoanthracene and benzopyrene; hence, a greater tendency for anthracene and phenanthrene to be lost by volatilization. Relative to volatilization, dredging losses of anthracene and phenanthrene were small for Alternative II with controls. Since volatilization was insignificant for benzoanthracene and benzopyrene, dredging and effluent losses accounted for a larger portion of the total losses of these compounds for Alternative II with controls.

Alternative III

Figure 73 shows that PAH losses for Alternative III without controls were primarily associated with solids losses during dredging. Stack gas losses from the thermal desorption unit were second in relative significance for anthracene and phenanthrene, and leachate and volatile losses were relatively minor for these two chemicals. For benzoanthracene and benzopyrene, leachate losses and losses in the stack gas from the thermal desorption unit were relatively minor. Volatilization was negligible for benzoanthracene and benzopyrene.

Figure 74 shows the distribution of losses for Alternative III with controls (lined pretreatment facility). Figures 73 and 74 are similar because lining the pretreatment facility minimally reduces total losses and does not significantly alter the distribution of losses for Alternative III.

Anthracene and phenanthrene losses from the thermal desorption unit were significant relative to other losses, where as benzoanthracene and benzopyrene losses from the thermal desorption unit were insignificant relative to other losses. These differences can be explained on the basis of chemical properties. Anthracene and phenanthrene as previously discussed are more mobile and tend to sorb less than benzoanthracene and benzopyrene. Thus, sorption and retention in the carbon column treating stack gases from the thermal desorption unit were greater for benzoanthracene and benzopyrene than for anthracene and phenanthrene.

Alternative IV

Figure 75 shows that anthracene and phenanthrene losses for Alternative IV without controls were distributed among treatment, leachate, effluent, volatile, and dredging losses. Dredging losses were the least significant of all the losses for these chemicals. Effluent was the major, though not the dominant loss pathway, for anthracene. Losses from the thermal desorption unit

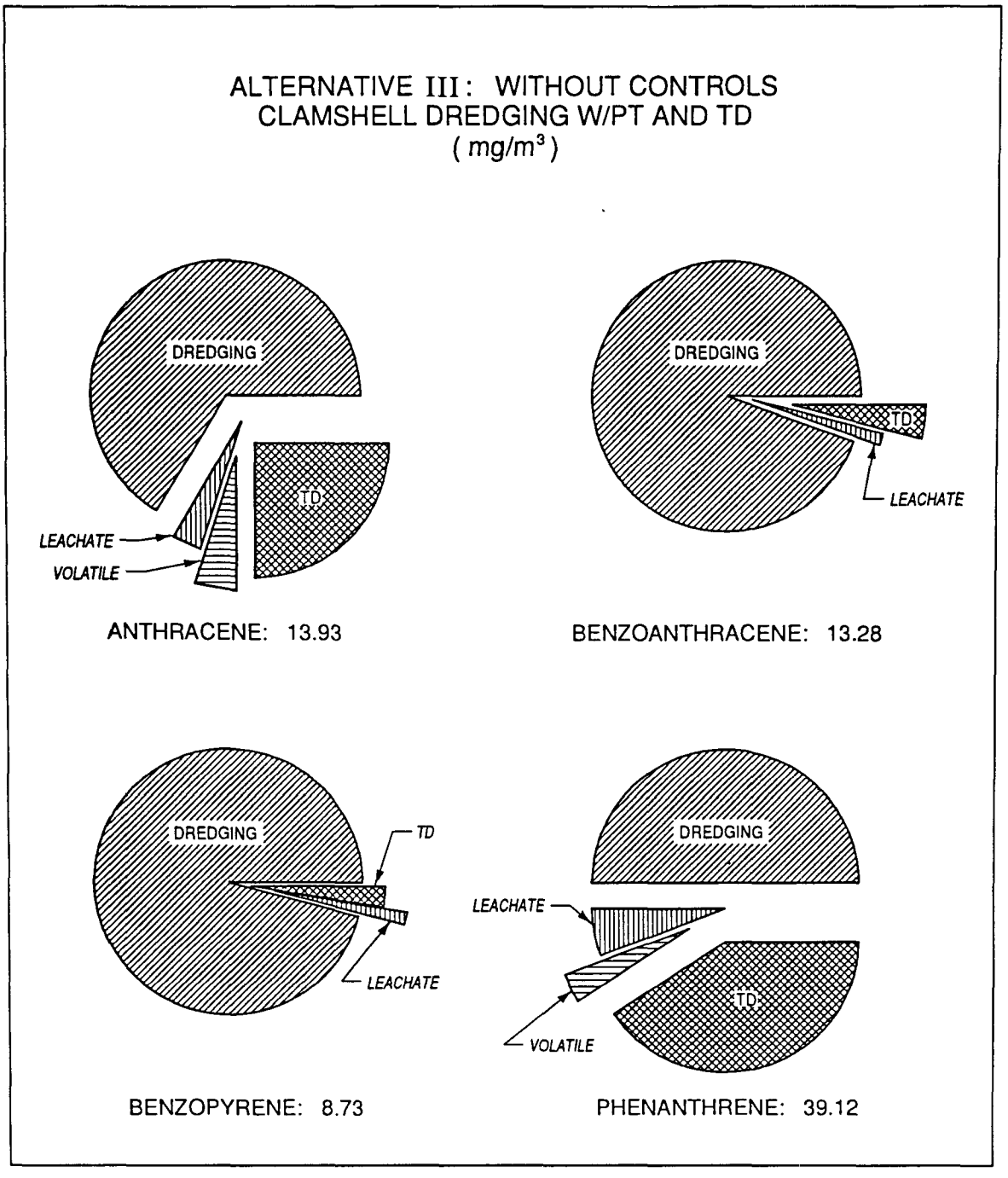


Figure 73. Alternative III without controls

were the major losses, though not the dominant loss, for phenanthrene. Benzoanthracene and benzopyrene were lost primarily through the effluent pathway.

Figure 76 shows the distribution of PAH losses for Alternative IV with controls (effluent treatment and lined pretreatment facility). Thermal desorption losses and volatile losses from the pretreatment facility were the most

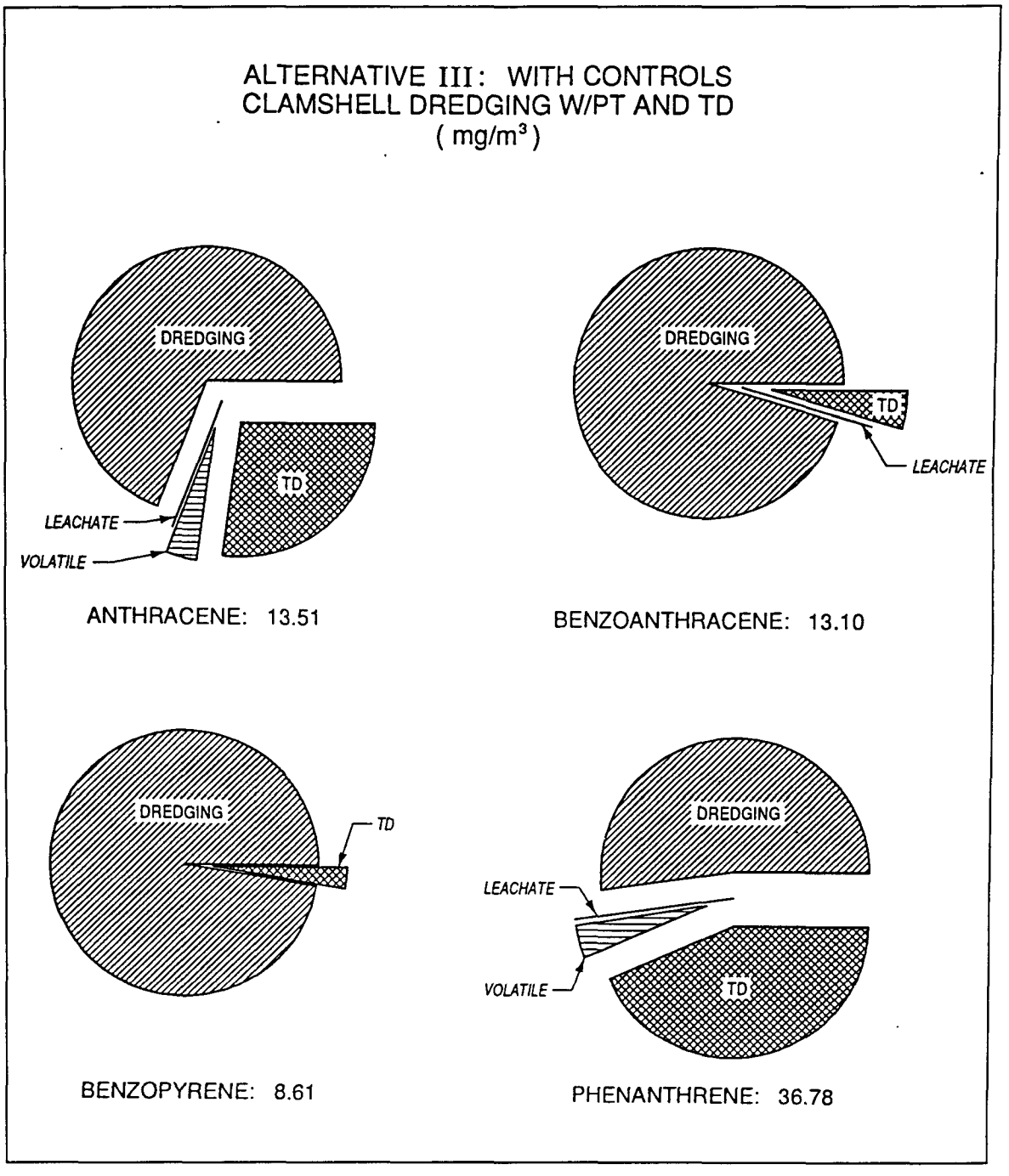


Figure 74. Alternative III with controls

significant losses for anthracene and phenanthrene. Effluent and dredging losses were the most significant losses for benzoanthracene and benzopyrene.

PAH losses for Alternative IV were reduced by 36 (phenanthrene) to 75 (benzoanthracene and benzopyrene) percent by implementing controls. Most of the reduction in losses were due to effluent treatment. Controls were more effective for benzoanthracene and benzopyrene than for anthracene and

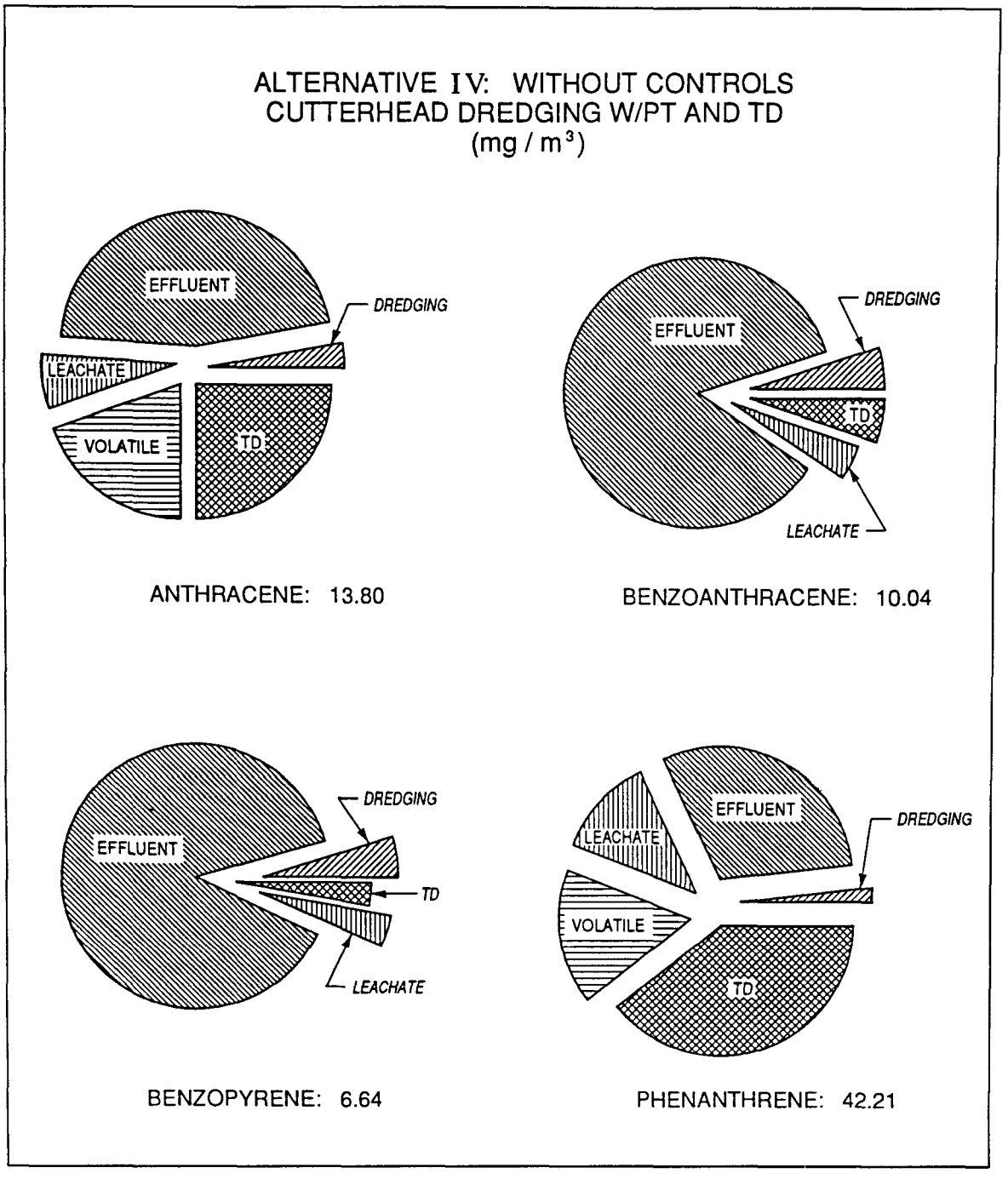


Figure 75. Alternative IV without controls

phenanthrene. The differences in control effectiveness was due to differences in the relative significance of effluent losses. Benzoanthracene and benzopyrene effluent losses comprised a greater share of the total losses in the without controls-alternative than effluent losses for anthracene and phenanthrene. Thus, effluent controls had more impact on those PAHs whose losses in the uncontrolled alternative were primarily effluent losses.

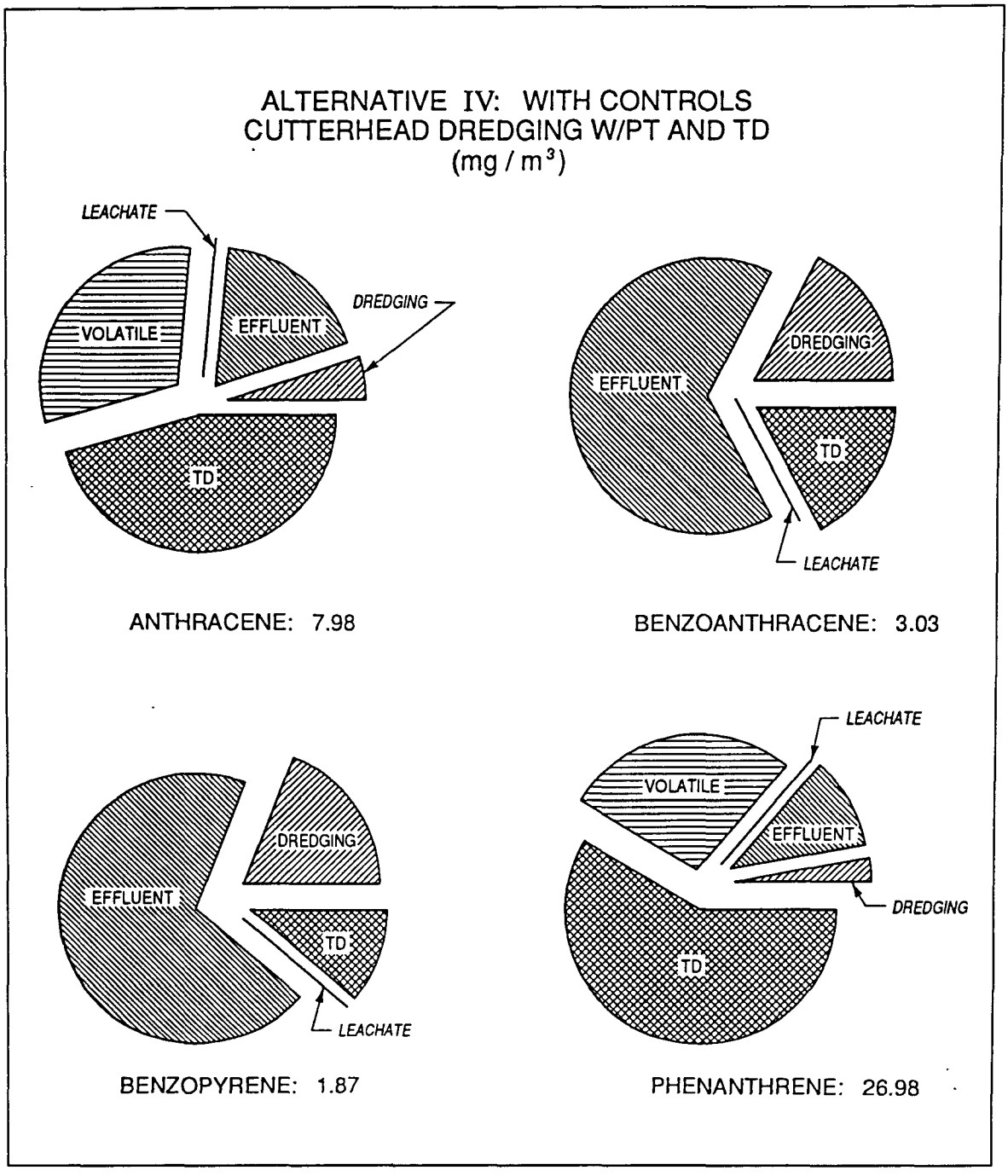


Figure 76. Alternative IV with controls

Effluent was a significant loss pathway for benzoanthracene and benzopyrene because other loss pathways such as volatilization and stack gas emission from the thermal desorption unit comprised relatively minor shares of the total losses. Anthracene and phenanthrene volatile losses from the pretreatment facility and stack gas emissions from the thermal desorption unit were more significant than effluent after treatment. Figure 76 suggests that further engineering controls could be chemical specific. For example, a cover for the

pretreatment facility could reduce anthracene and phenanthrene losses, but would have little effect on benzoanthracene and benzopyrene losses. Additional effluent treatment would reduce losses of all four PAHs, but would have a larger effect on the total losses for benzoanthracene and benzopyrene than on the total losses of anthracene and phenanthrene.

Summary

Several insights are offered by the example calculations. These insights are as follows:

a. Mechanical dredging followed by mechanical disposal in a CDF and hydraulic dredging followed by disposal in a CDF results in approximately the same PAH losses when leachate and volatile losses are neglected. In the former, PAHs are lost primarily at the point of dredging. In the latter, PAHs are lost in the effluent. The total mass loss is about the same. However, engineering controls are more practical for effluent than for dredging.

b. When leachate and volatile losses are considered, mechanical dredging and mechanical disposal in a CDF result in lower PAH losses than hydraulic dredging and disposal in a CDF. The primary difference is in leachate losses. Leachate losses are higher for the hydraulic dredging option because hydraulic dredging adds water to the sediment that is not removed during sedimentation as effluent. This water bulks the sediment and, depending on site-specific foundation conditions and hydrologic factors, may drain by gravity into underlying soils. Leachate controls are, therefore, more likely to be cost-effective for the hydraulic dredging and disposal option than for the mechanical dredging and disposal option.

c. The significance of volatile losses is highly chemical dependent. Four PAHs were included in the example calculations. For two, anthracene and phenanthrene, volatile losses were significant for some alternatives. For the other two, benzoanthracene and benzopyrene, volatile losses were negligible or minor for every remediation option considered.

d. Certain conventional wisdoms about dredging and dredged material disposal may need to be revisited. It is often said that many contaminants strongly sorb to sediment solids and, therefore, are not mobile. The example calculations suggest that this may be somewhat misleading. No matter how large the distribution coefficient, reversible sorption implies a capacity and potential for desorption.

11 Summary and Recommendations

Conclusions

General

The primary objective of this report was presentation of techniques for estimating contaminant losses associated with various sediment remediation alternatives. Preproject estimation of contaminant losses conducted early in the planning process can indicate the relative merit of various remediation alternatives. Intuitively, the alternative that minimizes contaminant losses is the most environmentally protective alternative. Although risk assessment, economics, feasibility, and other factors must be considered to fully evaluate alternatives, preliminary screening or ranking of alternatives according to estimated contaminant losses has merit because it is contaminant loadings (losses) to the environment that result in exposure concentrations that impair environmental quality. In addition, contaminant loss estimates provide some of the input data needed to conduct risk assessments for remediation alternatives.

Many environmental regulatory agencies are beginning to emphasize assessment of total mass losses of contaminants in their evaluations of dredged material management alternatives. Existing procedures such as the Corps of Engineers Management Strategy (Francingues et al. 1985), the Dredged Material Alternative Selection Strategy (Cullinane et al. 1986), the General Decision-Making Framework (Lee et al. 1991), and the Interagency Technical Framework for Evaluating Environmental Effects of Dredged Material Management Alternatives (USEPA/USACE) use analyses of contaminant migration pathways to estimate environmental effects (for example, water column and benthic toxicities). Estimated effects are compared with criteria established by regulatory authorities to arrive at decisions regarding the suitability of an alternative, including the need for restrictions. When acceptable combinations of restrictions are difficult to identify, the existing procedures provide little guidance for objectively evaluating tradeoffs between alternatives, including the no-action alternative. The approach to comprehensive analysis of contaminant losses described in this report provides an objective, comparative

assessment methodology that engineers, scientists, planners, decision-makers, and others involved in evaluation of sediment remediation alternatives may find helpful.

Techniques are available for estimating contaminant losses associated with most sediment remediation components and contaminant migration pathways within remediation components. In some cases, a priori estimation techniques are available that do not require data other than sediment characterization data and other minimal project data. Pathway specific laboratory tests are available for some contaminant migration pathways that provide more reliable estimates of losses than the a priori techniques. A priori techniques are suitable for planning-level assessments. Techniques that use pathway-specific laboratory data provide the type of loss estimates often called for by regulatory agencies that evaluate proposed remediation projects.

Availability and relative reliability of contaminant loss estimation techniques are shown in Table 24. The state of development of predictive techniques for estimating contaminant losses from remediation components varies with the component and the loss pathway. For some remediation components, there are no pathway-specific tests available. In these cases, a priori techniques may be the only techniques available; however, a priori techniques are not always available for all pathways of all components. The confidence and accuracy of contaminant loss estimates depend on the state of development and the amount of field-verification data available.

Table 24
Availability and Relative Reliability of Contaminant Loss Estimation Techniques

Component or Alternative	Available	Reliability	Ease of Use
In Situ Capping	Yes	Moderate	Difficult
Open-Water Disposal/Capping	Yes	Moderate	Difficult
Dredging	Yes	Low	Moderately Difficult
Transportation	No	--	--
Confinement	Yes	Variable	Moderately Difficult
Treatment	Yes	High	Simple
No Action	Yes	High	Very Difficult

This report illustrates how overall pooled estimates for all pathways and remediation components can be used to compare sediment remediation alternatives in terms of effectiveness. Most of the available estimation techniques, however, are not sufficiently developed or field verified to warrant decision making on the basis of contaminant loss estimates alone. Even if the

estimation techniques were fully developed and field verified, it would not be prudent to use estimated contaminant losses alone for decision making.

Using the a priori techniques described and illustrated in this report, it is possible to identify major contaminant loss pathways for various alternatives. This information can then be used to identify needs for laboratory testing to provide sediment-specific parameters for refined estimates of contaminant losses. For example, if for alternative A, pathways i and j are shown to be relatively insignificant and pathway k is shown to be significant relative to pathways i and j, then laboratory determination of sediment-specific parameters for pathway k is indicated if the comparison of alternative A to other alternatives is to be refined. Thus, the a priori techniques described in this report can be used to allocate resources toward refining contaminant estimates and, hence, evaluation of remediation alternatives.

This report includes a set of example calculations. Most of the calculations were implemented on commercially available mathematical software that allows the user to present equations as if they were written on engineering paper. In one case, public domain software (the Hydrologic Evaluation of Landfill Performance computer model) was used to estimate leachate seepage from upland pretreatment and CDFs. Preparation of this report did not involve computer model development, and no code was written to implement any of the estimation techniques. Readers are directed to the fact that a single computer code is not available for implementation of the various estimation techniques described in this report.

In no case do the a priori techniques described in this report replace sound engineering practice. A priori evaluation of alternatives is one thing. Selection, recommendation, and funding of a preferred alternative is quite another. In the latter, the preferred alternative must stand on its own merit as environmentally protective and cost-effective. Substantial sediment-specific testing is usually required to clearly demonstrate that a given alternative is at once environmentally protective and cost-effective. No amount of a priori estimation of contaminant losses is sufficient for this task. This report provides a planning level assessment tool for narrowing the universe of available alternatives and, hence, the scope of sediment-specific testing required for decision making.

Nonremoval technologies

Estimating contaminant losses for nonremoval technologies is difficult due to lack of field databases and standard procedures for assessment for nonremoval technologies. Predictive models based on diffusion are conceptually applicable to most nonremoval technologies. However, predictive techniques are not available that account for many important aspects of remediation with nonremoval technologies.

Losses during placement of a cap, or injection of immobilization additives, or injection of reagents for chemical treatment can result in highly

localized-transient disturbances of contaminated sediment. These highly localized and transient disturbances can be as important, if not more important, than long-term diffusion losses. At present, highly localized-transient losses associated with nonremoval technology implementation cannot be predicted.

Once the implementation phase of a nonremoval technology is completed, diffusion is the major loss pathway in the absence of significant advection. Application of diffusion models to in situ capping is a recent development in contaminant loss estimation. The theoretical basis for diffusion modeling is well developed and confirmed in laboratory-scale simulations of in situ capping, but field verification data are nonexistent. Convection, bioturbation, and biodegradation are potentially important, depending on site characteristics. Convection and bioturbation effects can be avoided by careful planning, design, and preproject testing. For example, controls for bioturbation should be part of engineering design, and sites with significant groundwater movement through the sediment are not good candidates for in situ capping.

Dredging

Techniques for estimating sediment solids losses during hydraulic and mechanical dredging are available for conventional dredging equipment. Techniques are not available for innovative dredging equipment options. The available predictive techniques provide estimates of sediment losses in terms of mass loss per time or mass loss per in situ volume dredged. Exposure concentrations are not estimated. To estimate exposure concentrations, the predicted losses of sediment and associated chemical contaminants must be incorporated into water quality or exposure assessment models.

Techniques for estimating contaminant losses during dredging are still in an early stage of development. Field data on turbidity and suspended solids downstream of dredging operations are available, but measurement of losses at the point of dredging that gave rise to the reported data are largely lacking. Empirical correlations of sediment losses at the point of dredging with dredging operational parameters have been developed, but field validation data are scarce. The predictive techniques focus on losses at the point of dredging and are inherently a priori, although laboratory tests have been proposed. It is anticipated that the available correlations will be modified and improved as a result of ongoing studies.

Transportation

Techniques for estimating losses of sediments and associated chemical contaminants during transportation of dredged material are not available for most transportation modes. Pipeline breaks, scow spillage, and truck accidents can be expected, but the frequency of such events have not been documented, and there has been little effort to quantify the associated losses.

A priori predictive techniques for losses from scows due to volatilization are available.

Pretreatment and disposal facilities

Key contaminant migration pathways, techniques for estimating losses, and qualitative indications of predictive reliability for pretreatment and CDFs are identified in Table 25. Pretreatment and confined disposal are remediation components for which engineering controls on contaminant losses are most practical.

Table 25
Availability and Reliability of Contaminant Loss Estimation Techniques for Pretreatment and Confined Disposal Facilities

Pathway	Available	Reliability	Ease of Use
Effluent			
Hydraulic	Yes	High	Simple
Mechanical	No	--	--
Leachate	Yes	Moderate	Simple but Complicated
Volatilization	Yes	Low	Difficult
Runoff	Yes	High	Simple

Contaminant migration pathways for pretreatment and CDFs are similar because both facilities confine dredged material solids. There is always a potential for leachate and volatile loss pathways to be of concern when considering pretreatment and confined disposal. In addition, hydraulic placement of dredged material in pretreatment and CDFs will involve an effluent pathway.

The relative significance of these contaminant migration pathways is contaminant and facility design specific. Pathways involving movement of large masses of water, such as effluent from hydraulic filling and long-term leaching, have the greatest potential for moving significant quantities of soluble and slightly soluble contaminants. Pathways such as volatilization may also result in loss of organic chemicals during filling and storage.

A priori techniques are available for estimating losses via effluent, leachate, and volatilization from pretreatment and CDFs. However, there are few field verification data for the a priori techniques. For effluent resulting from hydraulic filling, laboratory tests are available that have been field verified. Confidence and accuracy in effluent predictions for hydraulic filling are consequently high. There are no techniques for estimating losses during mechanical filling of nearshore and in-water facilities.

Scientifically sound a priori and laboratory-based techniques are available for estimating leachate quality. To estimate leachate losses, leachate quality estimates must be coupled with computer models such as the Hydrologic Evaluation of Landfill Performance model to simulate site-specific hydrologic processes (precipitation, evaporation, infiltration, percolation, etc.). Leachate prediction techniques have not been field verified. Confidence in predictions is moderate relative to predictions for other contaminant migration pathways.

The only predictive techniques available for estimating volatile losses are a priori techniques. In cases where highly contaminated dredged material is disposed, volatile emissions should be evaluated to protect workers and others who could inhale contaminants released through this pathway. The a priori techniques were developed from chemical vapor equilibrium concepts and transport phenomena fundamentals. Predicted emission rates are primarily dependent on the chemical concentration in the dredged material, the surface area through which emission occurs, and climatic factors such as wind speed. Confidence in volatile emission calculations is low relative to predictions for other contaminant migration pathways.

Dredged material treatment

Estimation of losses associated with dredged material treatment processing follows standard engineering practice of conducting laboratory and pilot-scale treatability studies. Performance data generated by treatability studies usually provide the information on treatment process waste streams and residuals needed to estimate losses and additional treatment requirements. The pilot-scale treatability studies conducted in other elements of the ARCS program may be used in planning level evaluations of treatment alternatives and associated contaminant losses. Caution should be exercised in using these data to ensure that the treatment processes under consideration are applicable to the sediment to be remediated. Sediment characteristics—physical and chemical—and other site-specific factors can significantly affect implementability of a treatment process.

Effluent/leachate treatment

Effluent and leachate may be viewed as wastewaters and as such are amendable to conventional wastewater treatment processes. The available literature on wastewater treatment engineering provides information suitable for planning level assessments of contaminant losses associated with effluent and leachate treatment. As with dredged material treatment, before proceeding with design and final engineering calculations including contaminant losses, standard engineering practice involves conducting treatability studies. The performance data generated by treatability studies usually provide the information on treatment process waste streams and residuals needed to estimate losses and additional treatment requirements.

Example calculations

Estimates of contaminant losses for components of a remediation alternative for a specific project can be pooled to provide an estimate for the entire remediation alternative. Such estimates can then be used to rank alternatives for remediation of a specific site. This approach has been illustrated through a set of calculations for a sediment contaminated with PAHs in the Dead Man's Creek portion of the Buffalo River, New York.

The example calculations show how to normalize losses with respect to the volume of sediment to be remediated so that estimates for various pathways can be pooled and alternatives can be compared on a common basis. The remediation alternatives considered were as follows:

I Clamshell dredging with disposal in a CDF (with and without effluent and leachate controls).

II Cutterhead dredging with disposal in a CDF (with and without effluent and leachate controls).

III Clamshell dredging with stockpiling in a pretreatment facility (with and without effluent and leachate controls) followed by thermal desorption processing of the dredged material.

IV Cutterhead dredging with dewatering in a pretreatment facility (with and without effluent and leachate controls) followed by thermal desorption processing of the dredged material.

V In Situ capping.

Example contaminant-loss calculations showed that in situ capping (Alternative V) were less than the losses for all other alternatives for remediation of PAH-contaminated sediment in the Dead Man's Creek area of the Buffalo River. PAH losses associated with in situ capping were estimated to be 1,000 to 100,000 times less than the next best alternative. Loss estimates for in situ capping were significantly lower than any of the other alternatives because the only contaminant migration pathway included in the analysis of in situ capping was diffusion through the cap. Losses due to disturbance of contaminated sediment during cap placement, release of excess pore pressure during consolidation, and erosion by extreme flow events were not estimated. Subject to these limitations, the large difference between in situ capping and the other alternatives suggests that in situ capping can be a very effective means of sediment remediation. A cap can be armored to improve its stability, and in situ capping would generally only be considered for sites subject to weak erosive forces and no significant groundwater movement.

Among Alternatives I through IV, Alternative IV with loss control measures, provided the least return of contaminants to the environment. There was, however, very little difference in total contaminant losses between

disposal in a CDF and thermal desorption for low mobility contaminants such as benzopyrene and benzanthracene. Thermal desorption rather than confined disposal of benzanthracene, for example, provided only about 3 percent less return of the contaminant to the environment.

Thermal desorption of the more mobile contaminant, phenanthrene, however, resulted in a reduction of total phenanthrene losses by 56 percent over the CDF option. This comparison assumes that engineering controls over effluent and leachate losses from pretreatment and CDFs are in place. In the absence of such controls, the CDF option results in much higher contaminant losses of phenanthrene. Absence of controls increased the losses during application of the pretreatment/thermal desorption option by 56 percent, and the losses for disposal in a CDF were more than an order of magnitude larger.

Differences were evident between low and high mobility contaminants upon application of the clamshell or cutterhead dredging options. Clamshell dredging tends to release a greater quantity of resuspended sediments compared with hydraulic cutterhead dredging. Low-mobility contaminants such as benzanthracene and benzopyrene are strongly associated with the sediment particles and are therefore released in greater quantities by clamshell dredging. High mobility contaminants such as phenanthrene and anthracene, however, tend to exhibit greater losses during cutterhead dredging. The large quantities of water needed to dredge hydraulically significantly increase the mass of these compounds in the water phase and thus increase volatile, effluent and leachate losses of these more soluble compounds. Comparison of Alternative I and II with loss control measures, for example, shows approximately twice as much phenanthrene and anthracene lost during application of cutterhead dredging than during application of clamshell dredging, while significantly reducing losses for the less soluble benzanthracene and benzopyrene.

Although the magnitude of the losses during application of any of these alternatives is sensitive to the particular set of assumptions employed, the results clearly suggest that the optimum remedial alternative, that is the alternative leading to a minimum loss of contaminants, can be a strong function of the particular contaminant. In addition, the presumption that greater control of suspended particulate losses leads to greater control of contaminant losses is not entirely accurate.

Recommendations

Recommendations are provided for using contaminant loss estimates and for research needed to improve the reliability and accuracy of available estimation techniques and develop techniques where none are presently available.

Uses

The estimation techniques and the example approach to using contaminant loss estimates described in this report were designed for comparison purposes. Their best use is in relative comparison and ranking of alternatives and relative comparison of loss pathways for specific alternatives. Contaminant-loss estimation exercises conducted solely to justify a predetermined preferred alternative should be avoided. Specific recommendations for using contaminant-loss estimates are provided below.

a. Contaminant-loss estimates should be used in a preproject planning mode to help screen remediation alternatives. Contaminant-loss estimates alone are not sufficient for decision making.

b. Contaminant-loss estimates for a specific alternative should be used to indicate critical loss pathways where engineering controls should be considered and potentially provide the most return.

c. Contaminant-loss estimates for a specific alternative should be used to indicate where pathway specific laboratory testing is needed to improve estimates and provide information for evaluating the feasibility of engineering controls.

d. Contaminant-loss estimates should be used as input for risk assessment.

e. Contaminant-loss estimates for preferred alternatives should be used to demonstrate the merit of the preferred alternative relative to the no-action alternative and to indicate where engineering controls may provide benefit relative to the no-action alternative.

Research needs

Research needs for improving available contaminant-loss estimation techniques and providing estimation techniques where none are available are provided below in order of priority. The order of priority was developed within the context of freshwater sediment remediation and not maintenance dredging or remediation of estuarine sediments.

a. In situ capping—A priori and laboratory-based techniques for estimating contaminant losses during the implementation phase of an in situ capping project are a top research priority. The available contaminant loss estimation techniques for in situ capping account for diffusion losses alone. Loss estimates for in situ capping will, therefore, generally be lower than estimates for most other alternatives. However, implementation losses are probably much higher than diffusion losses and, if accounted for, could dominant the loss estimates for in situ capping. Improved techniques that account for implementation losses are needed to provide a more realistic preproject assessment of in situ

capping. Development of improved contaminant loss estimation techniques for in situ capping should be conducted such that the techniques are also applicable to capping dredged material. A field verification component will also be needed.

b. Dredged Material Treatment—Treatability studies sometimes do not provide sufficient data to fully evaluate contaminant losses. Laboratory- and pilot-scale treatability studies in which detailed data are obtained on all process streams are needed in order to fully evaluate contaminant losses and develop a database for preproject evaluation of probable contaminant losses. This information is needed to demonstrate the relative merit of treatment to other alternatives. Because treatability studies are very expensive, efforts should be made to obtain as much process and contaminant loss information as possible. Treatability studies that do not include a detailed materials balance should be avoided.

c. Volatile Emissions—Volatile emissions are potentially important from a worker health and safety viewpoint for highly contaminated sediments and dredged materials, as well as, a potentially important contaminant loss pathway. Research is needed to improve the available a priori estimation techniques and develop laboratory-based estimation techniques. A field verification component is also needed.

d. Leachate flow—The Hydrologic Evaluation of Landfill Performance computer model is an excellent model for estimating leachate flow from upland pretreatment and CDFs. Many of the assumptions on which the model is based are not readily applicable to nearshore and in-water facilities in which significant lateral seepage may occur. The model can be "tricked" to simulate losses through dikes, but a model designed to evaluate such problems would be preferred. A time-varying contaminant transport model that simulates fluctuating water levels and the attendant changes in hydraulic gradients is needed to fully evaluate leachate seepage in nearshore and in-water facilities.

e. Effluent Losses—Available estimation techniques are limited to hydraulic filling, and most of the laboratory and field data behind the available techniques are for inorganic contaminants. Additional field verification involving organic contaminants is needed to supplement the data on inorganic contaminants and provide a complete picture of the predictive capability of the modified elutriate test. In addition, research and development are needed to provide laboratory and a priori estimation techniques for effluent losses during mechanical filling of nearshore and in-water pretreatment and disposal facilities.

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Appendix A
Notation

a	empirical swing velocity significance factor
A	dredging area or area available for mass transfer
A_c	capped area
A_v	surface area of vessel
A_w	water surface area
b	empirical tangential velocity significance factor
B	Collins bucket parameter
C	solubility in water
CEF	contaminant containment efficiency factor for the effluent pathway
CEF_{EFF}	containment efficiency based on effluent pathway only
C_a	dissolved concentration of chemical in air
C_{at}	background concentration of chemical in air at the dredged material-air interface
C_A	water concentration of A
C_{AO}	pore water concentration in original sediment
C_c	colloid concentration in water (DOC)
C_d	dissolved chemical concentration in water
C^*_d	hypothetical dissolved chemical concentration in equilibrium with background air
C_{doc}	colloidal specie concentration
$C_{EFF,TOT}$	total concentration of contaminant i in effluent
$C_{INF,TOT}$	total concentration of contaminant i in influent
C_p	suspended solids concentration
C_{ps}	suspended solids contaminant concentration
C_{pw}	pore water concentration of A
C_{pw}^o	pore water concentration in original sediment
C_s	contaminant concentration in the sediment
C_{sorb}	concentration of contaminant sorbed to solid phase
C_{sL}	leachable metal concentration in dredged solids
C_{total}	whole water contaminant concentration
C_w	aqueous phase contaminant concentration
C^*	background water concentration above cap
C_w^*	hypothetical dissolved chemical concentration in equilibrium with background air

C^*_w	water concentration of A
d	effective diameter of sediment grains
d_p	particle diameter
D	depth of dredging
D'	ratio of colloidal specie diffusivity to
D_A	molecular diffusivity (1 = air, 2 = water, 3 = sediment pore spaces)
D_{A1}	molecular diffusivity of chemical A in air
D_{A2}	diffusivity of A in water
D_{A3}	effective diffusivity of chemical A
D_B	molecular diffusivity of chemical B in water
D_b	effective biotubation diffusion coefficient
D_{ch}	diameter of cutterhead
D_{eff}	effective diffusivity, bracketed term equation
D_F	fractional depth of cut as a function of cutterhead diameter
D_p	dispersion coefficient
D_v	effective diameter of the vessel
E_{A3}	effective dispersion coefficient in the medium
f_b	fraction of bucket dredge cycle on bottom
f_d	fraction of bucket dredge cycle for bucket insertion
f_o	fraction of bucket dredge cycle out of water
f_r	fraction of sediment resuspended during dredging
f_u	fraction of bucket dredge cycle for bucket withdrawal
F_D	cutterhead resuspension rate factor accounting for degree of burial
F_F	cutterhead resuspension rate factor accounting for other factors
f_{oc}	fraction organic carbon in sediment or dredged material
h_b	water depth (bucket dredging)
h_1	pond water elevation above base of dike
h_2	outside water elevation above base of dike
H	Henry's constant
H_1	head at crown of water table mound
H_2	head outside the confined disposal facility (CDF)
H_{ch}	height of cutterhead
i	contaminant index
K	hydraulic conductivity of the dike
K_{A1}	overall mass transfer coefficient based on air-side concentrations
K_{A2}	overall mass transfer coefficient based on water-side concentrations
K_b	benthic mass transfer coefficient
K_c	colloid-water partition coefficient of A
K_d	contaminant-specific equilibrium distribution coefficient
K_G	gas-side mass transfer coefficient
K_L	liquid-side mass transfer coefficient
K_O	colloid-water partition coefficient
K_{OC}	colloid-water partition coefficient of A
K_{OG}	overall gas-side mass transfer coefficient
K_{OL}	overall liquid-phase mass transfer coefficient

K_{ov}	overall mass transfer coefficient
L	horizontal distance separating surface of pond and surface of outside water body
L_{bc}	characteristic length of clamshell bucket
L_{Bio}	bioturbation layer thickness
L_{cap}	effective depth of cap diffusive layer
L_{ch}	length of cutterhead
L_v	vessel length
m	mass of contaminant released
M	weight of activated carbon
M_A	molecular weight of chemical A
M_B	molecular weight of chemical B
M_{cs}	mass of contaminant in the solid phase
M_{cw}	mass of contaminant in the aqueous phase
M_s	mass of solids
M_w	mass of water
n_e	instantaneous flux of chemical A through the dredged material
n_{EDM}	instantaneous flux of chemical A through the dredged material-air interface at time t
N_A	flux of contaminant A in free water
N_{ss}	steady-state flux
N_w	flux through air-water interface
p_{A_*}	background partial pressure of chemical A in air
p_A^*	partial pressure of component A in air at exposed surface (in equilibrium with dredged material)
p^*_A	pure component vapor pressure of chemical A
p_{A1}	partial pressure of component A in background air
P	total atmospheric pressure
q	discharge per unit length of dike
Q	volumetric flow of water
Q_d	volumetric flow of water through the averaging volume
R	universal gas constant, 82.1 atm
R_1	distance from center of CDF to edge of water table crown
R_2	distance from center of CDF to the dike
R_A	release rate of contaminant
$R_A(t)$	release rate of contaminant, at time t
$R_A(t \rightarrow \infty)$	release rate of contaminant, at steady-state
R_D	rate of contaminant release
$R_{D,b}$	contaminant release rate
$R_{D,ch}$	dissolved contaminant release rate for a cutterhead dredge
$R_{D,V}$	volatile chemical emission during dredging
$R_{d,b}$	dissolved contaminant release for a clamshell dredge
R_f	retardation factor as defined by Equation 53
R_p	release rate of resuspended particles
$R_{p,b}$	particle resuspension rate
$R_{V,es}$	volatile emission rate for chemical A from exposed sediment
$R_{V,esp}$	volatile emission rate from partially filled vessel, g/cm^2 sec
S	interphase contaminant transfer
S_c	Schmidt number

t	time
T	temperature
T_c	dimensionless cycle time
U	Darcy or superficial water velocity
U_d	net deposition velocity
V_c	tangential velocity of cutterhead relative to axis of rotation
V_{cb}	volume of the clamshell bucket
V_{curr}	water velocity
V_i	cutterhead hydraulic inlet suction velocity
V_s	swing velocity of cutterhead dredge
V_t	maximum tangential velocity of cutterhead relative to fixed axis
V_x	wind speed
v	average pore water velocity
v_1	wind velocity
v_2	water current velocity
v_3	deposition velocity of sediment particles
W_A	total contaminant concentration in sediment (dry basis)
W_{Ap}	contaminant concentration sorbed to sediment
x	amount of substance adsorbed
z	distance through water, into the sediment or cap
Z	water depth in meters
Z_h	distance from top of vessel, cm
α	height of area swept by cutterhead as fraction of cutterhead dimensions
β	length of area swept by cutterhead as fraction of cutterhead dimensions
ϵ	sediment porosity
ϵ_1	air-filled porosity
γ	Bohlen sweep area correction factor
π	3.14159...
ρ_1	air density
ρ_s	in situ bulk density of the sediment
ρ_w	water density
ρ_b	bulk density
ν	kinematic viscosity
ν_1	kinematic viscosity of air
τ	dispersivity
τ_{cb}	clamshell bucket dredge cycle time

Subscript

1	air
2	water
3	sediment

Appendix B
A Priori Estimation of Distribution Coefficients

Introduction

Application of methods for estimation of contaminant losses presented in the main body of this report often requires estimation of equilibrium distribution coefficients between sediment, water, and/or air media. The distribution coefficient is defined as the equilibrium ratio of the concentration of the contaminant in one phase divided by the concentration of the contaminant in an adjacent phase at equilibrium.

Distribution coefficients represent the maximum amount of contaminant that can be partitioned into an adjacent media given a concentration within the sediment or dredged material solids. In general, distribution coefficients are functions of temperature and concentration and the chemical properties of the adjacent phases. By definition, however, distribution coefficients are not functions of time nor the rate of mixing within the phases. Although the coefficient is generally a function of concentration, the available data rarely supports models that incorporate this behavior and the distribution coefficient is typically assumed to be independent of contaminant concentration. Thus, the equilibrium concentration in a phase is assumed to be linearly dependent on the concentration in the adjacent phase, or for partitioning between water and sediment solids (Equations 24, 25, and 26 of the main text)

$$C_w = \frac{C_s}{K_d}$$

and for partitioning between water and air (Equations 31 and 32 of main text).

$$C_A = HC_w$$

Of interest in this report is partitioning of contaminants between (a) sediment or dredged material solids and pore water, and (b) sediments or dredged material and adjacent air.

Contaminants of primary interest include metals and hydrophobic organic chemicals that tend to partition strongly to sediments and thus pose long-term sediment quality problems. Elemental species and hydrophobic organic materials partition to sediments by very different mechanisms. In addition, elemental species tend to be nonvolatile, and their partitioning from sediments to air or from water to air need not be considered. Thus, presentation of methods for the prediction of distribution coefficients of the materials of interest will be separated by type of contaminant and the environmental interface under consideration.

Hydrophobic Organic Species

Hydrophobic organic species are characterized by their low-water solubility. This class of compounds includes almost all hydrocarbons and substituted organic compounds except the simple alcohols and phenols. Observations have suggested that the partitioning of these compounds between water and a particular soil or sediment is largely controlled by the hydrophobicity of the compound, for example, as measured by the distribution coefficient of the compound between water and octanol (K_{ow}). In addition, observations have suggested that the partitioning of a particular hydrophobic organic to different soils or sediments is largely controlled by the organic carbon content of the solid phase. This is consistent with the concept of "like dissolves like" for defining the mechanism of sorption onto the sediment or soil phase. To a first approximation, the distribution coefficient of a hydrophobic organic compound between sediment or dredged material and water is given by

$$K_d = K_{oc} f_{oc}$$

where

K_{oc} = organic carbon-based distribution coefficient

f_{oc} = fraction organic carbon in sediment or dredged material

The organic carbon-based distribution coefficient (K_{oc}) is a measure of the hydrophobicity of the organic compound. The fraction organic carbon is a chemical property of the sediment or dredged material. This procedure for estimating the distribution coefficient thus separates the problem into determining a single parameter characterizing the chemical and a single parameter characterizing the sediment. The organic carbon-based distribution coefficient is determined by measuring the sorption of a particular compound on a sediment or soil and normalizing by the organic carbon in the solid phase.

Lyman, Reehl, and Rosenblatt (1990) indicate that measured K_{oc} values are reasonably constant for a given compound sorbing to different soils and sediments. The coefficient of variation of K_{oc} with different soils was 10 to 140 percent (Lyman, Reehl, and Rosenblatt 1990). Selected values of K_{oc} are included in Table B1 with values of other relevant physical properties. Data from this table should be used with care recognizing that other data sources might provide values that are orders of magnitude different. The values chosen for the table, however, were selected based on consistency with similar compounds and the availability of corroborating data, where possible.

Table B1
Physical Properties of Selected Compounds[1]

Compound	MW	P_v mm Hg	Water Solubility mg/ℓ	H_c atm-m3 mol	Log K_{oc}
Acenaphthene	154	0.005	3.47	2.9×10^{-4}	1.25
Aldrin	365	$2.3E \times 10^{-5}$	0.017	6.5×10^{-4}	2.61
Anthracene	178	2×10^{-4}	0.045	0.001	4.27
Benzo[a]pyrene	252	5.5×10^{-9}	0.004	4.6×10^{-7}	6
Chlordane	410	1×10^{-5}	0.056	9.6×10^{-5}	5.15
Chrysene	228	6.3×10^{-9}	0.002	9.5×10^{-7}	5.39
p,p'-DDT	354	1.9×10^{-7}	0.003	3×10^{-5}	5.38
Dieldrin	381	3×10^{-6}	0.2	7.5×10^{-6}	4.55
Fluoranthene	202	5×10^{-6}	0.26	5.1×10^{-6}	4.62
Fluorene	166	7×10^{-4}	1.6	9.7×10^{-5}	3.7
Hexachlorobenzene	285	1×10^{-5}	0.005	7.5×10^{-4}	3.59
Indenopyrene	276	1×10^{-10}?	0.062	5.9×10^{-10}	7.49
PCB-1242 (Avg)	261	4×10^{-4}	0.24	5.7×10^{-4}	3.71
PCB-1254 (Avg)	327	7.7×10^{-5}	0.057	5.8×10^{-4}	5.61
Pentachlorophenol	266	1.7×10^{-4}	20	3×10^{-6}	2.95
Phenanthrene	178	6.8×10^{-4}	1	1.6×10^{-4}	3.72
Pyrene	202	6.9×10^{-7}	0.135	1.4×10^{-6}	4.66
TCDD	322	1.4×10^{-9}	0.0002	3×10^{-6}	6.66

[1] From *Groundwater Chemicals Desk Reference* by Montgomery & Welkom, Lewis Publishers (1990). Henry's Law Constants (H) calculated from vapor pressure and solubility. A11 data are estimates at 25 °C.

K_{oc} can also be estimated on the basis of correlations, for example with solubility or the octanol-water partition coefficient (K_{ow}). For example, Curtis, Reinhard, and Roberts (1986) have presented the correlation

$$\log K_{oc} = 0.92 \log K_{ow} - 0.23$$

As indicated by this correlation, K_{oc} and K_{ow} tend to be the same order of magnitude. K_{ow} is a good indicator of the ability of a compound to partition between organic and water phases and has been correlated with bioconcentration and water solubility in addition to the sediment-water sorption coefficient. In addition, Hansch and Leo (1979) have developed a procedure for estimating K_{ow} using only the molecular structure of the compound. A K_{ow} for essentially any compound whose structure is known can be estimated by this method. Procedures and examples of various methods of estimating K_{ow}, K_{oc}, and K_d are detailed in Lyman, Reehl, and Rosenblatt (1990).

The use of the approach discussed above is limited to situations where sorption of the organic compound to the sediment is controlled by hydrophobic interactions. Hydrophilic compounds do not partition in the same manner as the hydrophobic compounds. In addition, at very low organic carbon contents, for example at 0.1 percent or less, direct sorption to mineral surfaces in the sediment or dredged material becomes important, and partitioning is no longer simply a function of organic carbon content. Organic acids and bases, phenolic compounds, and many pesticides can also deviate significantly from the behavior suggested above at a pH that causes significant ionization of the species. The acid dissociation constant (pK_a) is a convenient indicator of the extent of ionization. At a pH = pK_a, half of the compound is in its ionized state. At a pH = $pK_a + 2$, the concentration of the ionized form is 100 times that of the concentration of the neutral species and the reverse is true for pH = $pK_a - 2$. Thus 2,4,6-trichlorophenol, with a $pK_a = 7.42$ (Montgomery and Welkom 1990), will interact hydrophobically with soils or sediments and exhibit a K_{oc} of about 1000 at pH ≤ 6, while at pH > 9, essentially no sorption will be observed.

In addition to the limitations outlined above, the assumption of constant K_d typically limits the validity of the entire approach to low-contaminant concentrations. A critical sediment loading can be defined as the sediment concentration that is in equilibrium with a saturated water solution, i.e., water containing the compound at its solubility limit. Linear partitioning is typically not observed at sediment concentrations that are near the critical loading. In addition, under no circumstances should linear partitioning between sediment and water be applied at sediment concentrations that exceed the critical loading. It is possible to measure a sediment concentration that exceeds critical loading due to the presence of a separate nonaqueous phase or due to nonlinear partitioning. It is not possible, however, to achieve a truly dissolved concentration that exceeds the water solubility of the compound.

At low-sediment concentrations, for example, in sediment suspended in the water column, linear partitioning is also apparently no longer observed; distribution coefficients depend on sediment concentration, perhaps due to the presence of colloidal material (Gschwend and Wu 1985). Baker et al. (1991) observed that field data on partitioning to dilute suspended solids rarely fit the

linear partitioning model, perhaps due to the presence of colloids and the kinetics of solids uptake of the sorbing contaminant.

Elemental Species

The partitioning of metals and other elemental species between sediments or dredged material and water is much more complicated than that for hydrophobic organic species. The total concentration of an element in sediment or dredged material is the sum of that which is chemical bound in various geochemical phases (Brannon et al. 1976), physically sorbed, and dissolved in the interstitial waters. Generally, the chemically bound portion, which usually comprises 90 to 99 percent of the contaminant mass, is immobile and unavailable for partitioning into the aqueous phase under most environmental conditions. The physically sorbed portion is exchangeable through ion exchange, and the dissolved form is mobile. In reality, the elemental species exist in the sediment in a variety of forms. Brannon et al. (1976) identified at least the following sediment geochemical phases for sorbed and fixed contaminants:

a. Adsorbed on the surface of charged mineral and organic surfaces.

b. Oxides, hydroxides, and hydrous oxides of Mn and Fe.

c. Chemically bound in organic matter.

d. Chemically bound with sulfides.

e. Bound within the crystalline lattice (residual).

Brannon et al. (1976) devised a selective extraction scheme that treated sediment samples with increasingly harsh treatments to define the interstitial water, exchangeable, easily reducible, organic + sulfide, moderately reducible and residual fractions. The contaminants removed with each fraction were assumed to indicate the proportion of the original element in each of the chemical forms identified above. The exchangeable fraction, for example, was defined by the amount of the elemental species that could be extracted with ammonium acetate. Brannon et al. (1976) applied the selective extraction procedure to sediments from three areas, Ashtabula, Ohio (freshwater), Mobile Bay, Alabama (estuarine), and Bridgeport, Connecticut (saltwater). The exchangeable fraction of iron and manganese was found to correlate well with the interstitial water concentrations with an exchangeable fraction-water distribution coefficient of about 9 ℓ/kg for both.

Zinc and nickel correlated less well with the exchangeable concentration and exhibited an average exchangeable fraction-water distribution coefficient of 9 and 5, respectively. Copper and cadmium were not found in detectable quantities in the exchangeable fraction and interstitial water; concentrations were 100 to 1,000 times lower than for the other species. Thus, the

exchangeable fraction, as defined by the amount of contaminant extracted with ammonium acetate, was a reasonable indicator of the interstitial or presumed equilibrium water concentrations. This would assume no changes in the chemical state of the sediment. Oxidation of the sediment, for example, tends to mobilize many of the metals and other elemental species that might be present in the sediment.

Brannon, Myers, and Price (1992) and Environmental Laboratory (1987) conducted further tests to define distribution coefficients for elemental species in freshwater sediments at Indiana Harbor, Indiana, and Hamlet City Lake, North Carolina. Both sequential batch leach tests and continuous column leaching tests were employed. The combination of batch and continuous tests has several advantages over traditional procedures to determine mobile elemental fractions. The batch test ostensibly defines equilibrium conditions for a particular sediment to water ratio, while the continuous test should indicate the dynamics of the leaching process. However, further research is required to fully define the capabilities and procedures for conducting and analyzing batch and column leach tests.

Although definitive procedures for the a priori estimation of elemental distribution coefficients do not exist, general guidelines for the magnitude of these distributions coefficients do exist. The elemental partitioning studies (Brannon et al. 1976) and the batch equilibrium and column leaching studies (Environmental Laboratory 1987; Brannon, Myers, and Price 1992) indicate that distribution coefficients for most metals in freshwater sediments range from 1 to 10 ℓ/kg. Table B2 lists the observed range of distribution coefficients adapted from a table presented by Dragun (1988). The distribution coefficients summarized in Table B2 are the ratio of the total soil or sediment concentration (i.e., the sum of both exchangeable and chemically fixed elements) to the adjacent water concentrations. As indicated by the discussion above, it is believed that a more generally useful partition coefficient would be one based on the exchangeable concentration of the element.

Air-Water Partitioning

Essentially all of the elemental species of interest in sediments and dredged material are nonvolatile. Therefore, the discussion here will be limited to evaporation of hydrophobic organic species from water.

The general expression of equilibrium at a fluid-fluid interface is based on the concept of continuity of component activity across the interface. This is most easily expressed as continuity of fugacity, which is an effective pressure corrected for nonidealities.

The fugacity on each side of the interface for a contaminant i is written as the product of a standard state fugacity, the mole fraction of the contaminant and a correction for nonideality, the activity coefficient.

Table B2
Ranges for Distribution Coefficients for Various Soils and Clays (After Dragun 1988)

Element	Observed K_d ml/g	Logarithmic Mean	Standard Deviation (Error Factor)[1]
Ag	10-1,000	110	3.7
Am	1-47,230	810	20
As(III)	1-8.3	3.3	1.8
As(V)	1.9-18	6.7	1.6
Ca	1.2-9.8	4	2.2
Cd	1.3-27	6.7	2.5
Ce	58-6,000	1,100	3.7
Cm	93-51,900	3,300	6.7
Co	0.2-3,800	55	10
Cr(III)	470-150,000	2,200	3.3
Cr(VI)	1.2-1,800	37	9
Cs	10-52,000	1,100	6.7
Cu	1.4-333	22	3
Fe	1.4-1,000	55	5.5
K	2-9	5.5	1.6
Mg	1.6-13.5	5.5	1.6
Mn	0.2-10,000	148	15
Mo	0.4-400	20	8.2
Np	0.2-929	11	10
Pb	4.5-7,640	100	5.5
Po	196-1,063	550	2
Pu	11-300,000	1,800	10
Ru	48-1,000	600	2.7
Se(IV)	1.2-8.6	2.7	2
Sr	0.2-3,300	27	7.4
Tc	0.003-0.28	30	3
Th	2,000-510,000	60,000	4.5
U	11-4,400	45	3.7
Zn	0.1-8,000	16	6.7

[1] Standard deviation as a multiplicative factor of mean.

$$f_i = x_i \gamma_i f_i^o$$

where

f_i = fugacity of component i

x_i = mole fraction of i

γ_i = activity coefficient of component i

f_i^o = standard state fugacity of component i

At the air-water interface, the equality of fugacities implies

$$f_i^{air} = f_i^{water}$$

The relationship between concentrations (or mole fractions) of the contaminant across the air-water interface then depends on the specification of activity coefficients and standard state fugacities in each of the phases.

The standard state fugacity is normally taken as the pressure that would be exerted by the pure component (i.e., contaminant i) at the same temperature, pressure, and phase as the mixture. Thus the standard state fugacity of a component in air would be the pressure exerted by a pure component vapor in the atmosphere (i.e., 1 atm). In addition, since gases act ideally at low pressure, the activity coefficient in the atmosphere is 1. Similarly, the standard state fugacity of a component in water would be the pressure exerted by a pure component liquid at the desired temperature. This is just the pure component vapor pressure (or saturation pressure) at that temperature. Estimation of the activity coefficient in water is more difficult due to the typically large deviations from ideality (i.e., $\gamma_i^{air} \neq 1$). Hydrophobic organics exhibit a low solubility in water, and even a saturated water solution is not changed appreciably by the presence of the organic species. Thus, the water-organic interactions and the activity coefficient are essentially independent of concentration. In addition, a saturated solution exerts the same component pressure as a pure phase since the addition of any more of the component produces such a phase. Thus, continuity of fugacities across the air-water interface for a saturated water solution is described by

$$Y_i(1)P = P_v = X_{is}\gamma_i P_v$$

or

$$\gamma_i = \frac{1}{X_{is}}$$

where

y_i = mole fraction of i in the air (y_iP is the component partial pressure)

P = total pressure (1 atm)

P_v = pure component vapor pressure of i (= y_iP for saturated air)

x_{is} = mole fraction of i in water at solubility limit

γ_i = activity coefficient of i in water

Based on this approach, the relationship between air concentration (as measured by air phase mole fraction, y_i and water concentration (as measured by water phase mole fraction, x_i) is given by

$$y_iP = \frac{x_iP_v}{x_{is}} = Hx_i$$

where

$H = P_v/x_{is}$, a Henry's Law Constant

Thus, the air-water equilibrium for a hydrophobic organic compound is also governed by a linear partitioning law with an essentially constant distribution coefficient. This is valid as long as the water phase activity coefficient is independent of the concentration of the partitioning contaminant, an assumption that is generally good for low solubility, hydrophobic organic compounds. This approach cannot be applied to hydrophilic organic compounds such as phenols, low molecular weight alcohols, and organic acids or bases.

The Henry's Law Constant, H, is defined above as the ratio of a vapor pressure and the solubility in mole fraction units. It is often convenient to define solubility in concentration, or mass per volume units. The equivalent Henry's Law Constant (H_c) is the ratio of the pure component vapor pressure to the solubility in these concentration units. The relationship between the partial pressure in the air (y_iP) and the water concentration then becomes

$$Y_iP = H_cc_i$$

Henry's Law Constants reported in Table B1 are the H_c as defined here with atmospheres used as the unit of pressure and concentration measured in mole/cubic meter. The values for H_c shown in Table B1 were calculated from

vapor pressure and solubility data. This approach was taken because several independent measurements of solubility and vapor pressure are reported by Montgomery and Welkom (1990), whereas typically only a single Henry's Law Constant value is reported. It is also convenient at times to use concentration units in the air phase. The Henry's Law Constant is then dimensionless if the same mass and volume units are used to define the concentrations in the air and water phases. Note also that although H was used to indicate unit of pressure/mole fraction and H_c units of pressure/concentration, this notation is not standardized. Errors have often resulted from the use of incorrect units for Henry's Law Constants, especially when this quantity is used in mass transfer two-layer resistance models.

Partitioning Between Sediments Or Dredged Material and Air

The partitioning of hydrophobic organics between exposed sediments or dredged material and air is generally equivalent to that defined in the previous section. As long as the volumetric water content of the sediments is more than a few percent, water will tend to coat the surface of the sediment particles. Phase equilibrium is then controlled by the water-air interface. The equilibrium interstitial water concentration as defined by the sediment-water equilibrium can then be used with a Henry's Law Constant as defined by the preceding section to determine the equilibrium partial pressure of the contaminant above the exposed sediment.

When the volumetric water content of the sediment or dredged material is less than a few percent, the equilibrium partial pressure of hydrophobic organics of the surface of the sediment begins to decrease dramatically. Direct sorption of the organic molecules onto the sediment surface can take place, a process which can significantly increase the capacity of the solid phase to retain contaminants. The partitioning between the air and sediment phase is largely a function of the exposed surface area of the sediment phase. Valasaraj and Thibodeaux (1992) have presented data on a number of hydrophobic organic compounds sorbing to dry, moist, and wet soils. On completely dry soils or sediments, sorption of vapors onto the soil surface is apparently controlled by the surface area of the sorbet and is a nonlinear function of concentration. In general, however, sediments exposed due to tides or dredged material in a confined disposal facility will rarely achieve the dry state necessary for this mechanism to become important. Even dryland soils are unlikely to be influenced by this process except in the upper few centimeters of soil. In addition, the assumption of water-wet sediment will provide a conservative upper bound to the equilibrium partial pressure of a contaminant above a dried sediment. For these reasons, equilibrium at a dry sediment-air interface will not be considered further.

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Appendix C
Input Parameters for Disposal From an Instantaneous Dump (DIFID), Disposal From Continuous Discharge (DIFCD), and Disposal From a Hopper Dredge (DIFHD) Models

Table C1
Model Input Parameters

Parameter	Models[1]	Units	Options[2]
Disposal Site Descriptions			
Descriptive title	I,C,H		
Gridpoints (left to right)	I,C,H		
Gridpoints (top to bottom)	I,C,H		
Distance between gridpoints	I,C,H	feet	
Constant water depth	I,C,H	feet	C
Gridpoints depths	I,C,H	feet	V
Points in density profile	I,C,H		
Depth of density point	I,C,H	feet	
Density at profile point	I,C,H	g/cc	
Bottom slope in x-direction	I,H	degrees	
Bottom slope in z-direction	I,H	degrees	
Site boundary grid locations	I,C,H		

(Sheet 1 of 4)

[1] The use of a parameter in the DIFID, DIFCD, and DIFHD models is indicated in the table by an I, C, or H, respectively.
[2] The use of a parameter for the constant depth option or variable depth option is indicated in the table by a C or V, respectively. Other optional uses for parameters are so indicated.

Table C1 (Continued)

Parameter	Models[1]	Units	Options[2]
Disposal Operation Descriptions			
Volume of material in barge	I	cu yd	
Discharge flow rate	C,H	cfs	
Radius of discharge	C,H	feet	
Discharge depth	C,H	feet	
Angle of discharge	C	degrees	
Dredge course	C	degrees	
Vessel speed	C	ft/sec	
Barge velocity in x-direction	I	ft/sec	
Barge velocity in z-direction	I	ft/sec	
Barge length	I	feet	
Barge width	I	feet	
Post-disposal depth	I	feet	
Bottom depression length in x-direction	I,H	feet	Optional
Bottom depression length in z-direction	I,H	feet	Optional
Bottom depression depth	I,H	feet	Optional
X-coordinate of disposal operation	I,C,H	feet	
Z-coordinate of disposal operation	I,C,H	feet	
Disposal duration	I,C,H	seconds	
Time from start of tidal cycle	I,C,H	seconds	
Number of hopper bins opening together	H		
Distance between bins	H	feet	
Disposal Site Velocity Descriptions			
Type of velocity profile	I,C,H		
Tidal cycle time of velocity if constant profile not used	I,C,H	seconds	V
Vertically averaged velocity in x-direction at gridpoints	I,C,H	ft/sec	V
Vertically averaged velocity in z-direction at gridpoints	I,C,H	ft/sec	V
Velocity in x-direction at upper point	I,C,H	ft/sec	C
Depth of upper point for x-direction velocity	I,C,H	feet	C
Velocity in x-direction at lower point	I,C,H	ft/sec	C
Depth of lower point for x-direction velocity	I,C,H	feet	C
Velocity in z-direction at upper point	I,C,H	ft/sec	C
Depth of upper point for z-direction velocity	I,C,H	feet	C

(Sheet 2 of 4)

Table C1 (Continued)

Parameter	Models[1]	Units	Options[2]
Disposal Site Velocity Descriptions (Continued)			
Velocity in z-direction at lower point	I,C,H	ft/sec	C
Depth of lower point for z-direction velocity	I,C,H	feet	C
Material Descriptions			
Water density at dredge site	I,C,H	g/cc	
Number of solid fractions	I,C,H		
Solid fraction descriptions	I,C,H		
Solid fraction specific gravity	I,C,H		
Solid fraction volumetric concentration	I,C,H	cu ft/cu ft	
Solid fraction settling velocity	I,C,H	ft/sec	
Solid fraction deposited void ratio	I,C,H		
Moisture content of material in barge as multiple of liquid limit	I		Cohesive
Bulk density of dredged material	I,C,H	g/cc	
Liquid phase contaminant concentration	I,C,H	mg/ℓ	Optional
Background contaminant concentration	I,C,H	mg/ℓ	Optional
Sediment contaminant concentration	I,C,H	mg/kg	Optional
Contaminant water quality criteria	I,C,H	mg/ℓ	Optional
Toxicity criteria	I,C,H	percent	Optional
Model Coefficients			
Settling coefficient	I,C,H		
Apparent mass coefficient	I,C,H		
Drag coefficient	I,C,H		
Form drag for collapsing cloud	I,C,H		
Skin friction for collapsing cloud	I,C,H		
Drag for an ellipsoidal wedge	I,C,H		
Drag for a plate	I,C,H		
Friction between cloud and bottom	I,C,H		
Horizontal diffusion dissipation	I,C,H		
Vertical diffusion coefficient	I,C,H		
Cloud/ambient density gradient ratio	I,C,H		
Turbulent thermal entrainment	I,H		
Entrainment in collapse	I,H		
Jet entrainment	H,C		
Thermal entrainment	H,C		
Entrainment by convection in collapse	C		
Entrainment due collapse of element	C		

(Sheet 3 of 4)

Table C1 (Concluded)			
Parameter	**Models[1]**	**Units**	**Options[2]**
Input, Output, and Execution Descriptions			
Processes to simulate	I,C,H		
Type of computations to perform for initial mixing	I,C,H		
Number of depths for initial mixing calculations	I,C,H		
Depths for initial mixing calculations	I,C,H	feet	
Duration of simulation	I,C,H	seconds	
Time steps for mixing calculations	I,C,H		
Convective descent output option	I,C,H		
Collapse phase output option	I,C,H		
Number of print times for initial mixing output	I,C,H		
			(Sheet 4 of 4)